Horizontal stress change of energy piles subjected to thermal cycles in sand

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1 **Abstract**

2 Horizontal stress changes of semi-floating energy piles subjected to cyclic thermal loading
3 are investigated through finite element analysis adopting the hypoplastic model. By using
4 the validated finite element model a parametric study is carried out, considering effects of
5 amplitude of thermal cycles, pile diameter, pile length and relative density of sand. It is
6 revealed that due to volumetric contraction of sand (loose or dense) at the interface under
7 temperature induced cyclic shearing, a reduction of horizontal stress occurs. The degree of
8 reduction in horizontal stress is most affected by the amplitude of thermal cycles and the
9 pile diameter.

10 **Keywords:** energy piles; cyclic; horizontal stress; sand.

1. Introduction

Energy piles have been used to exploit the shallow geothermal energy and gained growing popularity over the past few decades [1-3]. Due to some uncertainties related to the thermo-mechanical performance of energy piles, increasingly more investigations have been carried out. Centrifuge modelling of energy piles in saturated sand [4] and unsaturated silt [5] show that antecedent thermal heating improves the ultimate bearing capacity of energy piles. Full-scale field tests in sand carried out by Wang et al. [6] show that antecedent heating improves the shaft resistance of energy piles. While centrifuge test results from Goode and McCartney [5] show almost no effect of heating on the bearing capacity of energy piles in dry sand. Thermo-elastic numerical analysis carried out by Olgun et al. [7] shows that radial expansion of energy pile due to monotonic heating contributes limited increase in horizontal stress.

For normal operating conditions, however, energy piles are subjected to thermal cycles rather than monotonic heating [1, 9, 11]. Cumulative settlement behaviour was reported for semi-floating energy piles under thermal cycles experimentally [8-10] and analytically by modified load transfer method [11-12]. For end-bearing piles, thermal cycles only have slight effect on the pile head displacement according to test results from Stewart and McCartney [13].

One possible reason for the cumulative settlement of semi-floating energy piles is that when energy piles experience temperature cycles there are repeated pile deformations in both horizontal and vertical directions due to thermal expansion and contraction. This imposes cyclic compression, extension and shearing on soil at the interface, which can result in volumetric contraction of the soil [14]. Therefore, reduction of horizontal stress and shaft resistance is induced according to Boulon and Foray [15]. Consequently, additional pile settlement occurs to further mobilize the base resistance to compensate for the shaft resistance reduction. This change of horizontal stress may be estimated by equation (1) proposed by Boulon and Foray [15]

$$\Delta\sigma_h = 4\delta_h E_p / D \quad (1)$$

where δ_h is the change in thickness of the interface soil layer, E_p is the pressuremeter modulus of soil and D is the pile diameter. This equation is derived based on the elastic cavity expansion theory by assuming that the far-field soil behaves elastically. It can be readily deduced that any volume change (elastic or plastic) at the interface soil layer can result in a variation of horizontal stress. When used in practice, an effective pressuremeter modulus should be chosen to account for the stiffness degradation with strain.

The objectives of this study are (1) to validate the above discussed mechanism; (2) to quantify the horizontal stress change of semi-floating energy piles in sand subjected to thermal cycles

47 via finite element analysis.

48 **2. Numerical parametric study plan**

49 In this study, bored concrete piles embedded in dry Toyoura sand are simulated as
50 wished-in-place piles. Two series of numerical simulations were performed to investigate
51 effects of the amplitude of thermal cycles (ΔT) and the pile diameter (D). The amplitude of
52 thermal cycles and pile diameter affect the magnitude of pile deformations, and thus
53 volumetric contraction of soil at the interface [14]. For both series, three different relative
54 densities (D_r) are considered to simulate loose, medium-dense and dense sand, respectively.

55 To investigate the effects of pile length, another two series (i.e. Series 3 and 4) of numerical
56 analyses on pile length, ranging from 20 m to 35 m, were carried out. For Series 3, the pile
57 diameter is kept the same at 1.0 m; whereas for Series 4, the pile diameter is also increased
58 from 1.0 m to 1.75 m to maintain the ratio of pile length to pile diameter (L / D) equal to 20.

59 For all the cases analyzed, the overall factor of safety is taken as 2.0 according to the failure
60 criterion proposed by Ng et al. [16]. Full reversal thermal cycles ($\pm \Delta T$) are applied with
61 respect to the reference temperature. Details of the parametric study are listed in Table 1.

62 **3. Numerical modelling methodology**

63 **3.1. Constitutive models and model parameters**

64 In order to simulate the above discussed mechanism, primary requirement of the soil
65 constitutive model is to predict the volume change of soil under cyclic shearing. To achieve
66 this purpose, the incrementally nonlinear hypoplastic model for sand developed by Niemunis
67 and Herle [17] is adopted. Niemunis and Herle refined von Wolffersdorff's model [18] by
68 introducing a small elastic range to improve the simulation of sand behaviour under cyclic
69 loading. There is no explicitly defined yield surface for the hypoplastic model, and the
70 state-dependent dilatancy of sand behaviour can be well simulated. Niemunis and Herle's
71 model needs a total of 13 parameters. All the model parameters and their definitions are listed
72 in Table 2. Model parameters for Toyoura sand are based on the calibration by Herle et al. [19]
73 and Ng et al. [20].

74 It should be noted that the adopted hypoplastic model cannot consider thermal effects on soil
75 [21-23]. This study only focuses on the mechanical aspect of the fully-coupled
76 thermo-mechanical soil-structure interaction problem. Heat flow and thermal effects on soil
77 behaviour are not considered. It is expected that ignoring thermal effects on soil behaviour is
78 likely to underestimate horizontal stress reduction, especially at the first heating-cooling
79 cycle, within which thermally-induced volumetric contraction of sand and normally
80 consolidated clay mainly occurs [21-23].

81 The concrete energy pile itself is assumed to behave thermo-elastically. Typical parameters

for concrete used in this study are in accordance with BS EN 1992-1-2. [24]. Young's modulus, Poisson's ratio, linear coefficient of thermal expansion and density are 3×10^7 kPa, 0.2, 1×10^{-5} /°C and 2500 kg/m³, respectively.

3.2. Verification of soil constitutive model

To verify Niemunis and Herle's model, a series of cyclic torsional simple shear tests on Toyoura sand by Shahnazari [14] was back-analyzed using the finite element software Abaqus. Implementation of Niemunis and Herle's model in Abaqus is realized by Gudehus et al. [25] through the user subroutine UMAT.

Fig. 1 compares the numerical results to the experimental results of sand volume change under cyclic shearing. The horizontal and vertical axes represent the void ratio e and shear strain γ , respectively. All four tests (a-d) show similar trends between numerical and experimental results. Under cyclic shearing, there is a continuous volumetric contraction of sand, but at a reduced rate. Finally, a stabilized state is reached. Quantitatively, tests b and c show relatively good agreement. For test a (loose sand) the numerical simulation overestimates the stabilized void ratio, while for test d (dense sand) there is a slight underestimation. One possible reason for the discrepancies is that the predefined lower limit compression line in the hypoplastic model, which corresponds to the hypoelastic state, may not be well-represented.

100 Fig. 2 compares measured and computed shear stress (τ) - shear strain (γ) responses for test c.
101 Although the measured increase of shear stress with an increasing number of cycles is
102 under-predicted by the model, both of them show that the soil stiffness consistently increases
103 with an increasing number of cycles.

104 Fig. 3 shows the comparison of measured and computed stress ratio (τ / σ_n) versus dilatancy
105 ($- d\epsilon_v^p / d\gamma^p$) responses, where σ_n is the normal stress, $d\epsilon_v^p$ and $d\gamma^p$ are the plastic volumetric
106 strain increment and plastic shear strain increment, respectively. Positive volumetric strain
107 increment means contraction whereas negative is dilation. Regarding the shear strain
108 increment, it is positive for loading and negative for unloading. The computed stress ratio
109 versus dilatancy relationship shows a reasonably good agreement with the experimental
110 results [14]. Upon each reversal of loading direction, computed results show that the
111 dilatancy ratio increases first and then decreases, while immediate decrease in dilatancy ratio
112 was observed. However, this discrepancy should not affect major findings from this study.
113 The reason is what matters most for this study is the ultimate volumetric contraction of sand
114 induced by cyclic shearing. According to Fig. 1, the ultimate volumetric contraction is
115 simulated reasonably well.

116 3.3. Finite element mesh

117 Fig. 4 shows the dimensions and typical finite element mesh of the model domain. The

problem is modelled as axisymmetric. 4-node bilinear element CAX4 is adopted for both pile and soil with the aspect ratio of mesh elements around 1.0. As can be seen from the figure, the mesh adjacent to the pile (up to $2D$ in the horizontal direction) is refined to capture nonlinear soil response. The density of pile mesh, which is not shown in Fig. 4, is half that of the refined soil mesh. To eliminate possible boundary effects, outer width and height of the model domain are set to be 1 and 2 times of the pile length (L) according to [26]. The vertical displacements on bottom of the model domain and horizontal displacements on two sides of the model domain are constrained [20].

3.4. Pile-soil interaction

The pile-soil interaction is modelled as hard contact in normal direction, and the Coulomb frictional model is used in tangential direction. Two parameters are necessary to characterize the Coulomb frictional model, the friction angle and the allowable elastic slip to fully mobilize the friction angle. For bored concrete piles, the interface friction angle is assumed to be the same as the soil critical state friction angle [27]. According to the interface shear tests of Peng [28], the allowable elastic slip can be taken as 5 mm for rough surface conditions.

3.5. Simulation procedure

In the first step of numerical simulation, the equilibrium state under gravitational load is established with the coefficient of lateral earth pressure taken as 0.5 [20]. In the second step,

axial working load is applied and maintained constant throughout the analysis. The working load is calculated as the ratio of pile ultimate bearing capacity to the factor of safety. An independent run is performed to determine the pile ultimate bearing capacity based on the failure criterion proposed by Ng et al. [16]. In the third step, 50 full reversal ($\pm\Delta T$) heating-cooling cycles at constant amplitude are imposed on the pile. Temperature distribution along pile length is assumed to be uniform [2, 3].

4. Validation of finite element model

As this study focuses on the mechanical aspect of the fully-coupled thermo-mechanical problem, a pile subjected to mechanical cyclic loading was chosen to validate the finite element model. Tokimatsu et al. [29] reported a centrifuge test on a pile subjected to cyclic vertical loading in dry Toyoura sand with g-level of 30 g. The model pile had a diameter of 0.65 m and a length of 7.5 m in prototype. Toyoura sand was pasted on the model pile surface to create a rough surface condition. The relative density of sand was 90%. Displacement - control loading method was adopted. The applied amplitude of pile head displacement was gradually increased (1%, 5%, 10% and 15% of pile diameter) every three loading cycles as shown in Fig. 5(a). The centrifuge test was simulated to validate the finite element model. Fig. 5(b) compares the experimental and numerical results (prototype) of mobilized shaft resistance (Q_s) with the number of loading cycles (N). The mobilized shaft resistance for

154 compression and tension within each cycle are separately presented. It can be seen that
155 although there are some minor discrepancies between the experimental and numerical results,
156 the numerical simulation is able to capture the mobilized shaft resistance in compression as
157 well as in tension. The consistency between simulated and measured results gives confidence
158 that the finite element model developed is appropriate.

159 **5. Results**

160 In this section, energy pile response during thermal cycles is analyzed first, followed by
161 discussion on the horizontal stress change corresponding to the end of 50 thermal cycles. The
162 horizontal stress change under consideration represents the average value along the pile
163 length. In addition, the horizontal stress change is normalized by the initial value and negative
164 values indicate a decrease. The normalization of horizontal stress change is calculated
165 according to equation (2):

$$166 \quad \Delta\sigma_h = [(\sigma_h - \sigma_h^0)/\sigma_h^0] \cdot 100 \quad (2-1)$$

$$167 \quad \Delta\sigma_h^{50} = [(\sigma_h^{50} - \sigma_h^0)/\sigma_h^0] \cdot 100 \quad (2-2)$$

168 where σ_h is the horizontal stress; σ_h^0 and σ_h^{50} are the horizontal stresses before thermal cycles
169 and after 50 thermal cycles, respectively; $\Delta\sigma_h$ is the horizontal stress change at a given
170 thermal cycle, while $\Delta\sigma_h^{50}$ is the horizontal stress change after 50 thermal cycles.

The ultimate pile resistance, working load, ultimate shaft resistance and base resistance, and mobilized shaft resistance and base resistance at working load for all the cases analyzed are summarized in Table 3.

5.1. Energy pile response during thermal cycles

As the underlying mechanism is the same, one typical case is chosen to analyze the energy pile response during thermal cycles. Details of the typical case are $D_r = 85\%$, $D = 1$ m, $L = 20$ m and $\Delta T = 15$ °C.

Fig. 6 shows the change of horizontal stress $\Delta\sigma_h$ and the normalized pile head settlement ω_h against pile diameter D during thermal cycles. It can be seen that within each thermal cycle horizontal stress may increase or decrease due to thermal expansion or contraction of the pile diameter. The overall trend with thermal cycles is a continuous reduction of horizontal stress due to volumetric contraction of sand at the interface under cyclic shearing, resulting in shaft resistance reduction. To compensate for the loss of shaft resistance, additional pile settlement accumulates to further mobilize the base resistance, as demonstrated in Fig. 7(a).

Fig. 7(b) shows the evolution of shaft resistance (Q_S^M) and base resistance (Q_B^M) normalized by the ultimate shaft resistance (Q_S) and ultimate base resistance (Q_B) (see Table 3), respectively. What should be noted from this figure is that the mobilized base resistance continues to increase with thermal cycles. One reason is due to the continuous settlement of

energy piles. The other reason is that sand below the pile base is densified due to cyclic shearing, and thus the base resistance is improved. The second point is further demonstrated in the following section.

It should also be noted that, as thermal cycle continues, the rate of horizontal stress decrease and additional pile settlement reduces. The decrease of horizontal stress approaches a stabilized value and the pile head settlement levels off before the end of 50 thermal cycles as indicated by the asymptotic lines in Fig. 6. The axial working load is compressive in this study. Therefore, as long as the axial working load is less than the base resistance available to be mobilized, eventually the pile response will stabilize at the cost of additional settlement to mobilize the base resistance.

Fig. 8 shows the changes in horizontal stress ($\Delta\sigma_h$) and volumetric strain (ε_v) with thermal cycles (N) for soil elements adjacent to the pile shaft at different depths. Evolution of the average horizontal stress change over entire pile length and the volumetric strain of one soil element below the pile base are also included in this figure. It can be seen that except for the soil element at $Z = 20$ m, all other soil elements adjacent to the pile shaft experience continuous volumetric contraction, but at a reduced rate. Correspondingly, the horizontal stress decreases, but stabilizes at about 2% volumetric contraction. For the soil element at $Z = 20$ m, initially there is some dilation followed by continuous contraction, which leads to

an increase of horizontal stress at the beginning and then a decrease of horizontal stress. The average horizontal stress change over entire pile length reflects the global response and is more close to that of the soil element at $Z = 15$ m.

The soil element below the pile base (denoted by ‘Base’ in Fig. 8) also experiences volumetric contraction, which stabilizes roughly at the 15th thermal cycle. The enhanced base resistance due to the densification of soil below the pile base explains the relatively small pile settlement, despite a large reduction in shaft resistance.

5.2. Horizontal stress decrease after 50 thermal cycles

Fig. 9 shows the effect of amplitude of thermal cycles (ΔT) on the horizontal stress decrease after 50 thermal cycles ($\Delta\sigma_h^{50}$) for three different relative densities. It can be seen that as ΔT increases from 5 °C to 20 °C, $\Delta\sigma_h^{50}$ goes up from about 15% to almost 90%. According to Shahnazari [14], for the same number of cycles, cyclic shearing induced volumetric contraction of sand increases with the shearing amplitude. For energy piles under thermal cycles, the amplitude of cyclic shearing increases with ΔT . Therefore, a larger ΔT results in a larger volumetric contraction, and hence a higher reduction in horizontal stress according to equation (1).

It can also be seen from Fig. 9 that the relationship between $\Delta\sigma_h^{50}$ and ΔT is approximately bilinear. This bilinear trend is similar to the mobilization of active earth pressure with δ_h in

equation (1) being the displacement driving the mobilization. As δ_h exceeds some threshold value, the mobilized active earth pressure remains relatively constant.

Reduction in shaft resistance of piles under cyclic axial loading has been observed both in centrifuge (see Fig. 5) and field tests [30-31]. Poulos [30] reported up to 90% reduction in shaft resistance after 100 two-way loading cycles. Jardine and Standing [31] reported that 26 - 29% reduction in tension capacity occurs at a pile head displacement of 10% of pile diameter.

Fig. 10 presents the effect of pile diameter D on $\Delta\sigma_h^{50}$ for three different relative densities. As shown in the figure, there is some increase of $\Delta\sigma_h^{50}$ as pile diameter increases from 0.6 m to 0.8 m. One reason is that the magnitude of shearing imposed on soil at the interface increases with the pile diameter. The other possible reason is that the percentage of initial mobilized shaft resistance at working load to the ultimate shaft resistance increases with the pile diameter (see Table 3). According to findings from [8, 11, 12], the higher the load ratio, the more significant the influence of thermal cycles on the performance of energy piles. However, increasing the pile diameter larger than 0.8 m has little influence on $\Delta\sigma_h^{50}$. For these cases with pile diameter exceeding 0.8 m, $\Delta\sigma_h^{50}$ lies within the range from 85% to 90%. This trend is consistent with that shown in Fig. 9. Both the amplitude of thermal cycles and pile diameter can influence the amplitude of cyclic shearing imposed on soil at the interface. For

243 those cases with D larger than 0.8 m and ΔT being equal to 15 °C, the value of δ_h may exceed
244 the threshold value after 50 thermal cycles, and thus the mobilized active earth pressure
245 remains relatively constant.

246 Regarding the effect of relative density of sand, Fig. 9 shows that $\Delta\sigma_h^{50}$ is relatively
247 independent of the relative density of sand. Similar trend can also be observed in Fig. 10
248 when the pile diameter is larger than 0.8 m. For cases with pile diameter of 0.6 m, it seems
249 that $\Delta\sigma_h^{50}$ is larger for loose sand. The possible reason may be that for loose sand the shearing
250 amplitude corresponding to $D = 0.6$ m and $\Delta T = 15$ °C is large enough to get δ_h reach the
251 threshold value after 50 cycles. While for dense sand, a larger pile diameter or thermal cycle
252 amplitude is required.

253 Fig. 11 shows the effect of pile length (L) on the horizontal stress decrease after 50 thermal
254 cycles ($\Delta\sigma_h^{50}$). The computed results for $D = 1.0$ m (i.e. Series 3) show that the reduction of
255 horizontal stress is a nonlinear function of pile length. As the pile length increases from 20 m
256 to 35 m, $\Delta\sigma_h^{50}$ decreases from about -90% to -30%. The reason is that soil elements at greater
257 depths experience less volumetric contraction and hence more cycles are required to reach a
258 steady state as shown in Fig. 8. Results from Series 4, whereby the pile diameter is also
259 increased to keep $L / D = 20$, show that $\Delta\sigma_h^{50}$ is almost constant. This is due to that a larger pile
260 diameter leads to a larger magnitude of compression and extension in the horizontal direction

for a given ΔT . Therefore, more volumetric contraction and horizontal stress reduction are induced. This implies that horizontal deformations of energy piles play an important role in the reduction of horizontal stress.

6. Conclusions

This paper reports a numerical investigation of the performance of semi-floating energy piles subjected to thermal cycles in sand. The study focuses on the horizontal stress change resulting from sand response to cyclic shearing caused by repeated thermal expansion and contraction of energy piles. Based on the study presented the following conclusions may be drawn:

(1) During thermal cycles, repeated expansion and contraction of energy piles impose cyclic shearing on soil at the interface, which can induce volumetric contraction and horizontal stress decrease, resulting in shaft resistance reduction. As a consequence, additional settlement occurs to further mobilize the base resistance to balance the shaft resistance reduction. As thermal cycle continues, for energy piles under axial compression, the pile response approaches a stabilized state eventually.

(2) The magnitude of shearing imposed on soil at the interface is governed by the amplitude of thermal cycles and the pile diameter. For the worst case, the horizontal stress reduction after 50 thermal cycles may be up to 90%. For the range of pile length studied (from 20 m

to 35 m), the horizontal stress reduction after 50 thermal cycles is relatively independent of the pile length at the given ratio of pile length to pile diameter of 20. In addition, the stiffening of soil below the pile base can contribute to reduce the additional pile settlement.

(3) For practical design of energy piles in sand, the shaft resistance reduction due to horizontal stress decrease should be considered. In addition, the resultant additional settlement to further mobilize the base resistance should also be checked.

Acknowledgements

The financial support provided by the Research Grants Council (RGC) of the HKSAR (grant no. GRF 617213 and 16209415), the HKUST (grant no. FP204) and the National Science Foundation of China (grant no. 51378178) are gratefully acknowledged. The second author appreciates greatly the HKPFS scholarship offered by the RGC.

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Table 1. A summary of the parametric study

Series No.	Factor studied	L (m)	D (m)	ΔT (°C)	D_r
1	ΔT	20	1.0	From 5 to 20 at the interval of 2.5	25%, 55%, 85%
2	D	20	From 0.6 to 1.4 at the interval of 0.2	15	25%, 55%, 85%
3	L	From 20 to 35	1.0	15	55%
4		at the interval of 5	$L / D = 20$		

Table 2. Parameters of the hypoplastic model and their values for Toyoura sand

	Parameter definition	Parameter symbol	Parameter value
Parameters included in von Wolffersdorff's model	Critical state friction angle	φ_c'	31°
	Extrapolated maximum void ratio at vanishing pressure	e_{i0}	1.10
	Extrapolated minimum void ratio at vanishing pressure	e_{d0}	0.61
	Extrapolated critical state void ratio at vanishing pressure	e_{c0}	0.98
	Parameters controlling the isotropic compression line	h_s	2.6 GPa
		n	0.27
	Parameter affecting the peak strength ratio	α	0.14
	Parameter affecting the incremental stiffness	β	1.1
Additional parameters for Niemunis and Herle's model	Parameter controlling the initial shear stiffness upon 180° reversal of strain path	m_R	11
	Parameter controlling the initial shear stiffness upon 90° reversal of strain path	m_T	6
	Size of the inter-granular strain	R	2×10^{-5}
	Parameter controlling the evolution of inter-granular strain	β_r	0.1
	Parameter controlling the stiffness degradation with strain	χ	1.0

Table 3. A summary of ultimate pile resistance, working load and mobilized shaft resistance and base resistance at working load

L (m)	D (m)	D_r	Q (kN)	P (kN)	Q_s (kN)	Q_B (kN)	$(Q_s)^P$ (kN)	$(Q_B)^P$ (kN)	$(Q_s)^P / Q_s$ (%)	$(Q_B)^P / Q_B$ (%)	$(Q_s)^P / P$ (%)	$(Q_B)^P / P$ (%)
20	0.6	25%	1663	832	1444	219	779	53	54	24	94	6
	0.8		2343	1172	1950	393	1077	95	55	24	92	8
	1.0		3093	1547	2460	633	1416	131	58	21	92	8
	1.2		3903	1951	2977	926	1843	108	62	12	94	6
	1.4		4791	2395	3500	1291	2168	227	62	18	91	9
	0.6	55%	1972	986	1589	383	895	91	56	24	91	9
	0.8		2816	1408	2134	682	1273	135	60	20	90	10
	1.0		3765	1883	2685	1080	1705	178	63	16	91	9
	1.2		4807	2404	3247	1560	2146	258	66	17	89	11
	1.4		5963	2982	3818	2145	2662	320	70	15	89	11
	0.6	85%	2425	1212	1779	646	1058	154	59	24	87	13
	0.8		3543	1772	2367	1176	1557	215	66	18	88	12
	1.0		4823	2412	2966	1857	2057	355	69	19	85	15
	1.2		6270	3135	3582	2688	2629	506	73	19	84	16
	1.4		7890	3945	4201	3689	3290	655	78	18	83	17
25			5408	2704	4177	1231	2513	191	60	16	93	7
30	1.0	55%	7365	3683	5987	1378	3454	229	58	17	94	6
35			9641	4820	8134	1507	4640	180	57	12	96	4
25	1.25		7201	3600	5262	1939	3340	260	63	13	93	7
30	1.5	55%	12246	6123	9110	3136	5627	496	62	16	92	8
35	1.75		19207	9603	14473	4734	8921	682	62	14	93	7

Notes: L – Pile length; D – Pile diameter; D_r – Relative density; Q – Ultimate pile resistance; P – Working load; Q_s – Ultimate shaft resistance; Q_B – Ultimate base resistance; $(Q_s)^P$ – Mobilized shaft resistance at working load; $(Q_B)^P$ – Mobilized base resistance at working load.

Figure 1

Figure 2

Figure 3

Figure 4

Figure 5

Figure 6

Figure 7

Figure 8

Figure 9

Figure 10

Figure 11