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Research Paper

Geotechnical Study of Soil Foundation and Design of PCC Piles to Improve its Bearing Capacity, A case study: Ramps of Dawar El-Tawheed Bridge in Jizan City - kSA

Ali H. Mahfouz

Associate Professor, Geological and Geophysical Engineering Department, Faculty of Petroleum and Mining Engineering, Suez Canal University, 43721, Suez-Egypt

Associate Professor, Civil Engineering Department, Engineering College, Jazan University, 45142, Jazan-KSA

ABSTRACT

Construction of Dawar El-Tawheed bridge project in Jazan –Kingdom of Saudi Arabia is national and very important project for developing jazan. The soil profile of soft clayey silt is sandwiched between two layers of silty sand and located between 5.0 m to 11.0 m in depth. It has high water content, low SPT no., and low bearing capacity. Hence the clay layer induces high settlement due to surcharge application of earth embankment at ramp T1, ramp T2, and ramp T3 especially at heights from 9 m right 3 m and calculated settlement for embankment less than 3 m heights are accepted regarding Saudi Code for soil and foundation. The soil and groundwater have high content of sulfates and chlorides at the project site. It is strongly recommended to increase the bearing capacity of the foundation soil by using new technology named PCC piles that has economically advantageous and high workability. The technology is cast in-situ thin wall concrete pipe piles (PCC piles) that can save time of implementation and application cost of about 30% of other method of soil improvement. PCC piles may increase the bearing capacity to be 294 kN/m^2, 243 kN/m^2, and 242 kN/m^2 for ramp T1, ramp T2, and ramp T3 respectively.

Key words: clayey silt soil, bearing capacity, ramps settlement, soil improvement, geogrid, PCC piles.

1. INTRODUCTION

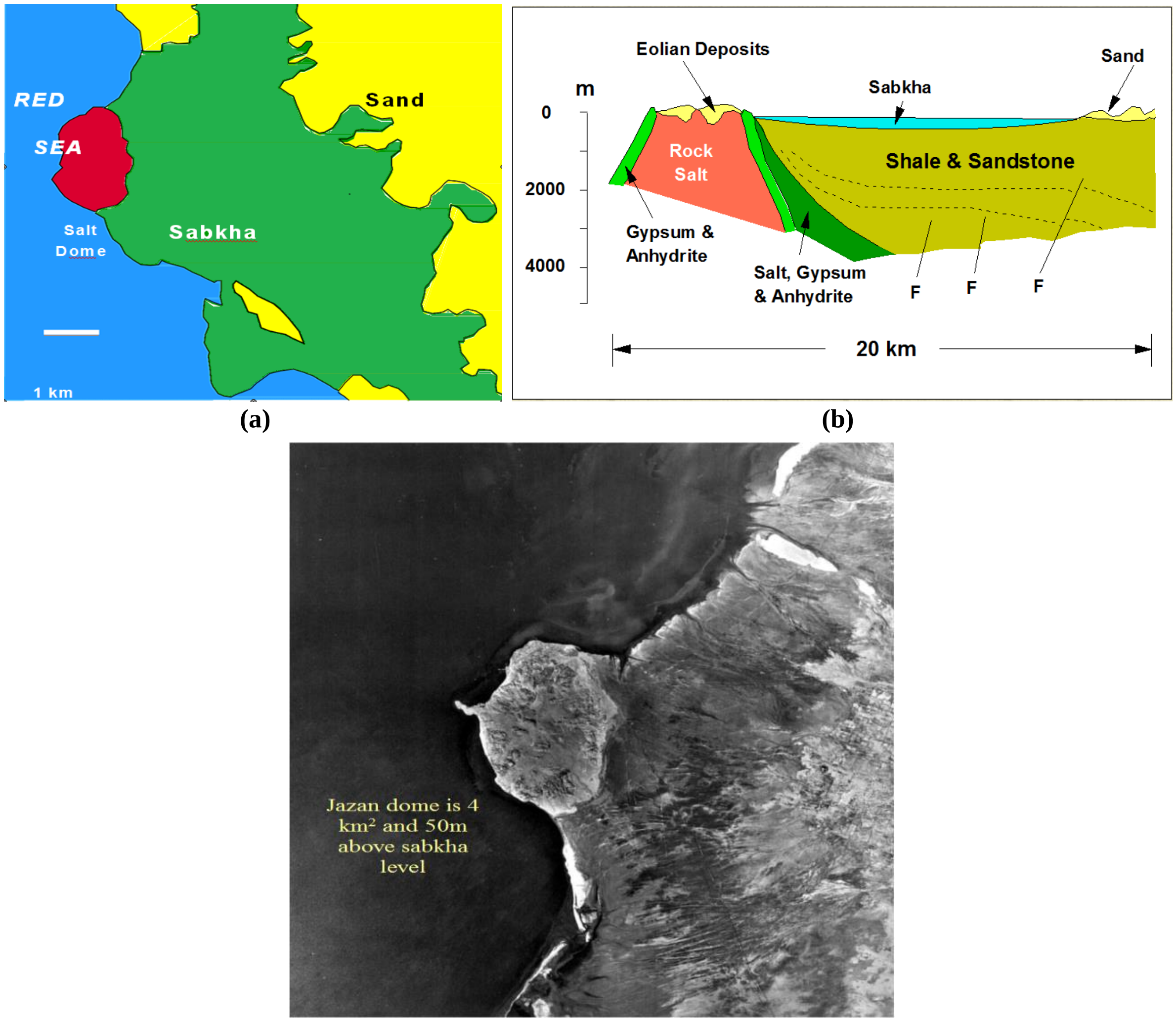
Geotechnical study is required for any engineering or building structure. The investigation may range from simple examination of the surface soils with or without a few shallow trial pits, to detailed study of the soil and ground water conditions by means of boreholes and in-situ and laboratory tests.

Geology is an important role in most areas of economic life, where the development of societies based on the branches of the science applications in life and become contributing to many services. Jazan region is located in the south west part of Saudi Arabia on the red sea (E: 42.0°-43.8° and N: 16.5°-17.0°). It`s area is 13,500 km2. Jazan region is part of Arabia shield which is a part of the Precambrian crustal plate and consists of igneous and metamorphic rocks. It is located in an active zone of earthquakes classified as zone 2B. One of the major problems in geotechnical earthquake engineering is the phenomenon of liquefaction of loose to medium-dense sands below the water table.

The dominant rocks are granite, basalts, diorite, gabbro and mica-schist. During the Tertiary period, the shield was separated from the adjacent African shield by a rift of earth`s crust that currently occupied by the red sea. Sedimentary coastal plain has formed on the area between the escarpment of the shield and the red sea. The climate of the Jazan region is considered arid with annual mean temperature 28°C, relative humidity 62% and annual precipitation 62 mm.

The landforms, developed in Jazan region, are mainly of alluvial nature, formed as a result of the downward transportation of soil material from the highlands by the many valleys and drainage channels that drain out in the sea. Moreover, Jazan embodies variant landforms such as marshland, coastal plain, alluvial plain and valleys.

The geotechnical aspects of Jazan soil were studied by many researches as Erol (1989), Shehata (1989), Al-Amoudi (1992), and Al-Amoudi et al. (1992). The city of Jazan is situated on an elevated terrain underlain by a salt dome measuring 4 km^2 in area and reaching about 50 m above sea level, (see Fig. 1).



(a) (b)

(c)

Fig. 1: (a) Simplified Geological Map, (b) Geologic section across Jazan [modified after Londry, 1979)] (c) Map of salt dome in Jazan city

2. BOREHOLES LOGGING

2.1. Project Description

The project is biggest national project in Jizan city- KSA is being solving traffic problem of crowding at different crosses there, see Fig. 2 which comprise the location map of DWAR ELTAWHEED bridge and its three ramps with layout of boreholes - sampling taken for field and laboratory tests. A 12- boreholes have be done at the ramps of Dwar El-Tawheed bridge project as 4 boreholes each ramp as depicting in location map, boreholes logs are shown in Fig. 3. Ramps dimensions are 17 m in width and 252 in length for ramp T1, 17 m in width and 235 m in length for ramp T2, and 6 m in width and 147 m in length for ramp T3. The maximum height of each ramp should be 9.0 m above the ground surface.

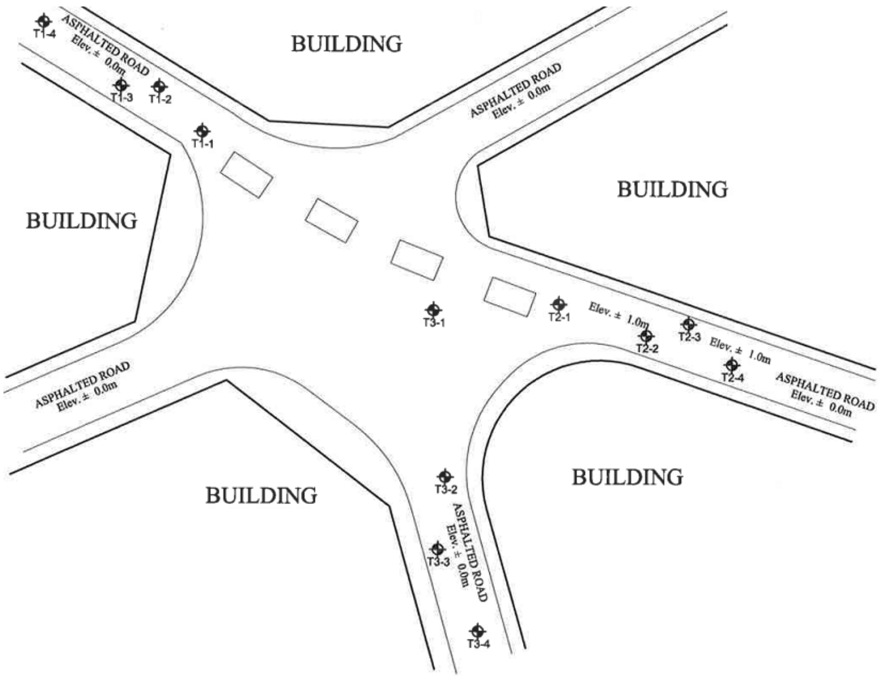
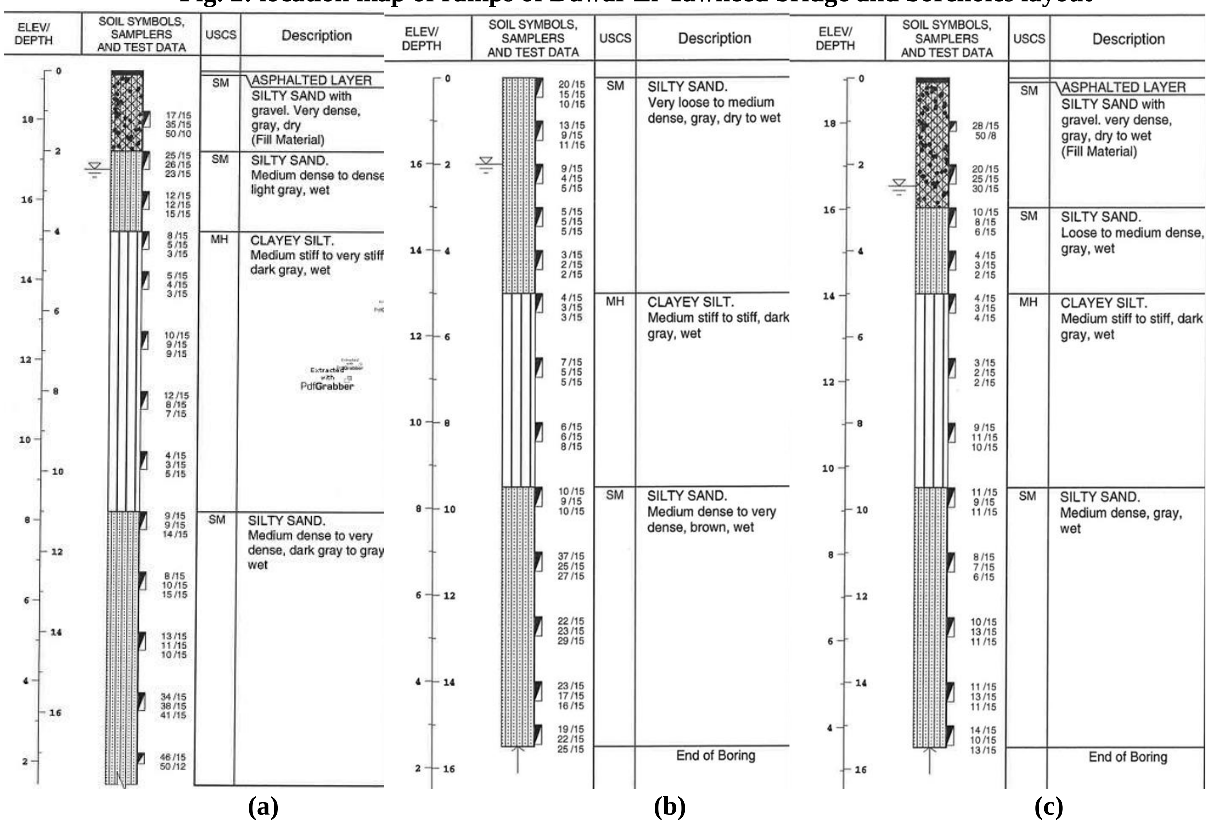


Fig. 2: location map of ramps of Dawar El-Tawheed bridge and boreholes layout



(a) (b) (c)

Fig. 3: borehole log at (a) T1-1, (b) T2-2, and (c) T3-2

3. FIELD AND LABORATORY TESTS

3.1. Field Tests

3.1.1. Natural Water Content

Natural water content (Wc %) has calculated at deferent depths of all boreholes for ramps T1, T2, and T3. Results are shown in Fig 4.

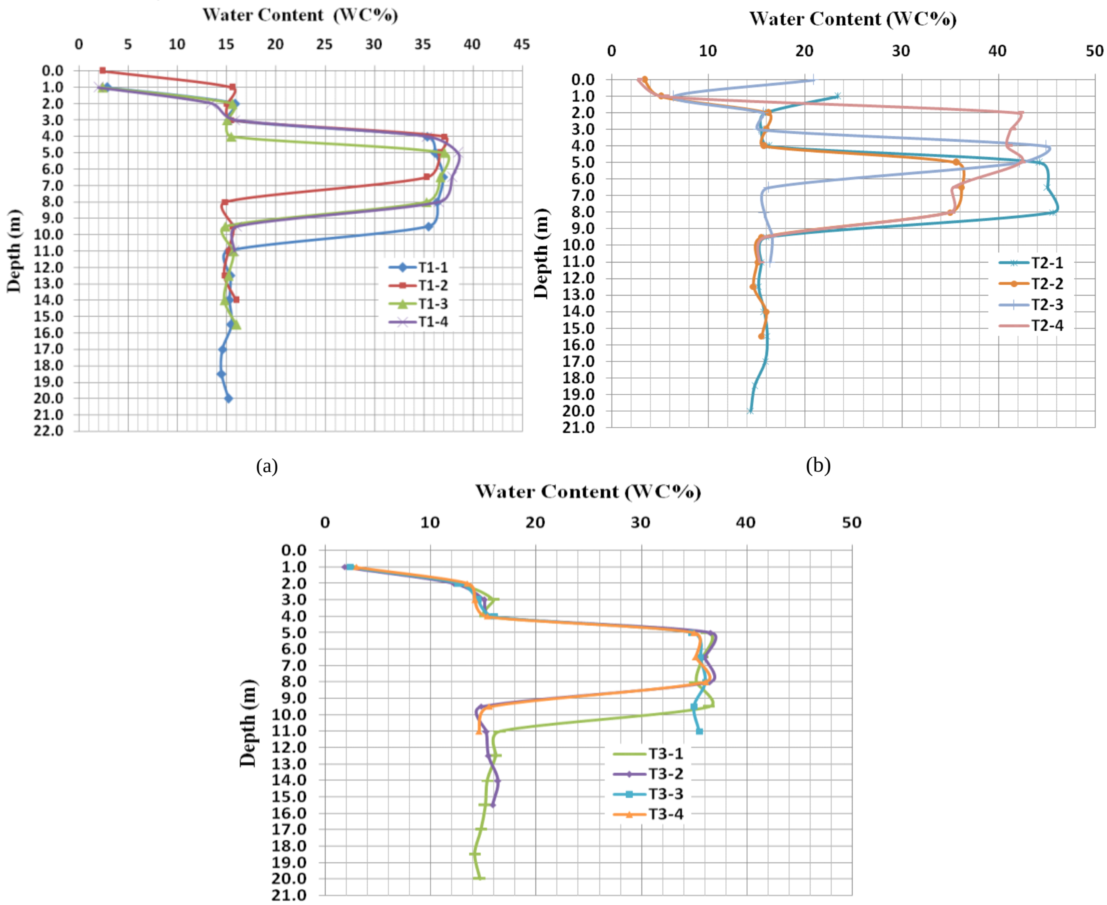


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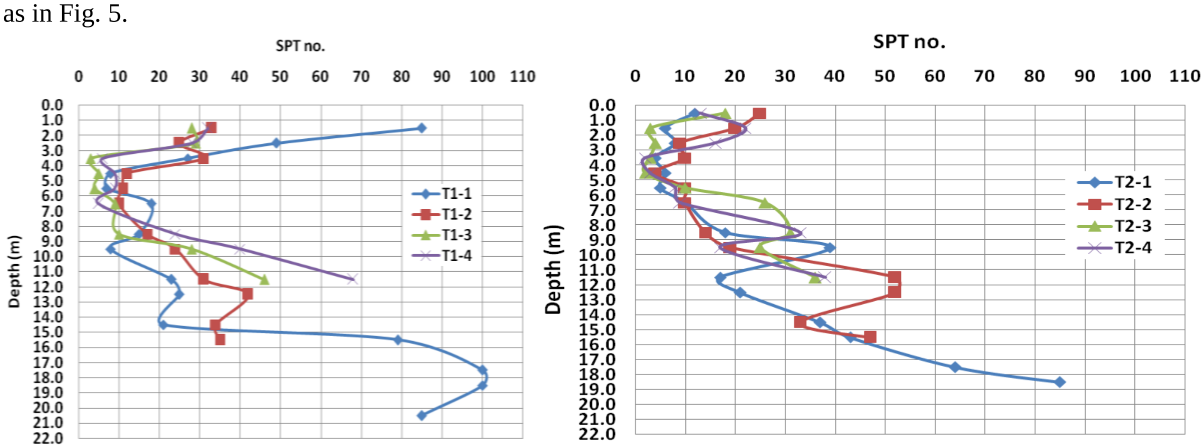
Fig. 4: Water content with depth at: (a) Ramp no. T1 (b) Ramp T2, and (c) Ramp T3

3.1.2. Standard Penetration Test (SPT)

This test has been carried out on all layers of soil at all boreholes with different depths, the results are shown as in Fig. 5.



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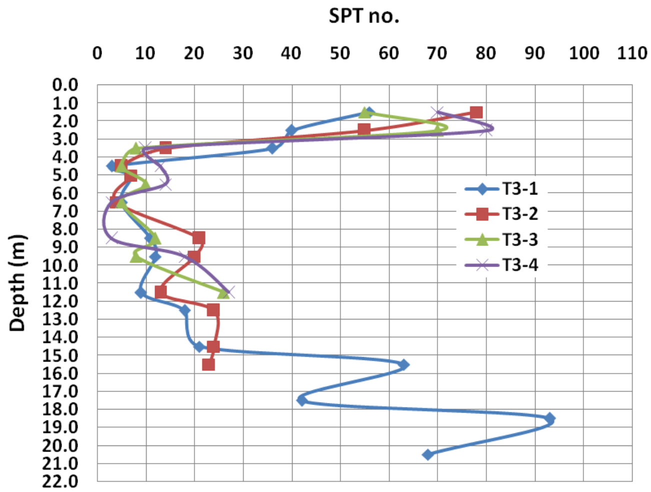


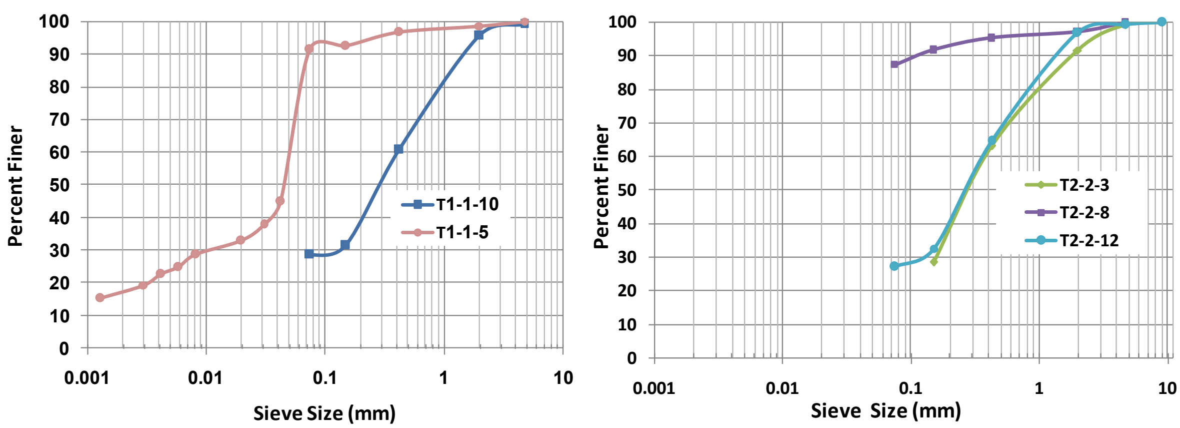
(c)

Fig. 5: Standard Penetration no. Vs. Depth at: (a) Ramp T1, (b) Ramp T2, and (c) Ramp T3

3.2. Laboratory Tests

3.2.1. Mechanical Sieve Analysis

Mechanical sieve analysis for all samples collected from boreholes at different depths of ramps T1, T2, and T3 are shown in Fig. 6.



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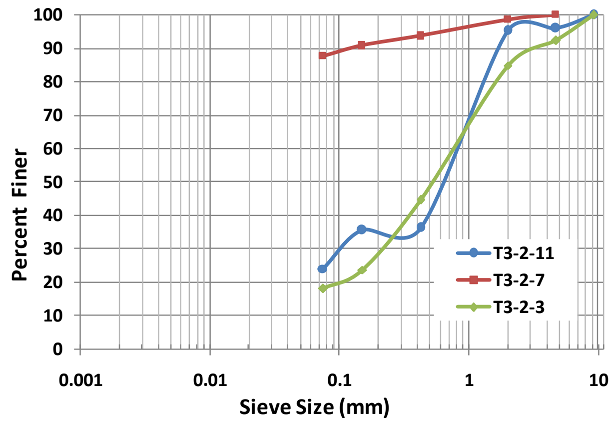


(c)

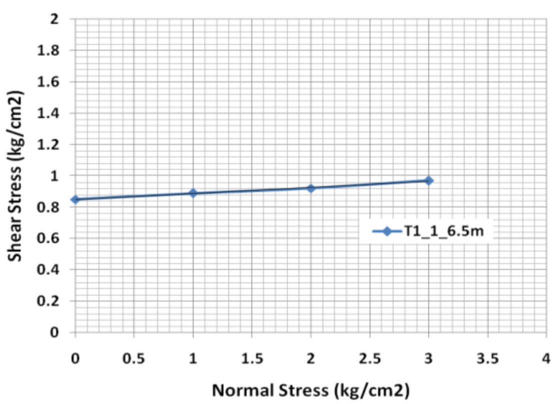
Fig. 6: Grain size distribution of soil at: (a) Ramp T1 (b) Ramp T2, and (c) Ramp T3

3.2.2. Direct Shear Test

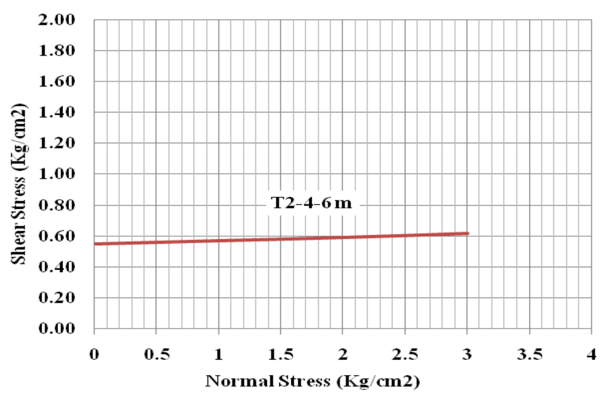
The test has been conducted on soil samples collected from boreholes at ramp T1, T2, and ramp T3 at different depths and the tests are performed under drained condition. Physical and mechanical properties of soil at Ramps are depicting in Table 1. Test results and relation curves are shown in Fig. 7.

Table 1: Mechanical and Physical properties of soil at Ramps T1, T2, and T3

Borehole No.	Depth (m)	Soil description	Φ (°)	Cohesion (C) (kg/cm^2)	Dry density (gm/cm^3)	Water content (Wc) %
T1-1	6.5	Clayey silt	2	0.85	1.34	37
T2-4	6	Fat Clay	1	0.53	1.40	42.3
T3-2	5	Fat Clay	3	0.51	1.32	36.6



(a)



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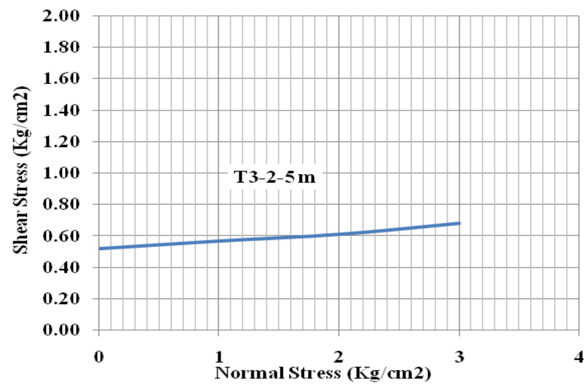


(c)

Fig. 7: Shear stress Vs. Normal stress curve of soil at: (a) Ramp T1 borehole no. 1 at depths 6.5 m, (b) Ramp T2 borehole no. 4 at depths 6 m, and (c) Ramp T3 borehole no. 2 at depths 5 m.

3.2.3. Water Content and Atterberg Limits

Results of natural water content and Atterberg limits are depicting in Table 2

Table 2: Results of Water Content and Atterberg Limits

Sample No.	Wc%	L.L	P.L	P.I	I_C	I_L
T1-1-05	36.2	52	35	17	0.93	0.071
T1-2-05	37.1	54	35	19	0.89	0.111
T1-3-07	36.1	51	34	17	0.71	0.124
T1-4-05	38.5	55	34	21	0.79	0.214
T1-4-08	16.1	20	18	2	1.95	-0.95
T2-1-06	45.1	63	32	31	0.58	0.423
T2-2-06	35.7	50	31	19	0.75	0.247
T2-3-05	44.9	60	31	29	o.52	0.479
T2-4-06	42.3	65	32	33	0.69	0.318
T2-4-08	35	52	32	20	0.85	0.150
T3-1-05	36.3	54	37	17	1.04	-0.041
T3-2-07	36.5	52	34	18	0.86	0.139
T3-3-04	16.1	19	17	2	1.45	-0.450
T3-3-06	35.6	53	33	20	0.87	0.130

3.2.4. Pocket Pentrometer Test

The test has been done on undisturbed soil samples collected at ramps T1, T2, and T3 at depths 5.5 – 6 m of each ramp and results recorded respectively as 0.48, 0.45, and 0.49 kg/cm^2. The results can be indicated as unconfined compressive strength.

3.2.5. Unconfined Compressive Strength Test

The test has been conducted regarding ASTM D 2166, the soil sample collected at ramp T2 borehole no. 4 with depth of 5.5 m. The result recorded unconfined compressive strength q_u = 1.11 kg/cm^2, see Fig 8.

3.2.6. Free Swelling Test

The test has been done on soil samples, data and results of free swelling test are recorded as in Table 3

Table 3: Free Swelling Test results

Sample no.	Depth (m)	Initial Volume (ml)	Final Volume (ml)	Free of swelling (%)
T1-3	5.0	10.0	12.0	20.0
T2-1	8.0	10.0	12.8	28.0
			Average	24.0

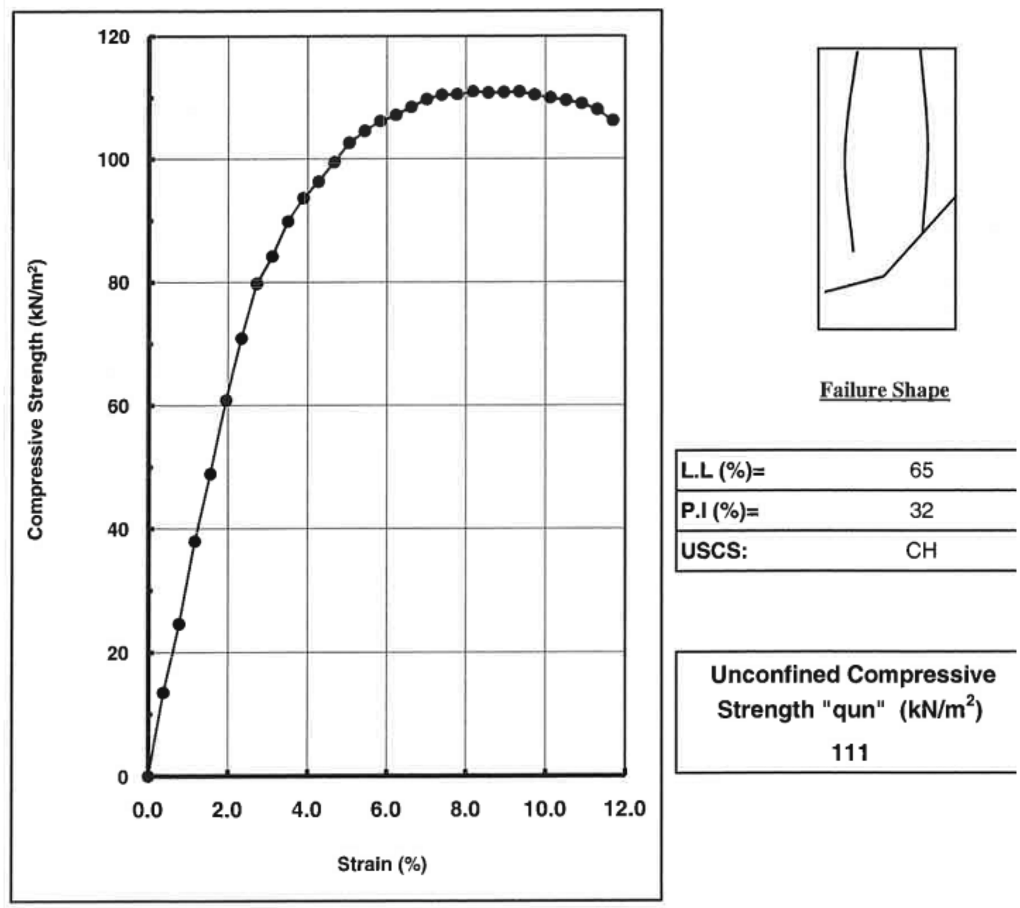


Fig. 8: Compression strength Vs. Strain curve on sample at ramp T2 borehole no. 4 at depth 5.5m.

3.2.7. Odeometer Consolidation Test

The test has been done on sample collected from borehole no 4 at ramp no. T2 at depth of 5.5m below the ground surface, soil can be classified as “Gray Fat Clay”. The test has been conducted as stress versus void ratio (see Fig. 9) regarding specification of ASTM D2435. Physical and mechanical properties of the soil sample are summarized as in Table 4

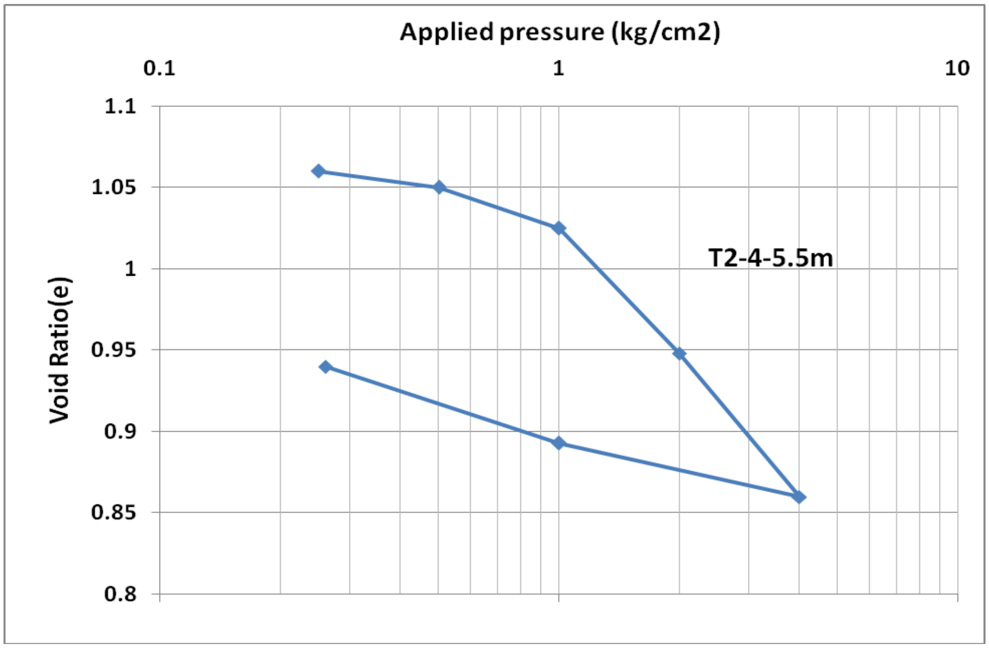


Fig. 9: Stress Vs. void ratio curve at Ramp T2 borehole no. 4 at depths 5.5 m.

Table 4: Physical and mechanical properties of soil at T2-4-5.5m

Initial Wc%	LL	PL	γ_b (t/m^3)	γ_d (t/m^3)	G_s	e_o	S_r (%)	C_c	C_s	P_o (kg/cm^2)	P_c (kg/cm^2)	OCR
40	65	33	1.79	1.28	2.65	1.067	99	0.289	0.063	0.65	0.68	1.05

3.2.8. Chemical analysis Test

3.2.8.1. Chemical Analysis Test on soil sample

The test has been done on soil sample that mixed with distilled water with ratio of water to soil as 2: 1 by weight. Results are shown in Table 5.

Table 5: Results of chemical analysis of soil sample.

Soil Sample No.	T1-2-4	T2-4-4
Depth (m)	4.00	3.00
pH	8.0	7.9
Total Sulphate%	0.0900	0.0870
Total Chloride%	0.1120	0.1260

3.2.8.2. Chemical Analysis Test on Groundwater Sample

The test has been done on groundwater sample that collected at ramp T1 and ramp T2. Results are shown in Table 6.

Table 6: Results of chemical analysis of groundwater sample.

Borehole No.	T1-2	T2-4
pH	8.0	8.2
Total Sulphate (PPM)	1070	1020
Total Chloride (PPM)	1465	1375

4. BEARING CAPACITY AND SETTLEMENT

4.1. Bearing Capacity Calculations

Ultimate Bearing capacity =(q_u) is calculated using eq. 1

$$q_u = (C.N_c.F_{cs}.F_{cd}.F_{ci}) + (q.N_q.F_{qs}.F_{qd}.F_{qi}) + (0.5\gamma BN_\gamma F_{\gamma s}.F_{\gamma d}.F_{\gamma i}) \quad (1)$$

C: cohesion.
q: effective stress at the level of the bottom of foundation.
γ: unit of soil.
B: width of foundation.
Sc,Sq,Sγ: shape factors.
dc,dq,dγ: depth factors.
ic,iq,iγ: inclination factors.

Foundation Information: Df=1m. Soil Information: Dw=2.5m, $\gamma_{water} = 10\,kN/m^3$, $\gamma_1 = 18\,kN/m^3$, $\gamma_2 = 18\,kN/m^3$, $\gamma_{1sub} = 8\,kN/m^3$, $\gamma_{2sub} = 8\,kN/m^3$.

Factor of safety (FS) = 4.00. Factor of shape is calculated as eq. 2

$$F_{cs} = 1 + \frac{N_q}{N_c} \times \frac{B}{L} \quad (2)$$

Calculations of bearing capacity for ramp T1, ramp T2, and ramp T3 are depicting in Table7:

Table 7: Bearing capacity results of ramp T1, ramp T2, and ramp T3

Element	Ramp No.		
	T1	T2	T3
Angle of friction $\phi^{(o)}$	0	0	0
Cohesion C (kN/m^2)	60	45	50
Width B (m)	17	17	6
Length L (m)	25	25	25
Gross ultimate bearing capacity q_u (kN/m^2)	366	279.4	286.8
Q (kN/m2)	18	18	18
Net ultimate bearing capacity (kN/m2)	366	279.4	308
Net Allowable (kN/m^2)	91.5	67	71.7

4.2. Settlement Calculations

Settlement calculations of soil layer formation at ramp T1, ramp T2, and ramp T3 have been estimated as shown in Table 8:

Table 8: Soil parameters

Soil Parameter	Clay	Silt
Average liquid limit	60%	50%
Average water content Wc (%)	41%	36%
Specific gravity Gs	2.65	2.68
Initial void ratio $e_\circ$	1.06	0.938
Compression index Cc	0.29	0.360
Recompression index Cr	0.060	0.048
Over consolidation ratio OCR	1.00	1.00

By using approximation method of stress distribution at each ramp, the stresses induced at ramp height of 1 m, 2m, 3m, … , and 9m can be estimated by using eq. 3 and eq. 4, compression index (Cc) may be calculated as eq. 5. Consequently, settlement can be calculated using eq. 6. Assuming that: $\gamma_b(sand) = 20kN/m^2$, $\gamma_b(clay) = 20kN/m^2$, hence for sand $e_\circ$=0.938, and for clay $e_\circ$=1.06.

$$\Delta\sigma_v = \frac{P \times L \times B}{(L+d)\times(B+d)} \qquad (3)$$

$$\sigma_v = 2\gamma b + \frac{1}{2}\gamma b + 2.5\gamma sub + \frac{7}{2}\gamma sub(clay) \qquad (4)$$

$$Cc = 0.009\,(LL-10) \qquad (5)$$

$$Sc = \frac{Cc}{1+e_\circ} \times Hc \log \frac{\sigma_v + \Delta\sigma_v}{\sigma_v} \qquad (6)$$

The estimated settlements at ramp T1, ramp T2, and ramp T3 are shown in Table 9:

Table 9: Calculation of settlement with embankment hight(Ramp no. T1, B=17m , L=25m)

	Average Sc (cm) at ramp No. (**allowed settlement≯10cm)**		
	T1	T2	T3
Height (m)	$B \times L$ (m) 17×25	$B \times L$ (m) 17×25	$B \times L$ (m) 6×25
1	5.10	4.23	3.10
2	9.58	8.28	5.99
3	13.75	11.88	8.72
4	16.40	15.10	11.27
5	20.78	18.08	12.93
6	23.78	20.85	16.03
7	26.88	23.18	18.20
8	29.45	25.30	20.14
9	31.98	27.55	22.26

5. DRIVEN CAST-IN-SITU CONCRETE THIN-WALL PIPE PILES (PCC)

Recently, one of the significant solutions for soft soil improvement is the use of thin-wall pipe pile using cast-in-situ concrete named PCC piles as a new technique. The new technology for PCC piles construction involves firstly driving a double walled open-ended steel casing by a vibratory driver with the protection of the tapered expendable driving shoe into soil stratum, secondly pouring concrete into the annulus void while vibratory system is withdrawing the steel casing forming the thin wall concrete pipe pile (PCC). After the PCC pile cured in place, a certain length (0.5 m) of soil plug at the pile head is excavated and concrete is poured to form a concrete pile cap. A layer of 30 to 50cm of broken gravels cushion with one or two layers of geogrid embedded in between is then placed at the pile cap. In such a way, most of loads of the embankment are taken by the rigid pile group, the geogrid embedded gravel cushion is acting as a rigid plate to increase the pile head load and reduce the surrounding soil settlement [Liu et al. (2007), Mahfouz (2007), Mahfouz (2011)].

The composite foundation consisting of PCC piles and the geogrid embedded gravel cushion is getting more popular now for soft ground road foundation improvement because of its low cost, high bearing capacity, embankment stability, and less post construction settlement. The static load test and field excavation can be concluded that the new type of pile is convenient to construct with high bearing capacity and reliable quality, which has great potential in practice (Liu et al. 2006, Xu et al. 2006).

The PCC pile equipment has been designed and the technology is applied at highways embankment projects improving the bearing capacity of the soft soil foundation and reducing settlements (Liu et al. 2004a, Liu et al. 2004b, Fei et al. 2004a, 2004b, 2004c). Also, application of PCC piles technique in sea embankment on soft soil foundation (Liu et al. 2005), for expressways Zhou et al. (2005). The effectiveness of using the PCC pile for reinforcing soft soil has been demonstrated by Zhitao et al.(2006). The advantages of the technique are: (1) Shortening the construction duration time and (2) good workability.

5.1. Construction Equipment

In view of the inadequacy of existing program-based approach and relying on their own strengths in geotechnical engineering research, a large-diameter cast-in-place concrete thin-wall pipe with vibration mode technology of vibrosinking machinery has been patented and developed by Hohai University – Nanjing, China in 2001 as shown in Fig. 10. Equipment components include:

1. General gantry tower: to drive pile, a vibration mode is applied to annular double-wall of the hollow case.

2. Lifting equipment: the lifting power is more than ordinary power to raise large scale pile.

3. Vibration hammer: the vibration hammer has a weight enough to drive the pile according to the piling rate requirements through additional pressure, so driving can be achieved quickly.

4. New double-walled steel casing: a combination of concentric annular pipe of 8mm steel thickness with different diameters. The inner and outer diameters of pipes are 0.76 and 1.00m, respectively. The inner pipe is open-ended whereas the annulus is fitted with a temporary conical-shaped driving shoe.

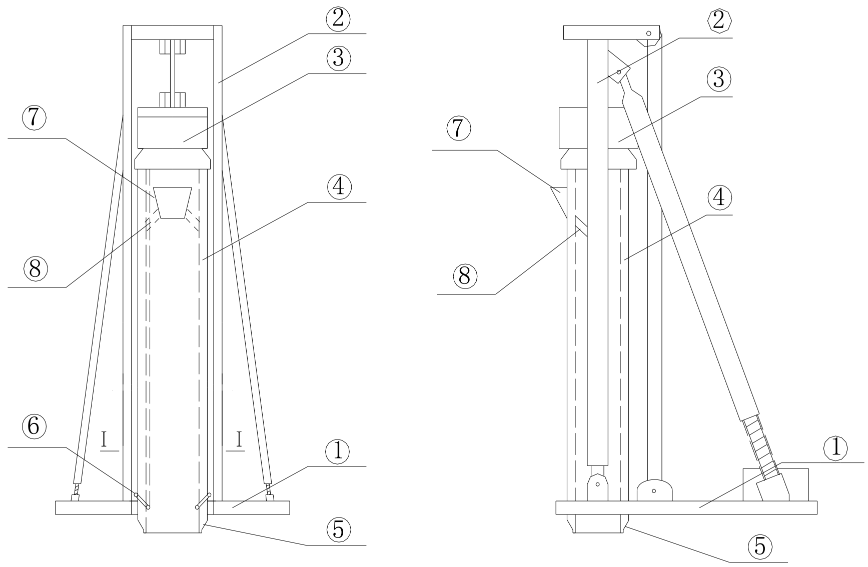


Fig. 10: Schematic Drawing of Construction Equipment

① base of the equipment (including windlass); ② Gantry crane; ③ vibration head; ④double-walled steel casing; ⑤ valve pile boots; ⑥ into mud mode; ⑦ feed inlet; ⑧ concrete shunt.

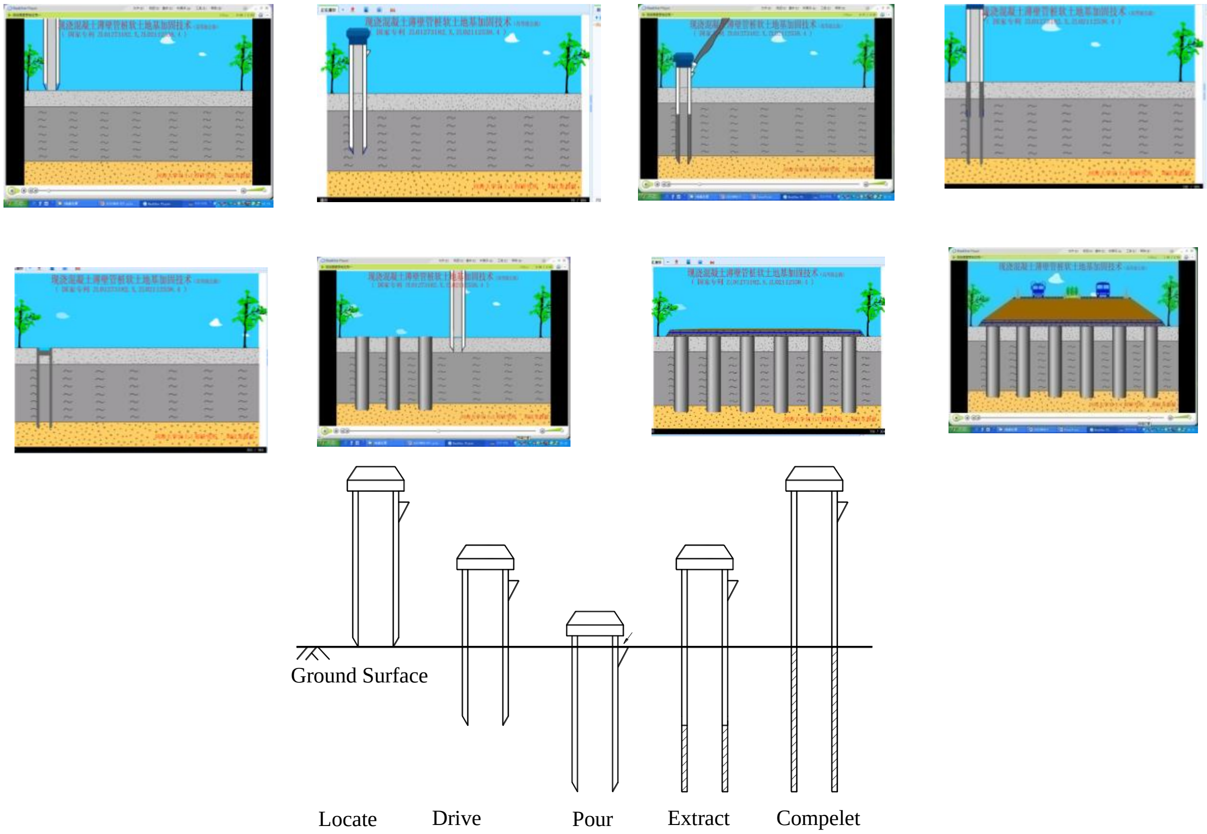
5.2. Principle and Mechanism of Piling

Construction machinery of in-situ thin-wall concrete pile is run by using vibratory hammering to the top of the pile casing forming annulus up to the designed depth, then pouring concrete into the hollow annulus; and finally vibratory is withdrawing the steel casing. In this way, an annulus concrete pipe pile can be formed with soil column inside the PCC pile. The vibratory hammer for pile driving consists of contra-rotating eccentric masses attached to the pile head. This vibratory system has quite heavy weight and very high frequency resulting high impact force to drive the steel casing into the soil quickly with a rate of 0.8-1.2 m/min. During the driving process, the soil is displaced into the inner pipe and outside the outer pipe creating 12 cm thick annulus for in situ concreting. As thin-wall vibration system makes the soil squeezing under the action of vibration force, hence the skin friction is declined sharply involving yield resistance and consequently high sinking rate.

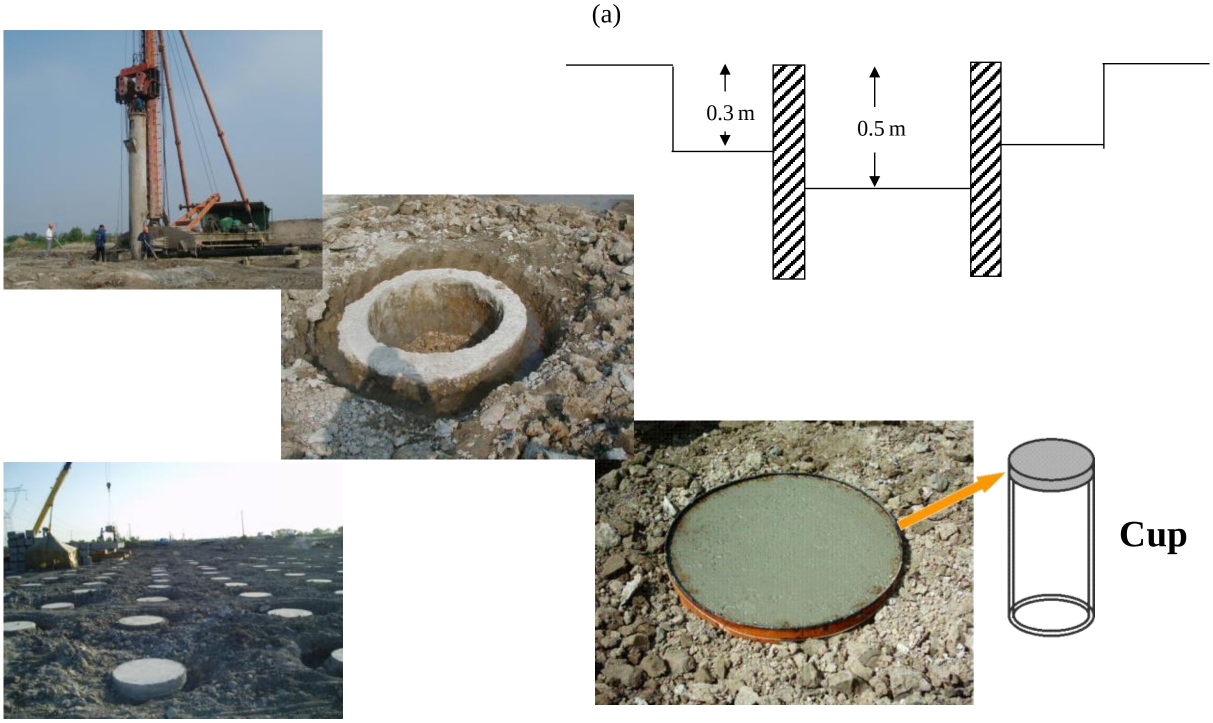
5.2.1. Casing, Vibration, and compaction

The construction process of PCC piles include: locating equipment in place, driving steel casing, pouring the annulus with concrete, pulling the steel casing, and moving to another spot. The construction flow chart is depicted in Fig. 11. During in situ concreting the annulus, the casing is withdrawn at a steady rate of 0.8–1.2 m/min. The pile casing should vibrate for 10 seconds before withdrawal. The withdrawing should be stopped temporarily with continuing vibration of the pile casing at every 1 m withdrawal for 5 to 10 seconds until the casing is completely withdrawn. This precaution may cause compaction of the PCC pile concrete and the squeezing pressure generated towards the inside and outside the thin wall casing is supporting the poured concrete forming thin wall thickness of the PCC pile. An appropriate concrete head varying from 0.3 to 0.5 m is always maintained within the annulus to provide stability, whereas the casing is withdrawn.

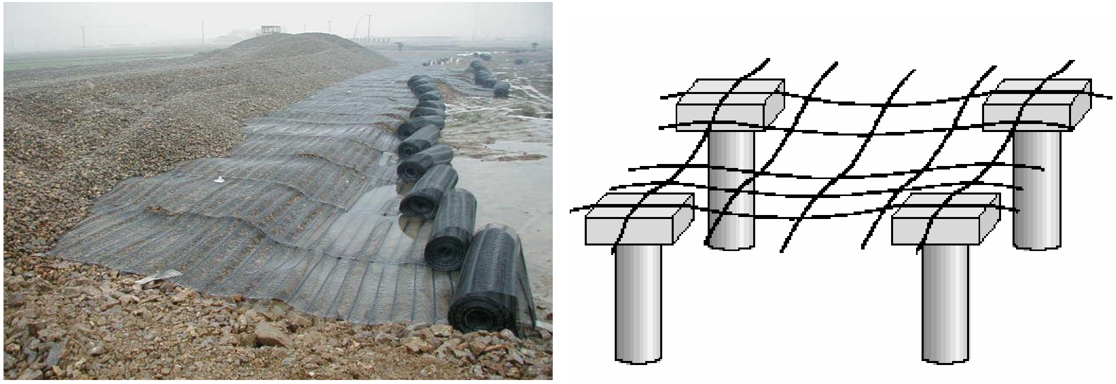
During penetration of the steel casing, large soil displacement occurs due to driving, vibrating and squeezing effects which make the surrounding soil being compacted to a certain degree. The disturbance extent depends on the wall thickness of the steel casing and the soil properties.


Ground Surface
Locate
Drive
Pour
Extract
Compelet

(a)


0.3 m
0.5 m
Cup

(b)



(c)

Fig.11: Pile Construction Flow: (a) driving and pouring concrete with pulling out pile casing, (b) pile caping, and (c) place gravel cushion and then geogrids

5.3. PCC piles design

5.3.1. Method for PCC Pile Design

Characteristic value of vertical bearing capacity of the PCC pile composite foundation **(f_{spk})** can be calculated as eq. 7.

$$f_{spk} = m\frac{R_a}{A_p} + \beta(1-m)\,f_{sk} \qquad (7)$$

Where: $m = \frac{d^2}{d_e^2}$

In which:

f_{spk} : Characteristic value of vertical bearing capacity in kPa.

m: The ratio of the pile area and soil area

d: The external diameter of PCC pile (m)

*d*e: The area of foundation that a single pile can handle (m)

*R*a: Characteristic value of vertical bearing capacity of single PCC pile (kN/m^2)

*A*p : The area of PCC pile cross section(including the area of soil in PCC pile) (m^2)

β: Reduction factor for the bearing capacity of soil around the pile

*f*sk: The bearing capacity of soil around the pile after the construction of pile (kPa)

5.3.2. Characteristic value of vertical bearing capacity of single PCC pile (Ra)

Characteristic value of vertical bearing capacity of single PCC pile (Ra) is estimated as eq. 8.

$$R_a = \frac{1}{K} Q_{uk} \qquad (8)$$

In which：

Q_{uk}: the ultimate bearing capacity of single PCC pile is calculated using eq. 9.

K: A safety factor（K=2）

$$Q_{uk} = u\sum_{i=1}^{n} q_{slk}\, l + \xi_p q_{pk} A_p \qquad (9)$$

5.3.3. ultimate bearing capacity of single PCC pile (Q_{uk})

In which:

u: The outer perimeter of PCC pile (m)
n: Number of soil levels that the PCC pile gets through
ξP: Reduction factor for the tip resistance of PCC pile
q_{sik}: The standard value of side resistance for i[th] soil layer (kPa)
q_{pk}: The standard value of tip resistance of bearing soil layer (kPa)
l_i: The length of i[ih] layer that PCC pile gets through (m)

5.3.4. Final settlement of the PCC pile composite foundation（S）

The final settlement *S* of the foundation can be calculated as in eq. 10.

$$S = S_1 + S_2 \tag{10}$$

In which:

$S1$: settlement within the depth of the PCC pile (mm), it is calculated by layerwize summation method as in eq. 11.
$S2$: settlement of sub-soil (mm)

$$S_1 = \psi_s S_1^{'} = \sum \frac{P_o}{\xi E_{si}} (z_i \bar{a}_i - z_{i-1} \bar{a}_{i-1}) \tag{11}$$

Where:
ψ_S : empirical coefficient for settlement
P0 : additional stress applied at the bottom of base
z_i : the distance from the bottom of base to i[th] soil layer
a_i: average additional stress coefficient in the range from i[th] soil layer to i-1[th] soil layer
E_{si}: the modulus of compression for the i[th] soil layer
ξ : improvement coefficient for Esi
f_{ak}: Characteristic value of vertical bearing capacity of the natural foundation

5.3.5. Parameters Adopted for PCC Pile Design

5.3.5.1. Soil parameters

Soil parameters of soil foundation of ramp T1, ramp T2, and ramp T3 are shown in Table 10.

Table 10: Soil parameters of soil foundation of ramp T1, ramp T2, and ramp T3.

Ramp No.	Layer No.	Soil type	Elevation (m)	Depth of i[th] soil layer (m)	Side resistance	Bearing capacity of soil around PCC pile (kPa)	Es (kPa)
T1	1	Fill	1.75	1.75	20	92	10
	2	Sand	3.75	2.00	40		7
	3	Silt	10.00	6.25	15		3.1
	4	Sand	17.00	7.00	55		12
T2	1	Fill	0.50	0.50	20	67	10
	2	Sand	1.75	1.25	40		7
	3	Clay	4.25	2.50	20		4.5
	4	Silt	10.00	5.75	15		3.1

	5	Sand	17.00	7.00	55		12
T3	1	Fill	2.25	2.25	20	72	10
	2	Sand	4.75	2.50	40		7
	3	Silt	10.75	6.00	15		3.1
	4	Sand	17.00	6.25	55		12

5.3.5.2. PCC Pile parameters

The pile length is 17m, external diameter is 1m, internal diameter is 0.76 m, and Wall thickness is 0.12 m, distance between two consecutive PCC piles is 3.3m, and layout pattern is quincunx. Characteristic value of settlement and vertical bearing capacity of the PCC pile composite foundation are summarized in Table 11.

Table 11: Designed bearing capacity and expected settlement of PCC pile

Ramp No.	Bearing Capacity (kN/m^2)	Settlement (mm)
T1	243.9	24.0
T2	243.4	21.4
T3	242.0	20.7

6. CONCLUSION AND RECOMMENDATIONS

6.1. Conclusion:

Construction of Dawar El-Tawheed bridge project in Jazan –Kingdom of Saudi Arabia is national project very important for developing jazan in different sectors of tourism, commerce, and industry. According previous results of this study, soil foundation of Dawar El-Tawheed bridge ramps is soft soil that has low bearing capacity and induces a high settlement under surcharge loads. The study may be concluded to the following points:

a. The soil profile of very soft fine grained soil located between 5.0 m to 11.0 m in depth has high water content, low SPT no.,

b. The soil has low bearing capacity of 92 kN/m^2, 67 kN/m^2, and 72 kN/m^2 at ramp T1 , ramp T2, and at ramp T1 respectively.

c. The clay layer induces high settlement more than 10 cm due to surcharge application of earth embankment at ramps T1, T2, and ramp T3 especially at heights from 9m to 3m.

d. The soil and groundwater have high salt content of sulfates and chlorides at the project site aggressively.

e. Applying PCC piles to improve foundation soil bearing capacity is reasonable where bearing capacity would be 243.9 kN/m^2, 243.4 kN/m^2,, and 242.0 kN/m^2 at ramp T1, ramp T2, and ramp T3, respectively.

f. The induced settlement of PCC piles would be 24.0mm, 21.4mm, and 20.7mm at ramp T1, ramp T2, and ramp T3, respectively.

6.2. Recommendations

Regarding results above it may be strongly recommended to apply the new technology of cast in-situ thin wall concrete pipe piles that named PCC piles to improve the foundation soil of Dawar El-Tawheed ramps because it might improve well its bearing capacity as well as it has economically advantageous, high workability, save concrete, and application cost is of about 30% of other method of soil improvement.

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