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EVALUATION OF SIDE RESISTANCE CAPACITY FOR DRILLED SHAFTS

Yit-Jin Chen*, Shiu-Shin Lin*, Hsin-Wen Chang*, and Maria Cecilia Marcos*

Key words: drilled shaft, load test, side resistance, case history.

ABSTRACT

This paper presents an extensive evaluation of axial side resistance of drilled shaft foundations. A wide variety of load test data are used and these data are divided into drained and undrained databases. Representative analytical models, including alpha (α), beta (β), and lambda (λ) methods, are examined in detail using both measured and predicted results to assess their relative merits for the drilled shaft design. Based on these analyses, the undrained shear strength (α-s_u) correlations have exhibited better statistics for undrained side resistance prediction, and the undrained strength ratio (α-USR) correlations can be adopted as an alternative analytical method, especially in the case of smaller s_u. For β method, the stress factors (K/K_o) are developed from the back-analysis of field load tests. However, the β method has presented more varying results for short shafts in both drained and undrained loading. Among all analytical methods, the λ method is relatively the less reliable prediction model. Specific design recommendations for side resistance analysis of drilled shaft are suggested.

I. INTRODUCTION

Drilled shafts (also called cast-in-place piles, drilled piers or bored piles) are frequently used as foundation for modern high-rise buildings, bridges, electrical transmission line structures, etc. Side resistance is an important source of drilled shaft capacity under axial loading, especially when the shaft is under the condition of uplift loading or considerably larger depth. Researches about this subject still have progressed during the past five decades. Based on soil conditions, the methods for analyzing side resistance are of two types: total stress analysis and effective stress analysis. These analytical methods can be further specified into the alpha (α), beta (β), and lambda (λ) methods. Table 1 lists the equations and the related factors for each method.

The α method is a conventional total stress analysis for the side resistance of drilled shaft foundations in cohesive soils. The side resistance capacity is related to the average soil undrained shear strength (s_u) by an empirical coefficient denoted as α, which is the adhesion factor. The original α [14] was based on empirical correlations of mean s_u over the foundation depth, using primarily driven pile data. The research from Stas and Kulhawy [13] showed that α method is meaningful for drilled shafts. They developed the correlation of α versus s_u for the drilled shaft design, however, the s_u values in their analysis were taken from random test types, as shown in Fig. 1.

Moreover, several researchers [7, 11, 12] also demonstrated that α is complexly related to other soil parameters such as the mean effective overburden stress ($\bar{\sigma}_{vm}$), the overconsolidation ratio (OCR), and the effective stress friction angle ($\bar{\phi}$). With these suggestions, Goh *et al*. [4] further carried out parametric studies using the trained neural network model and proposed that $\bar{\sigma}_{vm}$ can directly or indirectly influence α in designing drilled shafts.

The β method is an effective stress analysis which considers the frictional resistance for the soil-shaft interface. In this method, the side resistance is a function of horizontal effective stress ($\bar{\sigma}_{ho}$), effective stress friction angle (δ) for the soil-shaft interface, and shaft geometry. Kulhawy *et al*. [9] examined the available load test data and presented that the stress factor (K/K_o) is generally less than 1 and dependent on the construction method and its influence on the in-situ stress. They also suggested that the ratio of interface friction angle (δ) to soil friction angle ($\bar{\phi}$) is equal to 1 for drilled shafts.

Finally, the lambda (λ) method is a combination of total and effective stress analyses that can be used for cohesive soils. In this method, the side resistance is related to s_u and $\bar{\sigma}_{vm}$ by an empirical factor λ. The original empirical factor λ [15] was developed based on the database of driven pile data and is a function of the total depth of the pile. However, no further effort has been done in detail to examine its validity in the design of drilled shafts.

Although some of these analytical methods have been examined previously, it is reasonable to completely reassess these methods for the side resistance analysis of drilled shaft design, because new approaches have been developed, and more consistent procedures for assessing interpreted failure

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Table 1. Analysis equations of side resistance for drilled shafts.

Method	Equation[a]	Definition of factors
α	$Q_s(\alpha) = \pi B \sum_{n=1}^{N} \alpha_n s_{un} t_n$ (1)	α: empirical adhesion factor s_u: undrained shear strength
β	$Q_s(\beta) = \pi B \left(\frac{K}{K_o}\right) \sum_{n=1}^{N} \bar{\sigma}_{vn} K_{on} \tan\left[\bar{\phi}_n \cdot \frac{\delta}{\bar{\phi}}\right] t_n$ (2)	K: coefficient of horizontal soil stress K_o: in-situ K $\bar{\sigma}_v$: vertical effective stress $\bar{\phi}$: effective stress friction angle δ: interface friction angle for soil-shaft $\beta = K \tan\delta$
λ	$Q_s(\lambda) = \pi B \sum_{n=1}^{N} \lambda_n \left[\bar{\sigma}_{vn} + 2 s_{un}\right] t_n$ (3)	λ: empirical factor

[a]Q_s = capacity of side resistance; B = shaft diameter; N = number of soil layers; t = thickness.

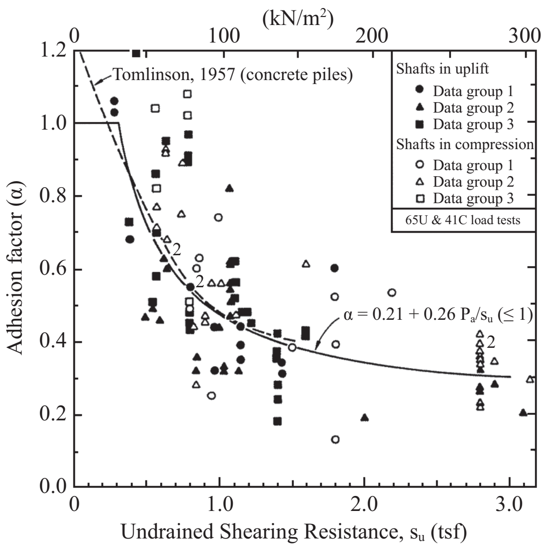

Fig. 1. α-s_u correlation [13].

load have been improved, additionally, numerous updated load test data have existed since those earlier studies. In this paper, a broad database is used to assess the relative merits and suitability of each analytical model using the most updated data and approaches. The results are compared statistically and graphically. After which, specific design recommendations for the use of side resistance in drilled shaft design are presented.

II. DATABASE OF LOAD TESTS

The database developed for this study is consisted of 222 field axial load tests conducted at 105 sites with a wide variety of soil profiles. All load test data were collected from published literature or load test reports. Based on primarily dominant soil conditions along the shaft length, axial load test data are grouped into drained or undrained loading. Among these load test data, 74 tests at 40 sites are grouped into drained loading and 148 tests at 65 sites are grouped into undrained loading. Load tests are divided into two groups, each based on the overall data quality, completing a total of eight-sub groups as follows: Group 1 in compression (denoted DC1 and UC1 for drained and undrained soils, respectively), Group 2 in compression (DC2 and UC2), Group 1 in uplift (DU1 and UU1), and Group 2 in uplift (DU2 and UU2). Group 1 includes those cases in which the complete load-displacement curve and geotechnical parameters were reported. Group 2 consists of all remaining cases, such as the groundwater table was not reported, the load test was stopped before interpreted failure load, or the geotechnical parameters were not measured over the entire depth, and therefore they have to be inferred from other tests. All of the selected tests were conducted on straight-sided drilled shafts. Based on the case history descriptions, it appears that the shaft constructions and test performances were of high quality. Consequently, these field data should reflect real situations and the results of the analyses could be representative for subsequent applications in engineering practice.

Tables 2 and 3 show the interpreted parameters and result of the analyses for the load tests with drained and undrained soils, respectively. The values of α, β_m, and λ are back-calculated using Eqs. (1)-(3) from the field load test results. The L_1-L_2 method [5, 6], which is a graphical construction method, is adopted to define the capacity in both uplift and compression. As shown in Fig. 2, the load-displacement curves can generally be simplified into three distinct regions: initial linear, curve transition, and final linear. Point L_1 (elastic limit) corresponds to the load (Q_{L1}) and butt displacement (ρ_{L1}) at the end of the initial linear region, while L_2 (failure threshold) corresponds to the load (Q_{L2}) and butt displacement (ρ_{L2}) at the initiation of the final linear region. Q_{L2} is defined as the "interpreted failure load or capacity" because beyond Q_{L2}, a small increase in load gives a significant increase in displacement. The updated research studies by Chen et al. [1] and Chen and Fang [2] defined that Q_{L2} occurs on average, at 10.6 mm (for drained loading) and 12.1 mm (for undrained loading) under uplift loading, while it occurs at 4%B for both drained and undrained compression

Table 2. Summary of analysis data for drained loading.

Site/Case No.	Q_s (kN)	$\bar{\sigma}_{vm}$ [a] (kN/m^2)	$\bar{\phi}_{tc}$ [b] (deg)	K_o [c]	β_m [d]	β_p [e]	Site/case No.	Q_s (kN)	$\bar{\sigma}_{vm}$ [a] (kN/m^2)	$\bar{\phi}_{tc}$ [b] (deg)	K_o [c]	β_m [d]	β_p [e]
DU1	334	27	36	1.4	1.42	1.05	DU21-2	52	42	36	1.3	0.97	0.59
DU2	58	15	35	1.0	0.76	0.72	DU21-3	74	76	35	0.9	0.63	1.02
DU3	362	14	36	3.1	2.97	2.43	DU21-4	90	76	35	0.9	0.63	0.70
DU4	484	42	35	1.1	0.93	0.79	DC1	488	42	33	0.9	0.60	1.56
DU5-1	24	13	36	3.1	1.36	1.64	DC2	978	42	36	2.1	1.57	0.69
DU5-2	68	19	37	2.5	1.50	1.38	DC3-1	1779	106	36	0.5	0.37	1.34
DU5-3	159	25	37	2.0	1.78	1.10	DC3-2	3202	99	37	0.7	0.54	1.61
DU6-1	650	32	36	2.1	0.94	1.11	DC4	548	50	30	1.8	1.07	0.93
DU6-2	650	32	36	2.1	0.94	1.11	DC5	1246	89	40	0.8	0.49	0.96
DU7-1	390	42	44	1.0	0.66	0.99	DC6	5907	96	34	0.7	0.44	-
DU7-2	498	42	44	1.0	0.77	0.99	DC7	4270	86	38	1.1	0.89	-
DU8-1	261	56	35	0.4	0.32	0.31	DC8-1	2755	89	40	0.7	0.59	1.00
DU8-2	278	56	35	0.4	0.28	0.31	DC8-2	3528	89	40	0.7	0.59	0.97
DU9-1	1028	29	40	3.6	3.03	2.93	DC9	144	50	35	1.1	0.75	1.02
DU9-2	1225	35	36	3.6	2.50	2.54	DC10-1	1440	127	36	0.5	0.37	0.92
DU10-1	605	20	44	-	6.08	-	DC10-2	1504	127	36	0.5	0.37	0.96
DU10-2	894	29	44	-	4.35	-	DC10-3	1280	118	36	0.5	0.37	0.96
DU11-1	205	38	35	-	0.83	-	DC10-4	1520	135	36	0.5	0.37	0.80
DU11-2	273	26	45	-	2.42	-	DC10-5	1696	127	36	0.5	0.37	1.08
DU11-3	180	25	40	-	1.73	-	DC11	4000	210	35	0.4	0.27	0.97
DU11-4	372	21	51	-	5.03	-	DC12	9963	108	43	1.1	0.75	1.72
DU11-5	115	30	31	-	0.87	-	DC13	1637	94	-	-	-	-
DU11-6	54	30	31	-	0.38	-	DC14	2562	50	38	2.2	1.77	0.98
DU11-7	67	30	31	-	0.48	-	DC15	6832	83	34	-	-	-
DU11-8	108	30	31	-	0.77	-	DC16-1	4494	61	-	-	-	-
DU11-9	141	44	31	-	0.45	-	DC16-2	2092	61			-	-
DU11-10	135	44	31	-	0.44	-	DC17-1	5160	90	-	-	-	-
DU11-11	211	44	31	-	0.68	-	DC17-2	5898	102	-	-	-	-
DU11-12	222	43	31	-	0.77	-	DC18	5696	254	43	1.5	1.02	1.02
DU11-13	47	25	33	-	0.40	-	DC19-1	1552	212	42	0.9	0.83	1.02
DU11-14	21	25	33	-	0.18	-	DC19-2	3465	230	42	0.9	0.83	1.09
DU11-15	24	25	33	-	0.21	-	DC19-3	1357	86	44	2.0	1.99	0.92
DU11-16	88	25	33	-	0.75	-	DC19-4	3059	86	44	2.0	1.99	-
DU12	2870	99	37	0.5	0.78	0.42	DC20	4520	177	37	0.5	0.39	1.02
DU13-1	729	42	43	1.3	1.74	1.25	DC21	6808	173	45	0.4	0.46	1.24
DU13-2	334	29	43	1.8	1.77	1.81	DC22-1	8838	311	35	0.4	0.22	0.79
DU14-1	564	49	35	1.5	1.67	1.08	DC22-2	7745	311	35	0.4	0.22	0.69
DU14-2	613	58	35	1.3	1.23	0.94	DC22-3	7935	311	35	0.4	0.22	0.71
DU14-3	522	70	35	1.0	0.72	0.72	DC23	24400	210	38	0.5	0.40	1.15
DU14-4	598	74	35	0.9	0.67	0.65	DC24	5600	113	45	0.5	-	-
DU14-5	683	77	35	0.8	0.64	0.58	DC25	9252	233	35	0.4	0.21	0.86
DU15-1	193	21	33	2.3	1.32	1.54	DC26	1067	70	-	-	-	-
DU15-2	362	27	34	1.9	1.52	1.32	DC27-1	2206	118	-	-	-	-
DU15-3	407	28	44	2.7	3.11	2.69	DC27-2	2170	118	-	-	-	-
DU16	974	35	-	-	2.66	-	DC28-1	676	66	39	0.7	0.58	1.20
DU17-1	2369	288	45	0.3	0.24	0.27	DC28-2	1210	100	43	0.6	0.58	0.94
DU17-2	2668	265	45	0.3	0.20	0.27	DC29	124	45	28	0.6	0.33	1.35
DU18-1	92	27	-	-	0.79	-	DC30	427	47	37	0.9	0.70	1.18
DU18-2	111	29	-	-	0.81	-	DC31	1370	85	45	0.7	0.72	0.87
DU18-3	113	31	-	-	0.71	-	DC32-1	9550	207	33	0.5	0.31	0.86
DU18-4	99	34	-	-	0.53	-	DC32-2	9128	207	33	0.5	0.31	0.83
DU18-5	112	27	-	-	0.96	-	DC33	7579	115	41	0.6	0.46	1.07
DU18-6	101	27	-	-	0.86	-	DC34-1	196	90	30	0.5	0.30	0.67
DU19	1950	36	44	5.5	6.40	5.52	DC34-2	233	90	30	0.5	0.30	0.69
DU20-1	365	90	30	0.5	0.37	0.31	DC34-3	350	90	30	0.5	0.30	0.83
DU20-2	435	90	30	0.5	0.38	0.31	DC35	4500	135	-	-	-	-
DU20-3	431	90	30	0.5	0.30	0.31	DC36-1	2215	65	-	-	-	-
DU21-1	82	42	36	1.3	0.89	0.97	DC36-2	2680	94	-	-	-	-

[a] $\bar{\sigma}_{vm}$ = mean effective overburden stress along shaft. [b] $\bar{\phi}_{tc}$ = effective stress friction angle ($\bar{\phi}$) for triaxial compression test. [c] $K_o = (1 - \sin\bar{\phi}_{tc})OCR^{\sin\bar{\phi}_{tc}}$, based on Mayne and Kulhawy [10]. [d] β_m was calculated using Eq. (10). [e] β_p was calculated using Eq. (9) and the suggested K/K_o values of this study.

Table 3. Summary of analysis data for undrained loading.

Site/Case No.	Q_s (kN)	$\bar{\sigma}_{vm}$ [a] (kN/m^2)	s_u(CIUC) (kN/m^2)	K_o [b]	α_{CIUC}	β_m [c]	β_p [d]	λ
UU1	525	37	102	2.76	0.61	1.71	1.71	0.26
UU2-1	334	19	307	5.11	0.23	2.98	2.89	0.11
UU2-2	356	19	307	5.11	0.25	3.65	2.89	0.12
UU3-1	301	25	68	3.67	0.63	1.75	2.37	0.27
UU3-2	319	25	68	3.67	0.67	1.87	2.37	0.28
UU4	290	40	59	-	0.56	3.09	-	0.21
UU5	1177	343	285	1.04	0.32	0.33	0.47	0.10
UU6	220	29	100	3.10	0.35	1.22	1.85	0.15
UU7	125	23	112	2.52	0.37	2.05	1.76	0.17
UU8	270	16	95	4.71	0.61	3.12	3.43	0.28
UU9-1	402	63	56	-	0.81	0.74	-	0.26
UU9-2	451	59	57	-	0.69	0.69	-	0.23
UU9-3	323	40	57	-	0.66	0.95	-	0.24
UU10-1	194	34	26	0.61	0.87	0.67	0.40	0.26
	194	34	32	0.61	0.71	-	-	0.23
UU10-2	182	34	26	0.61	0.88	0.68	0.40	0.27
	182	34	32	0.61	0.72	-	-	0.23
UU11	616	80	96	1.22	0.32	0.39	0.65	0.11
UU12-1	618	43	21	0.84	0.84	0.31	0.50	0.21
UU13-1	480	29	74	2.42	0.56	1.42	1.44	0.23
	480	29	178	-	0.26	-	-	0.11
	480	29	165	-	0.27	-	-	0.11
UU13-2	943	34	86	2.32	0.63	1.59	1.38	0.26
	943	34	180	-	0.30	-	-	0.14
	943	34	166	-	0.33	-	-	0.15
UU13-3	147	26	68	2.55	0.62	1.62	1.51	0.26
	147	26	179	-	0.25	-	-	0.11
	147	26	165	-	0.26	-	-	0.12
UU14-1	2044	57	58	0.90	0.64	0.65	0.68	0.21
UU14-2	2044	57	45	-	0.82	-	-	0.25
UU14-3	2044	57	53	-	0.70	-	-	0.23
UU15-1	448	29	182	1.80	0.40	2.48	1.41	0.18
UU15-2	723	44	182	1.41	0.42	1.73	1.11	0.19
UU16	8896	296	120	0.53	0.56	0.23	0.24	0.13
UU17	6316	285	120	0.50	0.40	0.19	0.25	0.09
UU18	11565	142	100	0.88	0.51	0.36	0.40	0.15
UU19	3558	75	90	0.88	0.40	0.47	0.38	0.14
UU20-1	614	51	94	1.01	0.42	0.77	0.60	0.16
UU20-2	667	51	94	1.01	0.45	0.83	0.60	0.18

Site/Case No.	Q_s (kN)	$\bar{\sigma}_{vm}$ [a] (kN/m^2)	s_u(CIUC) (kN/m^2)	K_o [b]	α_{CIUC}	β_m [c]	β_p [d]	λ
UC1	395	55	147	1.19	0.34	0.92	0.74	0.14
UC2	1516	54	90	0.93	0.56	0.95	0.53	0.22
UC3	8162	148	100	0.41	0.50	0.34	0.35	0.14
UC4	19706	236	120	0.50	0.50	0.25	0.25	0.13
UC5	13406	296	120	0.53	0.52	0.21	0.24	0.12
UC6	9528	197	120	0.89	0.45	0.31	0.38	0.12
UC7	11521	231	120	0.49	0.40	0.21	0.26	0.10
UC8	4448	90	100	0.79	0.40	0.45	0.37	0.14
UC9	290	39	24	0.74	0.88	0.54	0.44	0.11
UC10-1	178	30	132	2.14	0.27	1.19	1.38	0.10
	178	30	137	-	0.26	-	-	0.10
UC10-2	383	45	118	1.74	0.35	0.91	1.13	0.13
	383	45	109		0.38	-	-	0.14
UC10-2	890	67	136	1.45	0.45	0.91	1.01	0.17
	890	67	123	-	0.50	-	-	0.18
UC11-1	636	107	291	1.59	0.32	0.86	1.05	0.03
UC11-2	2500	119	303	1.40	0.41	1.04	0.93	0.10
UC11-3	2400	110	297	1.39	0.27	0.74	0.92	0.07
UC11-4	1180	105	340	1.40	0.29	0.89	0.93	0.04
UC11-5	1400	97	302	1.42	0.23	0.71	0.94	0.04
UC12-1	946	70	119	1.37	0.40	0.69	0.92	0.18
	946	70	94	-	0.51	-	-	0.22
UC12-2	1067	70	119	1.37	0.54	0.91	0.92	0.21
	1067	70	94	-	0.68	-	-	0.25
UC12-3	2130	105	137	1.27	0.37	0.48	0.63	0.18
	2130	105	120	-	0.42	-	-	0.20
UC13-1	41	28	48	2.00	0.63	1.07	1.29	0.24
UC13-2	74	36	50	1.85	0.71	0.99	1.15	0.26
UC13-3	122	40	57	1.83	0.80	1.15	1.09	0.30
UC13-4	129	41	68	1.83	0.68	1.14	1.09	0.26
UC14-1	2355	83	192	1.56	0.45	1.04	1.32	0.18
	2355	83	208	-	0.41	-	-	0.17
	2355	83	193	-	0.45	-	-	0.18
UC14-2	2028	83	192	1.56	0.37	0.86	1.32	0.15
	2028	83	208	-	0.34	-	-	0.14
	2028	83	193	-	0.37	-	-	0.15
UC15-1	329	60	140	1.11	0.31	0.70	0.72	0.12
	329	60	186	-	0.25	-	-	0.10
UC15-2	3053	160	260	0.99	0.40	0.71	0.67	0.17

[a] $\bar{\sigma}_{vm}$ = mean effective overburden stress along shaft. [b] $K_o = (1 - \sin\bar{\phi}_{tc})\mathrm{OCR}^{\sin\bar{\phi}_{tc}}$, based on Mayne and Kulhawy [10]. [c] β_m was calculated using Eq. (10). [d] β_p was calculated using Eq. (9) and the suggested K/K_o values of this study.

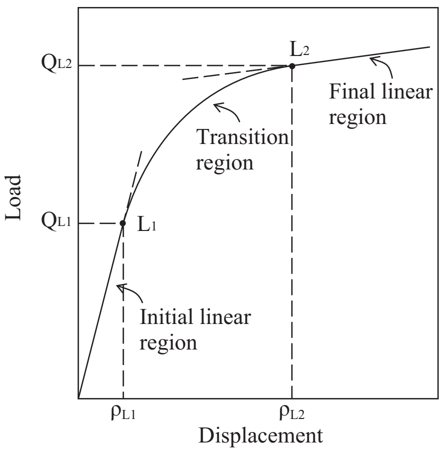

Fig. 2. Regions of load-displacement curve.

loading. They also developed the relationships between L_2 and other interpretation criteria from lower bound to higher bound. Therefore, these interrelationships among interpretation criteria are also used to infer the required L_2 if the load test data are insufficient or are terminated prematurely.

Table 4 lists the reference sources and soil strength test types of all load test case histories in Tables 2 and 3. These load tests were conducted on various types of soil around the world and at different times. For convenience, Table 5 lists the ranges of foundation geometry and the interpreted capacities from the database, along with the data standard deviation (SD) and the coefficient of variation (COV), which is the standard deviation divided by the mean. As can be seen, the ranges are broad, but the results for drained and undrained analyses are roughly comparable.

Table 4. Reference sources of shaft load tests in Tables 2 and 3.

Site No.	Test type		Reference sources
	s_u [a]	$\bar{\phi}$ [b]	
DU1, 15; UU2, 3	UC	SPT-N	Harza Engineering Co. (1978)"Foundation uplift tests – Missouri Basin Power Project East Transmission", for Basin Electric Power Coop., Bismarck, 26 p.
DU2	-	SPT-N	Ohio Edison Co. (early 1960s) "Test program proves feasibility of concrete cylinder anchors for steel transmission towers for Ohio Edison Co.", Akron, 8 p.
DU3	-	SPT-N	Virginia Electric and Power Co. (1967) "Tests on piles and caissons, slurry-Hopewell 230 KV; Bacons Castle, Virginia", 15 p.
DU4; DC1	-	SPT-N	Florida Testing Laboratories (1965) "Combined section test pile, tension and compression", for Florida Power Corp., St. Petersburg, 9 p.
DU5	-	SPT-N	Florida Testing Laboratories (1966) "Piles research for design parameters", for Florida Power Corp., St. Petersburg, 69 p.
DU6; DC2	-	SPT-N	Ismael, NF and Klym, TW (1979) "Uplift and bearing capacity of short piers in sand", J. Geotechnical Eng. Div., ASCE, 105(GT5), 579-594.
DU7	-	SPT-N	Stern, LI, Bose, SK and King, RD (1976) "Uplift capacity of poured-in-place cylindrical caissons", Paper A 76 153-9, IEEE PES Winter Meeting, New York, 9 p.
DU8; UU11	UC	SPT-N	Sowa, VA (1970) "Pulling capacity of concrete cast-in-situ bored piles", Canadian Geotechnical J., 7(4), 482-493.
DU9	-	SPT-N	Stone and Webster Engineering Corp. (1969) "Report on pull test on auger piles, Branchburg to Ramapo Line", Public Service Electric and Gas Co., Newark, 18 p.
DU10; UU15	CIUC	CPT	Briaud, J-L, Pacal, AJ and Shively, AW (1984) "Power line foundation design using the pressuremeter", Proc., 1st Intl. Conf. on Case Histories in Geotechnical Eng. (1), Rolla, 279-283.
DU11; UU9	DSS	CPT	Tucker, KD (1987) "Uplift capacity of drilled shafts and driven piles in granular materials", Foundations for Transmission Line Towers (GSP 8), Ed. J-L Briaud, ASCE, New York, 142-159.
DU12; DC3	-	SPT-N	HDR Infrastructure (1986) "Preliminary foundation report and summary of load test program", I-275, Howard Frankling Bridge, Tampa, 77 p.
DU13	-	CPT	Konstantinidis, B, Pacal, AJ and Shively, AW (1987) "Uplift capacity of drilled piers in desert soils: a case history", Foundations for Transmission Line Towers (GSP 8), Ed. J-L Briaud, ASCE, New York, 128-141.
DU14	-	SPT-N	Marsico, R, Retallack, RL and Tedesco, PA, (1976) "Report on pile testing for AEP transmission lines – Part I and II", Transactions on Power Apparatus and Systems, IEEE, 95(6), 1795-1813.
DU16	-	SPT-N	Pacific Power and Light Co. (1980) "Pier uplift test", Portland, 20 p.
DU17	-	SPT-N	Sacre, AS (1977) "Study of pullout resistance of drilled shafts", MS Thesis, Univ. of Texas, Austin, 164 p.
DU18; UU7, 8	UU	-	Paterson, G and Urie, RL (1964) "Uplift resistance tests on full sizes transmission tower foundations", Proc., CIGRE (1), Paper 203, Paris, 19 p.
DU19	-	SPT-N	Woodward-Clyde Consultants (1980) "Geotechnical investigation for the Miguel-Imperial valley 500 KV transmission line (Tower sites 25 through 213)", for San Diego Gas and Electric Co., San Diego, 62 p.
DU20; DC34	-	CPT	Chambon, P., Corté J-F (1991) Étude sur modèles réduits centrifugés. Application aux tunnels à faible profondeur en terrain meuble pulvérulent, Études et Recherches des Laboratoires des Ponts et Chaussées, série Géotechnique, GT 48, 163. Neves, M, Mestat, P, Frank, R and Degny, E (2001) "Research on the behavior of bored piles - I. in situ and laboratory experiments", CESAR-LCPC, 231, 39-54.
DU21	-	CPT	Sven, K, Johan, C and Lars, D (2006) "Tension test on bored piles in sand", Symp. Intl. ELU/ULS, Paris, 87-94.
DC4	-	SPT-N	Jain, GS and Gupta, SP (1968) "Comparative study of multi-underreamed pile with large diameter pile in sandy soil", Proc., 3rd Budapest Conf. on Soil Mechanics and Foundation Eng., Budapest, 563-570.
DC5	-	SPT-N	O'Neill, MW and Reese, LC (1978) "Load transfer in a slender drilled pier in sand", Preprint 3141, ASCE Spring Convention, Pittsburgh, 30 p.
DC6, 7	-	SPT-N	Reese, LG and O'Neill, MW (1988) "Field load tests of drilled shafts", Proc., 1st Intl. Geotechnical Seminar on Deep Foundations on Bored and Auger Piles,
DC8	-	CPT	Brusey, WG and York, DL (1991) "Pile test program at John F. Kennedy International Airport", Foundations Profondes, Ecole Nationale des Ponts et Chausses, Paris, 379-387.
DC9	-	CPT	Stuckrath, L and Descoeudres, F (1991) "Large-scale pile tests in an instrumented test pit", Foundations Profondes, Ecole Nationale des Ponts et Chausses, Paris, 301-308.
DC10	-	CPT	Roscoe, GH (1983) "Behavior of flight auger bored piles in sand", Proc., Intl. Conf. on Advances in Piling and Ground Treatment for Foundations, ICE, London, 241-250.
DC11	-	SPT-N	Azevedo, N, Jr. (1991) "Load transfer in bored pile in residual soil", Proc., 9th Pan-American Conf. on Soil Mechanics and Foundation Eng. (2), 569-576.
DC12	-	SPT-N	Farr, S, and Aurora, RP (1981) "Behavior of instrumented pier in gravelly sand", Drilled Piers and Caissons, Ed. MW, O'Neill, ASCE, New York, 53-65.
DC13	-	-	Reese, LC (1982) "Analysis of instrumented drilled shafts subjected to axial load, Sunshine Skyway Bridge", Report GR81-20, Geotechnical Eng. Center, Univ. of Texas, Austin, 69 p.
DC14	-	CPT	Fujita, K (1986) "Pile construction and related problems", Proc., Intl. Conf. on Deep Foundations (2), Beijing, 137-155.

Table 4. (Continued)

Site No.	Test type		Reference sources
	s_u[a]	$\bar{\phi}$[b]	
DC15	-	CPT	Long, JH and Reese, LC (1983) "Analysis of field tests of drilled shafts", Report to Farr Foundation Co., Univ. of Texas, Austin, 34 p.
DC16	-	-	Cernak, B (1976) "Time effect of suspension on behavior of piers", Proc., 6th European Conf. on Soil Mechanics and Foundation Eng. (1.1), Vienna, 111-114.
DC17	-	-	Schmertmann and Crapps, Inc. (1985) "I-595 Viaduct Load Test Program", Job 86095-3406, for Florida Dept. of Transportation, Gainesville, 54 p.
DC18	-	SPT-N	Baker, CN, Jr. (1991) "Prediction and performance of drilled shafts constructed under slurry", 16th Annual Deep Foundations Institute Conf., Chicago, 155-172.
DC19	-	SPT-N	Franke, E and Garbrecht, D (1977) "Test loading on 8 large bored piles in sand", Proc., 9th Intl. Conf. on Soil Mechanics and Foundation Eng. (1), Tokyo, 529-532.
DC20	-	CPT	Caputo, V and Viggiani, C (1988) "Some experiences with bored and auger piles in Naples area", Proc., 1st Intl. Geotechnical Seminar on Deep Foundations on Bored and Auger Piles, Ghent, 273-281.
DC21	-	SPT-N	Frank, R, Kalteziotis, N, Bustamante, M, Christoulas, S and Zervogiannis, H (1991) "Evaluation of performance of two piles using pressuremeter method", J. Geotechnical Eng., ASCE, 117(5), 695-713.
DC22	-	SPT-N	Chang, MF and Lo, SS (1990) "Behavior of some axially loaded cast-in-place piles", Proc., 10th Southeast Asian Geotechnical Conf. (1), Taipei, 327-332.
DC23	-	SPT-N	Hirayama, H (1990) "Load-settlement analysis for bored piles using hyperbolic transfer functions", Soils and Foundations, 30(1), 55-64.
DC24	-	CPT	Brandl, H (1985) "Bearing capacity of piers and piles with large diameters", Proc., 11th Intl. Conf. on Soil Mechanics and Foundation Eng. (3), San Francisco, 1525-1530.
DC25	-	CPT	Wang, YZ (1986) "Design and construction of large bored piles over Yellow River", Proc., Intl. Conf. on Deep Foundations (2), Beijing, 124-129.
DC26	-	CPT	Martins, FF and Martins, JB (1989) "CPT and pile tests in granitic residual soils", Proc., 12th Intl. Conf. on Soil Mechanics and Foundation Eng. (1), Rio de Janeiro, 529-531.
DC27	-	-	Maertens, J (1985) "Comparative tests on bored and driven piles at Kallo", Belgian Geotechnical Volume, Published for the 1985 Golden Jubilee, ISSMFE, 31-38.
DC28, 29, 30, 31	-	CPT	Burch, SB, Parra, F, Townsend, FC and McVay, MC (1988) "Design guidelines for drilled shaft foundations", Univ. of Florida, Gainesville, 141 p.
DC32	-	SPT-N	Diagnostic Engineering Consultants, Ltd. (2001). "Report on compression load test of bored piles for Taiwan high speed rail project", Taiwan.
DC33	-	SPT-N	Hai Tain Engeering Ltd. (2002). "Report on compression load test of bored piles at Daja township", Tai-Chung County, Taiwan.
DC35, 36	-	-	Thasnanipan, N, Maung, AW, Ganeshan, B and Teparaksa W (1998). "Design, construction and behavior of bored cast-in-situ concrete piles in Bangkok subsoils", Proc., 4th Intl. Conf. on Case Histories in Geotechnical Eng., Louis Missouri, 281-287.
UU1	UC	TC	Mckenzie, RJ (1971). "Uplift testing of prototype transmission tower footings", Proc., 1st Australia-New Zealand Conf. on Geomechanics (1), Melbourne, 283-290.
UU4	UC	TC	Los Angeles Department of Water and Power (1967) "Footing tests for Pacific Intertie D-C system transmission line", Los Angeles, 37.
UU5	UU	TC	Pearce, RA and Brassow, CL (1979). "Pull-out load test of a drilled pier in very stiff Beaumont clay", Symp. on Deep Foundation, Ed. FM Fuller, ASCE, New York, 331-342.
UU6	UU	SPT-N	Wichita Testing Co. (1977). "Final report – steel Tower foundation test", for Kansas Gas and Electric Co., Wichita, 36.
UU10	UU, UC	CPT	Tang, NC, Shen, HR and Liu, SG (1983). "Static uplift capacity of bored piles", Proc., Intl. Conf. on Advances in Piling and Ground Treatment, ICE, London, 197-202.
UU12; UC9	CIUC	TC	Radhakrishna, HS, Cragg, CBH, Tsang, R and Bozozuk, M (1986), "Uplift and compression behavior of drilled piers in Leda clay", Proc., 39th Canadian Geotechnical Conf., Ottawa, 123-130.
UU13	UU, UC	TC	Adams, JI and Radhakrishna, HS (1970). "Uplift resistance of augered footings in fissured clay", Ontario Hydro Research Quarterly, 22(1), 10-16.
UU14	CIUC UU, UC	TC	Ismael, NF and Klym, TW (1978). "Behavior of rigid piers in layered cohesive soils", J. Geotechnical Eng. Div., ASCE, 104(GT8), 1061-1074.
UU16; UC5	UU, UC	SPT-N	Diagnostic Engineering Consultants, Ltd. (2002). "Report on uplift and compression load test of bored piles for Makoto bank", Taiwan, Taipei.
UU17;UC6	UU	SPT-N	Fubon Bank Corp. (1996). "Report on uplift and compression load test of bored piles for Fubon bank", Taiwan, Taipei.
UU18	UU	SPT-N	Diagnostic Engineering Consultants, Ltd. (2002). "Report on uplift load test of bored piles for Hung-Sheng Debao", Taiwan, Taipei.
UU19; UC8	UC, UU	SPT-N	Diagnostic Engineering Consultants, Ltd. (2002). "Report on uplift load test of bored piles for Shin Kong Insurance Co.", Taiwan, Taipei.
UU20	UU	TC	Yajima, J, Aoki, Y, and Shibasaki, F (1993). "Uplift capacity of piles under cyclic load", Proc., 11th Southeast Asian Geotechnical Conf., Singapore, 601-604.

Table 4. (Continued)

Site No.	Test type		Reference sources
	s_u[a]	$\bar{\phi}$[b]	
UU21	UC	SPT-N	Reference not available for publication. Summary is given as Case No. 25 (1983). "Transmission line structure foundation for uplift-compression loading: load test summaries", Report EL-3160, EPRI, Palo Alto, 731 p.
UC1	UU, UC	SPT-N	Diagnostic Engineering Consultants, Ltd. (2002). "Report on compression load test of bored piles for Taipei Area", Taipei, Taiwan.
UC2	UU	SPT-N	Taiwan Keelung District Court (1995). "Report of compression load test of bored piles for new court building", Keelung, Taiwan.
UC3	UU	SPT-N	Diagnostic Engineering Consultants, Ltd. (1997). "Report on compression load test of bored piles for Yungho Sanshing new construction", Taipei, Taiwan.
UC4	UU	SPT-N	Diagnostic Engineering Consultants, Ltd. (2000). "Report on compression load test of bored piles for Taipei Ta-Chi Bridge", Taipei, Taiwan.
UC7	UC	SPT-N	Tzuchi Culture Center (2002). "Report on compression load test of bored piles for building of Tzuchi Culture Center", Taipei, Taiwan.
UC10	DSS, VST	TC	Watt, WG, Kurfurst, PJ and Zeman, ZP (1969). "Comparison of pile load test – skin friction values and laboratory strength tests", Canadian Geotechnical J., 6(3), 339-352.
UC11	UC	TC	Jelinek, R, Koreck, HW and Stocker, M (1977). "Load test on 5 large diameter bored piles in clay", Proc., 9th Intl. Conf. on Soil Mechanics and Foundation Eng. (1), Tokyo, 571-576.
UC12	UU,UC	TC	O'Neill, MW and Reese, LC (1970). "Behavior of axially loaded drilled shafts in Beaumont clay", Research Report 89-8, Univ. of Texas, Austin, 775. O'Neill, MW and Reese, LC (1972). "Behavior of bored piles in Beaumont clay", J. Soil Mechanics and Foundations Div., ASCE, 98(SM2), 195-213.
UC13	UC	TC	DuBose, LA (1955), "Load studies on drilled shaft", Proc., Highway Research Board (34), 152-162.
UC14	CIUC UU, UC	TC	Holtz, RD and Baker, CN (1972). "Some load transfer data on caissons in hard Chicago clay", Proc., ASCE Conf. on Performance of Earth and Earth Supported Structures (1), Purdue Univ., Lafayette, 1223-1242.
UC15	UU, UC	TC	Bhanot, KL (1968). "Behavior of scaled and full-length cast-in-place concrete piles", Ph.D. Thesis, Univ. of Alberta, Edmonton, 336.

[a]UC = unconfined compression test; CIUC = consolidated-isotropically undrained triaxial compression test; DSS = direct simple shear test; UU = unconsolidated-undrained triaxial compression test; PSC = plane strain compression test; VST = vane shear test. [b]TC = triaxial compression test; CPT = cone penetration test; SPT-N = standard penetration test.

Table 5. Range of geometry and test number of drilled shafts for analysis.

Soil type[a]	No. tests		Shaft geometry (m)		D/B	Interpreted Q_s[b] (kN)
			Depth, D	Dia., B		
D	74	Range	1.4-62.0	0.14-2.0	2.5-70.5	21-24400
		Mean	11.5	0.73	16.6	2061
		SD	11.3	0.36	13.4	3254
		COV	0.99	0.49	0.80	1.58
U	148	Range	1.6-77.0	0.18-1.8	1.6-64.2	41-21503
		Mean	14.9	0.82	17.3	2702
		SD	16.6	0.38	13.4	3983
		COV	1.11	0.46	0.78	1.47

[a]D = drained; U = undrained. [b]Q_s was interpreted from L_1-L_2 method.

III. EVALUATION OF SIDE RESISTANCE

1. α Method

1) α-s_u Correlations

The value of α can be back-calculated from the field load test results using Eq. (1) and is simplified as follows:

$$\alpha = Q_s(L_2) / [\pi B D s_u] \quad (4)$$

where $Q_s(L_2)$ = interpreted side resistance using the L_2 method, s_u = mean undrained shear strength over shaft depth (D), and B = shaft diameter. To standardize the α-s_u relationship, a unique test type of undrained shear strength from consolidated-isotropically undrained triaxial compression (CIUC)

Table 6. α-s_u correlations for different test and loading types.

s_u test type	Loading type	Regression equation	Statistics		
			n	r^2	SD
CIUC	Compression	$\alpha_{CIUC} = 0.29 + 0.19/[s_u(CIUC)/p_a]$	104	0.61	0.08
	Uplift	$\alpha_{CIUC} = 0.32 + 0.16/[s_u(CIUC)/p_a]$	44	0.65	0.12
	All data	$\alpha_{CIUC} = 0.30 + 0.17/[s_u(CIUC)/p_a]$	148	0.66	0.09
UU	Compression	$\alpha_{UU} = 0.32 + 0.17/[s_u(UU)/p_a]$	104	0.43	0.14
	Uplift	$\alpha_{UU} = 0.30 + 0.17/[s_u(UU)/p_a]$	44	0.69	0.15
	All data	$\alpha_{UU} = 0.32 + 0.16/[s_u(UU)/p_a]$	148	0.56	0.14
UC	Compression	$\alpha_{UC} = 0.34 + 0.17/[s_u(UC)/p_a]$	104	0.42	0.15
	Uplift	$\alpha_{UC} = 0.30 + 0.17/[s_u(UC)/p_a]$	44	0.57	0.16
	All data	$\alpha_{UC} = 0.34 + 0.16/[s_u(UC)/p_a]$	148	0.56	0.15

test [denoted s_u(CIUC)] is selected as the appropriate reference test, because it is quite common and of good quality test. To obtain s_u(CIUC), s_u values from all other test types are converted to "equivalent" s_u(CIUC) values. The procedures to convert are based on the conclusions from a research study done by Chen and Kulhawy [3] for unconsolidated-undrained triaxial (UU) and unconfined compression (UC) tests. They developed the strength interrelationships using theoretical base and laboratory test data from all over the world, so these correlations can be applied to general cohesive soils. The representative equations are as follows:

$$s_u(UU)/s_u(CIUC) = 0.911 + 0.499 \log [s_u(UU)/\bar{\sigma}_{vo}] \quad (5)$$

$$s_u(UC)/s_u(CIUC) = 0.893 + 0.513 \log [s_u(UC)/\bar{\sigma}_{vo}] \quad (6)$$

For consolidated-anisotropically undrained triaxial, plane strain, and direct simple shear tests, Kulhawy and Mayne [8] established the equation as follows:

$$s_u(CT)/\bar{\sigma}_{vo} = a_{TEST}\, a_{RATE}\, a_{OCR}\, s_u(CIUC)/\bar{\sigma}_{vo} \quad (7)$$

in which $\bar{\sigma}_{vo}$ = effective vertical overburden stress, $s_u(CT) = s_u$ for converted test type, and a = coefficients of correction factors for test type, strain rate during testing, and OCR.

Using the above analytical procedures, Fig. 3 illustrates the updated undrained shear strength correlation producing a regression equation as follows:

$$\alpha_{CIUC} = 0.30 + 0.17/[s_u(CIUC)/p_a]$$

$$(n = 148,\ r^2 = 0.66,\ \text{and SD} = 0.09) \quad (8)$$

where s_u is normalized by one atmospheric stress (p_a= 101.3 kN/m^2). It is apparent that the data distribution of updated α-s_u correlation is superior to the previous result in Fig. 1.

Although the α_{CIUC}-s_u(CIUC) equation in Fig. 3 is developed using regression analysis, it can be clearly observed that the α value in the regression line is conservative for $s_u(CIUC)/p_a < 1$, since most data points are seen above the regression line. In addition, Fig. 3 also shows that the α value presents a considerably steep slope in small value of $s_u(CIUC)/p_a$. Therefore, the use of α_{CIUC}-s_u(CIUC) correlation for design may tend to be conservative and sensitive in the area of smaller s_u.

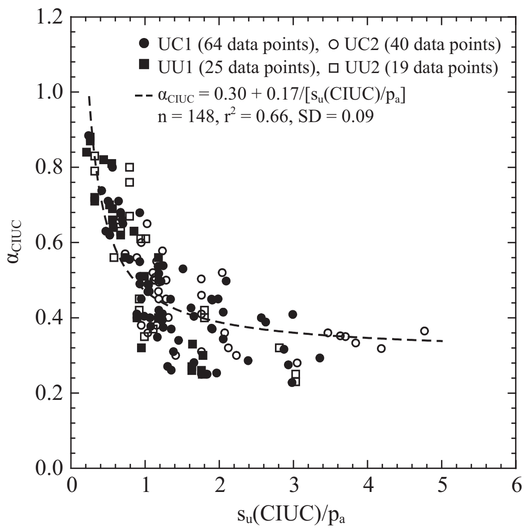

Fig. 3. α_{CIUC}-$s_u(CIUC)/p_a$ correlation.

Table 6 lists the comparisons of different loadings and test types of undrained shear strength. Comparing uplift and compression, the results show that the values are quite similar. Although a small difference exists, it is not significant enough to warrant differentiation. For convenience, both UU test, denoted α_{UU} versus s_u(UU), and UC test, denoted α_{UC} versus s_u(UC), are also shown in Table 6. As can be seen, α_{UU} and α_{UC} are somehow greater than α_{CIUC} because the test result from s_u(CIUC) is typically greater than s_u(UU) or s_u(UC) for general clays [3]. Furthermore, the α values from the UU or UC test present larger standard deviations (SD) and are less reliable than those from the CIUC test. This observation is consistent with the common supposition since the quality of the CIUC test is generally better than UU and UC tests.

Table 7. Summary of statistical data for Fig. 4.

USR	Statistics		
	n	r^2	SD
< 1.0	41	0.69	0.09
1.0-2.0	39	0.56	0.09
2.0-3.0	39	0.71	0.08
3.0-4.0	13	0.63	0.08
4.0-5.0	14	0.30	0.09
> 5.0	18	0.29	0.13

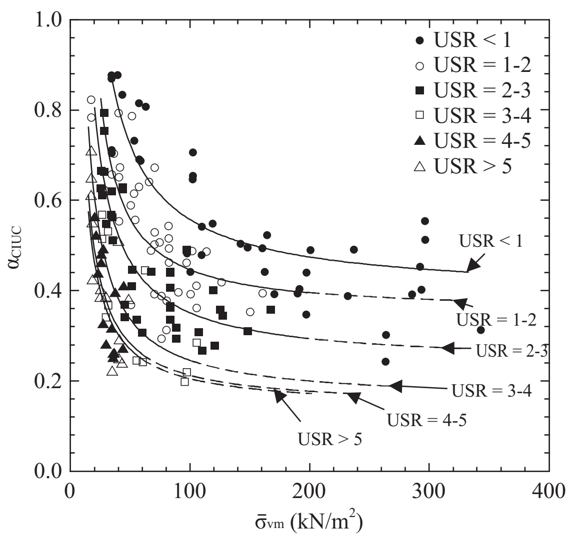

Fig. 4. α_{CIUC}-s_u(CIUC)/ $\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$ correlations.

2) α-$s_u/\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$ Correlations

Fig. 4 shows the results of undrained strength ratio correlation, α_{CIUC}-s_u(CIUC)/ $\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$. Meanwhile, Table 7 lists the statistical data for all individual curves in Fig. 4. These correlations are developed directly from the field load test database. Several points can be observed from Fig. 4 and Table 7. First, the coefficients of determination r^2 are large when USRs are small. The values of r^2 become smaller when USR is greater than 4, this may be due to the limited data distribution in the α_{CIUC}-s_u(CIUC)/ $\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$ correlation. Second, the difference of regression lines is minimal when USR is greater than 3. Furthermore, the regression lines are very close when USR is greater than 4. Third, the trend of α_{CIUC} decreases with increasing s_u(CIUC)/ $\bar{\sigma}_{vm}$ and $\bar{\sigma}_{vm}$. In addition, it can be noted that these data points present more consistent results for small $\bar{\sigma}_{vm}$. On the other hand, the data points are scattered or lacking when the value of $\bar{\sigma}_{vm}$ increases. By visual observation, some regression lines are also somehow conservative for small $\bar{\sigma}_{vm}$. The phenomenon is comparable with the result in Fig. 3.

Table 8. Statistical results of back-calculated K/K_o.

Mode[a]	Construction method	n	Mean	SD	COV
D	slurry	12	0.73	0.24	0.34
	casing	10	0.97	0.12	0.12
	dry	52	1.03	0.36	0.35
U	slurry	16	0.79	0.28	0.35
	casing	25	0.88	0.31	0.35
	dry	63	1.12	0.29	0.26

[a]D = drained; U = undrained.

The α_{CIUC}-s_u(CIUC)/ $\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$ correlations can be regarded as an alternative analysis for traditional α-s_u correlations, especially in the condition of having a small s_u. The α value on the α_{CIUC}-s_u(CIUC)/ $\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$ correlations is more precisely distinguished than using the conventional α_{CIUC}-s_u(CIUC) correlations. Therefore, the required α value can be more reasonably selected. It can be seen from the distribution of data in Fig. 4 and Table 7, that the suggested ranges of alternative values are (1) $\bar{\sigma}_{vm}$ < 200 kN/m^2 for USR < 3 and (2) $\bar{\sigma}_{vm}$ < 100 kN/m^2 for USR > 3, because the α_{CIUC}-s_u(CIUC)/ $\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$ correlations in these ranges have shown better statistical results.

2. β Method

1) Drained Load Tests

The approximate β (β_p) can be predicted from Eq. (2) and given as:

$$\beta_p = K_o\,(K/K_o)\,\tan(\bar{\phi}\ \cdot\ \delta/\bar{\phi}) \tag{9}$$

Meanwhile, the measured β (β_m) is also back-calculated from the field load test results using Eq. (2), as follows:

$$\beta_m = Q_s(L_2)\ /\ [\pi\ B\ D\ \bar{\sigma}_{vm}\ ] \tag{10}$$

where K = the operative horizontal stress coefficient, which depends on the original in-situ coefficient of horizontal soil stress (K_o), and the other terms have been defined previously. With assumptions of $\beta_m = \beta_p$ and $\delta/\bar{\phi} = 1$ for drilled shafts, the stress factor K/K_o for overall foundation depth can be back-calculated from Eqs. (9) and (10). Table 8 shows the statistical results of K/K_o for the different construction methods. For drained tests, the mean values of K/K_o are 0.73, 0.97, and 1.03 for slurry, casing, and dry construction, respectively. The results are obviously larger than in the previous study by Kulhawy *et al.* [9], which showed K/K_o = 2/3 for slurry, 1 for dry construction, and intermediate values between these ranges for casing construction under water.

Fig. 5 shows the comparison of measured and predicted drained β based on the suggested K/K_o values of this study and

Table 9. Comparison of β analysis using various K/K_o suggestions.

Reseachers[a]	Mode[b]	Regression analysis	Mean analysis
Kulhawy *et al.*	D	$\beta_m = 1.13\ \beta_p$ (n = 71, r^2 = 0.95, SD = 0.20)	Mean β_m/β_p = 1.13 (n = 74, SD = 0.26, COV = 0.23)
This study	D	$\beta_m = 1.01\ \beta_p$ (n = 71, r^2 = 0.95, SD = 0.21)	Mean β_m/β_p = 1.03 (n = 74, SD = 0.24, COV = 0 .24)
Kulhawy *et al.*	U	$\beta_m = 1.19\ \beta_p$ (n = 104, r^2 = 0.96, SD = 0.25)	Mean β_m/β_p = 1.13 (n = 104, SD = 0.27, COV = 0.24)
This study	U	$\beta_m = 1.00\ \beta_p$ (n = 104, r^2 = 0.95, SD = 0.29)	Mean β_m/β_p = 1.01 (n = 104, SD = 0.27, COV = 0.26)

[a]Kulhawy *et al*. [9].
[b]D = drained; U = undrained.

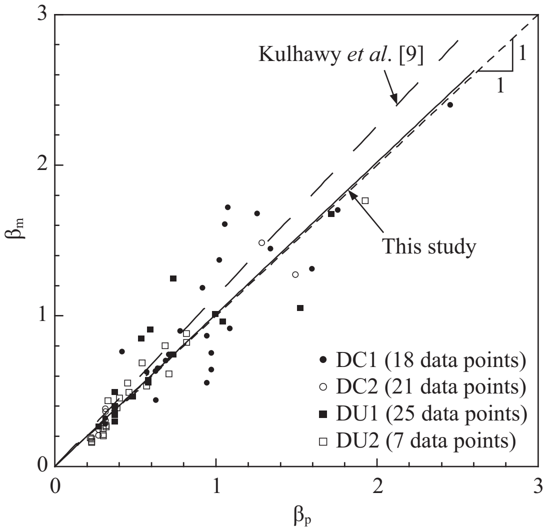

Fig. 5. Comparison of measured and predicted drained β.

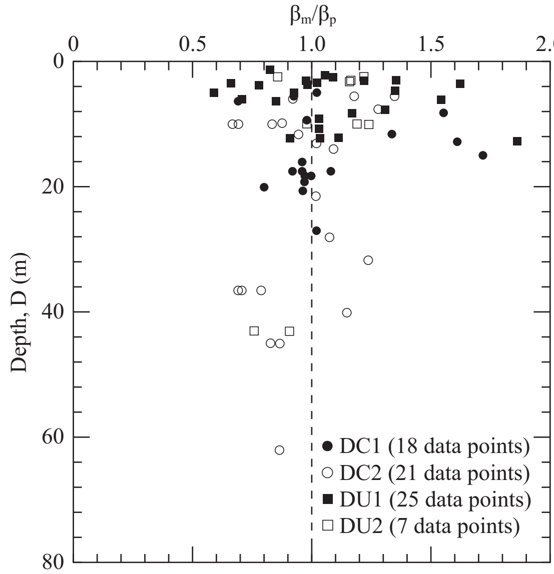

Fig. 6. Drained β_m/β_p versus depth.

previous research works. Table 9 summarizes the statistical data for both regression and mean analyses. It can be seen in Fig. 5 and Table 9, that the overall measured and predicted results of this study are more consistent than the previous study. From the result of mean β_m/β_p = 1.13, it is clear that the suggested K/K_o values by Kulhawy *et al*. [9] are underestimated. In this study, the predicted and measured drained β values are much closer. Therefore, the updated analysis for K/K_o is suggested because it has shown superiority over the previous study.

The relation of drained β_m/β_p versus depth is presented in Fig. 6. It can be seen that there is a wide range of β_m/β_p from 0.5 to 1.9 at shallow depths (D < 20 m); however, the data distribution becomes narrow (0.7-1.2) when shaft length is greater than 20 m. On average, for shaft length < 20 m, mean β_m/β_p = 1.06 with n = 59, SD = 0.25, and COV = 0.24, while for shafts longer than 20 m, mean β_m/β_p = 0.91 with n = 15, SD = 0.16, and COV = 0.18. Therefore, it can be observed that the predicted drained β method may be more reliable for shafts greater than 20 m in length based on these available load test case histories.

2) Undrained Load Tests

A similar evaluation is done for undrained β analysis. Table 8 also shows the statistical results of back-calculated K/K_o for the different construction methods of the undrained tests. The mean values of K/K_o are 0.79, 0.88, and 1.12 for slurry, casing, and dry construction, respectively. These results are larger than those in the previous study [9].

Table 9 also summarizes the statistics using different suggested values of K/K_o for the undrained tests. It can be seen that the measured and predicted results from this study present more consistent results as well. The reasons for the improvement in the prediction are the same as in the drained tests. Figs. 7 and 8 present the results of the comparison of the measured and predicted β and the relation of undrained β_m/β_p versus depth, respectively. On average, the mean undrained β_m/β_p is 1.01 and $\beta_m = 1.00\ \beta_p$ for the regression analysis. The average predicted undrained β is consistent with the measured undrained β.

The relation of undrained β_m/β_p versus depth also presents a wide range of values (0.6 -1.8) at shallow depths (D < 20 m), but the data distribution becomes narrow (0.7-1.3) when shaft length is greater than 20 m. On average, for short shafts, mean β_m/β_p = 1.02 with n = 78, SD = 0.29, and COV = 0.29, while for long shafts, mean β_m/β_p = 0.98 with n = 26, SD = 0.17, and COV = 0.17. Therefore, as in drained tests, the predicted

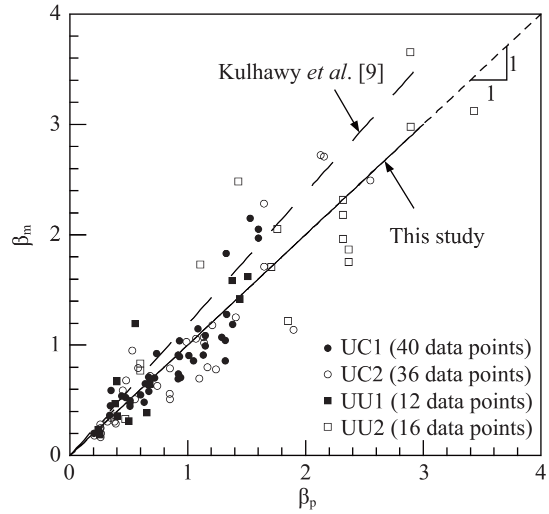

Fig. 7. Comparison of measured and predicted undrained β.

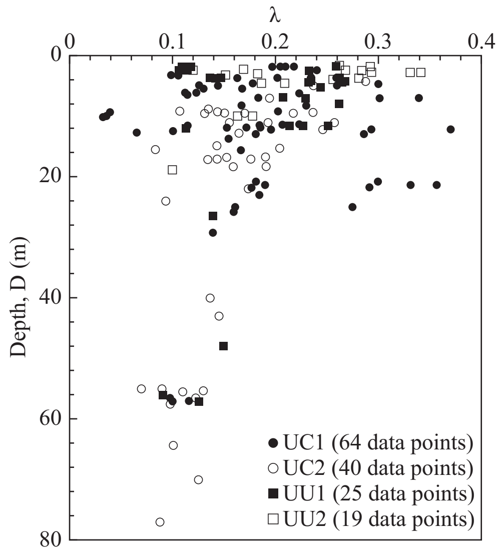

Fig. 9. λ versus depth.

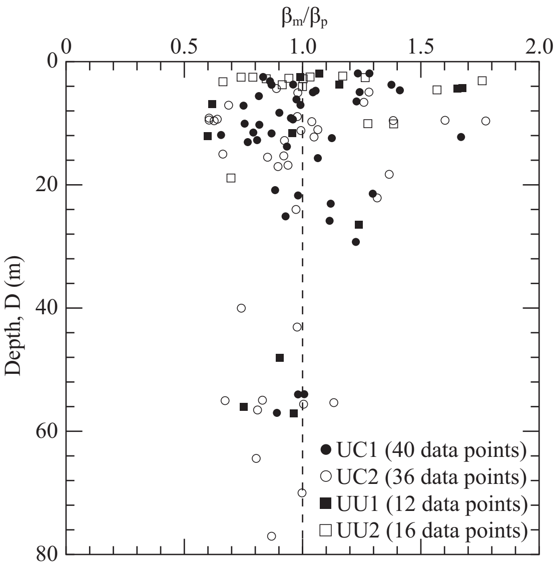

Fig. 8. Undrained β_m/β_p versus depth.

undrained β method is also somehow more reliable for shafts greater than 20 m in length based on these available load test case histories.

3. λ Method

The measured λ can be computed from Eq. (3) and given as:

$$\lambda = Q_s(L_2) / [\pi B D (\bar{\sigma}_{vm} + 2 s_u)] \quad (11)$$

in which all terms have been defined previously. Fig. 9 shows the results of λ versus shaft depth. It can be seen that the range of data points is very wide (0.05-0.35) for short shafts (D < 30 m) with mean λ = 0.19, n = 131, SD = 0.07, and COV = 0.36. For long shafts, (D > 30 m), the statistical results are mean λ = 0.11, n = 17, SD = 0.02, and COV = 0.20. Based on the available data points, the λ value seems convergent to a consistent value when the shaft depth is greater than 30 m. However, more load tests data are needed to examine whether this phenomenon is a significant issue or just a database issue. According to these statistics, it is obvious that the COV of the λ method is larger than that of the β method. Therefore, the prediction of side resistance of the λ method is generally less reliable than α and β methods.

IV. DESIGN RECOMMENDATIONS

For undrained loading, both α and β methods present more reasonable results based on the data analyses. The traditional α-s_u correlations in Table 6 are proposed for the prediction of undrained side resistance. The new α_{CIUC}-s_u(CIUC)/$\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$ correlations can be used as an alternative analytical method, especially in the case of smaller s_u. For β analysis, the suggested stress factors K/K_o are 0.79, 0.88, and 1.12 for slurry, casing, and dry construction, respectively, but the predicted β method presents somehow to be more variable for short shafts. However, α method has a more favorable data distribution along the regression line. The prediction of side resistance of the λ method is generally less reliable than α and β methods.

For drained loading, the analysis of the β method is reasonable and the suggested values of K/K_o are 0.73, 0.97, and 1.03 for slurry, casing, and dry construction, respectively. However, the predicted β method presents somehow to be less reliable for short shafts.

V. SUMMARY AND CONCLUSIONS

A wide variety of load test data were used for the evaluation of side resistance of drilled shafts under axial loading. Representative analytical models were examined in detail using both measured data and predicted results. Based on the evaluation, the following design recommendations for engineering practice are proposed.

(1) For undrained loading, the total stress analysis method, α_{CIUC}-s_u(CIUC) correlation, presents more reliable results.
(2) The new correlation, α_{CIUC}-s_u(CIUC)/$\bar{\sigma}_{vm}$ - $\bar{\sigma}_{vm}$, which is developed using field load test data, can be regarded as an alternative analysis method for the drilled shaft design, especially in the case of smaller s_u.
(3) For drained loading, the effective stress β analysis is a suitable method. Based on the analyses of the available database, it is reasonably consistent for long shafts, but has a wide range of results for short shafts. A similar situation is found for undrained loading.
(4) The suggested stress factors K/K_o can substantially improve the prediction model of β analysis. For drained loading, the suggested values of K/K_o are 0.73, 0.97, and 1.03 for slurry, casing, and dry construction, respectively; for undrained loading, the suggested values of K/K_o are 0.79, 0.88, and 1.12 for slurry, casing, and dry construction, respectively.
(5) The range of the λ method is wide and COV is large for the analysis of side resistance. Thus, the λ method for cohesive soils presents less reliable results.

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