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GROUND IMPROVEMENT OF A COAL ASH STORAGE POND USING SAND COMPACTION PILES

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ABSTRACT: Effects of sand compaction piles on ground improvement of a coal ash pond and axial performance on four drilled shafts through improved coal ash layer at a power plant in southern Taiwan is studied in this paper. Effects of the sand compaction piles improvement is evaluated by comparing the SPT-N values before and after ground improvement. The load versus displacement relation at head, the axial force along the shaft, the t-z curves and/or the toe q-z curve, which are the main concerns of shaft load test results, are presented and discussed in the paper. Applicability of the β value used by O'Neill and Reese (1999) for estimating shaft skin friction resistance of soil with no cohesion through improved coal ash layer is evaluated by comparing to the test measured results.

Keywords: Sand compaction pile, ground improvement, coal ash, drilled shafts

INTRODUCTION

A 135-hectare power plant located in the southwest part of Taiwan was built in the end of the 70's. Ninety-nine hectares of the total area was reclaimed from the sea using hydraulic fill dredging. The thickness of the reclaimed area varies from 2 to 5 meters. A coal ash storage pond is located on the west side of the power plant. The size of the pond is around 830 meters long and about 190 meters and 130 meters wide on the north and south end, respectively. Total area of the pond is about 12.84 hectares. The coal ash was produced by the power plant and was transported via wet disposal of slurry, sea water and coal ash mixture to the pond. In the early stage of the power plant's operation, the coal ash was stored in open space. To satisfy environmental requirement, four coal domes were built at the location of the coal ash pond for future coal ash storage. Each dome can store about 17 to 18 thousand tons of coal ash, and has an inner diameter of 120 meters with height of 59 meters. The main structures in the dome include an 18-meter high reinforced concrete cylindrical wall, a 41-meter high steel dome cap, a 28.5-meter high steel central column and a reinforced concrete transportation tunnel, which is 11.6 meters in width and 5.8 meters in height. Because of under lowland condition, ground improvement technique using sand compaction piles (SCP) was conducted for the foundations of the whole coal ash pond area. In fact, reclaimed ground improved by SCP has also been adopted in some cases, such as the research results of Tsuboi et al. (2002; 2001; 2000; 1998 and 1997). Fuchida and Akiyoshi (2003) used similar technique for preventing damage of soil-pile foundation super structure system. Kimura et al. (2000) improved soft ground by using solidified coal ash on marine environment. The stress state and volume change properties of the ground improved by SCP was studied by Priyankare et al. (2005).

Drilled shaft foundations, 1.2 meters in diameter and 36 meters in length were also used underneath the cylindrical wall of the domes. In addition to its use as a countermeasure against liquefaction, the purpose of SCP was also aim to provide good ground capacity to support aforementioned structures and to provide better shaft/soil resistance around the improved area.

Available researches such as Aboshi et al. (1991) and Barksdale and Takefumi (1991) often focus on sandy soil with fine content less than 20 percent and small replacement ratio of sand compaction piles. Effectiveness of SCP improvement for coal ash storage pond with higher fine content is not yet available. The purpose of the paper is to study the efficiency of the SCP for coal ash pond and the performance of the drilled shafts through SCP improved coal ash layer using axial shaft load test. The site, boring locations and shaft load testing locations are illustrated in Fig. 1. In the following, a description of the site and the measured soil material properties are presented first. A brief explanation of

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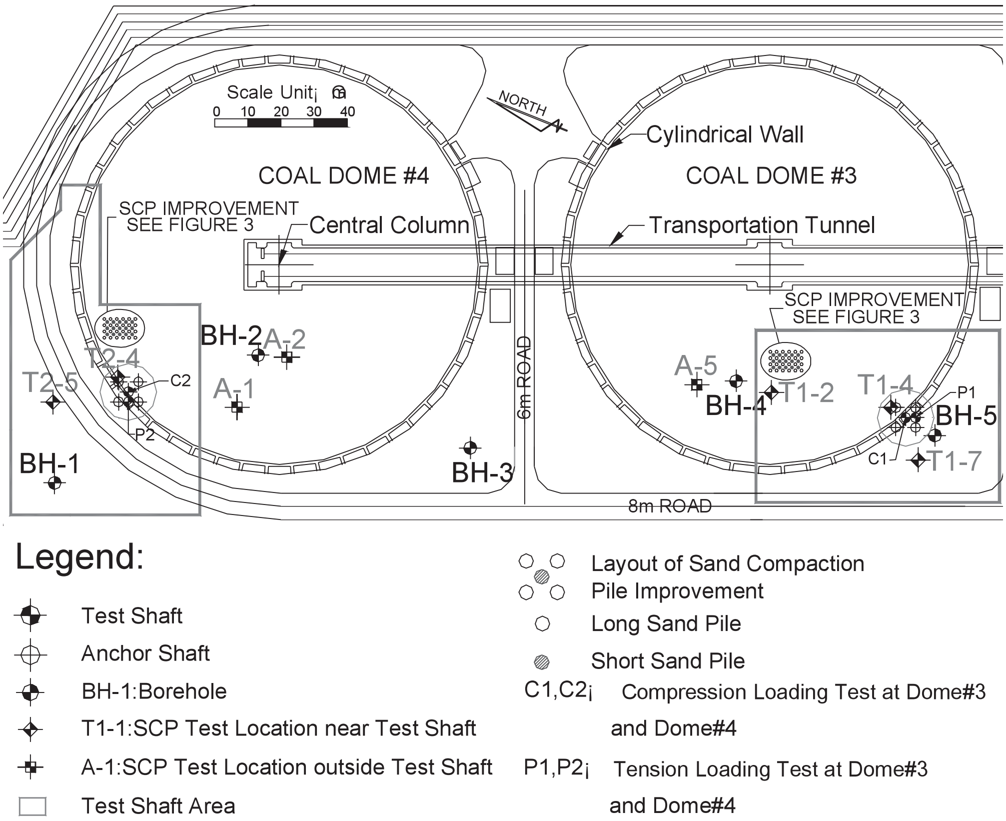


Fig. 1 Site, boring location and drilled shafts load test location

Table 1 Soil properties before ground improvement

Layer of stratum	Class	SPT-N	Thickness	γ_t	γ_d		Total stress		Effective stress	
Depth		Value	(m)	(kN/m^3)	γ_{dmax} (kN/m^3)	γ_{dmin} (kN/m^3)	Su (kN/m^2)	ϕ deg	C' (kN/m^2)	ϕ' deg
1.Black/gray coal ash, with occasional gravel GL= 4.2-6.8m	SF	1~ 5 (3.2)	4.8~ 8 (5.9)	12.8~ 6.0 (14.2)	9.5~ 0.1 (9.8)	7.0			16.7*	29*
2.Gray silty sand, with occasional silty clay and gravel GL= 8.9-9.3m	SM	11~ 18 (15.8)	2.5~ 4.8 (3.3)	18.2~ 0.7 (19.4)	16.1~ 6.8 (16.5)	12.7~ 3.0 (12.9)			5.9*	30*
3.Gray with occasional Yellowish brown silty clay and occasional silty sand GL=13.4-19.9m	CL	1.5~ 17 (5.2)	4.4~ 10.6 (6.2)	17.7~ 0.4 (19.3)	16.1~ 6.8 (16.5)	12.7~ 3.0 (12.9)	22.6**			
4.Gray silty sand/silty clay GL=19.5-38.1m	SM-CL	5~ 41 (16.6)	6.1~ 18.2 (10.5)	17.8~ 1.5 (19.7)			26.5**		2.9*	33*
5. Gray silty sand, with occasional silty clay/clayey silt End of GL= 50m	SM	6~ 71 (41.8)	2.4~ >10	18.0~ 2.2 (19.6)					14.7*	

Note: E.L.=+4.0m=G.L.=+0.0m

*Direct shear test (DS)

**Saturated Unconsolidated Undrained Test (SUU)

Table 2 Physical properties of coal ash

	SPT-N Value	ω(%)	γ_t (kN/m^3)	G_s	FC(%)	e	D_{10}	D_{30}	D_{50}	D_{60}	C_u	C_d
Max.	5.0	90.40	15.99	2.26	88.60	1.98	0.009	0.024	0.056	0.094	28.1	2.5
Min.	1.0	38.10	13.44	2.11	55.80	0.84	0	0.008	0.006	0.024	5.5	0.5
Average	3.4	62.90	14.25	2.22	74.68	1.52	0.006	0.016	0.031	0.052	10.3	1.1

Table 3 Sieve analysis results of the sand used for sand piles

Sieve no.	1”	1/2”	#4	#16	#50	#100	#200
Sieve diameter	25mm	12.5mm	4.75mm	1.18mm	300.m	150.m	75.m
Passing weight (%)	97-100	90-100	70-100	20-75	3-25	0-10	0-3

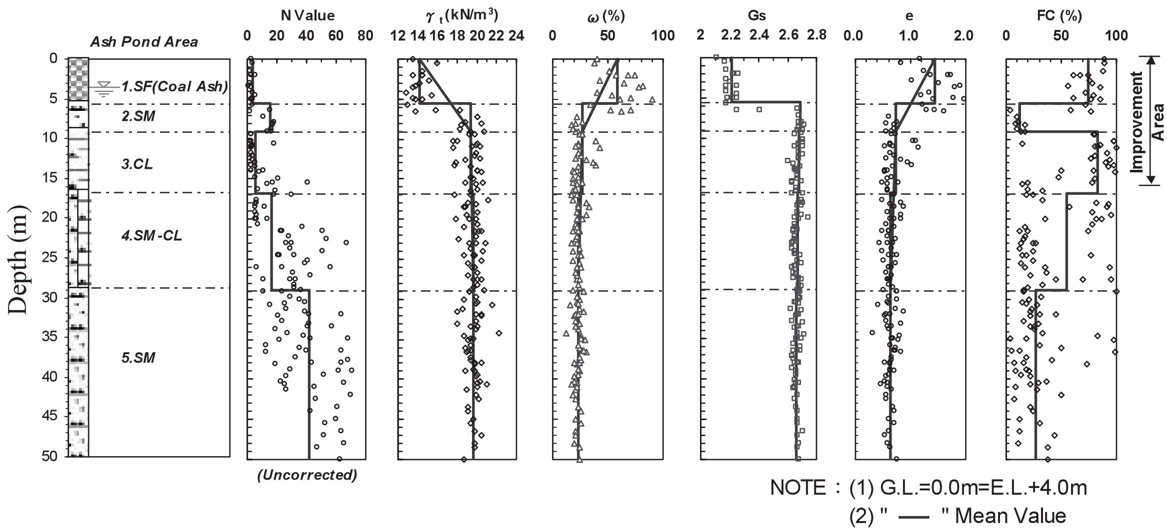


Fig. 2 Typical soil properties of the coal ash pond before ground improvement

design considerations and ground improvement procedures is then followed by a comparison on the changing of Standard Penetration Test (SPT)-N values before and after improvement. Subsequently, two compression (C1 and C2) and two other tension (P1 and P2) axial shaft load test results at dome #3 and dome #4 site (Fig. 1) are compared via load-displacement relation, t-z curves along shaft and/or q-z curve at toe.

SITE DESCRIPTION

Before ground improvement, an extensive program of site investigation, including drilling, sampling, in-situ penetration, and laboratory test was performed. Typical geological stratum of the site can be roughly divided into five layers as given in Table 1. The soil from ground surface down consists of 2 to 5-meter thick of coal ash, nearly 5 meters of silty sand layer, 7 meters thick of low plasticity clay and 13 meter thick of SM-CL medium or CL clay. Beneath the clay layer, silty sand layer is encountered. Other information regarding typical soil conditions of the site is also shown in Fig. 2, which gives the SPT-N values, soil unit weight, water content, void ratio and fine soil contents of each soil layer. Liquid limit and plasticity index of the CL (layer 3) layer are between 27 ~ 72 and between 7~34, respectively. For the SM-CL layer (layer 4), the liquid limit and the plasticity index are between 30~54, and 11~28, respectively. In addition, physical properties of the coal ash are listed in Table 2. Compare to the rest of the soil conditions given in Fig. 2, the coal ash layer appears to have a relatively low SPT-N value, low unit weight, high water content, low specific gravity, high void ration and high fines content. Ground water level of the site is between 3 to 6 meters below ground surface.

GROUND IMPROVEMENT BY SAND COMPACTION PILE

A triangular grid pattern spaced 1.5 meters center-to-center of 70cm diameter sand piles was used throughout the works of coal ash pond ground improvement. Two

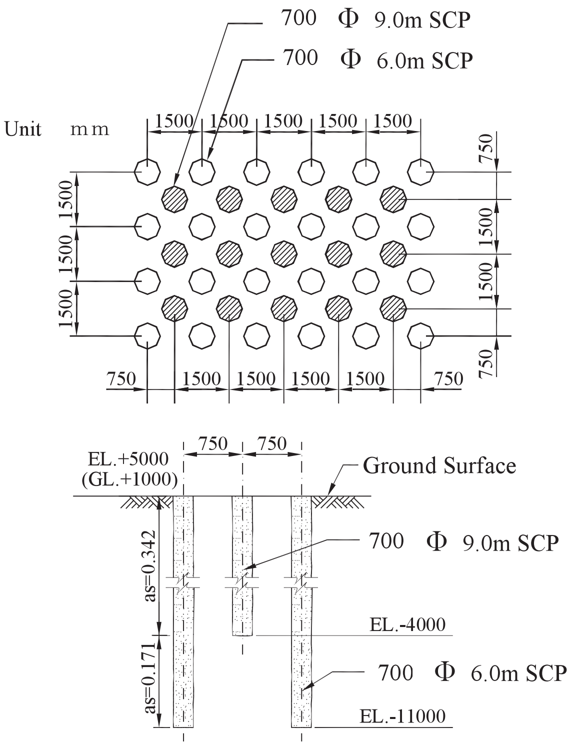


Fig. 3 Layout of sand compaction piles

different lengths, 16 meters and 9 meters sand piles, were alternately used in the improved zone (Fig. 3). The sand pile replacement ratio is 0.342 for the top 9 meters and is 0.171 for the clay layer between 9 and 16 meters below ground surface. Results of sieve analysis of the sand used for sand pile are given in Table 3. The 16-meter long sand piles were installed first before installing the 9-meter long piles. A 400 mm tube was installed to the desired depth using a vibrator. The sand is densified by repeatedly extracting and re-penetrating the vibrating pipe as it is withdrawn from the ground. The casing is first pulled up 3.7 meters using the crane and then re-penetrated back down 2.7 meters. This up and down procedure is repeated until the casing is completely withdrawn from the ground.

The effectiveness of improvement is checked by means of SPT, conducted seven days after installation of every 500 sand piles. Two locations, at intermediate points between piles and center of one of the sand piles were selected for SPT.

Comparison on SPT-N values obtained before and after improvement at coal dome #3 and #4 is shown in Figs. 4 and 5, respectively. In general, SPT conducted at the center of sand piles all appeared to have higher N-value than those conducted at intermediate points between piles. However, the SPT-N values conducted at intermediate points between piles at clay layer of depth between 10 and 15 meters showed less significant improvement. On the average, construction of SCPs results in the SPT-N value increasing by a factor of 5.6. The relationship between original (before ground improvement) SPT-N value and inter-pile N-value after improvement, N1, is shown in Fig. 6, in which various replacement ratio as employed in Japan (Aboshi et al. 1991) for sandy soil are also given for comparison. The commonly used replacement ratio between 0.05 and 0.2 of sandy soils does not apply for coal ash because of higher fine content. Fig. 7 shows the relationship between fine particle content and inter-pile N-value after improvement N1. Experience observed by Aboshi et al. (1991) shows that the higher the value of FC the smaller the value of N1, as shown in Fig. 6(a). The experience for coal ash pond also shows the decreasing of N with increasing fine content FC in the normal scale, but not in the semi-log scale (Fig. 6(b)). Because of higher sand pile replacement ratio adopted in the case, the data points from this study are all located above the trend lines.

DRILLED SHAFT AXIAL LOAD TESTS

Shafts adopted for cylindrical wall foundations are cast-in-place Portland cement concrete piles of 1.2 meters in diameter and 36 meters in depth with reinforcement provided by rebar cages shown in Fig. 8. Structural analysis of the cylindrical wall structure, considering possible strong wind effects, shows part of

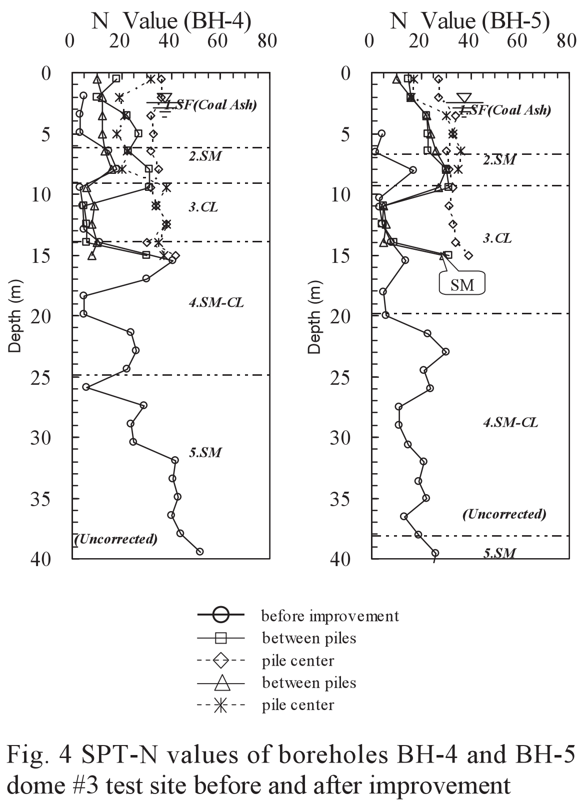


Fig. 4 SPT-N values of boreholes BH-4 and BH-5 of dome #3 test site before and after improvement

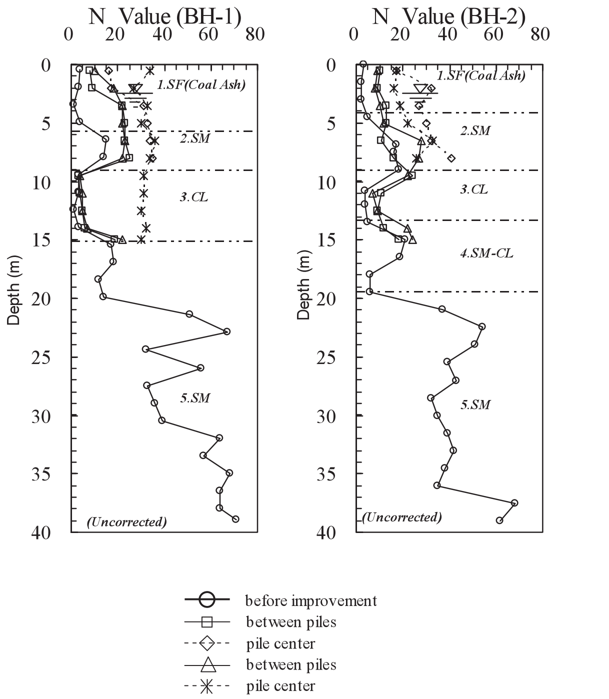
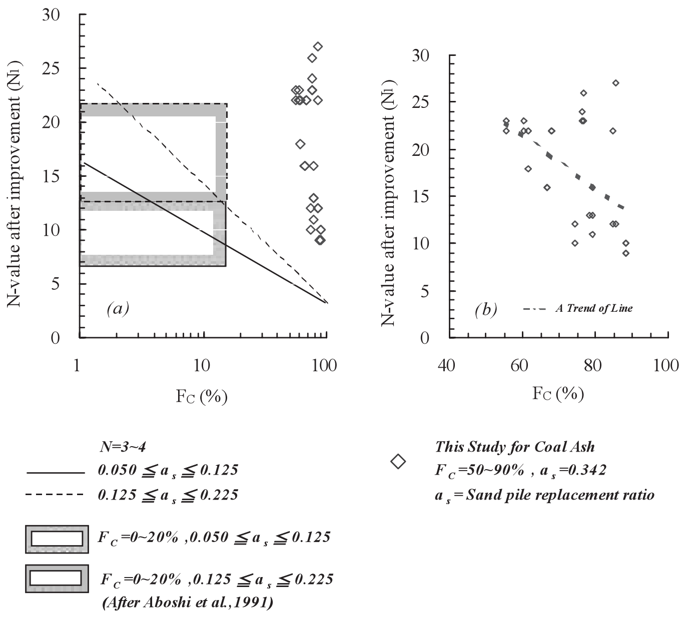


Fig. 5 SPT-N values of boreholes BH-1 and BH-2 of dome #4 test site before and after improvement



(a) (b)

Fig. 6 Relationship of SPT-N values before and after SCP improvement

the piles will be subjected to tensile loading. Hence, at each dome site, one compression and one tension shaft load test were conducted. To evaluate the total load carried at different depths along the shaft, twenty-four rebar cages were installed at eight different locations along the shaft. These gauges were attached to the rebar gage in sets of three at each depth and were protected. In addition, two telltales were also installed for to measure the displacement of the shaft close to shaft toe. Casings throughout the full soil layers and drilling under the drilling slurry were performed to prevent the soil from collapsing. Hammer grab was used for soil excavation. The tremie method was used for shaft concreting, with slump between 18 and 22cm. The design 28-day unconfined compressive strength of the concrete is 30380 kN/m^2.

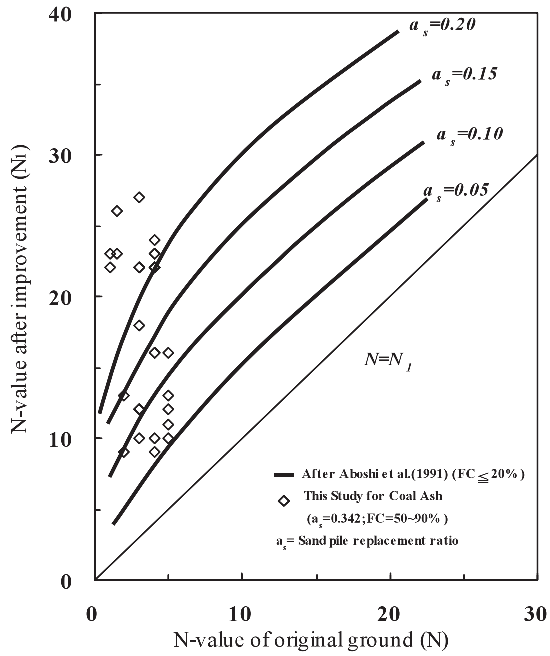


Fig. 7 Relationship of SPT-N values after improvement versus fine content FC

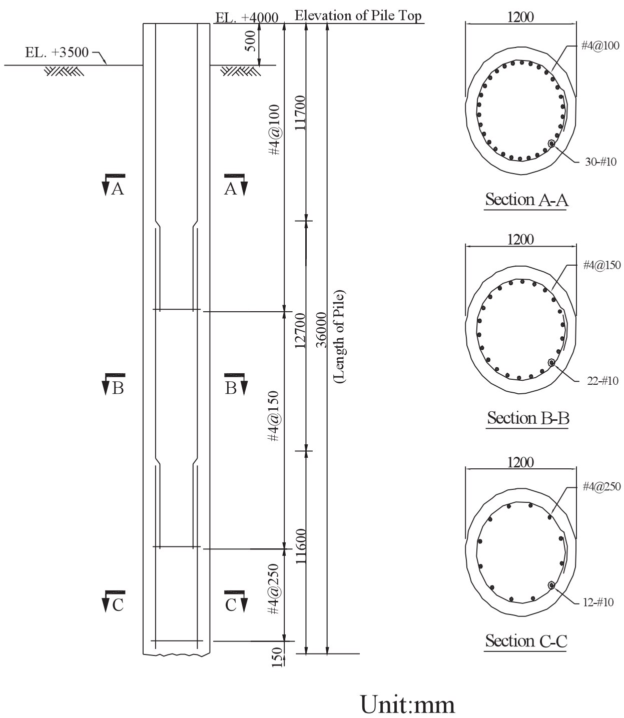


Fig. 8 Reinforcement arrangement of the tested shafts

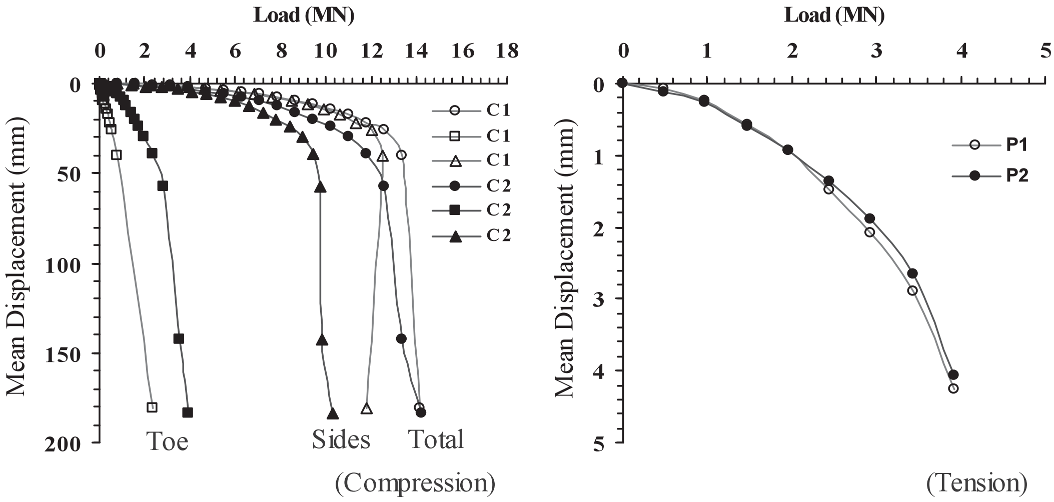


Fig. 9 Pile head load versus displacement relationship

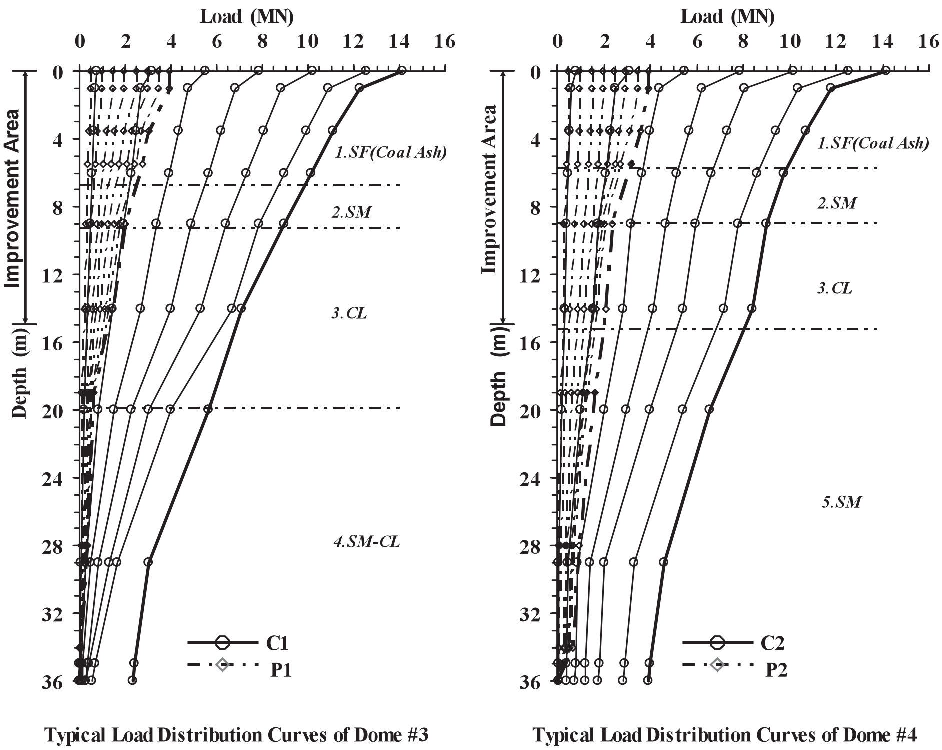
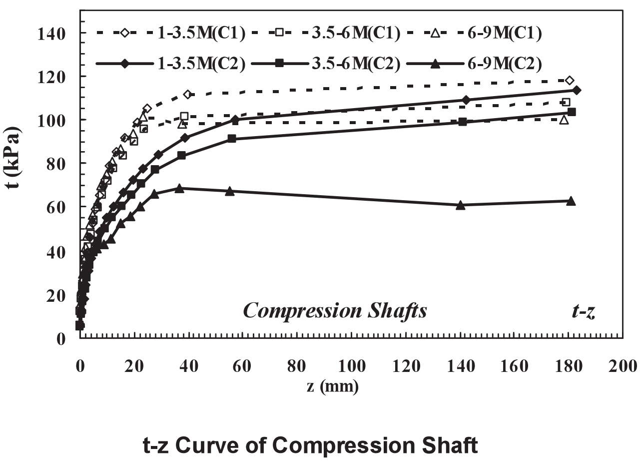


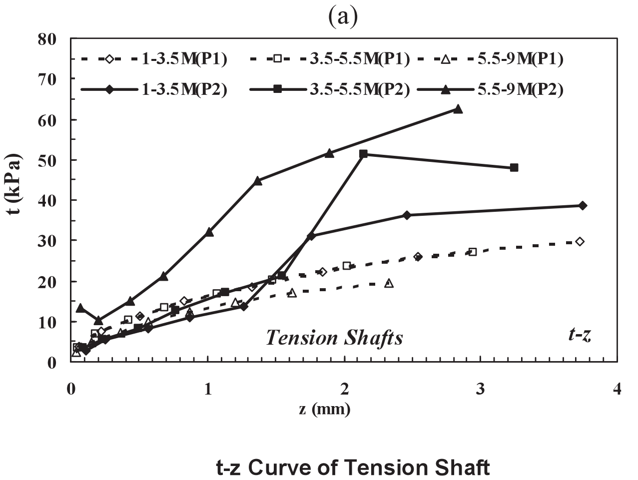
Fig. 10 Load distribution curves along the tested shafts

Compression loading test of shaft followed the pattern of ASTM D1143-81 (1994), using the optional quick-load test procedure. Settlement readings were taken immediately after each load increment and at one minute and then at every two minutes before increasing the load. When the maximum load was attained, the loading was maintained until the settlement rate was less than 0.25mm/hr before unloading. Then, the maximum loading was unloaded by four decrements. The testing procedures suggested by ASTM D3689-90 (1995) were followed for shaft tension load test.

The load-displacement relations at the pile head for the tested shafts at dome #3 and dome #4 are shown in Fig. 9. For tension test shafts P1 and P2, the load versus displacement relationships is almost identical at both sites. However, the compression test shaft C1 at dome #3 appears to have higher side resistance and lower base capacity. The skin friction resistance of both compression shafts takes up to 83 percent and 72 percent of the total load for the shafts at dome #3 and dome #4, respectively. Fig. 10 shows the transferred axial force in the shaft as a function of depth. Compare to the load displacement relation of Fig. 9 for the compression shafts C1 and C2, the maximum applied load 14.13 MN and 14.20 MN at dome #3 and dome #4 are all beyond the yield point. The trends of load distribution of four shafts are similar except the compression shaft at dome #4 takes higher toe resistance. For compression test shafts C1 and C2, the skin friction resistance through SCP improved area carries up to 56 percent and 68 percent of the total load carried by the shafts at dome #3 and dome #4 site, respectively. For tension test shafts P1 and P2, much higher skin friction has been mobilized at SCP improved zone. A set of side resistance, *t*, versus local shaft movement, *z*, curves at improved area based on measured results is shown in Fig. 11. For compression test shafts, the C1 pile gives slight higher unit side resistance. However, the t-z curves of C1 and C2 at depth of 6 to 9 meters shows softening after reaching the peak resistance at displacement of 21mm and 34mm of shafts at dome #3 and dome #4 site, respectively. For tension test pile, the P2 shaft gives higher resistance than that of the P1 shaft. Also, the side resistance of compression shafts is higher than that of the tension test shafts.



(a)



(b)

Fig. 11 t-z curves at SCP improved area of compression shafts of C1 and C2 and tension shafts of P1 and P2

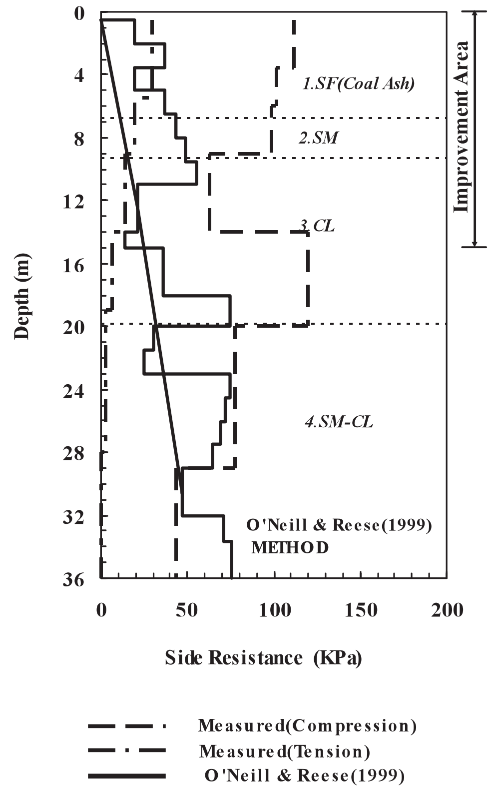


Fig. 12 Measured and estimated skin resistance along shafts

In general, the α -method (Tomlinson 1957), and the β -method are the most often used methods for pile shaft skin frictional resistance estimation along depth. In addition to the site ground conditions, due to the physical characteristics of coal ash, especially after sand compaction piles improvement, pile friction resistance through this improved coal ash layer is treated as cohesionless soils. Hence, the β -method is considered in the paper. For cohesionless soil, the current FHWA guidelines (O'Neill and Reese 1999) suggest the unit side shear as

$$f_S = \beta \cdot \sigma_v^{'} \qquad (1)$$

where *z* is the midheight depth in meters; σ_v' is effective vertical stress at midheight; and β can be expressed as

$$\beta = 1.5 - 0.245[z(m)]^{0.5} \qquad (2)$$

for $SPT\ N_{60} \geq 15$ or for $SPT\ N_{60} < 15$

$$\beta = [N_{60}/15]\{1.5 - 0.245[z(m)]\}^{0.5} \qquad (3)$$

Estimation of shaft skin resistance along depth interpreted using β -method (O'Neill and Reese 1999) is shown in Fig. 12. The measured resistance is the average value of the shaft test results obtained from both sites.

The resistance estimated by β method appears in between tension and compression test results. The resistance estimated by β method appears in between tension and compression test results.

The β versus $\sqrt{z}$ relationship via the shafts tested data and the O'Neill and Reese (1999) is shown in Fig. 13. Significant difference of the β versus $\sqrt{z}$ relationship is observed between the tested results and that of the O'Neill and Reese (1999). Due to the low unit weight of the coal ash for effective stress calculation, the estimated results by using β-method tends to underestimate shaft frictional resistance.

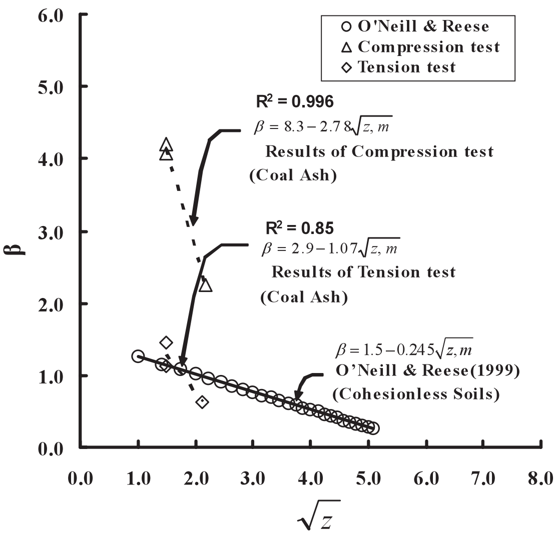


Fig. 13 β versus $\sqrt{z}$ relationship

CONCLUSIONS

From the study in this paper, the following observations can be summarized:

1. Conventional procedure of SCP has a limit when applied to the coal ash pond which contains high fine particle content. A high sand replacement ratio up to 0.342 was used in this project.
2. Construction of SCPs in this project results in the SPT-N value increasing by a factor of 5.6.
3. Coal ash pond after improved by sand compaction piles can also provide good shaft frictional resistance between drilled shaft and surrounding improved coal ash.
4. The β versus $\sqrt{z}$ relationship used by O'Neill and Reese (1999) for drilled shaft skin friction resistance calculation is different from either compression or tension shaft load test results.

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