Shaft Resistance of Shaft-grouted Bored Piles and Barrettes Recently Constructed in Ho Chi Minh City

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ABSTRACT: Recent years, several high-rise buildings have been constructed in Ho Chi Minh city, the largest and most dynamic city in Vietnam. The city is located in the Saigon-Dongnai River delta, where, especially in the central districts, bored piles and barrettes for the high-rise buildings need to be large and socketed in alluvial deposits at large depths. Shaft-grouting technique has been recently applied to increase shaft resistance of the bored piles and barrettes. This paper briefly presents latest shaft grouting technique applied to bored piles and barrettes in the city. A database of head down and bidirectional tests on well-instrumented grouted and not-grouted bored piles and barrettes was analysed to evaluate the enhancement of shaft resistance. Correlations between the ultimate unit shaft resistance (r_u) with the SPT N_{60} value indicated that the r_u-value of grouted piles in both clayey and sandy soils was on average two times larger than that of not grouted piles. Estimated r_u-values obtained from β-method recommended in practice compared well with those obtained from the instrumented piles.

KEYWORDS: Bored piles, Barrettes, Shaft Grouting, Shaft resistance

1. INTRODUCTION

Ho Chi Minh (HCM) city, the largest city in Vietnam, is located in Saigon-Dongnai River delta, where the soft alluvial deposits often extend to great depths. This means that the conventional bored piles or barrettes for high-rise buildings in the city must be sufficiently large and socketed in good bearing stratum at great depths, causing the pile foundation work very expensive. To reduce the number of piles for a project or reduce the pile length, shaft-grouting recently has been applied to bored piles and barrettes of a number of high-rise buildings in the city. Although the grouting technique can be applied to enhance capacity of both shaft and toe resistances, this paper addresses only the techniques and procedures to enhance shaft resistance.

The shaft-grouting method was first applied to steel piles in offshore foundation (Gouvenot and Gabaix 1975) and then to bored piles and barrettes (e.g., Stocker 1983). The method has been recently applied to bored piles and barrettes for high-rise buildings in some Asian cities, such as Bangkok (Littlechild et al. 1998), Hong Kong (Plumbridge et al. 2000, Chan et al. 2004, Sze and Chan 2012), and HCM (Phan and Pham 2013, Nguyen and Fellenius 2015, and Nguyen et al. 2016). The method has been increasingly used in Vietnam these days, especially in HCM city where skyscrapers are being constructed intensively in the central areas having deep alluvial deposits.

Although shaft-grouting method is increasingly applied in HCM city, there hasn't been a comprehensive study on the effectiveness of the method following the primary study of Phan and Pham (2013). The two authors presented a study on the effectiveness of the method applied to some instrumented bored piles and barrettes constructed in the city and proposed rough correlations to estimate the enhanced shaft resistance of the piles. However, a key drawback of this study was that the rough correlations were simply drawn from all data points obtained from the test piles without methodically defining whether shaft resistance at the strain gauge levels was fully mobilized.

The key objectives of this paper are to briefly introduce latest grouting technique applied in HCM city and, more important, to quantitatively examine the enhancement of shaft resistance of shaft-grouted bored piles and barrettes in the city. Toward this end, a database of static loading tests (head-down tests) and bidirectional tests on a number of well-instrumented grouted and not grouted bored piles and barrettes was established and analyzed.

2. BRIEF ON SHAFT GROUTING METHOD

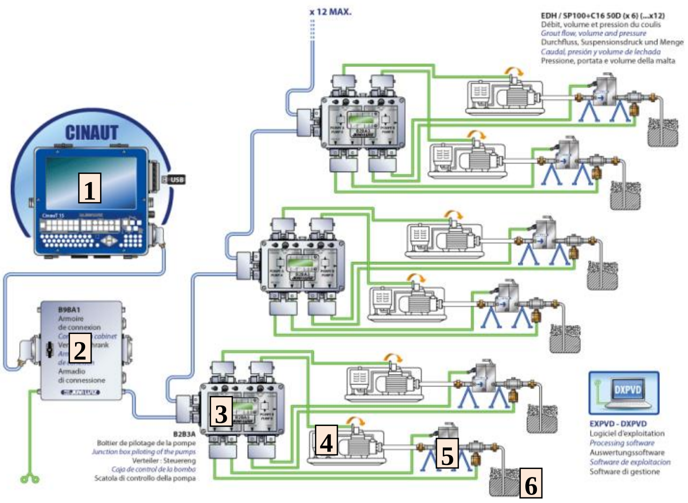
2.1 Principle of the method

A typical configuration of modern grouting system being used by FECON South (FCS) is illustrated in Figure 1(a). The shaft grouting system applied to bored piles and barrettes is in principle similar to the common jet grouting system in practice. However, instead of using jetting rods connected to boring/pushing machine, a double packer threaded into grouting tubes ("tube à manchettes"), which are attached to the external face of steel cage, is used to jet the grout through specific prefabricated manchettes (holes) along the tubes. The interval of the manchettes is typically of 1.0 m. Grouting tubes are often mild steel tubes covered by the rubble sleeves typically of 42 to 50 mm internal diameter. Figures 1(b) and 1(c) show some photos of the grouting system of FCS in practice.

Figure 2(a) schematically shows a soil-pile section with the arrangement of grouting tube around the steel cage. At a grouting point, after being lowered down and inflated, the double packer jets grout through the manchette and concrete cover to form the grout layer between soil and pile surface. Figures 2(b) and 2(c) show the working principle of a double packer (discussed in more detail in next section) and a photo of an actual double packer used by FCS at construction sites, respectively.

2.2 Mechanism of the increase in shaft resistance

The fundamental mechanisms of the increase in shaft resistance by the method have been summarized by Stocker (1983), Troughton and Stocker (1994) and Chan et al. (2004). Some key points are as follows: (*i*) the concrete on the perimeter pile is cracked and pushed against the surrounding soil with the grout bracing the pile against the soil; (*ii*) the increased lateral stress causes a local increase in soil density in the interface zone of the pile, which has become softened or loosened by the pile construction process; (*iii*) in granular soils, cementation of the soil particles in the interface zone may occur due to the infiltration of grout into the pores of the soil; (iv) voids, fissures, cavities or wash outs may be filled with grout providing an improved contact between the pile and the soil.



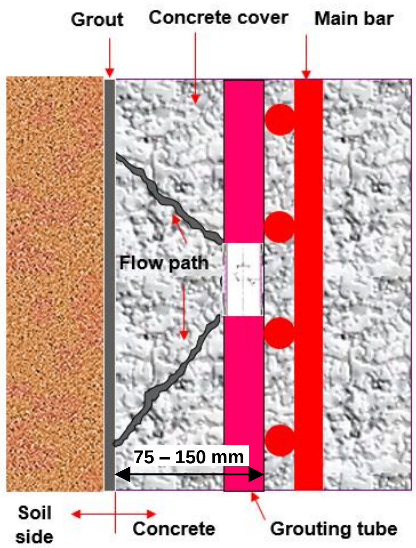
1) Data processing unit, (2) Junction box, (3) Flowmeter, (4) grouting pump; (5) pressure balance; (6) bored piles)

(a)

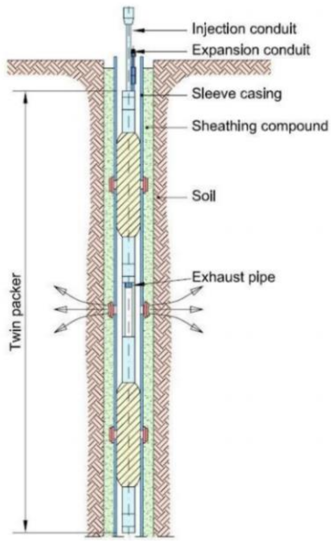
(b)

(c)

Figure 1 Configuration and equipment for shaft grouting: (a) configuration of grouting system being used by FCS; (b) part of grouting system of FCS; (c) FCS engineers with grouting specialist



(a)



(b)

(c)

Figure 2 (a) Schematic illustration of pile section with grouting tube and flow paths; (b) configuration of a typical double packer (http://www.rkc.net.in/GROUTING-C.html); (c) photo of an double packer used by FCS at the sites

2.3 Grout material and grouting procedures

The grout used for the projects had cement, water, and bentonite proportions of 100 kg, 66.7 litres, and 1.5 kg, respectively as recommended by some specifications (e.g., Arup 2017). A small amount of additives (e.g., Bentoncry 186 and Daracern 100) was also used. The mixing of grout was closely controlled in accordance with the BS EN 197-1 (2011) standard. Note that the mixed grout slurry should pass through a nominal 1.2 mm sieve before injection and should be injected within 45 minute after mixing. Following these proportions and procedures, the 14- and 28-day strength of a grout cube (1,000 cm^3) should be at least 18 MPa and 25 MPa, respectively.

Grouting procedures include two main steps: water cracking and shaft grouting. Before shaft grouting, the manchette is first cracked by applying water pressure to make flow paths in the concrete cover (Figure 2(a)). The process comprises lowering the double packer into the grouting tube to the lowest manchette position. The packer is then inflated to seal the portion of the grouting tube (Figure 2(b)). Water is then pumped with sufficient pressure through the tube to cause the concrete cover to crack and, thus, open flow paths for the grout. (Note that this procedure should be implemented within 24 hours after concreting to avoid concrete attaining too high strength). After the flow paths in the concrete cover had been established, the grouting procedure, with the use of cement grout, was implemented using the same process as used for the cracking of the concrete cover. This procedure was sequentially repeated from the lowest to the highest manchette positions. To illustrate the process, Figures 3(a) and 3(b) show photos of shaft grouting performed at Empire City and Friendship Tower projects in HCM city.

Pile sections to be grouted depend mainly on the stiffness of soil layers: the higher the soil stiffness, the better the shaft resistance enhanced. The upper layers, which are typically soft clays or loose sands, are therefore often not considered in shaft grouting.

In current practice (no standard exists yet), the target grout volume per manchette shall be 35 litres/m^2 of the surface area of the pile. If the target grout volume is not achieved, then, the manchette above, below, or to the sides shall be injected with additional grout, if necessary, in order to ensure that the minimum average grout take over the shaft grout zone is greater than 25 litres/m^2. The grouting at a manchette is stopped when the target volume or the maximum pressure (typically 4.0 MPa) of the system is reached, whichever comes first.

(a) (b)

Figure 3 Performance of shaft grouting by FCS: (a) steel cage with grouting tubes; (b) grouting in progress

3. CASE STUDIES

3.1 Project information

Well-instrumented test piles from six high-rise building projects in HCM city were used for the cases included in this study. The project locations and their main foundation information are given in Figure 4 and Table 1, respectively. As shown in Figure 4, all the project locations are along or near Saigon River in central region of the city, which belongs the Saigon-Dongnai River delta. Soil profiles at the project sites are typically characterized as alluvial deposits with alternating sand and clay layers extending to very great depths.

Figure 4 Locations of the study projects

As listed in Table 1, pile toe depths were designed to bear in dense sand at depth of 80 m or more. Among the projects selected, Landmark 81 (81-floor skyscraper), the tallest building in Vietnam (one of top 10 tallest buildings in the world constructed in 2018), is a great project with many records in foundation works. The foundation works of the projects were implemented by well-known international contractors (Bachy Soletance and Bauer) and one domestic (FECON South).

3.2 Instrumented test piles

A total number of fourteen well-instrumented test piles from the six projects were taken into analysis, of which five were barrettes and nine were bored piles as noted in Table 2. All the piles were instrumented with 7 to 12 levels of strain gauges (Geokon vibrating wire sensors) along the pile depths. Additionally, six test piles, marked with § symbol, were instrumented with two to three O-cells at some depth above the pile toes. Among the test piles, ten were shaft-grouted at some sections (i.e., not fully grouted) and the others were plain (not grouted).

As common practice in the country, thin-walled tube sampling method was used to take undisturbed samples of clayey soils for laboratory tests and the SPT was the key test used for obtaining disturbed samples and SPT *N*-index for establishing strength of sandy soils. The SPT *N*-index is therefore the key parameter herein used to analyze the shaft resistance of the test piles. At each test site, a soil profile with SPT *N*-indices adjacent to the test pile was carefully plotted as typically shown in Figure 5 for pile TP2 at Landmark 81 project. Note from Table 1 and Figure 4 that, due to deep alluvial deposits at the sites, the pile toes of the test piles were unable to reach bearing layers such as gravel or soft rock but the piles were terminated in medium to dense sand layers. Figure 6 shows photos from the installation of test pile TN6 (instrumented with three O-cells) at Vinhomes-Bason project.

Table 1 Brief information of foundations of the projects

No	Project name	No. of El. floor	No. of B. floor	Address	Pile type	Pile length (m)	Main soil layers (down to pile toe)
1	Landmark 81	81	3	Binh Thanh	Barrette	83.0 – 88.0	Soft organic silty clay, clayey sand, fine to medium sand, coarse sand
2	Vinhomes Bason	50	3	District 1	Bored/Barrette	60.0 – 69.0	Medium dense sand, stiff to hard clay, dense sand
3	Friendship building	21	4	District 1	Bored	65.0 – 80.0	Sandy clay, clayey sand, silty clayey sand, sand with silt
4	Empire City	35	2	District 2	Bored	60.0 – 62.0	Soft organic clay, silty clay with fine sand, medium dense sand
5	Eximbank building	35	4	District 1	Barrette	60.0 – 82.0	Medium dense sand , hard silty clay, dense to very dense sand
6	German House	25	4	District 1	Bored	70.0 – 80.0	Medium dense clayey sand, hard clay, dense/very dense sand

Note: Foundation works of projects 1 & 2 were carried out by Bachy Soletanche Vietnam; 3 & 4 by FCS; and 5 & 6 by Bauer Vietnam; El. floor = Elevated floor, B. floor = Basement floor.

Table 2 Details of instrumented test piles from the projects

No	Project name	Test pile name	Pile type	Slurry type	Pile length (m)	Diameter/ Size (m)	R_{max}/R_D (MN)	IST/SG	Grouted depths (m)	Curing time (day)	Test date
1	Landmark 81	TP1§	Barrette	Polymer	85.0	1.0×2.8	83.8/32.0	Y(8)/N	-	29	May 20, 2015
		TP2§	Barrette	Polymer	80.0	1.0×2.8	93.2/35.8	Y(12)/Y	37.0-43.0¶	27	May 13, 2015
2	Vinhomes Bason	TN6§	Barrette	Polymer	69.0	0.8×2.8	64.0/30.0	Y(8)/Y	58.0-68.0	22	Feb 27, 2016
3	Friendship Building	TP1	Bored	Polymer	79.0	1.5	30.0/15.0	Y(10)/N	-	32	Jan 30, 2018
		TP2	Bored	Polymer	64.0	1.2	31.5/10.5	Y(10)/Y	41.0-63.0	27	Jan 20, 2018
4	Empire City	TSBP1-MU4	Bored	Bentonite	62.0	1.2	26.0/13.0	Y(9)/Y	34.0-61.0	22	Jun 29, 2017
		TSBP2-MU4	Bored	Bentonite	62.0	1.2	26.0/13.0	Y(9)/Y	34.0-61.0	26	Jul 6, 2017
		TSBP1-MU7	Bored	Bentonite	62.0	1.2	25.0/12.5	Y(7)/Y	34.0-61.0	23	Jul 14, 2017
		TSBP4-MU7	Bored	Bentonite	62.0	1.2	25.0/12.5	Y(7)/Y	34.0-61.0	27	Jul 21, 2017
		TSBP7-MU7	Bored	Bentonite	62.0	1.2	35.0/12.5	Y(7)/Y	34.0-61.0	68	Dec 27, 2017
5	Eximbank Building	TP1§	Barrette	Polymer	65.3	0.8×2.8	59.6/25	Y(8)/Y	26.6-64.3	24	Sep 20, 2013
		TP2§	Barrette	Polymer	85.3	0.8×2.8	65.1/25	Y(10)/Y	65.1-83.7	25	Sep 24, 2013
6	German House	P9§	Bored	Polymer	72.8	1.5	26.4/15.1	Y(11)/N	-	27	Dec 30, 2014
		P40§	Bored	Polymer	79.8	2.0	29.2/20.9	Y(9)/N	-	27	Jan 5, 2015

Note: §The piles instrumented with two to three O-cells at some level above the pile toe; R_{max} = the maximum load applied from static load test; R_D = design (allowable) load; IST = instrumented (strain gauges were installed along the steel cage of the pile); SG = Shaft-grouted; Y = Yes, the number in the parentheses indicate number of strain gauge levels; N = No; ¶ three more sections at depths 50.0 - 57.0 m, 58.3 - 64.5 m, and 71.0 - 76.0 m were also grouted.

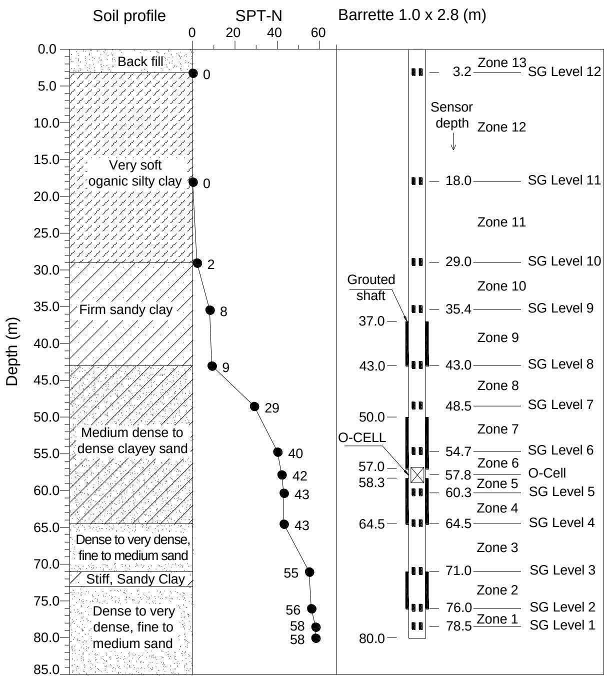


Figure 5 Soil profile and test pile TP2 at Landmark 81 project

Figure 6 Installation of the instrumented barrette (TN6) at Vinhomes - Bason project

4. SHAFT RESISTANCE

4.1 Mobilization of shaft resistance

For strain-gauge instrumented piles the full mobilization at any strain gauge records is best evaluated from the tangent-stiffness curve ($\Delta Q_{PH}/\Delta\varepsilon$) versus strain ($\varepsilon$) (Fellenius 1989, 1991) as typically shown in Figure 7, where Q_{PH} is the applied load at the pile head. The soil at the strain gauge is considered fully mobilized, i.e., ultimate shaft resistance reached, when the tangent stiffness approaches a linear line at large strain. To determine r_u-value at a fully mobilized level, a curve of mobilized shaft resistance (r_s) versus the applied load was constructed, in which the r_s value was estimated as follows:

$$r_s = \frac{\Delta Q_{SG}}{\Delta A_c} \tag{1}$$

where ΔQ_{SG} is load increment in a unit pile length that houses the strain gauge (SG). Herein, ΔQ_{SG} was obtained from the load distribution curve along depth resulted from the head-down test; and ΔA_c is unit circumferential area of the pile at the strain gauge level. The r_u was then determined as the peak or stabilized r_s value of the r_s vs. Q_{PH} curve.

For piles additionally instrumented with O-cells, the full mobilization at a strain gauge level can directly be examined from the *t-z* curve since the displacement (z) at the gauge is obtained from the experiment. Alternatively, the r_u value at a gauge level can also be determined from the mobilized shaft resistance (r_s) (by Eq. 1) versus the induced load (Q_{SG}) as typically shown in Figure 8 for pile TP2 of Landmark 81 project (Figure 5). The soil is considered fully mobilized when the r_s-Q_{SG} curve shows a peak (for dense sands and over consolidated (OC) clays) or when the mobilized shaft resistance (r_s) value becomes relatively stabilized at large load (for loose sands and normally consolidated (NC) clays). Accordingly, the soils at the strain gauge levels shown in Figure 8 were fully mobilized and the r_u values were determined as the peak or stabilized r_s values of the curves.

Following the procedures discussed above, soils at 70 out of 125 strain gauge levels (of 14 piles in total) were fully mobilized. Among these, results at 5 gauge levels at or very near ground surface were excluded because the SPT N and vertical effective stress (σ'_{v0}) values at these levels were too small to make any reliable correlations.

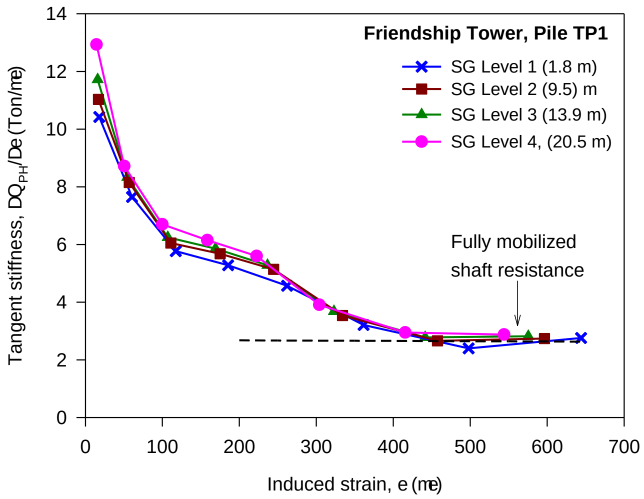


Figure 7 Indication of fully mobilized shaft resistance at strain gauge levels determined from tangent stiffness

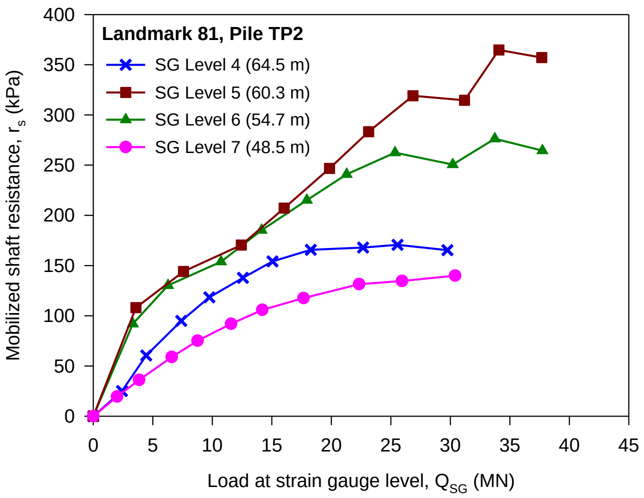


Figure 8 Mobilized shaft resistance at some strain gauge levels of pile TP2 instrumented with O-cells

Thus, a total of 65 fully mobilized data points were finally taken into analysis, in which soils at 23 and 42 levels were classified as clayey soil and sandy soil, respectively. Among the 65 fully mobilized data points, soils at 21 strain gauge levels were grouted and 44 strain gauge levels were not grouted.

4.2 Analysis results and discussion

As stated previously, the SPT *N*-index was used to evaluate strength parameters of soils at the projects. In general, the ultimate (unit) shaft resistance (r_u) is often correlated with the SPT *N*-index according to the following simple form.

$$r_u(kPa) = kN_{60} \tag{2}$$

where k = correlation factor; N_{60} = corrected *N*-index for 60% efficient energy and other factors (Skempton 1986). Note using the uncorrected *N*-index in Eq. (2), i.e., r_u (kPa) = kN, (Phan and Pham 2013, Sze and Chan 2012) is not encouraged, because the raw index (N) contains characteristics of local equipment and testing procedures and thus such a correlation is incomparable with others from different SPT performances. Figure 9 shows a correlation between N_{60} and r_u for all grouted and not grouted data points of clayey soil. For the not grouted data points, k varies significantly from 3.0 to 20.0 but can roughly be divided into two groups based on the N_{60}-indices. For very soft to soft clays, $N_{60} < 8$ (Clayton 1993), the factor is almost 20. For firm to hard clays ($N_{60} > 8$), the factor ranged mainly from 3.0 to 8.0, giving an average value of 4.8.

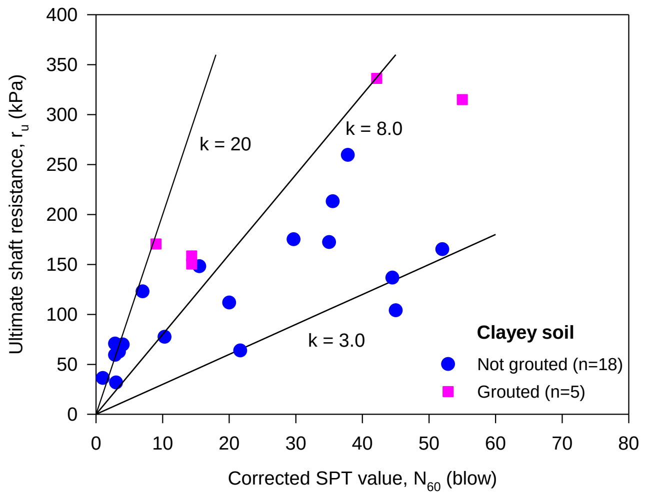


Figure 9 Correlation of r_u and N_{60} for clayey soil

The factor of grouted data points (n = 5) in firm to hard clays varies from 6.0 to 18.0, giving an average value of 10.0. Although more data points in clayey soil are needed to make strong correlations, the data set herein suggested that in firm to hard clays the r_u-value of grouted piles would roughly be two times larger than that of not grouted piles. Similar to Figure 9, Figure 10 shows a correlation between N_{60} and r_u for all grouted and not grouted data points of sandy soil. As shown, the grouted and not grouted data points fall into two distinct groups. For data points from not grouted piles, the factor ranges narrowly from 2.0 to 4.5, resulting in an average of 3.6. For data points from grouted piles, the factor ranges from 4.5 to 12.0 with an average of 6.8. On average, for sandy soil, the r_u-value of grouted piles would also be about twice that of not grouted piles. The k values obtained from this study are compared with those from some other studies as shown in Table 3.

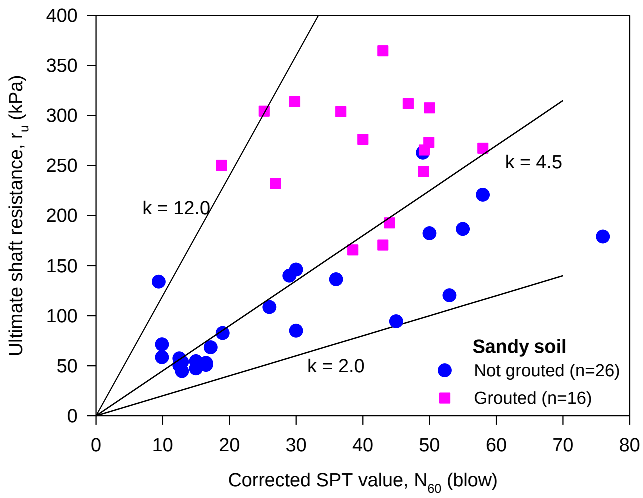


Figure 10 Correlation of r_u and N_{60} for sandy soil

It is interesting to examine how the effective stress method (the β–method) works for the sandy soil in this study. In this method, the r_u-value is proportional to the vertical effective stress (σ'_{v0}) in the following form:

$$r_u = \beta\sigma'_{v0} \tag{3}$$

where β is the proportionality coefficient of shaft resistance.

Table 3 A summary of some correlations between SPT N and ru values

No.	Reference	Location	Soil type	*k* value (avg)		$(r_{u,G}/r_{u,N})_{avg}$
				Not grouted	Grouted	
1	This study	HCM City	Sandy soil	2.0 – 4.5 (3.6)	4.5 – 12.0 (6.8)	1.9
			Clayey soil	3.0 – 8.0 (4.5)	6.0 – 18.0 (10)	2.2
2	Phan and Pham (2013)	HCM City	Sandy soil	2.2 – 9.7 (5.0)	4.4 – 9.7 (8.0)	1.6
			Clayey soil	2.3 – 6.7 (4.7)	4.6 – 8.8 (8.0)	1.7
3	Sze and Chan (2012)	Hong Kong	Alluvial deposits	0.8 – 1.0	3.5 – 6.0	-
			Complete decomposed granite	0.8 – 1.2	2.5	-
			Weathered sedimentary rocks	0.6 – 1.2	1.6 – 5.0	-
4	Littlechild et al. (1998)	Bangkok city	Sandy soil	1.3 – 4.2 (2.7)	2.7 – 7.6 (4.9)	2.0
			Clayey soil	1.6 – 4.9 (4.2)	4.2 – 8.9 (6.6)	2.0

Note: $(r_{u,G}/r_{u,N})_{avg}$ = the average resistance ratio of grouted over not grouted; $r_u = kN_{60}$ for this study and $r_u = kN$ for the others

A correlation between σ'_{v0} and r_u and for the fully mobilized data points in sandy soil is shown in Figure 11. For the not grouted piles β ranges mostly from 0.15 to 0.38 with an average value of 0.25, whereas for the grouted shafts the value varied from 0.35 to 0.9, with an average value of 0.53, N.B., the analyses assume that the grouting resulted in no change of pile circumference. Similar to the change of *k* value, the average β value from grouted shafts is roughly two times larger than that from not grouted shafts.

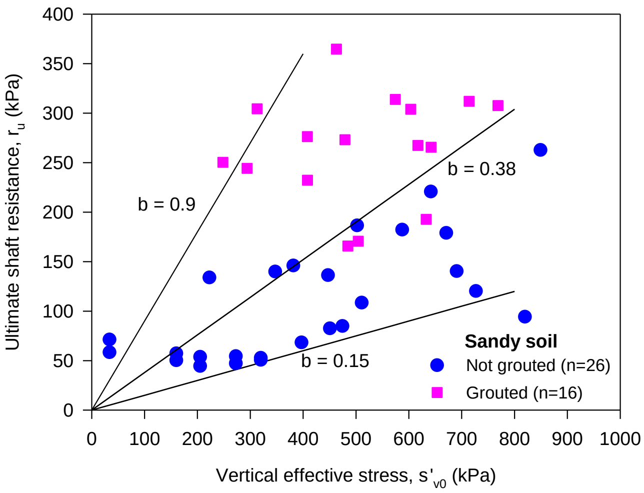


Figure 11 Correlation between r_u and σ'_{v0} for sandy soil (Eqs. 3 and 4)

It is also interesting to see how the experimental r_u-values obtained from the test piles compared with those estimated from the β–method (the effective stress method). Theoretically, the β–coefficient can be estimated by the following equation:

$$\beta = K\tan\delta = \left[\left(1-\sin\phi'\right)OCR^{\sin\phi'}\right]\tan\delta \qquad (4)$$

where K is coefficient of earth pressure; OCR (= σ'_p/σ'_{v0}) is the overconsolidation ratio of soil; ϕ' is effective friction angle of soil and is evaluated as ϕ' = 20+[15.4$(N_1)_{60}$]$^{0.5}$ (Mayne et al. 2001); and δ friction angle between soil and pile surface. The preconsolidation stress (σ'_p) of soil can be estimated as (Brown et al. 2010):

$$\sigma'_p = 0.47\,p_a\left(N_{60}\right)^m \qquad (5)$$

where p_a =100 kPa is the atmospheric pressure and the exponent m is 0.6 for clean quartzitic sand and 0.8 for silty sands.
Typical values of m = 0.8 and δ = 0.75ϕ′ were selected to evaluate σ'_p, β, and then the r_u using Eqs. (3) to (5). The experimental and estimated r_u-values in the sandy soil were then correlated as shown in Figure 12. The figure indicates that although the correlation is somewhat scattered, the estimated r_u-values compare well with the experimental values. Figure 13 shows correlations between σ'_{v0} and estimated r_u as well as experimental r_u. It is interesting to note that the estimated values fall well into the range experimental values (β = 0.15 to 0.38), resulting an average of 0.28.

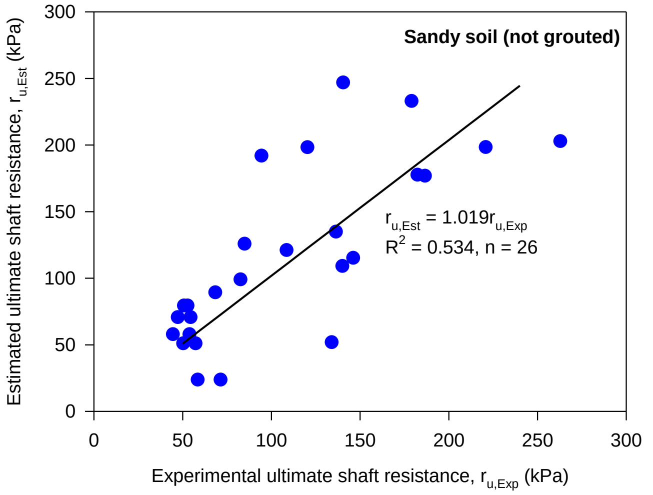


Figure 12 Correlation between experimental and estimated r_u values for not grouted sandy soil

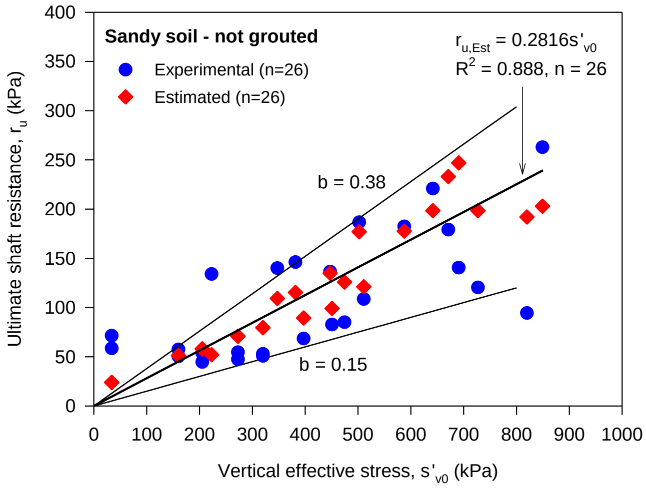


Figure 13 A comparison of correlations between σ'_{v0} and experimental and estimated r_u

5. CONCLUSIONS

This paper first briefly presents latest shaft grouting technique applied to bored piles and barrettes in HCM city, where the piles in central areas are often installed to very great depths. A database of head down and bidirectional tests on fourteen well-instrumented grouted and not grouted bored piles and barrettes was then analysed to evaluate the enhancement of shaft grouting. For this, correlations between the ultimate unit shaft resistance (r_u) and the SPT N_{60}-indices (i.e., $r_u = kN_{60}$) were analysed. Some key conclusions from the study can be drawn as follows: (1) for not grouted piles in soft clays ($N_{60} < 8$), the factor k is as large as 20, however in firm to hard clays ($N_{60} > 8$) the factor varies mainly from 3.0 to 8.0, resulting an average value of 4.8. For grouted piles in firm to hard clays, the factor varies from 6.0 to 18.0, giving an average value of 10.0. It is thus concluded that in firm to hard clays the r_u value of grouted piles would roughly be two times larger than that of not grouted piles; (2) For not grouted piles in sandy soil, the factor varies narrowly from 2.0 to 4.5, resulting in an average value of 3.6. For grouted piles in the same soil type, the factor varies from 4.5 to 12.0 with an average value of 6.8. Thus, it can be concluded that for sandy soil, the r_u value of grouted piles would also roughly be two times larger than that of not grouted piles; These findings (i.e., grouted r_u is about two times larger than not grouted r_u for both clayey and sandy soils) are similar to that obtained from the study of Littlechild et al. (1998) for Bangkok soil; (3) it is also found that the estimated r_u values obtained from β method recommended in Brown et al. (2010) are well comparable with the values obtained from the instrumented piles.

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