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Side Resistance of Drilled Shafts in Weak Fine Grained Sedimentary Rock

Timothy D. Stark[1], Ahmed H. Baghdady[2], Abdolreza Osouli[3], Heather Shoup[4], and Michael A. Short[5]

[1] Professor of Civil and Environmental Eng., Univ. of Illinois at Urbana-Champaign, tstark@illinois.edu, corresponding author.
[2] Project Manager, Langan International, LLC, 300 Kimball Drive, 4th Floor, Parsippany, NJ 07054, abaghdady@langan.com.
[3] Associate Professor of Civil and Environmental Eng., Southern Illinois Univ. at Edwardsville, aosouli@siue.edu.
[4] Central Office Geotechnical Engineer, Central Bureau of Materials Illinois Dept. of Transportation, Heather.Shoup@illinois.gov.
[5] Materials Engineer, District 3, Illinois Dept. of Transportation, Michael.Short@illinois.gov.

ABSTRACT

Load transfer mechanism in side resistance of rock socketed drilled shafts has been studied for the past four decades using results of axial load tests and theoretical methods. Various models for prediction of side resistance have been proposed. Only few studies (e.g., Horvath et al. 1983; Rowe and Armitage 1987; Hassan et al. 1997, Miller 2003, Abu-Hejleh et al. 2003) have been completed on socket side resistance of drilled shafts in weathered and fractured fine-grained rock. These studies, however, were based on only a limited number of load test data. A survey of current predictive models has been conducted. This survey shows most of the current models include strong and intact rocks in their databases. Almost all of the current models use a power function to correlate side resistance of rock socket to rock unconfined compressive strength. A database of side resistance of large diameter drilled shafts in only weak fine-grained rocks, such as, weak shales, mudstones, and siltstones (i.e., Intermediate Geomaterial first introduced by O'Neil et al. 1996, Hassan et al. 1997, and O'Neill and Reese 1999) has been complied in this study. The range of weak rocks considered herein corresponds to an unconfined compressive strength of 0.48 to 4.8 MPa. Analysis of this database shows that a linear model best predicts the side resistance of drilled shafts in weak fine-grained sedimentary rocks.

Key words: Rock socketed drilled shafts, side resistance, weak fine-grained sedimentary rocks, unconfined compressive strength

INTRODUCTION

Drilled shafts are commonly used to support large structural loads where surficial soils do not provide sufficient bearing capacity for shallow foundations. To support these loads, drilled shafts are often socketed into weak fine-grained rocks, which are at the boundary between clays and rock that have unconfined compressive strengths of 0.48 to 4.8 MPa (10 ksf to 100 ksf) (Kulhawy and Phoon, 1993; Hassan et al. 1997). A rock socketed drilled shaft distributes applied axial loads to side and tip resistance. Allocation of axial load between these two components of resistance depends on relative stiffness of the shaft concrete and the surrounding rock, length of the rock socket, and allowable axial displacements. Drilled shafts in weak sedimentary rocks obtain most of their axial capacity by mobilizing side resistance along the drilled shaft/socket interface (Horvath, 1978; Horvath and Kenney, 1979; Horvath, 1982). Side resistance is usually mobilized at small displacements along the shaft/socket interface, and it remains constant after failure (Rosenberg and Journeaux 1976).

Since the 1960s, many full−scale load tests have been conducted on drilled shafts socketed in rock. However, only Williams (1980a) compiled a database that focuses on drilled shafts in weak fine-grained rocks. Therefore, most available design methods were developed using databases that include load tests in both weak and strong rocks.

Only a few researchers (e.g., Miller 2003; Abu-Hejleh et al. 2003; Abu-Hejleh and Attwooll 2005) have studied the applicability of available predictive models to drilled shafts in weak fine-grained rocks. Although their work provides valuable information on this matter, their databases include a limited number of load tests against which predictive methods for estimating side resistance can be evaluated.

In this paper, some of the available predictive models for side resistance are reviewed and compared to the available load test data developed herein, and then the axial load transfer via side resistance is discussed for weak fine-grained rocks. The paper then presents recommendations and an empirical design correlation for predicting side resistance of drilled shafts in weak sedimentary fine-grained rocks.

AVALAIBLE SIDE RESISTANCE MODELS

Analytical studies and load test measurements show that side resistance accounts for a large percentage of mobilized axial capacity of drilled shafts socketed in weak fine-grained rocks (Horvath and Kenney 1979). Therefore, many designers prefer to design drilled shafts to take axial loads in side resistance, as opposed to accounting for combined side and tip resistance (Miller 2003). In important projects, full–scale load tests can be used to determine side resistance of the rock socket. In small projects, load tests can be cost prohibitive so predictive models are used for determination of side resistance. Many of these predictive models, however, are developed based on load tests that include both weak and strong rocks (e.g., Rosenberg and Journeaux 1976; Horvath and Kenney 1979; Williams et al. 1980; Rowe and Armitage 1987; Carter and Kulhawy 1988; Prakoso 2002; Kulhawy et al. 2005). Load transfer in side resistance, however, is different for weak and strong rocks. This important concept has been stressed by Teng (1962) where he differentiates between load transfer in hard and soft rocks and is further discussed by Kulhawy et al. (2005). It is, therefore, important for designers to be familiar with the background of available predictive methods. **Table 1** summarizes the common predictive models for side resistance (f_s) of drilled shafts in rocks mainly using the unconfined compressive strength (q_u) and atmospheric pressure (p_a).

Table 1. Available side resistance predictive methods in rocks.

Method	Reference	Predictive Equation	Remarks
1	Rosenberg and Journeaux (1976)	$\frac{f_{s,max}}{p_a} = 1.09 * (\frac{q_u}{p_a})^{0.5}$	p_a = atmospheric pressure (101,325 Pa or 101.3 kPa) & q_u = 0.48 to 34.5 MPa
2	Horvath and Kenney (1979)	$f_s = \alpha * \sqrt{q_u (MPa)}$	α = 0.2 to 0.25 & q_u = 0.33 to 41.3 MPa
3	Reynolds and Kaderabek (1980)	$f_s = 0.014 * q_u$	Median q_u = 1.5 MPa
4	Williams et al. (1980)	$f_s(MPa) = 0.44 * (q_u)^{0.35}$	q_u = 0.38 to 99.9 MPa
5	Rowe and Armitage (1984)	$f_s = 0.45 * \sqrt{q_u (MPa)}$	q_u = 0.41 to 40.5 MPa & rock sockets with grooves less than 10 mm deep)
6	Rowe and Armitage (1984)	$f_s = 0.6 * \sqrt{q_u (MPa)}$	q_u = 0.41 to 40.5 MPa & rock sockets with grooves greater than 10 mm deep)
7	Miller (2003)	$f_s = 0.4 * \sqrt{q_u (MPa)}$	Three Missouri shale sites
10	Kulhawy et al. (2005)	$\frac{f_{s,max}}{p_a} = 1.0 * (\frac{q_u}{p_a})^{0.5}$	Prakoso (2002) load test database
	McVay et al. (1992)	$f_{s,max}(MPa) = 0.05 * (q_u)^{0.5} * (q_t)^{0.5}$	q_t = splitting tensile strength of rock
	Meigh and Wolski (1979)	$f_{s,max}(MPa) = 0.25 * q_u$	0.5 MPa < q_u < 0.7 MPa
	Meigh and Wolski (1979)	$f_{s,max}(MPa) = 0.55 * (\frac{q_u}{p_a})^{0.6}$	0.7 MPa < q_u < 12.7 MPa
	Kulhawy and Phoon (1993)	$\frac{f_{s,max}}{P_a} = \Psi * (\frac{S_u}{p_a})^{0.5}$	Ψ = rock socket roughness factor = 3 for artificially roungend socket, = 2 for normal drilling, and = 1 for smooth or smeared sockets
	Carter and Kulhawy (1988)	$\frac{f_{s,max}}{P_a} = 1.42 * (\frac{q_u}{p_a})^{0.5}$	
	Gupton and Logan (1980)	$f_{s,max}(MPa) = 9.6x10^{-3} * q_u (MPa)$	
	Abu-Hejleh & Attwooll (2005)	$f_{s,max}(MPa) = 3.1x10^{-4} * q_u (MPa)$	0.5 MPa < q_u < 0.7 MPa

Abu-Hejleh & Attwooll (2005)	$\frac{f_{s,max}}{P_a} = 1.42 * (\frac{q_u}{p_a})^{0.5}$	1.1 MPa $< q_u <$ 5.0 MPa

Horvath and Kenney (1979) include an adhesion or interface friction factor termed, α. The adhesion factor is used in the following expression to calculate the unit side resistance, f_s, using the undrained shear strength, s_u:

$$f_s = \alpha * s_u \tag{1}$$

The adhesion factor is an empirical and dimensionless factor that relates the percentage of s_u that can be mobilized in terms of side resistance. **Eq. (1)** is a total stress analysis of the side resistance referred to as the alpha method (Reese and O'Neil, 1989).

SIDE RESISTANCE DATABASE

Predictive models for the design of drilled shafts in rocks are empirical. Many of these predictive models were developed based on databases consisting of load tests on drilled shafts in different types of rocks. Therefore, the applicability of these predictive models needs to be evaluated for weak fine-grained sedimentary rocks.

A database of drilled shaft side resistance in weak fine-grained rocks was compiled from published literature in this study. The database includes over 45 relevant drilled shaft load tests since 1960 with 54 values of side resistance. These load tests were performed in the United States, Europe, and Australia. The dimension of the rock sockets used for these drilled shafts vary considerably. The side resistance data are grouped according to their size based on recommendations of Horvath and Kenney (1979) and a summary is given in **Table 2**. **Table 3** summarizes the database according to rock type and unconfined compressive strength (q_u) of the rocks included in the database.

Table 2. Summary of load test methods in drilled shaft database.

Test Description	**Method of Measurement of Side Resistance**		
	Conventional Load Test	**Osterberg Load Test**	**Other**
Large scale piers (diameter > 0.41 m)	12	21	15
Small scale piers (diameter < 0.41 m)	3	—	1

Table 3. Summary of rock types and their unconfined compressive strength.

Rock Type	**No. of Tests**	**Unconfined Compressive Strength Range (MPa)**	**Unit Side Resistance Range (MPa)**
Shale	29	0.13 – 3.10	0.05 – 1.10
Mudstone	19	0.57 – 3.49	0.12 – 1.05
Claystone	6	0.40 – 3.06	0.12 – 0.91

This database was used herein to evaluate current design methods and to develop design recommendations for drilled shaft in weak fine-grained sedimentary rocks. This database was also used to study the load transfer mechanism in side resistance of drilled shafts in weak sedimentary rocks and is summarized in **Table 4**. In **Table 4**, $f_{s,max}$ is the maximum unit side resistance estimated from the load test data collected before test termination with units of stress, e.g., kPa, and q_u is the unconfined compressive strength of the rock in the vicinity of the strain gauges used to calculate $f_{s,max}$. Another parameter considered from the load test information is the Rock Quality Designation (RQD) described by Deere and Deere (1988). RQD is calculated by dividing the length of rock core that is at least 100 mm long by the total length of the core drilled on a particular run or sample (Deere and Deere, 1988). This side resistance database includes 93 values of unit side resistance from more than 65 drilled shaft load tests. The side resistance database in **Table 4** includes the following information:

- Load tests conducted using Osterberg loadcell (O-Cell), Ring Cells, and conventional top-loaded drilled shaft load tests.
- Drilled shafts embedded in weak shales, claystones, and mudstones.
- Drilled shaft diameters (D) range from 0.33 to 2.0 m.
- Most of the drilled shafts sockets were drilled normally. Only a few of the drilled shafts had artificially roughened socket walls that increased side resistance.
- The ratio of drilled shaft vertical movement to diameter is less than 1.7%.

Table 4. Unit side resistance database for drilled shafts in weak fine-grained based sedimentary rocks.

Index	Reference	Geomaterial Type	$f_{s,max}$ (MPa)	q_u (MPa)	Shaft D (m)	RQD (%)	Test Method	Remarks
1	Matich and Kozicki (1967)	Brown to gray shale and massive	> 0.31	0.69	0.61	__	Pull-out test	Artificially roughened
2	Corps of Engineers (1968)	Clay-shale	> 0.27	0.73	__	__	__	__
3	Geoke and Hustad (1979): Shaft 1	Gray clay-shale (Caddo formation)	0.36 @ 0.25 in	1.03	0.76	__	Compression test	Drilled with rock auger
4	Geoke and Hustad (1979): Shaft 2	Gray clay-shale (Caddo formation)	0.22 @ 0.25 in	0.76	0.76	__	Compression test	Drilled with rock auger
5	Wilson (1976) Port Elizabeth, south Africa: West pile	Mudstone from Uitenhage series of Cretaceous system	0.18 @ 0.47 in	1.09	0.90	__	Pull-out test	Concrete defects due to water entering shaft
6	Wilson (1976) Port Elizabeth, south Africa East pile	Mudstone from Uitenhage series of Cretaceous system	0.12 @ 0.12 in	1.09	0.90	__	Pull-out test	Concrete defects due to water entering shaft
7	Mason (1960): PC25 USA	Weak shale	0.42	1.50	0.61	__	Compression test	__
8	Johnston and Donald (1979) Melbourne (F2)	Weathered Melbourne mudstone	0.94	1.93	1.19	__	Compression test	__
9	Brown and Thompson (2008)	Claystone	> 0.46 @ 0.13 in	2.07	0.71	__	Compression test	__
10	Brown and Thompson (2008)	Clay-shale	0.34 @ 0.61 in	2.07	0.51	__	Compression test	__
11	Loadtest (2008) IL 5 over IL 84	Shale	0.07 @ 0.44 in	0.27	1.07	__	Compression test	__
12	Loadtest (2008) IL 5 over IL 84	Shale	0.13 @ 0.44 in	0.56	1.07	__	Compression test	__
13	Loadtest (2008) IL 5 over IL 84	Shale	0.64 @ 0.45 in	2.67	1.07	__	Compression test	__
14	Loadtest (1996) FAU 6265	Shale	0.05 @ 0.1 in	0.13	62	__	Compression test	__

Index	Reference	Geomaterial Type	$f_{s,max}$ (MPa)	q_u (MPa)	Shaft D (m)	RQD (%)	Test Method	Remarks
15	Pells et al. (1978) PC 29	Weathered Melbourne mudstone	0.79	2.21	1.09	__	Compression test	__
16	Millar (1976): City Center Perth, W.A.	King Park shale	> 1.10 @ 31.75 mm	3.06	0.69	__	Compression test	Drilled under bentonite
17	Millar (1976): Telephone Exchange, Perth, W.A. (TP1)	King Park shale	> 0.30 @ 31.75 mm	1.00	0.66	__	__	__
18	Millar (1976): Telephone Exchange, Perth, W.A. (TP2)	King Park shale	5.51 @ 4.06 mm	2.68	0.79	__	__	__
19	Johnston and Donald (1979) Flinders St., Melbourne (F1)	Weathered Melbourne mudstone	10.15	3.06	1.20	__	__	__
20	Walter et al. (1997)	Mudstone	0.60	3.20	0.90	__	Down-hole jack	__
21	Williams and Pells (1981)	Shale	1.10	3.10	0.69	__	__	Drilled and cast under bentonite
22	Williams and Pells (1981)	Shale	0.72	2.70	0.79	__	__	__
23	Williams (1980a): PS1 Stanley Ave., Melbourne	Weathered Melbourne mudstone	> 0.56	0.83	0.66	__	Compression test	Drilled normally
24	Williams (1980a): PS3 Stanley Ave., Melbourne	Weathered Melbourne mudstone	0.51	0.57	1.12	__	Compression test	Roughened
25	Williams (1980a): PS12 Stanley Ave., Melbourne	Weathered Melbourne mudstone	0.41	0.59	0.34	__	Compression test	Drilled with core barrel
26	Williams (1980a): PS14 Stanley Ave., Melbourne	Weathered Melbourne mudstone	0.50	0.58	0.39	__	Compression test	Roughened
27	Williams (1980a): PS15 Stanley Ave., Melbourne	Weathered Melbourne mudstone	0.41	0.60	0.39	__	Compression test	Roughened

Index	Reference	Geomaterial Type	$f_{s,max}$ (MPa)	q_u (MPa)	D (m)	RQD (%)	Test Method	Remarks
28	Williams (1980a): PS 16 Stanley Ave., Melbourne	Weathered Melbourne mudstone	> 0.36	0.58	0.39	__	__	Roughened
29	Williams (1980a): M1 Middleborough Rd. Melbourne	Weathered Melbourne mudstone	0.60	2.46	1.21	__	__	Drilled with bucket auger
30	Williams (1980a): M2 Middleborough Rd. Melbourne	Weathered Melbourne mudstone	0.64	2.30	1.30	__	__	Roughened
31	Williams (1980a): M3 Middleborough Rd. Melbourne	Weathered Melbourne mudstone	0.71	2.30	1.23	__	__	Drilled with bucket auger
32	Williams (1980a): M4 Middleborough Rd. Melbourne	Weathered Melbourne mudstone	0.62	2.34	1.35	__	__	Roughened
33	Williams (1980a) Pile WG303/2 Melbourne	Slightly weathered Melbourne mudstone	0.85	3.49	__	__	__	Roughened
34	Leach et al. (1976): Pile A, Kilroot, N. Ireland	Mudstone	0.21 @ 5.84 mm	0.80	0.74	__	__	Drilled with auger
35	Leach et al. (1976): Pile B, Kilroot, N. Ireland	Mudstone	0.12 @ 13.97 mm	0.92	0.74	__	__	Drilled with auger
36	Aurora and Reese (1976): MT1, Montopolis	Clay-shale	0.41	1.42	0.74	__	Conventional	Drilled with auger, dry
37	Aurora and Reese (1976): MT2, Montopolis	Clay-shale	0.37	1.42	0.18	__	Conventional	Drilled with auger, dry
38	Aurora and Reese (1976): MT3, Montopolis	Clay-shale	0.69	1.42	0.75	__	Conventional	Drilled with auger, dry
39	Aurora and Reese (1976): DT1, Dallas	Clay-shale	0.28 @ 5.08 mm	0.61	0.18	__	Conventional	Drilled with auger, dry
40	LT-8718-2, Scandia, KS Socket (Loadtest, 2001a)	Gray to dark gray shale with limey seams	0.15 @ 19.81 mm	0.62	1.83	40	Osterberg Loadcell (O-Cell)	Drilled with auger

Index	Reference	Geomaterial Type	$f_{s,max}$ (MPa)	q_u (MPa)	Shaft D (m)	RQD (%)	Test Method	Remarks
41	LT-9048 Route 116 Over the Platte River, Plattsburg, MO (Loadtest, 2004)	Gray silt shale	> 0.72 @ 16.76 mm	2.20	1.22	__	O-Cell	Drilled with auger, Dry
42	LT-8718-1, Scandia, KS US 36 Over Republican River Socket (Loadtest, 2001b)	Dark gray shale (Graneros shale formation)	0.18 @ 43.94 mm	0.94	1.83	49	O-Cell	Drilled with auger
43	LT-8854, Des Moines, IA I-235 Over Des Moines River Socket (Loadtest, 2002)	Clay-shale	0.62 @ 21.84 mm	2.69	1.07	93	O-Cell	Drilled by auger and core barrel
44	LT-8816, Osborne County, Kansas US 281 Over Solomon River Socket (Loadtest, 2001c)	Gray to dark gray chalky shale	0.52 @ 18.29 mm	2.37	1.07	80	O-Cell	Drilled with rock auger
45	LT-8733: Pier 1 West, Wakarusa, KS US 75 at 77th Street Socket (Loadtest, 2001d)	Gray shale with limestone lenses	> 0.41 @ 5.08 mm	1.03	1.83	__	O-Cell	Drilled in dry with auger

Index	Reference	Geomaterial Type	$f_{s,max}$ (MPa)	q_u (MPa)	Shaft D (m)	RQD (%)	Test Method	Remarks
46	Brown and Thompson (2008)	Weathered shale	0.95 @ 0.01 mm	2.21	1.80	__	O-Cell	__
47	Miller (2003): Lexington, MO TS-1A, O-Cell to SG 2	Hard gray clayshale	0.73 @ 0.01 mm	2.13	1.11	__	O-Cell	Drilled normally
48	Miller (2003): Lexington, MO TS-2, Lower to Upper O-Cell	Hard gray shale to clayshale	0.73 @ 0.01 mm	2.25	1.17	__	O-Cell	Drilled normally
49	Miller (2003): Grandview, MO SG 5 to SG 6	Gray thinly laminated clayshale	0.36 @ 0.02 mm	0.93	1.98	__	O-Cell	Drilled normally
50	Abu-Hejleh et al. (2003): I-225	Soil-like claystone	> 0.12 @ 40.64 mm	0.40	1.07	__	O-Cell	Slightly roughened
51	Abu-Hejleh et al. (2003): I-225	Soil-like claystone	> 0.17 @ 40.64 mm	0.59	1.07	__	O-Cell	Slightly roughened
52	Abu-Hejleh et al. (2003): I-225	Soil-like claystone	> 0.15 @ 40.64 mm	0.48	1.07	__	O-Cell	Slightly roughened
53	Abu-Hejleh et al. (2003): County line	Soil-like claystone	> 0.16 @ 0.02 mm	0.50	1.22	__	O-Cell	Slightly roughened
54	Abu-Hejleh et al. (2003): Franklin	Very hard sandy claystone	> 0.91 @ 0.01 mm	3.06	1.07	__	O-Cell	Wet

LOAD TRANSFER MECHANISM

The load transfer mechanism in rock socketed drilled shafts is a function of rock q_u, rock socket nominal diameter, and magnitude of drilled shaft displacement. Understanding the load transfer mechanism(s) is necessary for identifying the important factors that should be included in a predictive model. This paper focuses on only load transfer in side resistance.

Analytical studies and load test measurements (Moore 1964; Gibson 1973; Osterberg and Gill 1973; Aurora and Reese 1976; Ladanyi 1977; Geoke and Hustad 1979; Horvath and Kenney 1979; Rowe and Armitage 1987; Brown et al. 2010) indicate that side resistance contributes significantly to the axial capacity of drilled shafts socketed in weak fine-grained rocks until large displacement or slip occurs at the shaft/socket interface. Some of the major factors in side resistance load transfer such as construction method, socket diameter, shaft displacement, rock type, and unconfined compressive strength, are discussed in this section.

Effect of Construction Methods

Construction techniques have a large influence on the mobilized side resistance in drilled shafts (Reese and O'Neil, 1988). For example, if the sides of the socket are roughed due to the auger, the concrete can better adhere to the rock walls and provide greater side resistance than smooth walls. An empirical adhesion factor is used to quantify the level of adhesion between the socket walls and drilled shaft concrete. A higher adhesion factor means a greater interlock between the rock and concrete and usually reflects that some construction technique was used to increase the bond between the rock walls and the concrete, such as a "tooth" being added to the edge of the auger. A roughed socket can also prevent a post–peak reduction of side resistance with shaft displacement (Williams et al. 1980).

Drilled shafts with concrete defects such as water in the shaft preventing full adherence of the concrete to the rock walls, the concrete not being vibrated sufficiently to make contact with the rock walls, or the concrete being contaminated by soil as the casing is withdrawn of if a casing is not used, also can decrease side resistance. **Figure 1** shows the adhesion factors derived from the side resistance database and Index numbers shown in **Table 4.** The adhesion factors range from 0.1 to 0.9 with a lot of the data around an adhesion factor of 0.3. This means that only 30% of the unconfined compressive strength is being mobilized in side resistance. This data demonstrates the range of impact different construction techniques have on the mobilized side resistance as discussed in the following sub-sections.

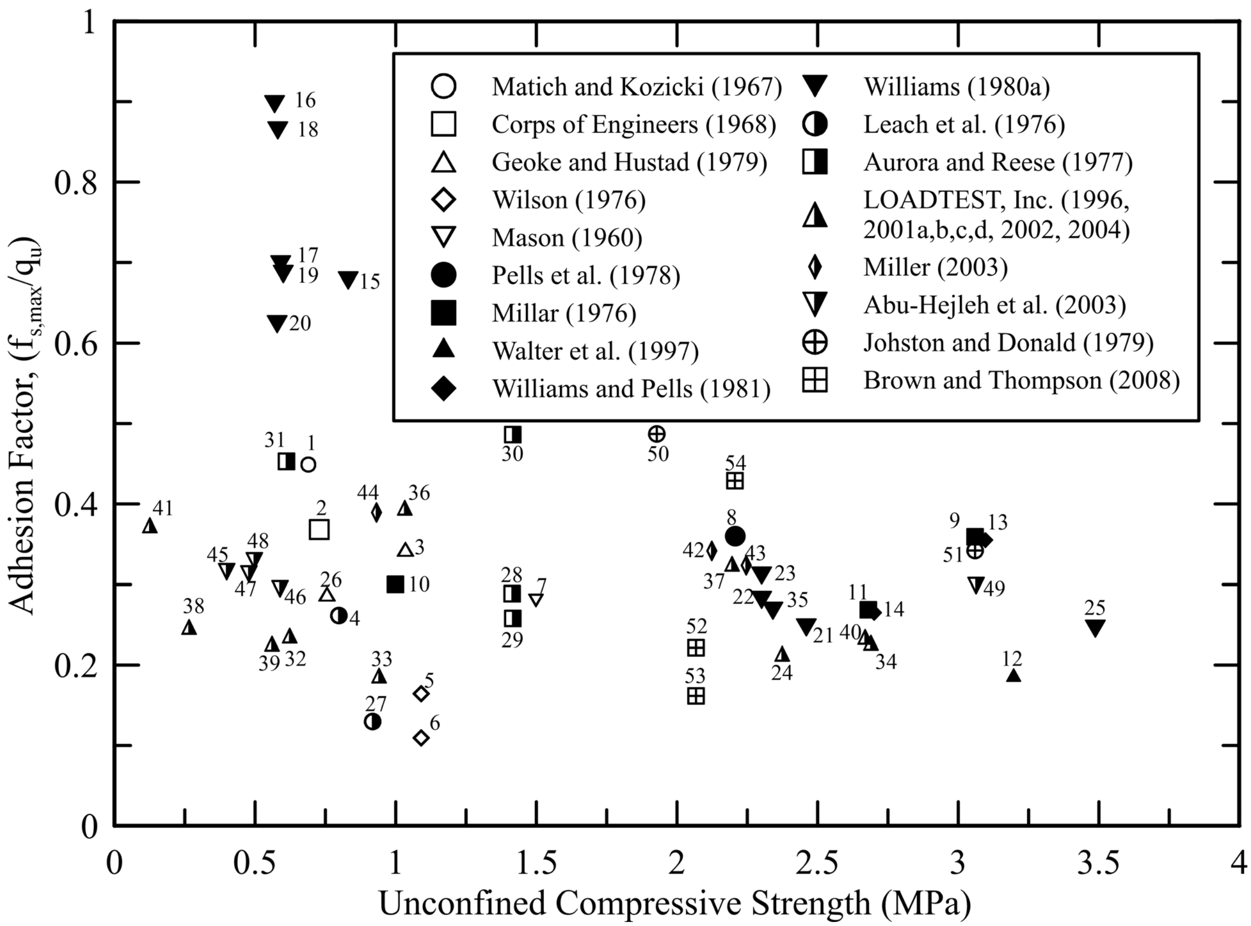

Figure 1. Load test database for unit side resistance with various construction techniques with Index numbers shown in Table 4.

Artificially Roughened Rock Sockets

Artificially roughened rock sockets are beyond the scope of this paper, however, examples of the usefulness of this technique is discussed in this section using load test measurements. For example, data points labeled 15 to 20 in **Figure 1** were derived from static load tests performed on drilled shafts socketed in Melbourne mudstone with artificially roughened rock sockets (Williams 1980a and b). These tests correspond to an adhesion factor of 0.6 to 0.7. The data point labeled #1 in **Figure 1** (adhesion factor of about 0.45) is from Matich and Kozicki (1967) and also was obtained from a static load test with a roughened socket. These data points represent normalized side resistance with artificially roughened sockets and indicate that side resistance can be increased for drilled shafts in weak rocks if the socket or boring walls are roughened by mechanical means, as compared to normally constructed rock sockets that exhibit smoother walls. Williams et al. (1980) suggest that conventional drilling with a bucket auger device also produces a roughened socket walls.

Concrete Defects

Data points labeled 5 and 6 in **Figure 1** (adhesion factor between 0.1 and 0.2) were obtained from two static load tests on drilled shafts at Port Elizabeth, South Africa (Wilson 1976). There was a concrete defect in these drilled shafts due to water entering the shaft hole while concrete was being placed. This defect in concrete adherence and curing caused a significant reduction in the mobilized unit side resistance in these drilled shafts.

Effect of Drilled Shaft Socket Diameter

Figure 2 plots adhesion factor versus shaft diameters from 0.125 m to 1.875 m for drilled shafts in weak fine-grained rocks and indicates that the adhesion factor is unaffected by drilled shaft diameter. Horvath and Kenney (1979) point out "…within the size range for large diameter socketed piers (D > 0.4 m (16 in)) the effect of socket diameter [on adhesion factor] appears to be negligible..." This finding is in agreement with conclusions of Williams et al. (1980) and Brown et al. (2010) and is supported by the data shown in **Figure 2**.

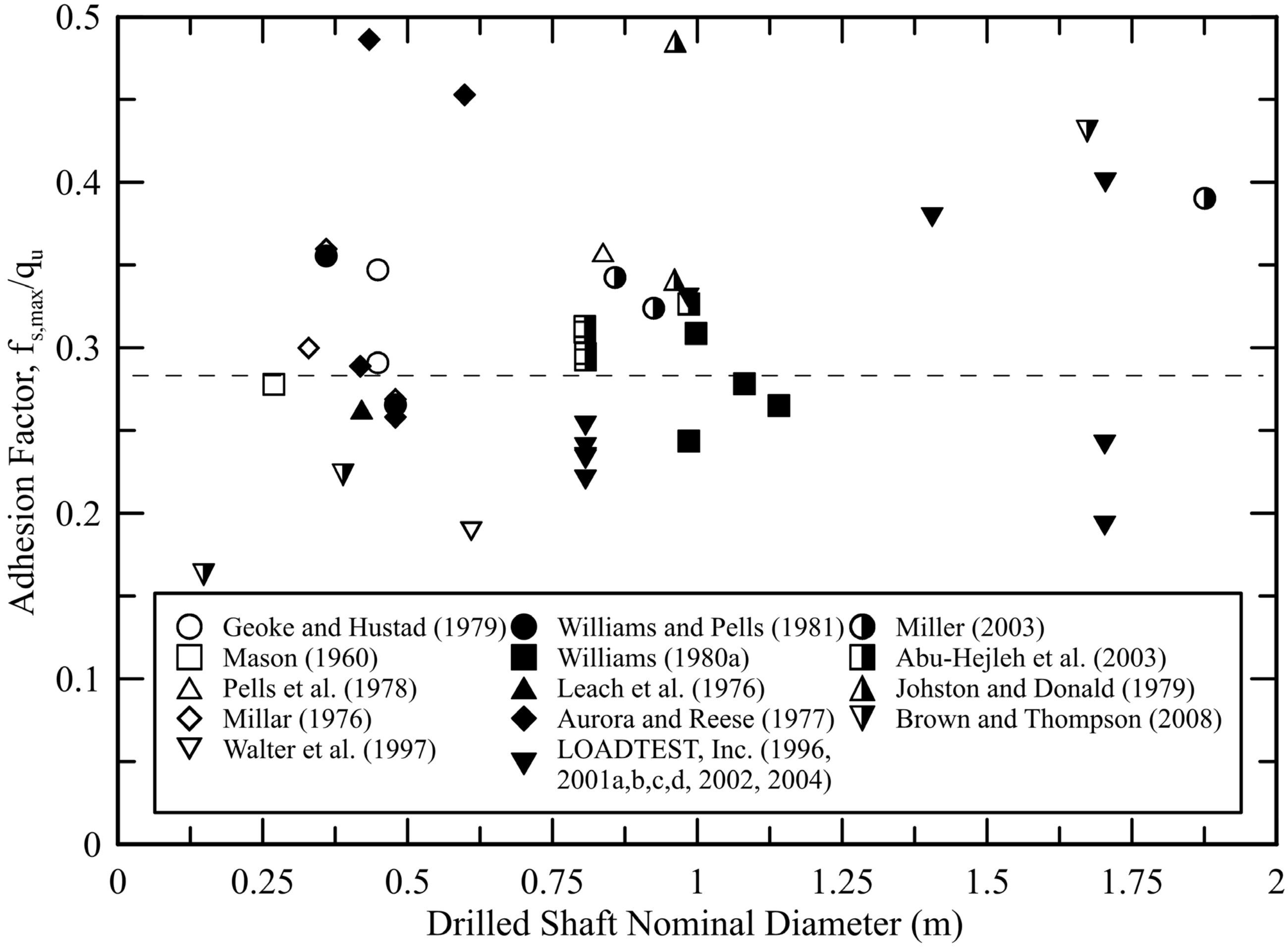

Figure 2. Effect of shaft diameter on adhesion factor or maximum side resistance.

Effect of Drilled Shaft Displacement

Figure 3 presents a relationship between drilled shaft diameter and drilled shaft displacement required to mobilize $f_{s,max}$. This data was obtained from the side resistance database described in **Table 4**. **Figure 3** shows that shaft displacements of less than 25 mm (1 inch) are generally required to mobilize the maximum side resistance along the shaft/rock interface and therefore, it is assumed that full side resistance is mobilized in drilled shafts in weak rocks for design purposes because of the small displacement required.

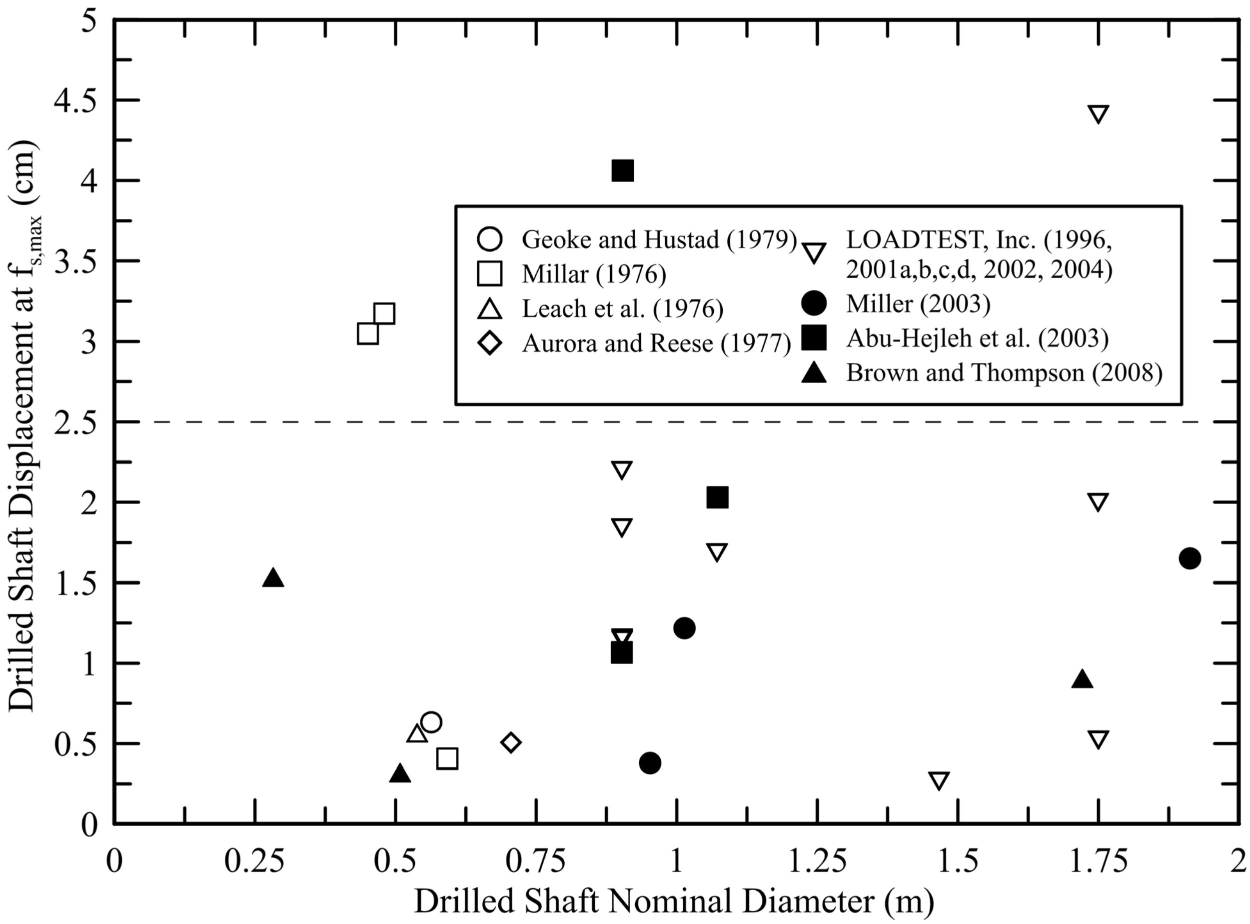

Figure 3. Effect of shaft displacement on mobilized side resistance.

Figure 4 shows the ratio of residual side resistance to peak or maximum side resistance ($f_{s,max}$) versus drilled shaft displacement after $f_{s,max}$ is reached. This figure shows that side resistance of drilled shafts in weak rocks remains near the maximum value even after a post–peak shaft displacement of 35.6 mm (1.4 inches) is mobilized if the socket walls are relatively rough. This means there is little post–peak decrease in side resistance with increasing drilled shaft displacement. This is also in agreement with observations of Williams and Pells (1981). This conclusion is significant for drilled shaft design because it means both side and tip resistance could be used in design because there is little post–peak decrease in side resistance with increasing displacement, which is needed to mobilize the full tip resistance. If there was a large post–peak decrease in side resistance, a designer could not use the maximum value of side and tip resistance in design because it would overestimate the total resistance available.

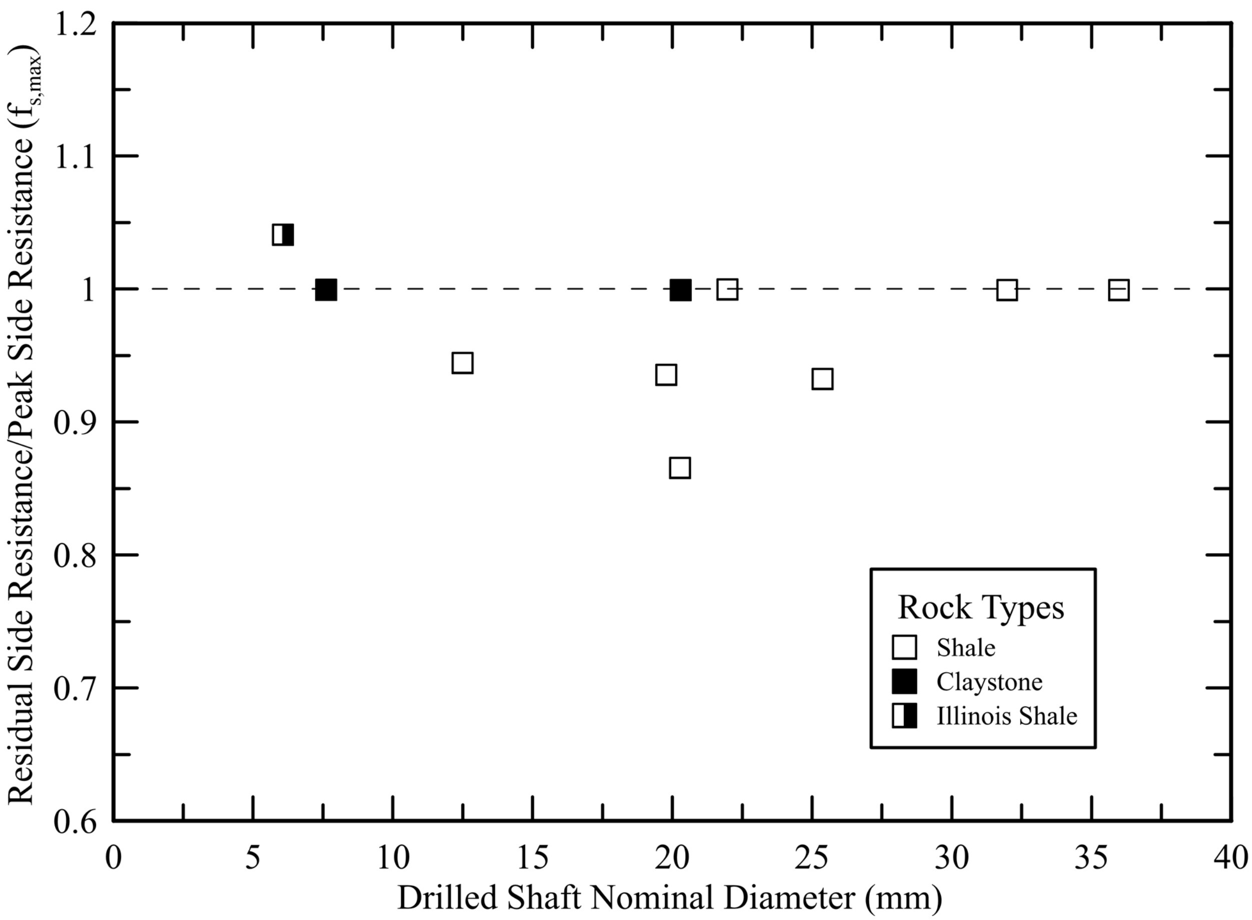

Figure 4. **Effect of post–peak shaft displacement on maximum side resistance.**

Effect of Rock Type and Unconfined Compressive Strength

Figure 5 shows that drilled shaft side resistance in weak fine-grained rock increases with the rock q_u. Therefore, rock q_u is one of the major factors that control the load transfer mechanism in side resistance. **Figure 5** further shows measured side resistance for various rock types and side resistance can be modeled using a linear trend line. Therefore, a single linear function is proposed herein to model side resistance load transfer in weak fine-grained rock sockets below.

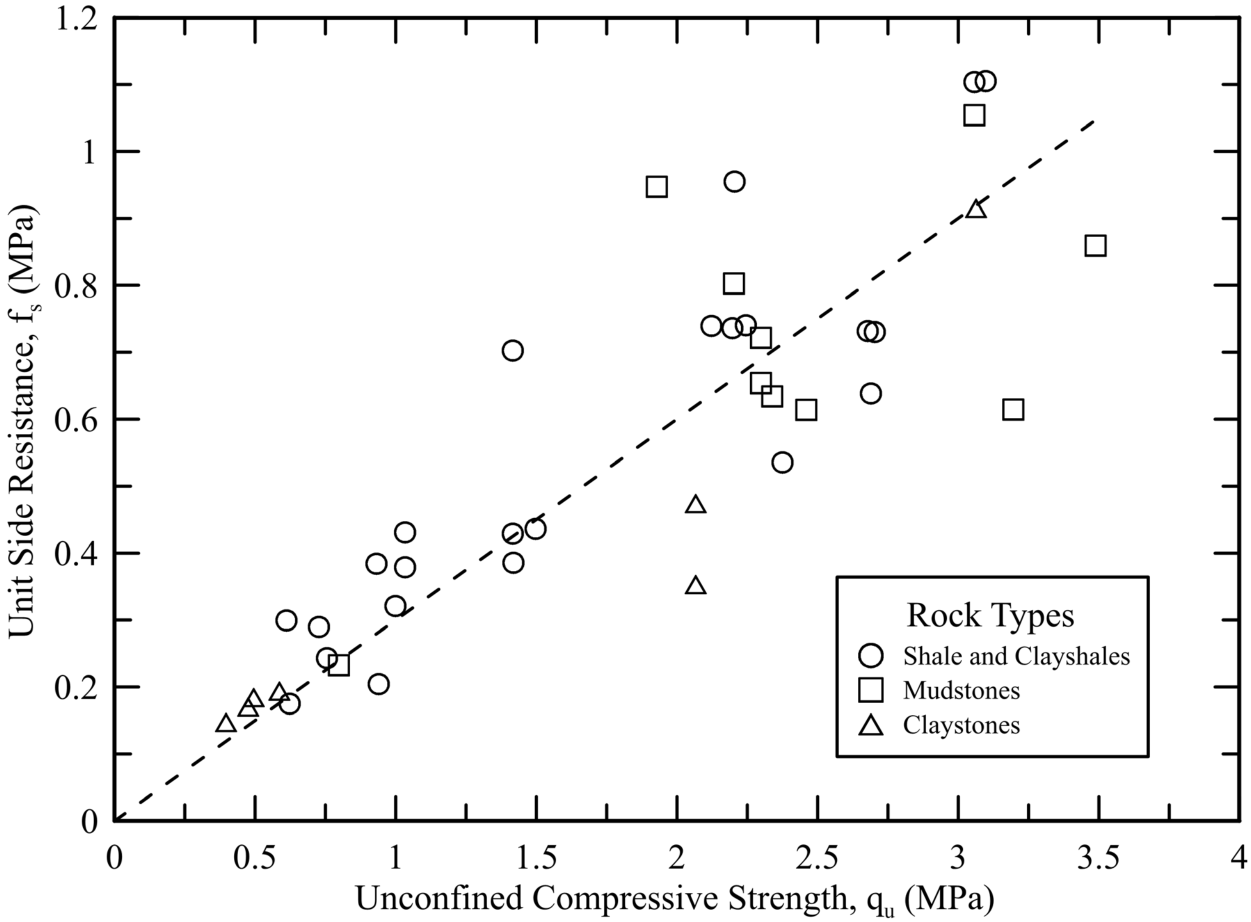

Figure 5. Effect of rock type and unconfined compressive strength on side resistance.

EVALUATION OF CURRENT DESIGN METHODS

Soil constitutive models could be used to study load transfer mechanism(s) in axially loaded drilled shafts. However, these models require values of cohesion intercept, friction angle, normal stiffness, and some quantitative measure of dilatancy of the weak rock(s) involved. Such information is not routinely collected in field or laboratory tests (Carter and Kulhawy 1988). For this reason, available predictive models are mainly empirical, using data that is readily available from field drilling and sampling and laboratory testing. These empirical models, however, are not always developed for drilled shafts in weak fine-grained rocks. Therefore, these models should be evaluated against a database that includes only weak fine-grained based rocks (0.48 MPa $<q_u<$ 4.8 MPa or 10 ksf $<q_u<$100 ksf) such as the one compiled in this study and presented in **Table 4**.

Table 5 shows the design equation and the mean and coefficient of variance (COV) of the predicted to measured unit side resistances for the drilled shaft database developed herein. In other words, the design equations in **Table 5** and a value of q_u for the weak rock at the elevation of each strain gage in the load test shown in **Table 4** were used to estimate the unit side resistance for each of the load tests shown in **Table 4**. The predicted values of side resistance were then divided by the strain gage derived side resistance values, which were calculated using the strain reading at each of the elevations. This produced a ratio of predicted to measured side resistance at various depths. If this prediction ratio equals unity (1.0), the predictive method exactly predicts the mobilized side resistance at that elevation. From these ratios of predicted to measured side resistance, the mean and standard deviation were computed (see **Table 5**). Once the mean and standard deviation were computed, the COV for each predictive method was computed by dividing the standard deviation of the predicted value by the mean of the ratios of predicted to measured side resistance. This mean and COV are the values shown in **Table 5** and indicate that some of the predictive methods overestimate the unit side resistance, i.e., ratio greater than unity (1.0), and some underestimate, i.e., ratio less than unity (1.0). **Table 5** also shows that the majority of these models are not accurate because some ofthe COV values are large, i.e., 0.45 to 0.62.

Table 5. Statistics for unit side resistance predictive models based on loadtest database presented in Table 4.

Predictive Method	Predictive Equation	Mean of ratios of predicted to measured side resistance	COV of predicted to measured ratios
Rosenberg and Journeaux (1976)	$f_s(\mathrm{MPa}) = 0.36*(q_u)^{0.52}$	1.25	0.50
Horvath and Kenney (1979)	$f_s = 0.2*\sqrt{q_u(\mathrm{MPa})}$	0.69	0.51
Williams et al. (1980)	$f_s(\mathrm{MPa}) = 0.44*(q_u)^{0.35}$	1.49	0.58
Rowe and Armitage (1987)	$f_s = 0.45*\sqrt{q_u(\mathrm{MPa})}$	1.54	0.51
Reynolds and Kaderabek (1980)	$f_s(\mathrm{MPa}) = 0.014*q_u$	1.04	0.25
Miller (2003)	$f_s = 0.4*\sqrt{q_u(\mathrm{MPa})}$	1.37	0.51
Kulhawy et al. (2005)	$f_s/P_a = (q_u/P_a)^{0.5}$	1.10	0.51
Carter and Kulhawy (1988)	$\frac{f_{s,max}}{P_a} = 1.42*(\frac{q_u}{P_a})^{0.5}$	1.55	0.51
Kulhawy and Phoon (1993)	$\frac{f_{s,max}}{P_a} = \Psi*(\frac{S_u}{P_a})^{0.5}$	1.55	0.51

Evaluation of Power Functions

The power function predictive models of Rosenberg and Journeaux (1976), Miller (2003), and Kulhawy et al. (2005) are superimposed as lines on the measured values of unit side resistance from the loadtest database shown in **Table 4** in **Figure 6**. **Figure 6** shows that these predictive models overestimate the side resistance for weak rocks with q_u less than 1.9 MPa (40 ksf) and underestimate side resistance for weak rocks with q_u greater than 1.9 MPa (40 ksf). Therefore, power functions exhibit a poor fit to the measured values of unit side resistance and are not recommended. Piecewise functions are more accurate than power functions; however, they occasionally underestimate the unit side resistance. Furthermore, the same level of accuracy can be obtained in design by using a simple linear function as a prediction method. As a result, it is recommended that a linear function be used to predict unit side resistance for drilled shafts constructed in weak Illinois shales as shown below in **Eq. (2)**.

Some reasons for the lack of agreement between the power function predictive models of Rosenberg and Journeaux (1976), Miller (2003), and Kulhawy et al. (2005) and the measured data in **Figure 6** are discussed in this paragraph. For exmple, the model by Rosenberg and Journeaux (1976) model includes strong rocks and thus the stronger rocks affect the mathematical form chosen for their model. Some of the measured side resistance data used in development of the Miller (2003) model were obtained from strain gages that are close to the Osterberg load cell (O-Cell), which can cause higher values. Abu-Hejleh et al. (2003) wrote, "…Findings of ongoing NCHRP project 21−08 suggest that the distribution of side resistance is expected to be biased toward higher values nearest the O−Cell…" This is due to higher strains occurring near the O−Cell that leads to an overestimate of the rock socket side resistance. Therefore, it is more appropriate to use an average unit side resistance along the entire rock socket which requires not using data from strain gages near the O−Cell. Review of load test data presented by Miller (2003) supports this hypothesis and suggests that the predictive model of Miller (2003) could have been affected by the unit side resistance values obtained from strain gages near the O−Cell, leading to their selection of a power function to fit the data. Kulhawy et al. (2005) base their method on a load test database collected by Prakoso (2002), which includes rocks with q_u as high as 95.6 MPa (2,000 ksf), which can overestimate rock socket side resistance in weak fine-grained rocks.

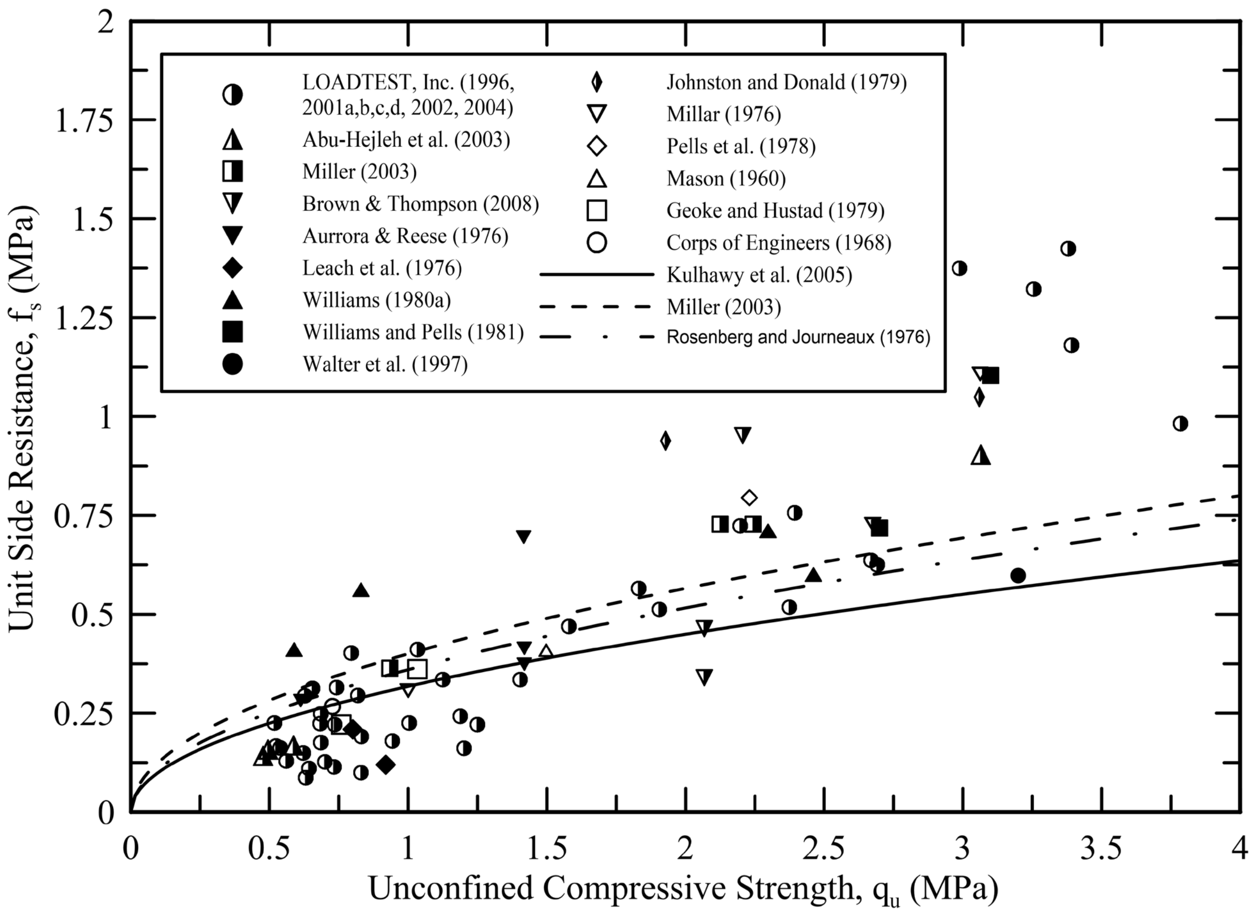

Figure 6. **Comparison of predictive models and side resistance database.**

Figure 7 compares the predicted values of side resistance, R_p, using the predictive models in **Table 5** with the measured values of side resistance, R_m, the from the loadtest database values side resistance in **Table 4**. **Figure 7** shows the various power functions do not accurately predict the measured side resistance of drilled shafts in weak fine-grained rocks because the data points are not in agreement with the three lines that represent the range of the predicted values. For example, the line labelled $R_p = R_m$ represents a ratio of the predicted (R_p) to mobilized or measured (R_m) side resistance of unity (1.0) or perfect agreement. The other two lines correspond to R_p being 0.5 x R_m and 2.0 x R_m to represent a represent the range of the predicted values from one-half of R_m to two times R_m. None of these three lines capture the distribution of the measured unit side resistance values so a linear function is proposed in **Eq. (2)** below.

Examination of the load test database presented in **Table 4,** the statistics in **Table 5**, and the measured values of unit side resistance shown in **Figure 8** suggest that a linear function provides a better fit to the observed trend of unit side resistance and q_u for weak fine-grained sedimentary rocks. Other investigators (e.g., Reynolds and Kaderabek 1980; Gupton and Logan 1984; Abu-Hejleh et al. 2003) also use a linear function and the statistics in **Table 5** show the COV is the lowest (0.25) for the linear function proposed by Reynolds and Kaderabek (1980). As a result, a linear predictive model is presented in **Figure 8** and **Eq. (2)** below instead of a power function.

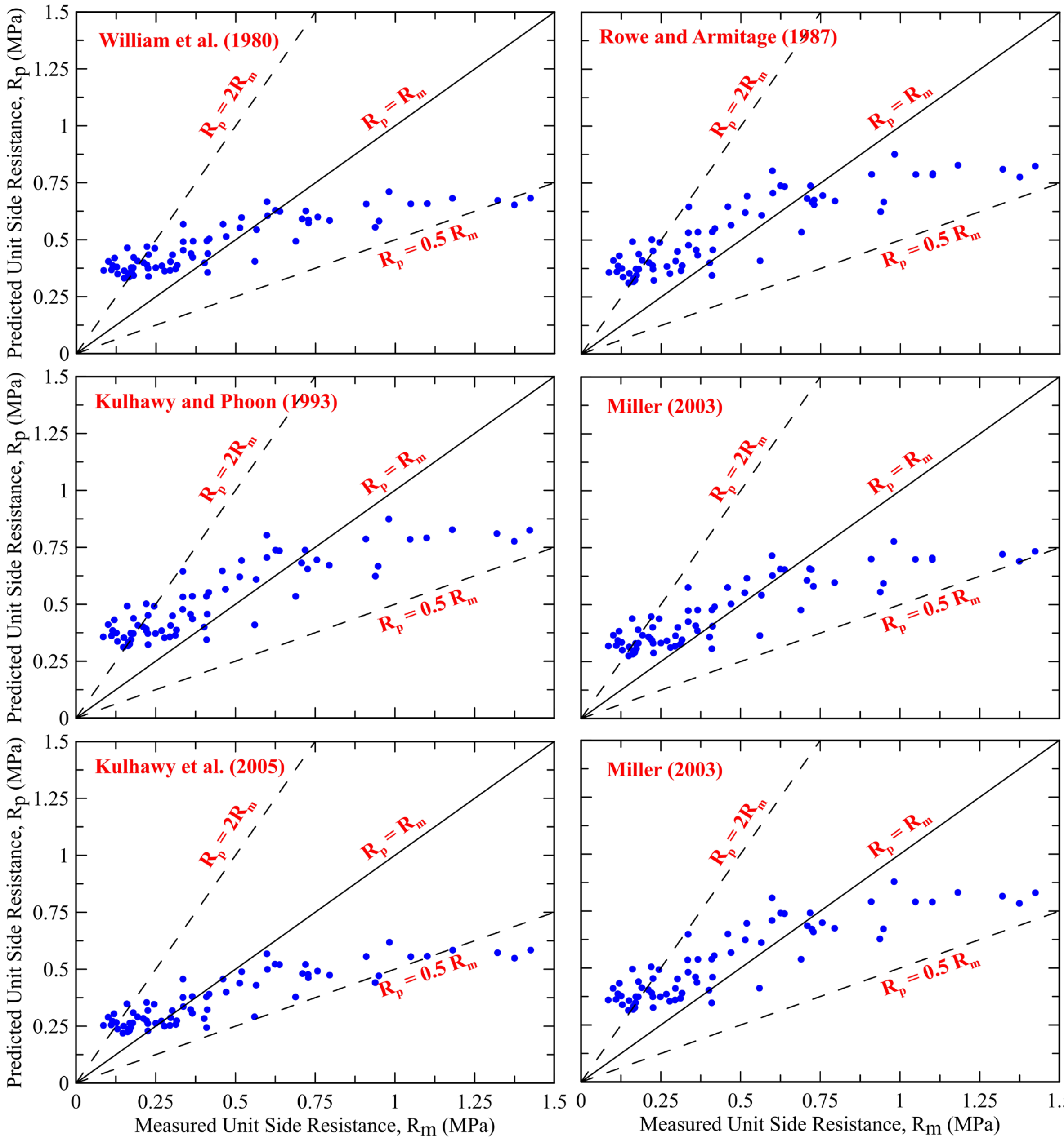

Figure 7. Comparison of predictive models in Table 5 with measured side resistances.

DESIGN RECOMMENDATIONS

The database developed herein indicates that q_u can be used to estimate the mobilized unit side resistance in drilled shafts socketed in weak fine-grained rocks because load transfer is not significantly affected by drilled shaft geometry (e.g., drilled shaft diameter). **Figure 3** also shows that the ultimate side resistance of drilled shafts in weak fine-grained rocks is often mobilized at relatively small vertical displacement, i.e., less than 25 mm. **Figure 4** shows that unit side resistance does not experience a significant post–peak decrease in side resistance with increasing shaft displacement so the linear function can be independent of axial displacement. Review of existing methods indicate that drilled shafts in weak shales, mudstones, and claystones exhibit similar behavior for side resistance which is also shown in **Figure 5** and the load test database presented in **Table 4**. Therefore, the proposed design method uses q_u of the weak fine-grained rock to predict unit side resistance for several types of weak fine-grained sedimentary rock.

The side resistance database (**Table 4**) was used to select representative and applicable load test data for developing an empirical design method for drilled shafts in weak fine-grained rocks. Regression analyses were used to determine the best fit line to the selected side resistance data. **Figure 8** shows this best fit line that relates measured unit side resistance to q_u for the design of drilled shaft rock sockets in weak fine-grained rock. Considering the data in **Figure 5** and **Figure 8**, the proposed side resistance relationship in **Eq. (2)** is valid for $q_u \leq 1.5$ MPa because at higher values of q_u the data scatter increases significantly. This scatter is not desirable for design so the side resistance relationship in **Eq. (2)** is limited $q_u \leq 1.5$ MPa. This also explains why the trend line in **Figure 8** is dashed at values of $q_u \geq 1.5$ MPa. As a result, one of the other side resistance relationships shown in **Table 1.**

"The trend line stops at a UCS of 3.5 MPa because the MSPT correlation for UCS as an applicable range of 0.48 to 4.8 MPa. Site-specific MSP and UCS testing should be conducted to verify the linear trend in Figure 8 is applicable."

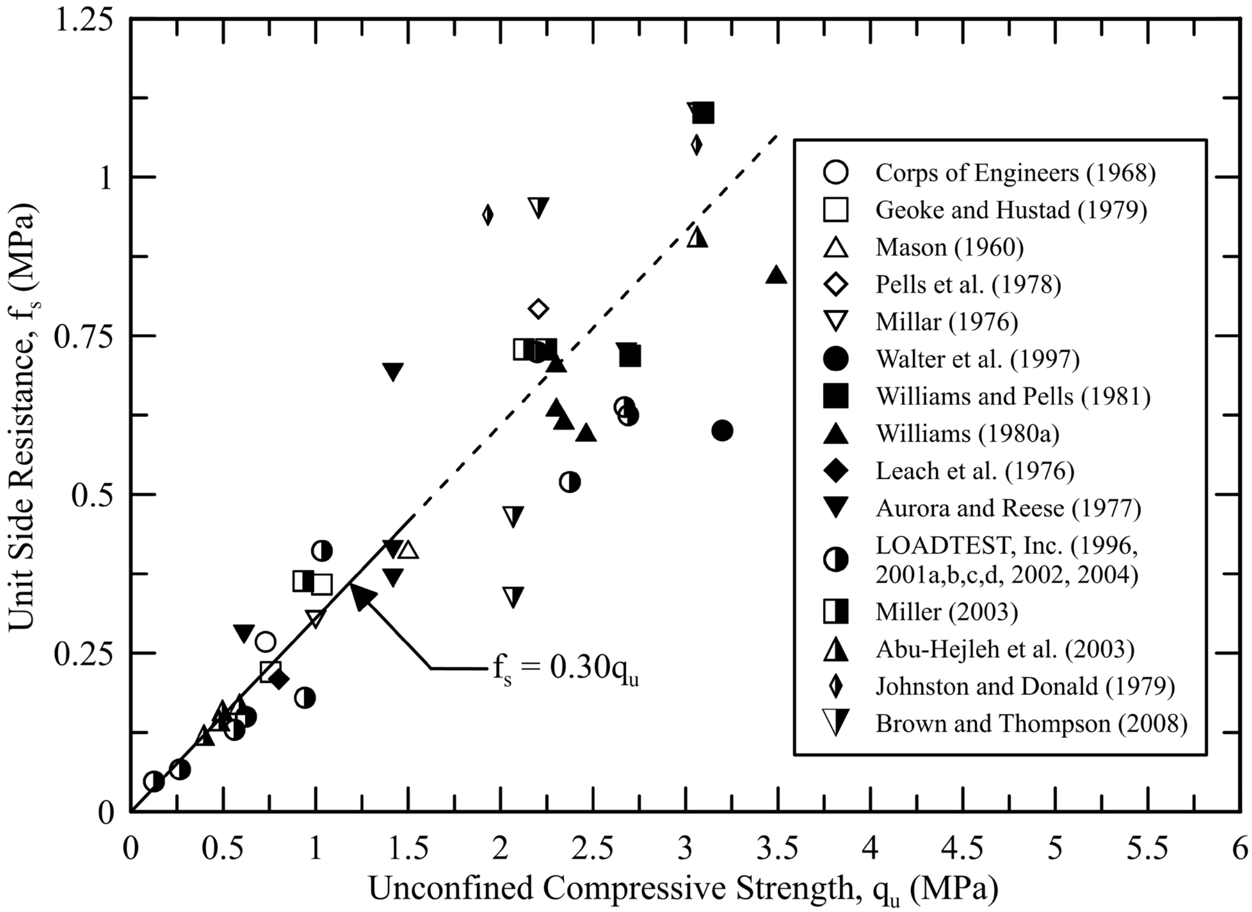

Figure 8. **Predictive method for unit side resistance of drilled shafts in weak rocks, using a linear function to fit the load test data.**

Other researchers suggest a linear function, or equation, to predict unit side resistance in weak rocks (e.g., Reynolds and Kaderabek 1980; Gupton and Logan 1984; Abu-Hejleh and Attwooll 2005) but recommend different coefficients or adhesion factors than the following expression or consider a different range of q_u for the rock socket. For example, Reynolds and Kaderabek 1980; have a similar adhesion factor but their database has a median q_u of 1.5 MPa instead of the weak rocks ($0.48 < q_u < 4.8$ MPa) considered herein. The proposed side resistance predictive method, f_s, uses an adhesion factor of 0.3 (see **Figure 1**) and average q_u along the rock socket wall with a limiting value of 1.4 MPa (30 ksf):

$$f_s(MPa) = \alpha * q_u \leq (1.4MPa) \quad (2)$$

where:

f_s = unit side resistance in socketed weak rocks (MPa/ksf) for q_u < 1.5 MPa;
q_u = average q_u of rock along socket wall (MPa/ksf)
α = 0.30 = empirical adhesion factor, dimensionless.

SUMMARY

Current empirical methods for estimating drilled shaft side resistance in weak fine-grained rocks (unconfined compressive strength of 0.48 to 4.8 MPa) are reviewed and the range of load test procedure, rock type, and q_u vary considerably. As a result, a static load test database was developed (**Table 4**) to evaluate the precision and accuracy of current predictive methods for weak fine-grained rock with qu between 0.48 and 4.8 MPa (10 and 100 ksf). Drilled shaft diameters in the new database range from 0.33 to 1.98 m (13 to 78 in.) for the unit side resistance database.

The load test database shows that load transfer in side resistance is independent of shaft diameter and only a small shaft displacement is required to mobilize full side resistance. Therefore, the proposed design method correlates unit side resistance to only q_u and an empirical adhesion (see **Eq. (1)**) to satisfactorily predict the mobilized side resistance in weak rocks with a limiting q_u value of 1.4 MPa (30 ksf). Based on the data herein, drilled shaft design can use both side and tip resistance because there is little post–peak decrease in side resistance with increasing displacement.

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REFERENCES

Abu-Hejleh, N., and W. J. Attwooll, *Colorado's Axial Load Tests on Drilled Shafts Socketed in Weak Rocks: Synthesis and Future Needs*, Final Contract Report No. CDOT-DTD-R-2005-4, Colorado Department of Transportation, Denver, CO, September 2005, 178 p.

Abu-Hejleh, N., M. W. O'Neil, D. Hanneman, and W. J. Attwooll, *Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rocks*, Final Contract Report No. CDOT-DTD-R-2003-6, Colorado Department of Transportation, Denver, CO, July 2003, 199 p.

Aurora, R. P., and L. C. Reese, *Behavior of Axially Loaded Drilled Shafts in Clay-Shales*, Final Contract Report No. CFHR 3-5-72-176-4, Center for Highway Research, The University of Texas at Austin, Austin, TX, March 1976, 184 p.

Brown, D. A., and R. Thompson, *Drilled shaft load test – Assessment, Sumter County, Alabama*, Letter Report to The State of Alabama Highway Department, Montgomery, Alabama, 2008, 46 p., http://danbrownandassociates.com/wp-content/uploads/2008/06/brf-00008536-sr8-drilled-shaft-load-test-assessment-sumter-county-al_brownthompson_11feb08.pdf.

Brown, D. A., J. P. Turner, and R. J. Castelli, *Drilled Shafts: Construction Procedure and LRFD Design Methods*, Final Contract Report No. FHWA-NHI-10-016, Federal Highway Administration, Washington, D.C., May 2010, 970 p.

Canadian Geotechnical Society, *Canadian Foundation Engineering Manual*, Friesens Corporation, Altona, MB, 2006, 488 p.

Carter, J. P., and F. H. Kulhawy, *Analysis and design of drilled shaft foundations socketed into rock*, Final Contract Report No. EL-5918, Electric Power Research Institute, Palo Alto, August 1988, 190 p.

Corps of Engineers, "Investigations for Building Foundations in Expansive Clays," Vol. 1, US Army Engineer District, Forth Worth, TX, 1968.

Deere, D. U., and D. W. Deere, "The Rock Quality Designation (RQD) Index in Practice," *Rock Classification Systems for Engineering Purposes*, ASTM STP 984, L. Kirkaldie, ed., American Society for Testing and Materials, Philadelphia, 1988, pp. 91–101.

Geoke, P. M., and P. A. Hustad, "Instrumented Drilled Shafts in Clay-shale," *Proceedings of Symposium on Deep Foundation*, E. M. Fuller, ed., ASCE National Convention, Atlanta, 1979, pp. 149–164.

Gibson, G. L., Field and laboratory investigation for design of rock-socketed caissons for a multi-storey building founded in sound limestone, Master of Engineering thesis, Carleton University, Ottawa, 1973.

Gupton, C., and T. Logan, "Design guidelines for drilled shafts in weak rocks of south Florida," *South Florida Annual ASCE Meeting*, 1984.

Hassan, K. M., M. W. O'Neil, S. A. Sheikh, and C. D. Ealy, "Design Method for Drilled Shafts in Soft Argillaceous Rocks," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 123, No. 3, 1997, pp. 272–280.

Horvath, R. G., *Field load test data on concrete-to-concrete rock bond strength drilled pier foundations*, Publication 78-07, Toronto, University of Toronto, 1978.

Horvath, R. G., and T. C. Kenney, “Shaft resistance of rock socketed drilled piers,” *Symposium on Deep Foundations*, ed. F.M. Fuller, Atlanta, October 1979, pp. 182–214.

Horvath, R. G., Drilled piers socketed into weak rock – method of improving performance, PhD Dissertation, University of Toronto, Ontario, 1982.

Horvath, R. G., T. C. Kenney, and P. Kozicki, “Methods for improving the performance of drilled piers in weak rock,” *Canadian Geotechnical Journal*, Vol. 20, No. 4, 1983, pp. 758-772.

Jaeger J. C., N. G. W. Cook, and R. W. Zimmerman, *Fundamentals of Rock Mechanics*, Blackwell Publishing, Ltd., Malden, MA, 2007, 475 p.

Johnston, I. W., and I. B. Donald, *Rock socket pile tests*, Final Contract Report No. 78/6/G, 703 Flinders Street–Spencer Street Overpass. Melbourne underground rail loop project, Monash University, Melbourne, Australia, 1979.

Kulhawy, F. H., and K. K. Phoon, “Drilled shaft side resistance in clay soil to rock,” In *Design and Performance of Deep Foundations: Piles and Piers in Soil and Soft Rock*, P. P. Nelson, T. D. Smith, and E. C. Clukey, eds., New York, October 24–28, 1993, pp. 172–183.

Kulhawy, F.H., W.A. Prakoso, and S.O. Akbas, “Evaluation of Capacity of Rock Foundation Sockets,” *40th U.S. Symposium on Rock Mechanics*, G. Chen, S. Huang, W. Zhou, and J. Tinucci, eds., Anchorage, Alaska, June 2005, 8 p.

Leach, B. A., J. W. Medland, and H. B. Sutherland, “The ultimate bearing capacity of bored piles in weathered Keuper marl,” *6th European Conference on Soil Mechanics and Foundation Engineering*, Vienna, Vol. 3, 1976, pp. 507–514.

Ladanyi, B., “Friction and end bearing tests on bedrock for high capacity socket design: discussion,” *Canadian Geotechnical Journal*, Vol. 14, 1977, pp. 153-155.

LOADTEST, Inc., *Report on drilled shaft load testing at CH-15 over the Illinois River*, Final Contract Report No. LT-8276, Gainesville, FL, September 1996, 71 p.

LOADTEST, Inc., *Report on drilled shaft load testing at East test shaft—US 36 over Republican River*, Scandia, KS, Final Contract Report No. LT-8718-2, Gainesville, FL, April 2001a, 58 p.

LOADTEST, Inc., *Report on drilled shaft load testing at West test shaft—US 36 over Republican River*, Scandia, KS, Final Contract Report No. LT-8718-1, Gainesville, FL, April 2001b, 59 p.

LOADTEST, Inc., *Report on drilled shaft load testing at Test Shaft #1—US 281 over Solomon River*, Osborne County, KS, Final Contract Report No. LT-8816, Gainesville, FL, October 2001c, 54 p.

LOADTEST, Inc., *Report on drilled shaft load testing at Pier 1 West—US 75 at 77th Street*, Wakarusa, KS, Final Contract Report No. LT-8733, Gainesville, FL, February 2001d, 57 p.

LOADTEST, Inc., *Report on drilled shaft load testing at Dedicated Test Shaft-I 235 over Des Moines River*, Final Contract Report No. LT-8854, Gainesville, FL, November 2002, 74 p.

LOADTEST, Inc., *Report on drilled shaft load testing at Test shaft- Route 116 over Platte River*, Final Contract Report No. LT-9048, Gainesville, FL, September 2004, 55 p.

LOADTEST, Inc., *Report on drilled shaft load testing at Test Shaft #1-IL5 over IL 84*, Final Contract Report No. LT-9405, Gainesville, FL, April 2008, 65 p.

Mason, R. C., "Transmission of high loads to primary foundations by large diameter shafts," paper to the ASCE Convention, New Orleans, 1960.

Matich, M. A. J., and P. Kozicki, "Some load tests on drilled cast-in-place concrete caissons," *Canadian Geotechnical Journal*, Vol. 4, No. 4, 1967, pp. 367–375.

McVay, M.C., F. C. Townsend, and R. C. Williams, "Design of socketed drilled shaft in limestone," *Journal of Geotechnical Engineering*, Vol. 118, No. 10, 1992, pp. 1626–1637.

Meigh, A. C., and W. Wolski, "Design parameters for weak rock," *7th European Conference on Soil Mechanics and Foundation Engineering,* Brighton*, 1979,* pp. 59–79.

Millar, W. R., *Results of pile tests on city centre and telephone exchange sites*, Perth, 1976.

Miller, A. D., Prediction of Ultimate Side Shear for Drilled Shafts in Missouri Shales, MS thesis, University of Missouri–Columbia, Missouri, MO, August 2003, 393 p.

Moore, W. W., *Foundation design*, ASCE, Civil Engineering, 1964, pp. 33-35.

O'Neil, M. W., and L. C. Reese, *Drilled Shafts: Construction Procedures and Design Methods*, Final Contract Report No. FHWA-IF-99- 025, Federal Highway Administration, Washington, D.C., 1999, 758 pp.

O'Neil, M. W., F. C. Townsend, K. H. Hassan, A. Buller, and P. S. Chan, *Load Transfer for Drilled Shafts in Intermediate Geomaterials*, FHWA Publication No. FHWA-RD-9-172, Department of Transportation, Federal Highway Administration, McLean, VA, 1996, 790 pp.

Osterberg, J. P., and S. A. Gill, "Load transfer mechanism for piers socketed in hard soils or rocks," *9th Canadian rock mechanics symposium,* Montreal*,* 1973.

Pells, P. J. N., D. J. Douglas, B. Rodway, C. Thorne, and B. K. McMahon, *Design loading for foundations on shales and sandstone in the Sydney region*, Research Report No. R 315, The University of Sydney, Sydney, March 1978.

Prakoso, W. A., Reliability-based design of foundations in rock masses, PhD dissertation, Cornell University, Ithaca, 2002.

Reese, L. C. and M. W. O'Neill, *Drilled shafts: construction procedures and design methods*, Report No. FHWA-HI-88-042, Washington, DC, 1988, 564 p.

Reese, L. C. and M. W. O'Neill, "New Design Method for Drilled shafts from Common Soil and Rock Tests," Proceedings of Foundation Engineering" Current Principles and Practices, ASCE Fred Kulhawy Ed., 1989, pp. 1026 – 1039.

Reynolds, R. T., and T. J. Kaderbeck, "Miami limestone foundation design and construction," *Journal of the Geotechnical Engineering Division*, Vol. 107, No. GT7, 1980, pp. 859–872.

Rosenberg, P., and N. L. Journeaux, "Friction and end bearing tests on bedrock for high capacity socket design," *Canadian Geotechnical Journal*, Vol. 13, No. 3, 1976, pp. 324–333.

Rowe, R. K., and H. H. Armitage, *Design of piles socketed into weak rocks*, GEOT -11-84, London, University of Western Ontario, 1984.

Rowe, R. K., and H. H. Armitage, "A design method for drilled piers in soft rock," *Canadian*

Geotechnical Journal, Vol. 24, No. 1, 1987, pp. 126–142.

Teng, W. C., *Foundation Design*, Prentice-Hall, Englewood Cliffs, NJ, 1962.

Walter, D. J., W. J. Burwash, and R. A. Montgomry, "Design of large-diameter drilled shafts for the Northumberland Strait bridge project," *Canadian Geotechnical Journal*, Vol. 34, No. 4, 1997, pp. 580-787.

Williams, A. F., I. W. Johnston, and I. B. Donald, "Design of socketed piles in weak rock," *The International Conference on Structural Foundations on Rock,* P. J. N. Pells, ed., Sydney, May 7–9, 1980, pp. 327–347.

Williams, A. F., *The design and performance of piles socketed into weak rock,* PhD dissertation, Monash University, Melbourne, Australia, 1980a.

Williams, A. F., "Principle of side resistance development is rock socketed piles," *Third Australia–New Zealand Conference on Geomechanics*, Wellington, May 12–16, 1980b, pp. 87–94.

Williams, A. F., and P. J. N. Pells, "Side resistance of rock sockets in sandstone, mudstone, and shale," *Canadian Geotechnical Journal*, Vol. 18, No. 4, 1981, pp. 502–513.

Wilson, L. C., "Tests of bored piles and driven piles in cretaceous mudstone at Port Elizabeth, South Africa," *Geotechnique*, Vol. 26, No. 1, 1976, pp. 5–12.

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Side Resistance of Drilled Shafts in Weak Fine Grained Sedimentary Rock

Timothy D. Stark[1], Ahmed H. Baghdady[2], Abdolreza Osouli[3], Heather Shoup[4], and Michael A. Short[5]

Figure Captions:

Figure 1. Load test database for unit side resistance with various construction techniques.

Figure 2. Effect of shaft diameter on adhesion factor or maximum side resistance.

Figure 3. Effect of shaft displacement on mobilized side resistance.

Figure 4. Effect of post-peak shaft displacement on side resistance.

Figure 5. Effect of rock type and unconfined compressive strength on side resistance.

Figure 6. Comparison of predictive models and side resistance database.

Figure 7. Comparison of predictive models of Table 5 with side resistance database.

Figure 8. Predictive method for unit side resistance of drilled shafts in weak rocks, using a linear function to fit the load test data.

Table Captions:

Table 1. Available side resistance predictive methods in rocks.

Table 2. Summary of load test methods.

Table 3. Summary of rock types and their unconfined compressive strength.

Table 4. Unit side resistance database for drilled shafts in weak clay based sedimentary rocks.

Table 5. Statistics for unit side resistance predictive models based on database presented in Table 4.