Evaluation of side resistance of driven precast concrete piles

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Abstract. This study critically evaluates the axial side resistance of driven precast concrete (PC) piles. A wide range of load test data are classified into drained and undrained databases and subsequently used in the investigation. Each database is further divided into: (1) compression and uplift loading and (2) round and square cross sections. Measured and predicted results are both applied to examine the representative analytical models, including the alpha (α), beta (β), and lambda (λ) methods. The statistical results show that the range of values of the empirical coefficient α is wide. The predicted results of the β method are underestimated in drained loading, but they are reasonable in undrained loading. The relationship between λ and pile depth is also developed. Based on the analyses, the relative merits of the three analytical models are established, and designs for analyzing the side resistance of driven PC piles are recommended.

1. Introduction
Driven precast concrete (PC) piles are widely used as deep foundations for high rise buildings, towers, highway structures and others. Side resistance is an important source of driven PC pile capacity under axial loading, especially when the pile depth is considerably large or for piles under the condition of uplift loading in which the tip resistance is negligibly small. Research on side resistance of deep foundations has been progressing over the years. O’Neill [1] summarized results of significant recent research on a few aspects of side resistance for driven and drilled shafts while Lutenegger [2] focused on the importance of Standard Penetration Test for estimating driven pile side resistance. Another research [3, 4] provided an extensive evaluation of side resistance for drilled shafts using representative analytical models.

Analytical methods can be specified into alpha (α), beta (β), or lambda (λ) methods. Table 1 lists the analysis models and the related variables for each model. The α method [5] is a conventional total stress analysis for driven piles in cohesive soils. The side resistance is related to the average soil undrained shear strength (su) by an empirical coefficient denoted as α, which is the adhesion factor. The original α [5] for concrete piles was based on empirical correlations of mean su over the foundation depth while Stas and Kulhawy [6] developed α-su correlation for drilled shafts shown in Figure 1. In addition, some researchers [3, 4] develop the α-su correlation for a standardized undrained shear strength value from consolidated-isotropically undrained triaxial compression (CIUC) for drilled shafts. Furthermore, several researchers [7-9] also demonstrated that α is complexly related to other soil parameters such as the mean effective overburden stress ($\bar{\sigma}_{om}$), overconsolidation ratio (OCR), and effective stress friction angle ($\phi_e$).
Figure 1. α - su Correlation [6]

Table 1. Analytical Models for Side Resistance Analysis

| Method | Analytical model$^a$ | Definition of factors |
|--------|----------------------|-----------------------|
| α      | $Q_α(α) = p\sum_{n=1}^{N} α_n s_n t_n$ | $α =$ empirical adhesion factor |
|        |                      | $s_n =$ undrained shear strength |
|        |                      | $K =$ coefficient of horizontal soil stress |
|        |                      | $K_o =$ in-situ K |
| β      | $Q_β(β) = p\left( \frac{K}{K_o} \sum_{n=1}^{N} σ_{vn} K_n \tan \left[ \phi_n - δ \right] \right) t_n$ | $σ_v =$ vertical effective stress |
|        |                      | $ϕ =$ effective stress friction angle |
|        |                      | $δ =$ soil-shaft interface friction angle |
|        |                      | $β = K \tan δ$ |
| λ      | $Q_λ(λ) = λ p(σ_{vm} + 2s_u) D$ | $λ =$ empirical factor |

The β method [10] is an effective stress analysis which considers frictional resistance of the soil-shaft interface. In this method, side resistance is a function of the horizontal effective stress ($σ_h$), effective stress friction angle ($δ$) for the soil-shaft interface, and pile geometry. Previous study [11] examined available load test data and presented that the stress factor ($K/K_o$) is dependent on the construction method and its influence on in-situ stress. They suggested that for small displacement piles, $K/K_o$ is in the range of 0.75 to 1.25, whereas for large displacement piles, $K/K_o$ is in the range of 1.0 to 2.0. They also suggested that the ratio of interface friction angle ($δ$) to soil friction angle ($ϕ$) is in the range of 0.8 to 1.0 for precast piles.

Lastly, the λ method [12] is a combination of total and effective stress analyses that can be used for cohesive soils. In this method, side resistance is related to $s_u$ and $σ_{vm}$ by an empirical factor $λ$. The original $λ$ was developed based on a database of driven pipe pile data and is a function of the total depth of the pile.

Although numerous methods have been critically evaluated for side resistance analysis of driven piles, more consistent approaches have been developed for assessing soil design parameters [13] that warrant a complete re-assessment of side resistance behavior of driven PC piles. In addition, more updated load test data exist nowadays that can be utilized for verifying estimated results. Furthermore, the effects of shape and installation methods on side resistance are of equal importance to examine. A broad database is utilized to assess the relative merits and suitability of each analytical model. Results
are compared statistically and graphically. Subsequently, specific design recommendations are given for the use of these models to driven PC pile side resistance design.

2. Database of load tests

The load test data were collected from geotechnical literature and load test reports. The database developed in this study consists of 234 field axial load test conducted in 99 sites. Among these data, 72 sites with 154 load tests are loaded in compression while 27 sites with 80 load tests are loaded in uplift. The case histories cover a range of soil profiles and pile shapes. These load tests were conducted around the world at different points in time. The soil profile is categorized herein as drained or undrained, based on the predominant soil condition along the pile depth. The piles are round or square in cross-section. The load tests are divided into two groups, based on the loading condition, while each group is further subdivided using the two profile types and two cross-sections, with a total of eight categories.

For group 1 in compression: (1) drained compression with round piles (DCR) has 10 sites with 37 tests; (2) drained compression with square piles (DCS) has 23 sites with 44 tests; (3) undrained compression with round piles (UCR) has 18 sites with 37 tests; and (4) undrained compression with square piles (UCS) has 21 sites with 36 tests. For group 2 in uplift: (5) drained uplift with round piles (URU) has 7 sites with 27 tests; (6) drained uplift with square piles (DUS) has 10 sites with 31 tests; (7) undrained uplift with round piles (URR) has 3 sites with 11 tests; and (8) undrained uplift with square piles (UUS) has 7 sites with 11 tests. All of the load tests have almost complete geological data and load-displacement curves, and all were conducted on straight-sided, driven PC piles. Hence, these field data should reflect common field situations and can be representative for subsequent applications in engineering practice.

The basic information for the DCR, DCS, UCR, and UCS tests are given in Tables 2 to 5, respectively, whereas the information for the DUR, DUS, UUR, and UUS tests are given in Tables 6 to 9, respectively. The values of $\alpha$, $\beta_m$, and $\lambda$ were back-calculated using Eqs. 1 to 3 and the field load test results. The $L_1-L_2$ method [14, 15] which is a graphical construction method was adopted to measure the axial compression capacity. The method presents reasonable results for pile design based on previous studies [16-18].

**Table 2. Basic Information and Side Resistance Analysis Result for Drained Compression Round Section (DCR) Tests**

| Site & Pile no. | Test site/Soil description along pile depth | Depth, $D$ (m) | Dia., $D$ (m) | $K_a$ | $\beta_p$ | $Q_{cap}$ (kN) | $Q_{cap}$ (kN) | $\beta_m$ | $\beta_p$ | $(K/K_m)_f$ |
|---------------|------------------------------------------|----------------|--------------|------|---------|----------------|----------------|---------|---------|-------------|
| DCR_1 | Arkansas; fine & silty sand | 13.7 | 0.41 | 0.431 | 0.261 | 356 | 1292 | 3.63 | 0.947 | 3.63 | 3.17 |
| DCR 2-1 | Dramen, Norway; medium to coarse sand | 8.0 | 0.28 | 0.670 | 0.304 | 86 | 203 | 2.36 | 0.718 | 2.36 | 2.09 |
| DCR 2-2 | 16.0 | 0.28 | 0.544 | 0.247 | 257 | 347 | 1.35 | 0.333 | 1.35 | 1.20 |
| DCR 2-3 | 7.5 | 0.28 | 0.677 | 0.308 | 78 | 155 | 1.98 | 0.610 | 1.98 | 1.76 |
| DCR 2-4 | 11.5 | 0.28 | 0.583 | 0.265 | 160 | 250 | 1.57 | 0.415 | 1.57 | 1.39 |
| DCR 2-5 | 15.5 | 0.28 | 0.544 | 0.247 | 242 | 355 | 1.47 | 0.362 | 1.47 | 1.30 |
| DCR 2-6 | 19.5 | 0.28 | 0.513 | 0.253 | 380 | 516 | 1.36 | 0.344 | 1.36 | 1.20 |
| DCR 2-7 | 23.5 | 0.28 | 0.472 | 0.258 | 551 | 734 | 1.33 | 0.344 | 1.33 | 1.17 |
| DCR 3 | Spain; fine silty sand | 18.0 | 0.91 | 0.420 | 0.261 | 1431 | 2260 | 1.58 | 0.413 | 1.58 | 1.38 |
| DCR 4-1 | 38.0 | 0.60 | 0.409 | 0.262 | 3007 | 5665 | 1.88 | 0.493 | 1.88 | 1.65 |
| DCR 4-2 | 27.0 | 0.50 | 0.454 | 0.260 | 1316 | 2504 | 1.90 | 0.494 | 1.90 | 1.67 |
| DCR 4-3 | 27.0 | 0.60 | 0.454 | 0.260 | 1579 | 4175 | 2.64 | 0.687 | 2.64 | 2.32 |
| DCR 4-4 | 27.0 | 0.60 | 0.454 | 0.260 | 1579 | 4559 | 2.89 | 0.750 | 2.89 | 2.54 |
| DCR 4-5 | 25.0 | 0.60 | 0.454 | 0.260 | 1379 | 3934 | 2.87 | 0.746 | 2.87 | 2.52 |
| DCR 4-6 | 25.0 | 0.60 | 0.454 | 0.260 | 1370 | 4225 | 3.08 | 0.801 | 3.08 | 2.71 |
| DCR 4-7 | 22.0 | 0.50 | 0.454 | 0.260 | 904 | 1926 | 2.13 | 0.554 | 2.13 | 1.87 |
| DCR 4-8 | 30.0 | 0.50 | 0.454 | 0.260 | 1599 | 3864 | 2.42 | 0.628 | 2.42 | 2.12 |
| DCR 5-1 | Kaohsiung, Taiwan; silty sand | 16.0 | 0.40 | 0.517 | 0.285 | 360 | 364 | 1.01 | 0.255 | 0.89 | 0.89 |
| DCR 5-2 | Taiwan; clayey silty sand | 20.0 | 0.40 | 0.517 | 0.252 | 548 | 525 | 0.96 | 0.242 | 0.96 | 0.85 |
| DCR 5-3 | 30.0 | 0.60 | 0.503 | 0.254 | 1778 | 1749 | 0.98 | 0.250 | 0.98 | 0.87 |
| DCR 5-4 | 27.0 | 0.60 | 0.503 | 0.254 | 1461 | 1387 | 0.95 | 0.242 | 0.95 | 0.84 |
| DCR 5-5 | 22.0 | 0.30 | 0.503 | 0.254 | 497 | 1726 | 3.48 | 0.884 | 3.48 | 3.07 |
| DCR 5-6 | 22.0 | 0.40 | 0.503 | 0.254 | 662 | 1836 | 2.77 | 0.705 | 2.77 | 2.45 |
| DCR 5-7 | 22.0 | 0.50 | 0.503 | 0.254 | 828 | 2588 | 3.13 | 0.795 | 3.13 | 2.76 |
| DCR 5-8 | 34.0 | 0.60 | 0.503 | 0.254 | 2245 | 4320 | 1.92 | 0.489 | 1.92 | 1.70 |
| DCR 5-9 | 34.0 | 0.60 | 0.503 | 0.254 | 2245 | 3309 | 1.47 | 0.375 | 1.47 | 1.30 |
| DCR 6-1 | Changbin, Taiwan; silty sand | 25.0 | 0.60 | 0.431 | 0.261 | 1322 | 4007 | 3.03 | 0.792 | 3.03 | 2.66 |
| DCR 6-2 | 23.0 | 0.50 | 0.431 | 0.261 | 943 | 3130 | 3.32 | 0.867 | 3.32 | 2.91 |
| DCR 6-3 | 23.0 | 0.50 | 0.431 | 0.261 | 943 | 2983 | 3.16 | 0.826 | 3.16 | 2.77 |

*K_s* = in-situ K; *β_p* = predicted beta value; *Q_{esp} = predicted compression side resistance; *Q_{scm} = measured compression side resistance; *β_m* = measured beta value; *(K/K_{in}) = back-calculated stress factor

**Table 3. Basic Information and Side Resistance Analysis Result for Drained Compression Square Section (DCS) Tests**

| Site & Pile no. | Test site/Soil description along pile depth | Depth, D (m) | Dia., B (m) | K_s | β_p | Q_{esp} (kN) | Q_{scm} (kN) | Q_{scm}/Q_{esp} | β_m | β_m/β_p | (K/K_{in}) |
|---------------|------------------------------------------|--------------|-------------|-----|-----|-------------|-------------|---------------|-----|---------|------------|
| DCS 1 | Florida; sand | 18.8 | 0.762 | 0.389 | 0.261 | 1414 | 4213 | 2.92 | 0.762 | 2.92 | 2.54 |
| DCS 2 | Sweden; sand | 12.8 | 0.235 | 0.727 | 0.398 | 300 | 328 | 1.09 | 0.434 | 1.09 | 0.96 |
| DCS 3 | Portugal; clayey sand | 6.0 | 0.350 | 0.815 | 0.452 | 201 | 636 | 3.17 | 1.431 | 3.17 | 2.78 |
| DCS 4-1 | Kuwait; silty sand | 12.1 | 0.300 | 0.420 | 0.261 | 283 | 298 | 1.05 | 0.275 | 1.05 | 0.92 |
| DCS 4-2 | 9.3 | 0.300 | 0.409 | 0.262 | 176 | 181 | 1.03 | 0.269 | 1.03 | 0.90 |
| DCS 5 | Kuwait; sand | 8.7 | 0.300 | 0.389 | 0.261 | 154 | 393 | 2.55 | 0.667 | 2.55 | 2.22 |
| DCS 6 | USA; fine sand | 27.0 | 0.355 | 0.454 | 0.260 | 971 | 3119 | 3.21 | 0.834 | 3.21 | 2.82 |
| DCS 7 | Georgia; coarse sand | 15.2 | 0.406 | 0.459 | 0.301 | 680 | 2186 | 3.21 | 0.968 | 3.21 | 2.80 |
| DCS 8-1 | Tidewater, Virginia; silty sand | 21.0 | 0.360 | 0.420 | 0.261 | 917 | 1975 | 2.15 | 0.563 | 2.15 | 1.88 |
| DCS 8-2 | 21.0 | 0.360 | 0.454 | 0.260 | 646 | 1639 | 2.54 | 0.659 | 2.54 | 2.23 |
| DCS 8-3 | 26.0 | 0.360 | 0.472 | 0.270 | 828 | 3127 | 3.78 | 1.020 | 3.78 | 3.32 |
| DCS 9-1 | Iraq; sand | 11.0 | 0.285 | 0.568 | 0.325 | 389 | 803 | 2.06 | 0.671 | 2.06 | 1.81 |
| DCS 9-2 | 15.0 | 0.285 | 0.466 | 0.259 | 364 | 1229 | 3.37 | 0.874 | 3.37 | 2.97 |
| DCS 10 | China; sandy loam | 25.0 | 0.450 | 0.454 | 0.260 | 1558 | 2363 | 1.52 | 0.394 | 1.52 | 1.33 |
| DCS 11-1 | Beijing; silty sand | 10.0 | 0.300 | 0.437 | 0.261 | 490 | 1673 | 3.42 | 0.892 | 3.42 | 2.99 |
| DCS 11-2 | 12.0 | 0.300 | 0.437 | 0.261 | 383 | 1055 | 2.75 | 0.718 | 2.75 | 2.41 |
| DCS 12 | USA; sand and silt | 38.0 | 0.510 | 0.420 | 0.261 | 3053 | 7040 | 2.31 | 0.603 | 2.31 | 2.02 |
| DCS 13-1 | Ontario-site A; sand, silt, and clay with some gravel | 11.4 | 0.305 | 0.379 | 0.261 | 467 | 1313 | 2.81 | 0.734 | 2.81 | 2.44 |
| DCS 13-2 | 11.2 | 0.305 | 0.379 | 0.261 | 453 | 1463 | 3.23 | 0.843 | 3.23 | 2.81 |
| DCS 13-3 | 8.5 | 0.305 | 0.379 | 0.261 | 246 | 710 | 2.89 | 0.755 | 2.89 | 2.51 |
| DCS 13-4 | 8.4 | 0.305 | 0.379 | 0.261 | 245 | 710 | 2.90 | 0.756 | 2.90 | 2.52 |
| DCS 13-5 | 12.5 | 0.305 | 0.384 | 0.261 | 545 | 1463 | 2.68 | 0.700 | 2.68 | 2.33 |
| DCS 13-6 | 15.1 | 0.305 | 0.346 | 0.258 | 749 | 1314 | 1.75 | 0.453 | 1.75 | 1.52 |
| DCS 14-1 | Ontario-site B; silty sand | 34.8 | 0.305 | 0.431 | 0.261 | 1150 | 2435 | 2.12 | 0.553 | 2.12 | 1.85 |
| DCS 14-2 | 16.5 | 0.305 | 0.442 | 0.261 | 300 | 820 | 2.73 | 0.712 | 2.73 | 2.39 |
| DCS 15-1 | Atlantic Coastal | 13.0 | 0.254 | 0.518 | 0.270 | 168 | 422 | 2.52 | 0.680 | 2.52 | 2.22 |
| Pile no. | Site & description | Depth, D (m) | Dia, B (m) | OCR | $K_o$ | $\beta_p$ | $Q_{uap}$ (kN) | $Q_{um}$ (kN) | $Q_{um}/Q_{uap}$ | $S_C$ | $\alpha$ | $\lambda$ |
|---------|---------------------|--------------|-----------|-----|-------|--------|-------------|-------------|-----------------|------|-------|------|
| UCR 1   | Boston; clay        | 45.5         | 0.41      | 2   | 0.664 | 0.379 | 4914       | 4325        | 0.88            | 158  | 0.47  | 0.14 |
| UCR 2   | Boston; clay        | 41.8         | 0.31      | 2   | 0.721 | 0.353 | 2826       | 1921        | 0.68            | 85   | 0.56  | 0.13 |
| UCR 3   | Philippines; sand   | 57.0         | 0.41      | 1   | 0.470 | 0.258 | 3129       | 2039        | 0.65            | 88   | 0.41  | 0.09 |
| UCR 4-1 | Puerto Rico; silty clay with sand | 21.65 | 0.30 | 0.192 | 0.730 | 1782 | 1854 | 1.04 | 183 | 0.50 | 0.19 |
| UCR 4-2 |                      | 19.8         | 0.30      | 8   | 1.392 | 0.914 | 1893       | 1042        | 0.55            | 199  | 0.28  | 0.11 |
| UCR 4-3 |                      | 22.9         | 0.30      | 8   | 1.372 | 0.964 | 2617       | 2275        | 0.87            | 271  | 0.39  | 0.17 |
| UCR 5   | Brazil; silty clay  | 40.0         | 0.42      | 1   | 0.470 | 0.258 | 5352       | 3136        | 0.59            | 198  | 0.30  | 0.08 |
| UCR 6   | LA; sandy silty clay| 13.0         | 0.46      | 6   | 1.232 | 0.556 | 1165       | 1499        | 1.29            | 104  | 0.77  | 0.25 |
| UCR 6-2 |                      | 17.7         | 0.46      | 4   | 0.900 | 0.524 | 2121       | 2084        | 0.98            | 171  | 0.48  | 0.16 |
| UCR 7   | Texas; stiff clay   | 31.0         | 0.32      | 3   | 0.927 | 0.367 | 2341       | 2898        | 1.24            | 175  | 0.53  | 0.17 |
| UCR 8   | Brazil; clay silt   | 14.0         | 0.18      | 1   | 0.531 | 0.250 | 214        | 224         | 1.04            | 63   | 0.58  | 0.12 |
| UCR 9-1 | China; muck clay    | 38.2         | 0.40      | 1   | 0.500 | 0.255 | 2463       | 2550        | 1.04            | 95   | 0.56  | 0.14 |
| UCR 9-2 | China; sandy clay   | 24.8         | 0.40      | 5   | 1.103 | 0.606 | 2562       | 2613        | 1.02            | 119  | 0.70  | 0.22 |
| UCR 9-3 |                      | 19.6         | 0.30      | 7   | 1.318 | 0.725 | 1460       | 1500        | 1.03            | 113  | 0.72  | 0.24 |
| UCR 10  | Canada; silty clay  | 36.0         | 0.36      | 3   | 0.852 | 0.454 | 2510       | 3430        | 1.37            | 116  | 0.73  | 0.23 |
| UCR 11  | U.S.; clay          | 8.2          | 1.37      | 7   | 1.323 | 0.674 | 1753       | 2872        | 1.64            | 106  | 0.77  | 0.36 |
| UCR 12-1| Malaysia; marine clay| 35.5 | 0.25 | 0.531 | 0.250 | 865 | 545 | 0.65 | 38 | 0.81 | 0.07 |
| UCR 12-2|                      | 14.5         | 0.25      | 2   | 0.735 | 0.346 | 230        | 274         | 1.19            | 45   | 1.00  | 0.23 |
| UCR 12-3|                      | 23.5         | 0.25      | 1   | 0.531 | 0.250 | 404        | 441         | 1.09            | 59   | 0.63  | 0.13 |
| UCR 12-4|                      | 11.5         | 0.25      | 2   | 0.735 | 0.346 | 177        | 179         | 1.01            | 40   | 0.99  | 0.20 |
| UCR 13-1| Malaysia; silty clay| 14.5         | 0.30      | 6   | 1.215 | 0.668 | 1083       | 1310        | 1.21            | 114  | 0.84  | 0.28 |
| UCR 13-2| Malaysia; sandy clay| 14.2         | 0.35      | 5   | 1.103 | 0.606 | 1077       | 1589        | 1.48            | 105  | 0.41  | 0.35 |
| UCR 13-3| Malaysia; sandy clay| 18.7         | 0.25      | 3   | 0.828 | 0.472 | 1024       | 874         | 0.85            | 145  | 0.41  | 0.14 |
| UCR 14  | Malaysia; clayey silt| 28.5         | 0.40      | 1   | 0.459 | 0.259 | 2531       | 2163        | 0.85            | 159  | 0.38  | 0.10 |
| UCR 15-1| Malaysia; soft marine clay | 43.0  | 1.00 | 0.679 | 0.373 | 7405 | 6850 | 0.93 | 150 | 0.34 | 0.11 |
| UCR 15-2|                      | 57.5         | 1.00      | 2   | 0.604 | 0.397 | 11689      | 7909        | 0.68            | 150  | 0.29  | 0.09 |
| UCR 15-3|                      | 33.8         | 1.00      | 3   | 0.828 | 0.472 | 5701       | 5730        | 1.01            | 105  | 0.51  | 0.17 |

$K_o$ = in-situ $K$; $\beta_p$ = predicted beta value; $Q_{uap}$ = predicted compression side resistance; $Q_{um}$ = measured compression side resistance; $\beta_m$ = measured beta value; $(K/K_o)_{um}$ = back-calculated stress factor.

Table 4. Basic Information and Side Resistance Analysis Result for Undrained Compression Round Section (UCR) Tests.
Table 5. Basic Information and Side Resistance Analysis Result for Undrained Compression Square Section (UCS) Tests

| Site & Pile no. | Test site/Soil description along pile depth | Depth, D (m) | Dia. B (m) | OCR | K<sub>u</sub> | β<sub>p</sub> | Q<sub>u</sub><sup>d</sup> (kN) | Q<sub>u</sub><sup>e</sup> (kN) | Q<sub>u</sub><sup>d/e</sup> | α(CIUC) | Q<sub>u</sub> <sup>(CIUC)</sup> | λ<sup>b</sup> |
|----------------|------------------------------------------|-------------|------------|-----|-------------|----------|----------------|----------------|----------------|-----------|----------------|-------|
| UCS 1-1        | Singapore; marine clay                   | 26.0        | 0.280      | 2   | 0.735       | 0.346    | 1426          | 2038          | 1.43           | 46        | 1.54           | 0.38  |
| UCS 1-2        | Malaysia; silt and clay                  | 17.4        | 0.350      | 5   | 1.059       | 0.678    | 2513          | 3061          | 1.22           | 109       | 0.77           | 0.28  |
| UCS 2-2        | Singapore; clay and silt                 | 14.7        | 0.300      | 6   | 1.179       | 0.755    | 1712          | 2412          | 1.41           | 101       | 0.72           | 0.27  |
| UCS 2-3        | Mexico; clayey soil                      | 16.8        | 0.250      | 5   | 1.059       | 0.678    | 1673          | 2162          | 0.97           | 107       | 0.59           | 0.22  |
| UCS 2-4        | California; lean to fat clay             | 21.0        | 0.355      | 2   | 0.650       | 0.384    | 2629          | 2164          | 0.82           | 180       | 0.55           | 0.16  |
| UCS 2-5        | London; London clay                      | 8.5         | 0.305      | 14.4| 1.847       | 0.855    | 698           | 702           | 1.01           | 146       | 0.59           | 0.23  |
| UCS 2-6        | London; London clay                      | 4.4         | 0.305      | 25.3| 2.679       | 1.579    | 328           | 291           | 0.89           | 127       | 0.54           | 0.23  |
| UCS 2-7        | Guiana; Demerara clay                    | 12.0        | 0.305      | 5   | 1.129       | 0.531    | 345           | 280           | 0.81           | 39        | 0.76           | 0.19  |
| UCS 2-8        | Australia; silt clay                     | 39.0        | 0.350      | 1   | 0.378       | 0.200    | 2896          | 3292          | 1.14           | 116       | 0.66           | 0.15  |
| UCS 2-9        | Louisiana; clays and silts               | 42.0        | 0.275      | 1   | 0.485       | 0.257    | 3304          | 3806          | 1.15           | 118       | 0.89           | 0.20  |
| UCS 14         | India; soft to stiff clay                | 22.5        | 0.400      | 3.7 | 0.916       | 0.541    | 2888          | 1472          | 0.51           | 168       | 0.31           | 0.11  |
| UCS 15         | Carbondale; silty clay                   | 6.1         | 0.305      | 15  | 2.320       | 1.800    | 320           | 631           | 1.98           | 68        | 0.70           | 0.59  |

OCR = overconsolidation ratio; K<sub>u</sub> = in-situ K; β<sub>p</sub> = predicted beta value; Q<sub>u</sub><sup>d</sup> = predicted compression side resistance; Q<sub>u</sub><sup>e</sup> = measured compression side resistance; α(CIUC) = undrained shear strength from CIUC test; λ<sup>b</sup> = calculated lambda; α(CIUC) = calculated alpha; h<sub>e</sub> = calculated lambda.
Table 6. Basic Information and Side Resistance Analysis Result for Drained Uplift Round Section (DUR) Tests

| Site & Pile no. | Test site/Soil description along pile depth | Depth, D (m) | Dia., B (m) | $K_{so}$ | $\beta_p$ | $Q_{sup}$ | $Q_{sum}$ | $Q_{sum}/Q_{sup}$ | $\beta_{pm}$ | $\beta_{pm}/\beta_p$ | (K/Ko)$_{bs}$ |
|----------------|------------------------------------------|-------------|------------|----------|---------|-----------|-----------|-----------------|-----------|----------------|---------------|
| DUR_1          | Spain; silty sand                         | 18.0        | 0.91       | 0.412    | 0.262   | 1174     | 1986      | 1.69            | 0.434     | 1.66           | 1.45          |
| DUR_2-1        | Miliao, Taiwan; silty sand                | 25.0        | 0.50       | 0.441    | 0.261   | 1141     | 1429      | 1.25            | 0.325     | 1.25           | 1.09          |
| DUR_2-2        | Miliao, Taiwan; silty sand                | 27.0        | 0.50       | 0.441    | 0.261   | 1315     | 1799      | 1.37            | 0.355     | 1.36           | 1.19          |
| DUR_2-3        | Miliao, Taiwan; silty sand                | 23.0        | 0.50       | 0.441    | 0.261   | 979      | 1106      | 1.13            | 0.293     | 1.12           | 0.99          |
| DUR_2-4        | Miliao, Taiwan; silty sand                | 23.0        | 0.40       | 0.441    | 0.261   | 784      | 518       | 0.66            | 0.172     | 0.66           | 0.58          |
| DUR_2-5        | Miliao, Taiwan; silty sand                | 12.0        | 0.50       | 0.494    | 0.292   | 410      | 984       | 2.40            | 0.699     | 2.39           | 2.10          |
| DUR_2-6        | Miliao, Taiwan; silty sand                | 23.0        | 0.50       | 0.441    | 0.261   | 979      | 1534      | 1.57            | 0.407     | 1.56           | 1.37          |
| DUR_2-7        | Miliao, Taiwan; silty sand                | 25.0        | 0.50       | 0.441    | 0.261   | 1141     | 1841      | 1.61            | 0.419     | 1.61           | 1.41          |
| DUR_2-8        | Miliao, Taiwan; silty sand                | 12.0        | 0.50       | 0.494    | 0.292   | 410      | 1087      | 2.65            | 0.772     | 2.64           | 2.32          |
| DUR_2-9        | Miliao, Taiwan; silty sand                | 11.0        | 0.50       | 0.415    | 0.245   | 295      | 809       | 2.74            | 0.671     | 2.73           | 2.40          |
| DUR_2-10       | Miliao, Taiwan; silty sand                | 20.0        | 0.60       | 0.531    | 0.250   | 844      | 1483      | 1.76            | 0.455     | 1.82           | 1.61          |
| DUR_4-1        | Dramen, Norway; uniform loose normally consolidated sand | 8.0    | 0.28      | 0.710    | 0.307   | 78       | 79        | 1.01            | 0.326     | 1.06           | 0.94          |
| DUR_4-2        | Dramen, Norway; uniform loose normally consolidated sand | 16.0    | 0.28      | 0.562    | 0.243   | 243      | 257       | 1.06            | 0.270     | 1.11           | 0.99          |
| DUR_4-3        | Brazil; silty sand                       | 23.0        | 0.28       | 0.440    | 0.242   | 469      | 274       | 0.58            | 0.143     | 0.59           | 0.52          |
| DUR_5-1        | Vietnam; clayey sand                      | 41.0        | 0.90       | 0.359    | 0.189   | 2560     | 1652      | 0.75            | 0.124     | 0.76           | 0.67          |
| DUR_5-2        | Vietnam; clayey sand                      | 41.0        | 0.90       | 0.359    | 0.189   | 2560     | 1662      | 0.75            | 0.125     | 0.77           | 0.68          |
| DUR_6-1        | Vietnam; clayey sand                      | 20.4        | 0.60       | 0.532    | 0.249   | 1890     | 2171      | 1.15            | 0.297     | 1.19           | 1.06          |
| DUR_6-2        | Vietnam; clayey sand                      | 20.4        | 0.60       | 0.523    | 0.251   | 1902     | 2421      | 1.27            | 0.331     | 1.32           | 1.16          |
| DUR_6-3        | Vietnam; clayey sand                      | 20.4        | 0.60       | 0.574    | 0.240   | 1815     | 1231      | 0.68            | 0.172     | 0.72           | 0.64          |
| DUR_7-1        | Miliao, Taiwan; silty sand                | 23.0        | 0.50       | 0.441    | 0.261   | 981      | 1341      | 1.37            | 0.355     | 1.36           | 1.19          |
| DUR_7-2        | Miliao, Taiwan; silty sand                | 9.0         | 0.50       | 0.441    | 0.261   | 279      | 939       | 3.37            | 0.875     | 3.36           | 2.94          |
| DUR_7-3        | Miliao, Taiwan; silty sand                | 9.0         | 0.50       | 0.441    | 0.261   | 279      | 1006      | 3.61            | 0.937     | 3.59           | 3.15          |
| DUR_7-4        | Miliao, Taiwan; silty sand                | 14.0        | 0.50       | 0.441    | 0.261   | 670      | 1367      | 2.04            | 0.530     | 2.03           | 1.78          |
| DUR_7-5        | Miliao, Taiwan; silty sand                | 14.0        | 0.50       | 0.441    | 0.261   | 670      | 1485      | 2.22            | 0.576     | 2.21           | 1.94          |
| DUR_7-6        | Miliao, Taiwan; silty sand                | 11.0        | 0.50       | 0.441    | 0.261   | 498      | 1588      | 3.19            | 0.994     | 3.81           | 3.34          |
| DUR_7-7        | Miliao, Taiwan; silty sand                | 23.0        | 0.50       | 0.441    | 0.261   | 1856     | 1658      | 0.89            | 0.232     | 0.89           | 0.78          |

1 $K_{so}$ = in-situ $K$; 2 $\beta_p$ = predicted beta value; 3 $Q_{sup}$ = predicted uplift side resistance; 4 $Q_{sum}$ = measured uplift side resistance = $Q_w$/$W$; 5 $\beta_{pm}$ = measured beta value; 6 (K/Ko)$_{bs}$ = back-calculated stress factor.

Table 7. Basic Information and Side Resistance Analysis Result for Drained Uplift Square Section (DUS) Tests

| Site & Pile no. | Test site/Soil description along pile depth | Depth, D (m) | Dia. B (m) | $K_{so}$ | $\beta_p$ | $Q_{sup}$ | $Q_{sum}$ | $Q_{sum}/Q_{sup}$ | $\beta_{pm}$ | $\beta_{pm}/\beta_p$ | (K/Ko)$_{bs}$ |
|----------------|------------------------------------------|-------------|------------|----------|---------|-----------|-----------|-----------------|-----------|----------------|---------------|
| DUS_1          | Iraq; uniform                           | 11.0        | 0.285      | 0.455    | 0.260   | 312       | 408       | 1.31            | 0.341     | 1.31           | 1.15          |
| Site & Pile no. | Test site/Soil description along pile depth | Depth, D (m) | Dia. B (m) | OCR | $K_o$ | $\beta_p$ | $Q_{um}$ | $Q_{um}$ | $\sum_s$ | $\sum_C$ | $\sum_d(CIUC)$ | $\alpha(CIUC)$ | $\lambda$ |
|----------------|---------------------------------------------|-------------|-----------|-----|------|--------|--------|--------|--------|--------|----------|-----------|-------|
| UUR 1-1        | Negeri, Malaysia; clay                      | 17.5        | 0.50      | 1.062 | 0.243 | 984    | 600    | 0.61   | 37     | 0.67   | 0.099    |           |       |
| UUR 1-2        | Negeri, Malaysia; clay                      | 17.5        | 0.50      | 1.062 | 0.243 | 984    | 600    | 0.61   | 37     | 0.67   | 0.099    |           |       |
| UUR 1-3        | Negeri, Malaysia; clay                      | 7.5         | 0.50      | 0.741 | 0.342 | 245    | 360    | 1.47   | 27     | 1.24   | 0.271    |           |       |
| UUR 2-1        | Bangkok, Thailand; soft clay               | 20.0        | 0.40      | 1.04  | 0.368 | 173    | 350    | 1.04   | 39     | 0.43   | 0.092    |           |       |
| UUR 2-2        | Bangkok, Thailand; soft clay               | 20.0        | 0.40      | 1.04  | 0.368 | 173    | 350    | 1.04   | 39     | 0.43   | 0.092    |           |       |
| UUR 3-1        | China; silty clay                          | 43.0        | 0.50      | 1.72  | 0.081 | 941    | 631    | 0.67   | 20     | 0.59   | 0.039    |           |       |
| UUR 3-2        | China; silty clay                          | 43.0        | 0.50      | 1.72  | 0.081 | 941    | 631    | 0.67   | 20     | 0.59   | 0.039    |           |       |
| UUR 3-3        | China; silty clay                          | 43.0        | 0.50      | 1.72  | 0.081 | 941    | 786    | 0.84   | 20     | 0.70   | 0.049    |           |       |
| UUR 3-4        | China; silty clay                          | 43.0        | 0.60      | 1.72  | 0.081 | 1130   | 897    | 0.79   | 20     | 0.70   | 0.046    |           |       |
| UUR 3-5        | China; silty clay                          | 43.0        | 0.60      | 1.72  | 0.081 | 1130   | 927    | 0.82   | 20     | 0.72   | 0.048    |           |       |
| UUR 3-6        | China; silty clay                          | 43.0        | 0.60      | 1.72  | 0.081 | 1130   | 933    | 0.83   | 20     | 0.72   | 0.048    |           |       |

* $K_o = \text{in-situ } K_o$; $\beta_p = \text{predicted } beta \text{ value}; \ Q_{um} = \text{predicted uplift side resistance}; \ Q_{um} = \text{measured uplift side resistance} = Q_{c-W}; \ Q_{um} = \text{measured uplift side resistance} = Q_{c-W}; \ Q_{um} = \text{measured uplift side resistance} = Q_{c-W}; \ Q_{um} = \text{measured uplift side resistance} = Q_{c-W};$ $\sum_s = \text{undrained shear strength from CIUC test}$; $\alpha(CIUC) = \text{calculated alpha}$; $\lambda = \text{calculated lambda}$
Table 9. Basic Information and Side Resistance Analysis Result for Undrained Uplift Square Section (UUS) Tests

| Site & Pile no. | Test site/Soil description along pile depth | Depth, D (m) | Dia., B (m) | OCR a | K o b | β p c | Q sup d (kN) | Q sum e (kN) | Q sum/ Q sup f | s(CIUC) g | α(CIUC) h | λ i |
|----------------|-------------------------------------------|-------------|-------------|-------|-------|-------|-------------|-------------|-------------|----------|-----------|-----|
| UUS_1          | Mexico; clayey soil                        | 15.0        | 0.300       | 1     | 0.531 | 0.250 | 386         | 363         | 0.94        | 48.0     | 0.58     | 0.141 |
| UUS_2-1        | Canada-Site A; glacial clay                | 25.0        | 0.380       | 1     | 0.212 | 0.108 | 857         | 615         | 0.72        | 78.0     | 0.30     | 0.056 |
| UUS_2-2        | Canada-Site B; marine silt                 | 23.8        | 0.380       | 1     | 0.224 | 0.114 | 819         | 629         | 0.77        | 76.0     | 0.33     | 0.063 |
| UUS_3          |                                            | 47.2        | 0.380       | 1     | 0.100 | 0.047 | 1266        | 1090        | 0.86        | 50.0     | 0.44     | 0.034 |
| UUS_4          | Louisiana; clays and silt                  | 25.0        | 0.356       | 2     | 0.234 | 0.124 | 833         | 477         | 0.57        | 182.0    | 0.19     | 0.031 |
| UUS_5-1        | Illinois; silt clay                        | 6.4         | 0.305       | 12    | 1.744 | 0.923 | 265         | 381         | 1.44        | 75.0     | 0.56     | 0.356 |
| UUS_5-2        |                                            | 6.4         | 0.305       | 10    | 1.588 | 0.841 | 241         | 303         | 1.26        | 75.0     | 0.56     | 0.283 |
| UUS_5-3        |                                            | 6.4         | 0.305       | 10    | 1.588 | 0.841 | 241         | 191         | 0.79        | 75.0     | 0.45     | 0.178 |
| UUS_6-1        | Kinnegar N. Ireland; clayey silt           | 6.0         | 0.250       | 2     | 0.693 | 0.367 | 81          | 55          | 0.68        | 30.0     | 0.45     | 0.150 |
| UUS_6-2        |                                            | 6.0         | 0.250       | 2     | 0.693 | 0.367 | 81          | 52          | 0.65        | 30.0     | 0.43     | 0.145 |
| UUS_7          | Singapore; clay & silt                     | 11.6        | 0.320       | 2     | 0.693 | 0.367 | 616         | 597         | 0.97        | 121.0    | 0.44     | 0.173 |

OCR = overconsolidation ratio; K o = in-situ K; β p = predicted beta value; Q sup = predicted uplift side resistance; Q sum = measured uplift side resistance = Q l2 - W; s(CIUC) = undrained shear strength from CIUC test; α(CIUC) = calculated alpha; λ = calculated lambda

The load-displacement curve in Figure 2 can generally be simplified into three distinct regions: initial linear, curve transition, and final linear. Point L1 (elastic limit) corresponds to the load (Q L1) and butt displacement (ρ L1) at the end of the initial linear region, while L2 (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 interrelationships between L2 and other interpretation criteria from the lower to higher bounds that was previously developed [17] are used to infer the required L2 if the test data are insufficient or are terminated prematurely.

![Figure 2. Regions of Axial Load-Displacement Curve](image-url)
geometry, side resistance, and their statistics are summarized in Tables 10 and 11 for compression and uplift, respectively. As can be seen, the range of geometry is broad and the diameters for the four categories are roughly comparable.

### Table 10. Range of Driven Pile Geometry for Compression Side Resistance Analysis

| Data | Number of tests | Statistics | Pile geometry (m) | D/B | Side resistance (KN) |
|------|-----------------|------------|-------------------|-----|---------------------|
|      |                 |            | Depth, D | Diameter\(^a\), B |     |                     |
|      |                 | Range     | 7.5-40.0 | 0.28-0.91 | 11.4-133.3 | 78-3028 |
|      |                 | Mean      | 22.1     | 0.48      | 48.6       | 1062   |
|      |                 | COV       | 0.37     | 0.33      | 0.43       | 0.73   |
| DCR  | 37              | Range     | 6.0-48.5 | 0.24-0.76 | 17.1-121.2 | 154-3203 |
|      |                 | Mean      | 17.9     | 0.38      | 49.3       | 819    |
|      |                 | COV       | 0.56     | 0.30      | 0.52       | 0.89   |
|      |                 | Range     | 8.2-57.5 | 0.18-1.37 | 6.0-142.0  | 177-11689 |
| DCS  | 44              | Mean      | 26.9     | 0.50      | 63.7       | 3355   |
|      |                 | COV       | 0.48     | 0.57      | 0.55       | 0.82   |
|      |                 | Range     | 4.4-60.3 | 0.25-0.46 | 14.5-152.7 | 74-6827 |
| UCR  | 37              | Mean      | 18.7     | 0.33      | 56.6       | 1647   |
|      |                 | COV       | 0.61     | 0.17      | 0.61       | 0.81   |
| UCR  | 37              | Range     | 8.0-41.0 | 0.30-0.90 | 19.8-82.1  | 78-2560 |
|      |                 | Mean      | 19.8     | 0.50      | 39.7       | 1007   |
|      |                 | COV       | 0.42     | 0.30      | 0.34       | 0.69   |
| UCR  | 37              | Range     | 3.0-23.5 | 0.20-0.80 | 15.0-70.6  | 26-2190 |
|      |                 | Mean      | 14.5     | 0.40      | 38.3       | 561    |
|      |                 | COV       | 0.35     | 0.48      | 0.47       | 0.95   |
| UCUS | 37              | Range     | 7.5-43.0 | 0.40-0.60 | 15.0-86.0  | 81-1266 |
|      |                 | Mean      | 31.0     | 0.50      | 59.8       | 517    |
|      |                 | COV       | 0.46     | 0.14      | 0.41       | 0.75   |
| UUR  | 11              | Range     | 6.0-47.2 | 0.30-0.40 | 21.0-124.2 | 245-1130 |
|      |                 | Mean      | 16.3     | 0.30      | 47.3       | 830    |
|      |                 | COV       | 0.80     | 0.15      | 0.68       | 0.41   |

\(^a\) or width of square section; \(^b\) from \(\beta\) method

### Table 11. Range of Driven Pile Geometry for Uplift Side Resistance Analysis

| Data | Number of tests | Statistics | Pile geometry (m) | D/B | Side resistance (KN) |
|------|-----------------|------------|-------------------|-----|---------------------|
|      |                 |            | Depth, D | Diameter\(^a\), B |     |                     |
|      |                 | Range     | 8.0 - 41.0 | 0.30 - 0.90 | 19.8 - 82.1 | 78 - 2560 |
|      |                 | Mean      | 19.8     | 0.50      | 39.7       | 1007   |
|      |                 | COV       | 0.42     | 0.30      | 0.34       | 0.69   |
| DUR  | 27              | Range     | 3.0 - 23.5 | 0.20 - 0.80 | 15.0 - 70.6 | 26 - 2190 |
|      |                 | Mean      | 14.5     | 0.40      | 38.3       | 561    |
|      |                 | COV       | 0.35     | 0.48      | 0.47       | 0.95   |
| DUS  | 31              | Range     | 7.5 - 43.0 | 0.40 - 0.60 | 15.0 - 86.0 | 81 - 1266 |
|      |                 | Mean      | 31.0     | 0.50      | 59.8       | 517    |
|      |                 | COV       | 0.46     | 0.14      | 0.41       | 0.75   |
| UUR  | 11              | Range     | 6.0 - 47.2 | 0.30 - 0.40 | 21.0 - 124.2 | 245 - 1130 |
|      |                 | Mean      | 16.3     | 0.30      | 47.3       | 830    |
|      |                 | COV       | 0.80     | 0.15      | 0.68       | 0.41   |

\(^a\) or width of square section; \(^b\) from \(\beta\) method

\[
\alpha = \frac{Q_s (L_2)}{pD_{su}}
\]

in which \(Q_s (L_2)\) = interpreted side resistance using \(L_2\) method and \(s_u\) = mean undrained shear strength over the pile depth (D), and \(p =\) perimeter. To standardize the \(\alpha\)-\(s_u\) relationship for driven piles, the unique test type of undrained shear strength from consolidated-isotropically undrained triaxial compression (CIUC) [3] was adopted. The CIUC was selected as reference test because it is quite common and of good quality test. The \(s_u\) values from all other tests were converted to “equivalent”
s_u(CIUC). The procedures to convert are based on previous study [20] for unconsolidated-undrained triaxial (UU) and unconfined compression (UC) tests.

Figure 3 illustrates the correlations between $\alpha$ and undrained shear strength for compression and uplift loading, with regression equations given as:

**Compression:**

$$\alpha_{(CIUC)} = \frac{0.33 + 0.32}{s_u(CIUC)/p_a} \quad (n = 73; \text{SD} = 0.20; r^2 = 0.54) \quad (2)$$

**Uplift:**

$$\alpha_{(CIUC)} = \frac{0.22 + 0.14}{s_u(CIUC)/p_a} \quad (n = 22; \text{SD} = 0.25; r^2 = 0.32) \quad (3)$$

**All data:**

$$\alpha_{(CIUC)} = \frac{0.35 + 0.15}{s_u(CIUC)/p_a} \quad (n = 95; \text{SD} = 0.28; r^2 = 0.42) \quad (4)$$

The $s_u(CIUC)$ is normalized by $p_a$, which is the atmospheric stress (101.3 kN/m$^2$) in the same unit as $s_u$. As can be seen in Figure 3, the trends of round and square piles for compression loading are somewhat comparable. The same is true for uplift loading. Comparisons of compression and uplift $\alpha_{CIUC-s_u(CIUC)}$ correlations show some interesting points. First, the coefficient of determination ($r^2$) is larger for compression which may be due to the limited number of data used for uplift. Second, the compression data points are seen above uplift data points indicating that $\alpha_{CIUC}$ values are generally smaller for uplift for a range of $s_u$ values. Third, Figure 3 demonstrates that $\alpha$ for small values of $s_u(CIUC)/p_a$ produces steep regression lines for compression and uplift. Hence, the use of $\alpha_{CIUC-s_u(CIUC)}$ correlation for design may tend to be conservative and sensitive for relatively small $s_u$-values.

Based on the available load test data, $\alpha$ for compression is in the range of 0.28 to 1.77, whereas for uplift, $\alpha$ is in the range of 0.19 to 1.62.

![Figure 3. $\alpha_{CIUC} - s_u(CIUC)/p_a$ Correlations for Driven Piles](image)

The $\alpha_{CIUC-s_u(CIUC)}$ correlation for drilled shafts [4] is compared to the $\alpha_{CIUC-s_u(CIUC)}$ correlation developed in this study. The $\alpha_{CIUC-s_u(CIUC)}$ correlation by the previous study [4] was developed from compression and uplift data. For comparison purposes, the compression and uplift $\alpha_{CIUC-s_u(CIUC)}$ correlations were combined. The comparison is presented in Figure 4 wherein some significant points...
are observed. First, the coefficient of determination ($r^2$) is higher in drilled shaft than in driven pile. This implies that consistent $\alpha$ value can be expected from drilled shaft than from driven pile. This behavior can be attributed to the better adhesion of soil-pile interface of drilled shaft which is the result of its installation procedure. The effect of hammer pile driving on the pore water pressure may indirectly affect the soil-pile adhesion resulting to more variable behavior of driven pile. Second, for smaller $s_u$(CIUC)/$p_o$ (< 0.50), the regression lines of drilled shaft and driven pile are converging. However, for larger values of $s_u$(CIUC)/$p_o$, drilled shaft tends to produce smaller values of $\alpha$ than driven pile. However, the difference is small. Third, the range of $\alpha$ for drilled shaft is smaller (with a maximum of < 1.0) than for driven pile (with a maximum of < 1.8).

\[
\alpha_{CIUC} = 0.30 + 0.17 / \left[ s_u(CIUC)/p_o \right]
\]

$n=148$, SD=0.09, $r^2=0.66$

**Figure 4.** Comparison of Driven Pile and Drilled Shaft $\alpha_{CIUC}$ - $s_u$(CIUC)/$p_o$ Correlations

3.1.2. $\alpha_{CIUC}$ - $s_u$(CIUC)/$\bar{\sigma}_v$ - $\bar{\sigma}_v$ correlations. The undrained shear strength ratio correlation, $\alpha_{CIUC}$- $s_u$(CIUC)/$\bar{\sigma}_v$ - $\bar{\sigma}_v$, was developed in which the overburden pressure was taken into account. The correlations were developed directly from field load test database and are presented in Figures. 5(a) to 5(c) for compression, uplift, and all data combined. These figures also include the statistical data for the individual curve. Result for compression loading in Figure 5(a) shows that the regression lines are closer when the undrained shear strength ratio (USR) is greater than 1.0. This could be due to a more scattered data for USR <1.0. Although the data points are few for small values of $\bar{\sigma}_v$, the regression lines appear to be stiffer for these values. This behavior is similar to the result in Figure 3 that the design may tend to be conservative for small values of $\bar{\sigma}_v$. Furthermore, $\alpha_{CIUC}$ is decreasing with increasing $s_u$(CIUC)/$\bar{\sigma}_v$ and $\bar{\sigma}_v$. For uplift, the data were subdivided into two ratios only due to limited number of data points and the correlation is shown in Figure 5(b). Roughly, similar behavior is observed as in compression that the trend of $\alpha_{CIUC}$ is decreasing with increasing $s_u$(CIUC)/$\bar{\sigma}_v$ and $\bar{\sigma}_v$. The compression and uplift data were combined as shown in Figure 5(c) which indicates a wide variation of data for USR < 0.50.

The $\alpha_{CIUC}$ value from $\alpha_{CIUC}$- $s_u$(CIUC)/$\bar{\sigma}_v$ - $\bar{\sigma}_v$ correlations can precisely be distinguished than using the conventional $\alpha_{CIUC}$- $s_u$(CIUC) correlations. Therefore, the necessary $\alpha$ value can be reasonably selected. These correlations can be regarded as an alternative method of analysis for traditional $\alpha$ - $s_u$ correlations or in verifying the required value of $\alpha$ for design.

3.2. $\beta$ method

Approximate beta values can be predicted as follows:
\[ \beta_p = K_o \left( \frac{K}{K_o} \right) \tan \left[ \bar{\phi} \cdot \delta / \bar{\phi} \right] \]  \hspace{1cm} (5)

Then, the average \( \beta_p \) over the pile depth was calculated by weighted average. The average \( \bar{\phi} \) and \( K_o \) are shown in Tables 2 to 5 and Tables 6 to 9 for compression and uplift loading, respectively. In this study, \( \delta / \bar{\phi} \) was taken as 0.90, and therefore all cases used \( \delta / \bar{\phi} = 0.90 \) for this calculation. For \( K/K_o \), a value of 1.0 was adopted which is a value within the range of small to large displacement piles. The detailed analysis results for the DCR, DCS, UCR, and UCS tests are given in Tables 2 to 5, respectively, whereas the results for the DUR, DUS, UUR, and UUS tests are given in Tables 6 to 9, respectively.

![Graph of Compression Results](attachment:compression_graph.png)

![Graph of Uplift Results](attachment:uplift_graph.png)
3.2.1. Drained load tests. The result for the drained loading is summarized in Table 12 including the statistics. For group 1 (compression loading), the predicted side resistance \( (Q_{scm}) \) is compared to the measured side resistance \( (Q_{sum}) \). The mean side resistance ratios \( (Q_{scm}/Q_{scp}) \) are 2.25 and 2.44 for round and square cross section piles, respectively. The COVs for both results are more than 30%. The capacity ratios indicate an obvious underprediction of side resistance for both pile sections. For uplift, the predicted side resistance \( (Q_{sup}) \) likewise is compared to the measured side resistance \( (Q_{sum}) \) as shown in Table 12. The mean side resistance ratios \( (Q_{sum}/Q_{sup}) \) are 1.64 and 1.88, for round and square cross section piles, respectively. The COVs for both results are around 50%. As in compression, the capacity ratios indicate an obvious underprediction of side resistance for both pile sections.

![Figure 5. Correlations for (a) Compression Loading, (b) Uplift Loading, and (c) All data](image)

**Table 12. Statistics of \( Q_{sum}/Q_{scp} \) for Drained Load Tests**

| Data | Statistics | \( Q_{scm}/Q_{scp} \) | Data | Statistics | \( Q_{sum}/Q_{sup} \) |
|------|------------|----------------------|------|------------|----------------------|
|      | n          | 37                   | n    | 27         |                      |
| DCR  | mean       | 2.25                 | DUR  | mean       | 1.64                 |
|      | COV        | 0.37                 | COV  | 0.53       |                      |
|      | n          | 44                   | n    | 31         |                      |
| DCS  | mean       | 2.44                 | DUS  | mean       | 1.88                 |
|      | COV        | 0.31                 | COV  | 0.50       |                      |
|      | n          | 81                   | n    | 58         |                      |
| All data | mean   | 2.35                 | All  | mean       | 1.77                 |
|      | COV        | 0.34                 | data | COV        | 0.51                 |

Comparison of compression and uplift side resistances is shown in Figure 6. For comparison, round and square piles were combined since their behavior is somewhat comparable. Results of all data combined for compression and uplift are likewise presented in Table 12. The mean side resistance ratios are 2.35 and 1.77 for compression and uplift load tests, respectively. The COVs are 0.34 and 0.51 for compression and uplift, respectively. The regression lines shown in Figure 6 indicate that \( Q_{scm} = 1.96 \ Q_{scp} \) and \( Q_{sum} = 1.51 \ Q_{sup} \) and are in good agreement with the mean results. Apparently, compression loading exhibits greater Underestimation of side resistance than uplift loading.
The underprediction of side resistance can be attributed to several factors. Underestimation of soil parameters is one possible reason because due to pile driving, the fact that the soil surrounding the pile becomes denser may have been neglected. Another reason may be due to the overconsolidation at shallower pile depths in which $K_0$ may have been underestimated. The stress coefficient $K/K_0$ which was assumed to be equal to 1.0 may be another reason.

The effects of pile depth on side resistance are examined in Figure 7. In general, the behavior for compression and uplift is comparable. It can be observed that the ratio of $Q_{sm}/Q_{sp}$ generally decreases as the depth of the pile increases. However, for uplift square piles, the depth range is small and a wide variation is observed from the data points. The β method appears to be more consistent for long piles because it shows a wide range of results for short piles. In general, this phenomenon supports the above analysis that $K_0$ has been underestimated at shallower depths.
To verify the issue of underestimation of side resistance in drained soils, $K/K_o$ values were back-calculated utilizing the field load test data. As a first approximation, the measured beta ($\beta_m$) can be computed as follows:

$$\beta_m = \frac{Q_s(L_2)}{[pD\bar{\sigma}_v]}$$

In which $Q_s(L_2)$ = interpreted side resistance from L2 method and $\bar{\sigma}_v$ = mean vertical effective stress. Using Eqs. 5 and 6 and with the assumptions of $\beta_m = \beta_p$ and $\delta/\phi = 1.0$, the mean $K/K_o$ was back-calculated for the overall foundation depth of driven PC piles. Table 13 lists the statistical results for $K/K_o$. For compression, the $K/K_o$ values are 1.98 for round piles and 2.15 for square piles. For uplift, the $K/K_o$ values are 1.47 for round piles and 1.67 for square piles. Square piles yield somewhat larger stress factor which can be attributed to the larger perimeter of a square pile for a same area of a round pile. The larger perimeter can provide larger influence area of denser soil resulting to better side resistance.

The $K/K_o$ values for drilled shafts were recommended by a previous study [4]. It is suggested that for drained tests, $K/K_o$ are 0.73, 0.97, and 1.03 for slurry, casing, and dry construction respectively, whereas for undrained tests, $K/K_o$ are 0.79, 0.88, and 1.12 for slurry, casing, and dry construction respectively. Comparison of these values with the $K/K_o$ values developed for driven piles indicates that larger stress factor can be expected for driven PC piles. This can be attributed to the installation method of driven PC piles to which the driving procedure provides denser soil surrounding the pile.

### Table 13. Back-Calculated $K/K_o$

| Test type | Pile section | Statistics | $K/K_o$ |
|-----------|--------------|------------|---------|
| Compression | DCR         | mean       | 1.98    |
|            |              | COV        | 0.37    |
|            | DCS          | mean       | 2.15    |
|            |              | COV        | 0.31    |
| Uplift     | DUR          | mean       | 1.47    |
|            |              | COV        | 0.54    |
|            | DUS          | mean       | 1.67    |
|            |              | COV        | 0.50    |

#### 3.2.2. Undrained load tests.
A similar evaluation is done for undrained $\beta$ analysis. The detailed analysis results for undrained load tests are presented in Tables 4 and 5 and Tables 8 and 9 for compression and uplift loading and are summarized in Table 14 including the statistics. The mean side resistance ratios ($Q_{scm}/Q_{scp}$) are 1.05 and 1.04, for round and square cross section piles, respectively. The COVs for round piles is 29% while it is 30% for square piles. The mean predicted side resistance is in quite good agreement with the mean measured side resistance. For uplift loading, the mean side resistance ratios ($Q_{sum}/Q_{sup}$) are 0.86 and 0.88, for round and square cross section piles, respectively. The COVs for both sections are 30%. The mean predicted side resistance is in somewhat good agreement with the mean measured side resistance. Contrary to drained load tests, it appears that the $\beta$ method reasonably predicts the undrained side resistance of driven precast concrete piles.
Comparison of undrained compression and uplift side resistances is shown in Table 14 and Figure 8. Similar to drained tests, round and square piles were combined because their behavior is somewhat comparable. Results of all data combined for both compression and uplift in Table 14 indicate that the mean side resistance ratios are 1.05 and 0.87 for compression and uplift load tests, respectively. The COVs are 0.29 and 0.30 for compression and uplift, respectively. The regression analysis shown in Figure 8 indicates that $Q_{scm} = 0.92 Q_{scp}$ and $Q_{sum} = 0.81 Q_{sup}$. Uplift loading exhibits a slight overprediction of side resistance. However, in general, $\beta$ method can reliably be used in undrained side resistance analysis of driven PC piles.

As in drained loading, examination on the effects of pile depth to side resistance for undrained condition is explored and is shown in Figure 9. In general, the scatter is substantial for undrained loading. The effect of depth on side resistance is not explicitly defined by the data points due to a wide range of results throughout the depth with $Q_{sm}/Q_{sp}$ ranging from 0.5 to 1.50. Although it can also be noted that wider range (0.50 to 2.0) is observed for shallower depths (< 10 m). Hence, careful engineering judgment on the use of the $\beta$ method in undrained conditions is suggested.
Table 14. Statistics of $Q_{um}/Q_{up}$ for Undrained Load Tests

| Data | Statistics | $Q_{um}/Q_{up}$ | Data | Statistics | $Q_{um}/Q_{up}$ |
|------|------------|-----------------|------|------------|-----------------|
| UCR  | mean       | 1.05            | UUR  | mean       | 0.86            |
|      | COV        | 0.29            |      | COV        | 0.30            |
| UCS  | mean       | 1.04            | UUS  | mean       | 0.88            |
|      | COV        | 0.30            |      | COV        | 0.30            |
| All  | mean       | 1.05            | All  | mean       | 0.87            |
|      | COV        | 0.29            |      | COV        | 0.30            |

3.3. $\lambda$ method

Undrained load tests were evaluated using $\lambda$ method. The value of $\lambda$ for driven PC piles was back-calculated from field load test results as follows:

$$
\lambda = \frac{Q_u (L_2)}{pD(\sigma_{vm} + 2s_u)}
$$

In which all terms have been defined previously. Figures. 10(a) and 10(b) demonstrate the variation of $\lambda$ to pile depth for compression and uplift loading, respectively. Both figures show a somewhat wider scatter for square piles than for round piles for shorter pile depths ($D < 30$ m). The combined round and square result presents the mean $\lambda$ values for specified depths in the inclusive table. Result shows that $\lambda$ generally decreases with increasing depth. Since the behavior of compression and uplift data is comparable, these data were merged to provide a relation applicable to both test types. The relation is shown in Figure 10(c) indicating a range of $\lambda$ value for shorter piles (< 30 m) of 0.29 to 0.20 whereas for longer piles (> 30 m), $\lambda$ value is ranging from 0.20 to 0.11. The variation of $\lambda$ values can be adopted for side resistance analysis of driven PC piles. The $\lambda$ method was likewise adopted by previous research [4] for drilled shaft in cohesive soils. Their findings indicated that $\lambda$ method produces less reliable results when applied to drilled shafts. Comparison of the previous study [4] and this study reveals that lambda method is more applicable to driven piles than drilled shafts.
Figure 10. $\lambda$ versus Depth for (a) Compression (b) Uplift Loading, and (c) All Data

4. Conclusion
Compression and uplift field load test data were utilized to evaluate the side resistance of driven precast concrete piles. For drained loading, $\beta$ method was applied while for undrained loading, $\alpha$, $\beta$, and $\lambda$ methods were applied. Based on these analyses, the following conclusions emerge.

- For undrained loading, the $\alpha_{\text{CIUC}} - s_d(\text{CIUC})/p_e$ correlation is developed using field load test data and can be utilized for driven PC pile total stress analysis. For undrained loading, the correlation $\alpha_{\text{CIUC}} - s_d(\text{CIUC})/\sigma' - \sigma_e$ developed from field load test data can be regarded as an alternative base for driven PC pile total stress analysis.

- For drained loading, $\beta$ method underpredicts the side resistance. The method is more consistent for long piles because it shows a wide range of results for short piles. For undrained loading, $\beta$
method reasonably predicts the side resistance. The suggested stress factor $K/K_o$ for drained compression loading is 1.98 for round piles and 2.15 for square piles whereas for drained uplift loading, the suggested stress factor $K/K_o$ is 1.47 for round piles and 1.67 for square piles. These values can substantially improve the pile capacity prediction using $\beta$ method.

For undrained loading, the compression and uplift $\lambda$ versus depth relations for driven PC piles are developed that can be utilized for pile analysis and design. The $\alpha$, $\beta$, $\lambda$ methods can reasonably be applied in driven piles under undrained loading conditions, whereas, $\beta$ method can be suitable for driven piles in drained loading condition with the use of appropriate stress factors.

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