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UPLIFT CAPACITY OF A WELL-INSTRUMENTED HAND-DUG CAISSON IN TAICHUNG GRAVEL

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ABSTRACT

Hand-dug caissons are often used in Taichung City to serve as a structural element to counter the buoyance induced by ground water for high-rise buildings with multi-level basement. The subsurface of Taichung City consists predominately of very dense sandy gravel that cannot be sampled for routine laboratory tests, thus impeding the use of empirical equations to estimate the capacities of caissons for lack of strength or index parameters of gravels. This paper presented the pile loading test result of a well-instrumented hand-dug caisson to shed a light on the uplift behavior of caisson within Taichung gravel. It is found that ultimate side friction of this caisson is about 25% higher than the maximum values stipulated in local design code, and the mobilized side friction is likely a function of the overburden pressure. Using the β-method as a basis, the ratio of ultimate skin friction to the overburden pressure of this test is calibrated against the published average curve. The calibrated curve was found shifting to the right of the original curve, indicating that Taichung gravel is perhaps more capable of providing a higher uplift capacity than average gravels.

Key words: Uplift capacity, pile loading test, gravel, hand-dug caisson.

1. INTRODUCTION

Taichung City is currently the second largest metropolis in Taiwan with a population near 3 million. Geographically, Taichung City locates in the middle of a vast alluvium basin in central Taiwan, and the subsurface consists mainly of very dense sandy gravel with a thickness of more than 100 m. From a geotechnical point of view, this thick dense sandy gravel is a competent bearing layer that offers high bearing capacity for mat or raft foundations of high-rise buildings, and foundation settlement induced by structural loading is generally small. With a ground condition dominated by competent gravel layer, it appears that piling contractors should have little room for survival in Taichung City, but in reality, the reverse is true. The reason behind a booming Taichung piling industry is primarily a result of the insatiable need for multi-level underground parking for high-rise buildings. Self-weight of these underground parking is in general less than the uplift force induced by ground water, thus requiring the use of piles to provide additional resistance against buoyancy.

For years, geotechnical engineers are mesmerized with the problem in estimating the ultimate uplift capacity for piles fully embedded in Taichung gravel. As stipulated in the design code (MOI 2001), one can use empirical equations that rely on blow counts of Standard Penetration Test (SPT N) as a design parameter to achieve this end. But it turns out this approach yields an ultimate side friction of no more than 0.15 MPa, which is far below the actual magnitude as revealed by pile loading tests. Another approach outlined in design code would be to use shear strength parameters in conjunction with Mohr-Coulomb equation to calculate the side friction. This approach may be useful for clayey or sandy materials, but renders useless when Taichung gravel is encountered. As it is almost impossible to retrieve undisturbed gravel samples for laboratory testing to obtain representative shear strength parameters for Taichung gravel. Amid all these difficulties in estimating the uplift capacity of piles in Taichung gravel, it appears that the most reliable approach for pile design would be to conduct full scale pile loading tests, though at a stiff cost for small or mid-size development projects.

This paper presents the uplift test results of a fully instrumented hand-dug caisson within Taichung gravel. The uplift behavior of this test pile was closely observed in the field and side frictions mobilized along the shaft at different depths were calculated. An effort was made to compare the mobilized side friction against published data, and it is believed that the test results presented herein can serve as a reliable reference for designer to assess the uplift capacity of piles in Taichung metropolis for future projects.

2. PROJECT DESCRIPTION

The project site is located in down town Taichung and is about 9,100 m^2 in site dimension. The developer planned to build a 39-story high-rise landmark apartment with a 4-level basement for parking purpose. The excavation depth of the basement is about 17.9 m and the basement covers almost the entire footprint of the project site. The basement structure may subject to an uplift pressure of around 0.18 MPa should ground water level rise to the surface under torrential rains. Since the 39-story super-

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structure occupies only about 50% footprint of the project site, a large portion of basement simply does not have sufficient self-weight to counter the uplift pressure induced by high ground water table. Knowing the lack of sufficient structural weight, the structural engineer designed an array of hand-dug caissons underneath the base slab of raft foundation to tie-down the basement. However, the structural engineer had little information on the uplift capacity of hand-dug caisson, therefore a test program on the uplift capacity of hand-dug caisson was implemented prior to the design stage. The structural engineer would eventually finalize the size of all hand-dug caissons, including the depth and diameter, based upon the test results.

3. GROUND CONDITIONS

A routine site investigation program was carried out for this project site, which showed a typical subsurface profile of Taichung basin without yielding too much information about the engineering properties of Taichung gravel. As revealed by the site investigation result, subsurface of the project site consists of a 2 m ~ 3 m thick top soil overlying a very thick and dense sandy gravel to at least 40 m in depth. The top soil is a medium stiff to stiff silty clay with a typical undrained shear strength (s_u) of about 0.05 MPa, which is considered a competent bearing layer for footings of low-rise buildings. The underlying sandy gravel layer is very dense, and the blow counts (SPT N) are always higher than 100. For high-rise buildings with multi-story basement, foundation level is always situated within the very dense sandy gravel, and the properties of the sandy gravel are of primary concern. Geological survey conducted by petroleum industry revealed that the thickness of this dense sandy gravel layer may exceed 200 m at the center of Taichung basin. For engineers and contractors involved in the deep excavation or piling projects in Taichung, this thick sandy gravel is regarded as a more or less uniform material with high bearing capacity, though the engineering properties of this material are yet to be thoroughly studied.

A sieve analysis had been conducted to delineate the composition of Taichung Gravel at this project site. Backhoe was used to excavate a test pit and to retrieve a large quantity of sandy gravel for on-site sieve analysis (Fig. 1). The grain size distribution of this particular sieve analysis was presented in Table 1, which showed that the weight percentages of cobble, gravel, sand and fine content are 60%, 23%, 14%, and 3%, respectively. Cobbles over 0.7 m in size may also be encountered (Fig. 2). The grain size distribution shown in Table 1 is considered representative of Taichung gravel, as a majority of deep excavation projects in Taichung all encountered the same material (Fig. 3).

Table 1 Grain size distribution of sandy gravel at project site

Sieve Number	Weight (kN)	Percentage Retained (%)	Cumulative Retained (%)
24"	0	0.00	0.00
24" ~ 12"	1.51	7.06	7.06
12" ~ 6"	4.07	19.06	26.12
6" ~ 3"	7.25	34.00	60.13
3" ~ 1.5"	2.87	13.46	73.59
1.5" ~ 3/4"	1.85	8.66	82.24
3/4" ~ #4	0.18	0.83	83.07
#10	0.13	0.59	83.66
#20	0.16	0.77	84.42
#40	1.08	5.03	89.45
#60	0.68	3.21	92.65
#100	0.62	2.92	95.57
#200	0.34	1.58	97.15
Pan	0.61	2.85	100.00
Total	21.35		

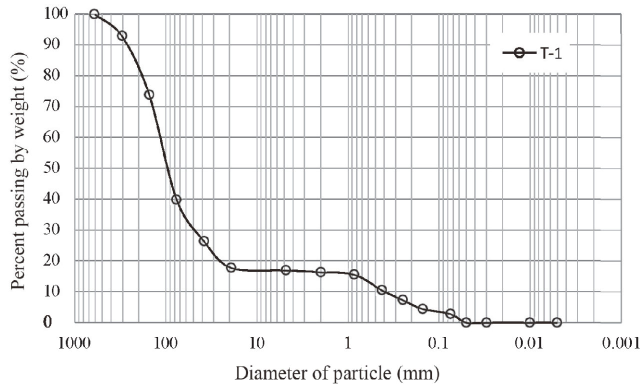


Fig. 1 Test pit excavation

Fig. 2 Cobble encountered in the test pit

Fig. 3 Typical deep excavation in Taichung

By looking at the grain size distribution and the field photos, it is apparent that retrieving undisturbed Taichung gravel samples for routine laboratory strength tests would not be feasible, which leads to great uncertainties in determining the shear strength parameters of Taichung gravel, and that in turn casts doubts in the excavation and foundation designs. Efforts had been made to back-calculate the effective friction angle (ϕ') of Taichung gravel by using a numerical tool in conjunction with the observed wall displacement curves of a 21.3 m deep excavation in Taichung (Chu *et al.* 1995). To match the relatively small displacements of the retaining wall, an effective friction angle of 66° was used in the back-analysis, and this number is considered by many designers as non-conservative. Another study utilizing plate loading test results as a basis for back-calculation listed the effective friction angle of Taichung gravel at 54.3° (Lin *et al.* 1998), which is still regarded as an overly optimistic number among design engineers. For routine excavation or foundation designs, 40° is a generally accepted number as the effective friction angle of the Taichung gravel, though sometimes a conservative number of 35° is used for certain risky projects. For this project site, the soil parameters for foundation and excavation designs are presented in Table 2. Shear strength parameters for gravel listed in Table 2 are mainly based upon experience rather than test results, and these numbers are considered conservative.

4. TEST PROGRAM

As mentioned above, the structural engineer had to know the uplift capacity of hand-dug caisson before the foundation design can be finalized, therefore a single set of uplift test was carried out following the completion of site investigation program. Layout of the uplift capacity test was illustrated in Figs. 4 and 5. The top soil, which consists of fill material and stiff clay, was first removed to expose the very dense gravel layer underneath. A hand-dug caisson (TPT1) 1.5 m in diameter and 6 m in depth was then constructed in the field to serve as the test pile (Figs. 6 and 7). It can be seen clearly in these photos that shaft surface of the hand-dug caisson is very rough, which is considered beneficiary to the development of side friction when loaded. Since the sandy gravel is a very competent bearing layer, reaction piles were replaced by two piers 1.5 m × 1.5 m in dimension and 2 m in depth as shown in Fig. 8. A total of 24 rebar strain gages were installed within the hand-dug caisson to measure the strain variation of rebars when subjected to uplift loading (Fig. 9). The installation depths of rebar strain gages are listed in Table 3 while the field installation photos are shown in Fig. 10. Other instruments, including LVDTs, load cell at top of the caisson and 4 extensometers, were also installed on top or within the caisson (Table 3). Procedure A of ASTM3689-07, which is a quick test procedure, was followed to conduct the uplift test on TPT1. A maximum uplift load of 25 MN was specified in the test program with the intention to fail the caisson at its ultimate uplift capacity.

Table 2 Simplified soil parameters of the project site

Layer	Description	Depth (m)	Thickness (m)	SPT N	γ_t^* (MN/m^3)	ω_n (%)	S_u (MPa)	c'^* (MPa)	ϕ'^* (°)	E^* (MPa)
1	Topsoil	2.3 ~ 3.4 (2.7)	2.7	–	0.018	23.8	0.05	0	30	20
2	Gravel	3.4 ~ 40.0	–	>100	0.021	–	–	0	45	> 125

Note: 1. ( ) means the average value, * values are based on experience.
2. Assume the water level at GL.–5m and GL.–3m in normal condition and torrential rains, respectively.

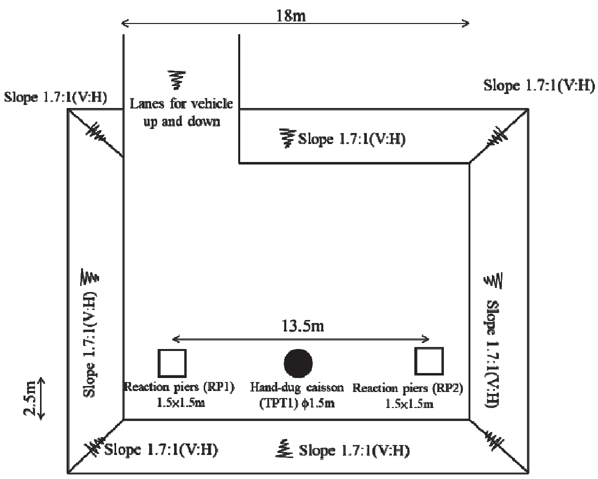


Fig. 4 Plan layout of the uplift test

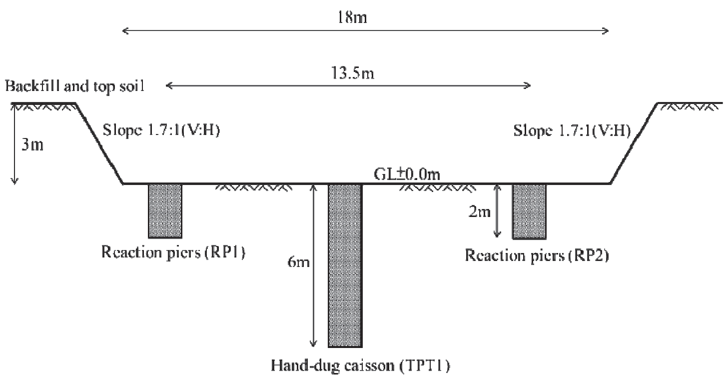


Fig. 5 Profile of the uplift test

Fig. 6 Construction of the hand-dug caisson (1)

Fig. 7 Construction of the hand-dug caisson (2)

Fig. 8 Construction of the reaction pier

Fig. 9 Rebar strain gages of the test pile

Fig. 10 Installation of the reinforcement cage together with the strain gages

Table 3 Instruments installed within the hand-dug caisson

Instruments within the caisson	Rebar strain gages	4 at GL.–0.18m
		4 at GL.–1.175m
		4 at GL.–2.17m
		4 at GL.–3.15m
		4 at GL.–4.14m
		4 at GL.–5.235m
	Extensometers	1 at GL.–3.15m
		1 at GL.–4.14m
		2 at GL.–5.235m
Instruments at top of the caisson	LVDT	4 at top of TPT1, 2 at top of reaction piers
	Load cell	6 at top of TPT1

5. TEST RESULTS

Though a maximum uplift load of 25 MN was planned, the test pile apparently reached its ultimate capacity at just around 5.9 MN, which is less than 25% of the expected capacity. The load-displacement curve is presented in Fig. 11, showing that the test pile could not provide further uplift resistance once the applied load reached 5.9 MN. At 5.9 MN, the test crew were unable to maintain the applied load at 5.9 MN, and the test pile continued to rise at an accelerating rate. The test was terminated when the pile top reached an upward displacement around 150 mm. After fully unloading, a residual displacement of 105 mm was observed. Details regarding the uplift displacements of test pile and settlements of reaction piers are listed in Table 4 for reference.

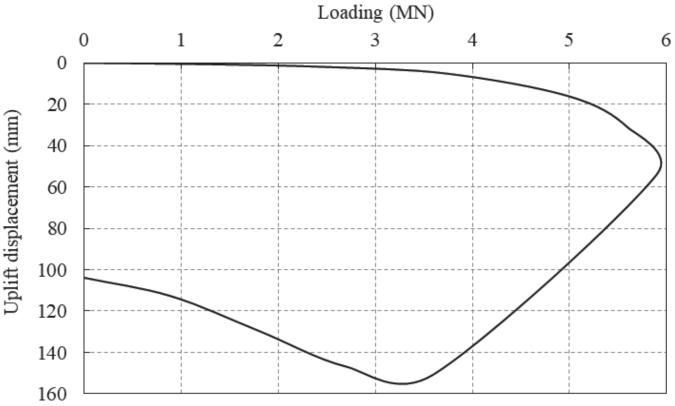


Fig. 11 Load-displacement curve of the test pile

Table 4 Uplift displacements of test pile and settlements of reaction piers

Loading (MN)	Settlement of reaction pier (mm)				Uplift displacement of TPT1 (mm)			Average uplift displacement at top of TPT1 (mm)
	RP1		RP2					
	S5	S8	S6	S7	GL.−3.15m	GL.−4.14m	GL.−5.235m	
1.25	0.18	0.19	0.22	0.26	0.28	0.12	0.11	0.6
2.5	0.48	0.51	0.54	0.65	1.26	1.17	1.1	1.95
3.75	0.87	0.99	1.01	1.02	3.99	3.73	4.12	5.2
5.0	2.09	2.37	1.94	1.46	14.68	14.25	14.88	16.18
5.6	4.38	4.77	3.17	2	29.08	28.82	29.6	31.17
5.9	8.3	9.14	5.04	1.68	50.73	51.39	52.15	53.83

The strain gage readings had further been processed to calculate the axial forces developed within the test pile. Assuming no slippage occurred between rebars and surrounding concrete during all loading stages, the strains of rebar and concrete at the same cross section are assumed to be the same. Knowing the strain levels, cross sectional areas and Young's moduli of rebars/concrete, it is then a straight forward calculation to estimate the axial load at a certain cross section of the test pile. In this case, rebar strain gages were installed at 6 cross sections as indicated in Table 3, enabling the engineer to plot the variation of axial load along the shaft of test pile as shown in Fig. 12.

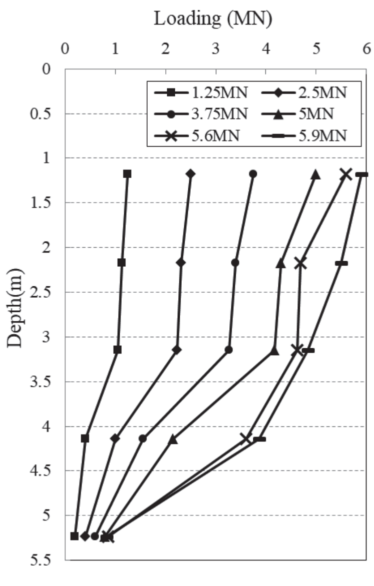


Fig. 12 Variation of axial load along the shaft of test pile

Further assuming that the difference in axial load of two adjacent cross sections is the side friction developed between these two sections, it is then possible to calculate the variations of side friction along the shaft at all loading stages. The results are presented in Fig. 13 and Tables 5 ~ 7. In the context of pile behavior, the curves shown in Fig. 13 are referred to as the *t-z* curves. As shown in Fig. 13, the *t-z* curves showing the development of side friction versus shaft displacement are not necessarily smooth ones as speculated, which adds to the difficulties in interpreting the results.

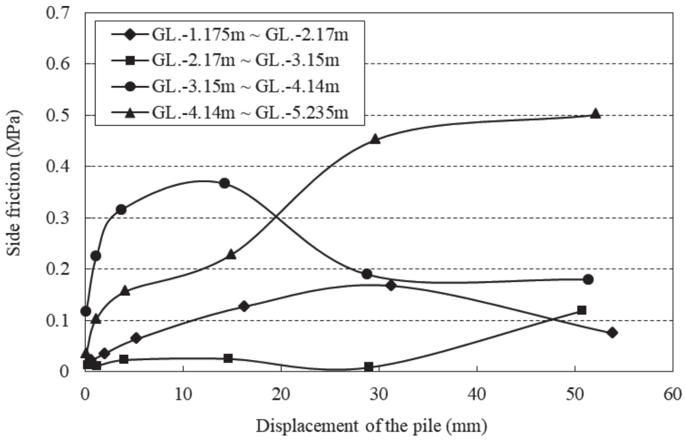


Fig. 13 Variation of side friction with the development of shaft displacement (GL.±0 is at top of the hand-dug caisson)

Table 5 Stresses of rebar strain gages at different load levels

Depth (m)	Loading					
	1.25 MN	2.5 MN	3.75 MN	5.0 MN	5.6 MN	5.9 MN
GL.−1.175m	2.33	5.15	10.77	14.50	16.35	18.33
GL.−2.17m	2.17	4.86	7.92	11.10	12.67	16.86
GL.−3.15m	2.02	4.70	7.53	10.56	12.46	13.44
GL.−4.14m	0.76	1.93	3.06	4.50	8.54	9.45
GL.−5.235m	0.36	0.73	1.11	1.47	1.63	1.60

Unit of stress: MPa

Table 6 Axial forces at different load levels

Depth (m)	Loading					
	1.25 MN	2.5 MN	3.75 MN	5.0 MN	5.6 MN	5.9 MN
GL.−1.175m	1.25	2.50	3.75	5.0	5.6	5.9
GL.−2.17m	1.13	2.31	3.40	4.30	4.68	5.49
GL.−3.15m	1.06	2.24	3.27	4.17	4.63	4.84
GL.−4.14m	0.41	1.01	1.55	2.16	3.6	3.86
GL.−5.235m	0.20	0.40	0.60	0.79	0.86	0.83
Uplift displacement at top of TPT1 (mm)	0.6	1.95	5.2	16.18	31.17	53.83
Uplift displacement at bottom of TPT1 (mm)	0.11	1.1	4.12	14.88	29.6	52.15

Note: 1. Uplift displacement at top of TPT1 is the average value of 4 LVDTs. Uplift displacement at the bottom of TPT1 (GL.−5.235m) is the average value of extensometers.
2. Unit of axial force: MN.

Table 7 Side frictions at different load levels

Depth of soil (m)	Loading					
	1.25 MN	2.5 MN	3.75 MN	5.0 MN	5.6 MN	5.9 MN
GL.–1.175m ~ GL.–2.17m	0.02	0.03	0.06	0.13	0.17	0.08
GL.–2.17m ~ GL.–3.15m	0.01	0.01	0.02	0.03	0.01	0.12
GL.–3.15m ~ GL.–4.14m	0.12	0.23	0.32	0.37	0.19	0.18
GL.–4.14m ~ GL.–5.235m	0.04	0.10	0.16	0.23	0.45	0.50

Note: Unit of side friction: MPa.

6. ULTIMATE SIDE FRICTIONS

From Fig. 13, the side frictions mobilized along the shaft of test pile tend to fluctuate with the shaft displacements. At a certain depth, for example at GL.–4.14m to GL.–5.23m, the side friction may first reach its peak value and then drop to a low level. Or the side friction may keep increasing with the shaft displacement without a clear pattern. Rough surface of the hand-dug caisson may be a key factor leading to the erratic behavior of side frictions, as sudden slips might occur at the interface of caisson shaft and gravels.

Ignoring the erratic ups-and-downs of *t-z* curve in Fig. 13 and simply taking the maximum value as the ultimate side friction at that particular depth, the ultimate side frictions developed along the shaft are summarized in Table 8. The ultimate side frictions vary from a low value of about 0.12 MPa at GL.–2.17m ~ –3.15m to a high value of 0.5 MPa at GL.–4.14m ~ –5.23m, apparently without a clear pattern such as increasing with the depth. The irregularities of the ultimate side frictions may be attributed to the roughness of the shaft surface, as the rough surface of the caisson may lead to stress concentration at certain depths, and the measured rebar strains may not necessarily yield the representative average side friction at that specific depth. Since the results showed in Table 8 appears confusing, it would be much straight forward if we use the average side friction for the whole length of caisson as the subject of study. For a relatively short pile as TPT1, the average side friction for each loading stage is simply taken as the ratio between loading at top of the caisson and the circumferential area of caisson. As shown in Table 9, the average side friction gradually increased to an ultimate value of 0.19 MPa at the peak uplift load of 5.9 MN.

Table 8 Maximum side frictions of the test pile

	Depth of soil (m)	Maximum side frictions
Maximum side frictions	GL.–1.175m ~ GL.–2.17m	Up to 0.17 MPa and down to 0.08 MPa
	GL.–2.17m ~ GL.–3.15m	0.12 MPa
	GL.–3.15m ~ GL.–4.14m	Up to 0.37 MPa and down to 0.18 MPa
	GL.–4.14m ~ GL.–5.235m	0.50 MPa

Table 9 Average side friction of the test pile

	Loading					
	1.25MN	2.5MN	3.75MN	5.0MN	5.6MN	5.9MN
Average side friction	0.04	0.08	0.12	0.16	0.18	0.19

Note: Unit of average side friction: MPa.

This ultimate value of 0.19 MPa is 25% higher than the maximum value cited in the local design code, which stipulates that the side friction estimated by the use of empirical equation is not allowed to exceed 0.15 MPa even if the SPT N value is higher than 50.

7. COMPARISON WITH OTHER HAND-DUG CAISSON TEST RESULTS

There are published (Chen and Hsieh 2001) and unpublished uplift test results available for this study. The unpublished results were provided by local contractors that only addressed the size of test pile and its ultimate capacity. Basic information and uplift test results of these hand-dug caissons were compiled in Table 10, and the results showed that the average side friction varied from around 0.18 MPa to 0.5 MPa. Though all of the hand-dug caissons were embedded in Taichung gravel, they were constructed and tested at different elevations pending on the thickness of top soil and other construction factors. Owing to schedule issues, some of them were even constructed and tested while the basement excavation was in progress (Fig. 14). All these discrepancies in test condition, including different overburden pressure and excavation disturbance, are possible factors that affect the eventual test results. As for the TPT1 caisson presented in this paper, the average side frictions of 0.19 MPa is on the lower end when compared with other results. It is not clear why the result of TPT1 is at the lower end. The construction of TPT1 was well scrutinized, thus excessive disturbance of the ground is an unlikely factor leading to the

Table 10 Comparison with other hand-dug caisson test results

Test No.	PT-2	PT-3	PT-5	PT-7	A	B	C	D	TPT1
Diameter (m)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Length (m)	9	9	6	6	6	6	6	9	6
Ultimate load (MN)	19.6	21.3	9.8	17.8	9	5	11	17	5.9
Max. uplift (mm)	19.54	9.5	51.07	51.67	–	–	–	–	53.8
Avg. side friction (MPa)	0.46	0.5	0.35	0.49	0.32	0.18	0.39	0.4	0.19

Notes: 1. PT-2, PT-3, PT-5, and PT-7 are published data (Chen and Hsieh, 2001)
2. A, B, C, and D are unpublished data (personal communication)
3. TPT1 is this case study.

Fig. 14 An uplift pile loading test conducted at the bottom of basement excavation

comparatively low side friction. A comparatively low gravel content or smaller gravel size at the test site may lead to a test result that is at the lower limit, further implying that Taichung gravel is a non-uniform material in nature. Since no construction or test details are available for further assessment, these data can only be regarded as part of a growing statistic data base, and care must be exercised when referring to these data for actual design.

8. COMPARISON WITH DRILLED SHAFT TEST RESULTS

Asides from being constructed by conventional hand-dug technique, there were tension piles installed by full-cased drilling scheme in recent years. Though expensive in terms of construction cost, the full-cased drilling technique can be more versatile. Full-cased drilled shaft can be constructed under water and can reach depth in excess of 40 m, far more than the depth conventional hand-dug technique can reach. Another advantage of the full-cased drilled shaft is the construction crew no longer have to work under the pit, which is a risky maneuver by all means. However, some believed that the use of casing may create a smooth pile shaft that could perhaps reduce the side friction to a much lower level, which is definitely not a good outcome in using the full-cased drilling scheme.

Though rare, four uplift test results on full-cased drilled shaft are available to this study. Among which, three are published data (Chen and Hsieh 2001) while the remaining one is acquired through personal communication. The test details and results are summarized in Table 11, which shows that the ultimate side frictions are varying from 0.30 MPa to 0.52 MPa. From Table 11, it appears that the ultimate side frictions developed along the shaft of full-cased drilled shafts are more or less in the same range of hand-dug caissons, there is no discernable difference between the results of hand-dug caissons and full-cased drilled shafts. Of course, the data base is too small to make a conclusive comparison, but perhaps the surface of a full-cased drilled shaft is not very smooth as speculated.

One additional note on the full-cased drilled shaft test results is that two of the test piles (PT-1 and PT-4) were embedded at a certain depth rather than starting at the ground surface. For PT-1, tremie concrete was cast from GL.–26m to GL.–15m. While for PT-4, tremie concrete was cast from GL.–21m to GL.–15m. The purpose of casting concrete only to a certain depth instead of all the way to the ground surface is to simulate the field condition that the tension piles were actually installed underneath the base slab of basement. By doing so, these two test piles were actually under a better confinement than the other two test piles situated near the ground surface (PT-6 and PT-TB). The difference in confinement may lead to the difference in average side friction of these two types of test piles. For test piles under better confinement, the average side friction is about 0.5 MPa, which is about 40% higher than the other two test piles with lower confinement.

Table 11 Uplift test results of four full-cased drilled shafts

Test No.	PT-1	PT-4	PT-6	TP-TB
Diameter (m)	1.5	1.5	1.5	1.2
Length (m)	11	6	6	15
Installation depth (m)	GL.–15m~ GL.–26m	GL.–15m~ GL.–21m	GL.0m~ GL.–6m	GL.0m~ GL.–15m
Ultimate load (MN)	27	14.5	9.4	17
Max. uplift (mm)	15.2	10.45	62.2	12.5
Avg. side friction (MPa)	0.52	0.51	0.33	0.3

Notes: 1. PT-1、PT-4、PT-6 are published data (Chen and Hsieh 2001)
2. TP-TB is unpublished data (personal communication)

9. NUMERICAL SIMULATIONS

One intriguing question regarding the engineering properties of Taichung gravel is what the strength parameters, namely c' and ϕ', should be used for foundation or excavation design. One set of strength parameters with $c' = 0$ and $\phi' = 45°$ is often used for design, while others argued that a higher friction angle of $\phi' \geq 50°$ is possible. Field observations in deep excavation projects showed that Taichung gravel often has an extremely good self-standing ability (Fig. 3), implying that Taichung gravel not only has a high friction angle, but may also has an apparent cohesion as a result of cementation or interlocking effect. In other words, $c' \neq 0$ and $\phi' \geq 45°$ are very likely numbers for Taichung gravel. Currently, there are uncertainties in the selection of strength parameters for Taichung gravel, and the most probable c' and ϕ' values remain unresolved.

The uplift test results of this hand-dug caisson can serve as a good basis to delineate the strength parameters of Taichung gravel. Parametric studies using numerical tool such as PLAXIS can be carried out to fit the load-displacement curve with a series of possible c' and ϕ' combinations. Comparing the uplift test curve and the simulation curves can provide a certain confidence level regarding the probable values of strength parameters. There were no attempts to pursue an exact or near perfect fit of the test results in this paper, as the current simulations ignored many numerical details and focused only on the effects of varying strength parameters.

PLAXIS code (2013) in conjunction with simple Mohr-Coulomb model to simulate ground behavior is used in this study. The test pile is considered as a linear elastic material and simulated in an axis-symmetric manner. The presence of reaction piers is ignored for simplicity. The analysis domain is presented in Fig. 15. The domain boundaries on both sides of Fig. 15 are free to move in the Y-direction (roller-connected), while the bottom of analysis domain is considered as pin-connected. Basic input parameters, including unit weight, Young's modulus, shear modulus and Poisson's ratio for topsoil,

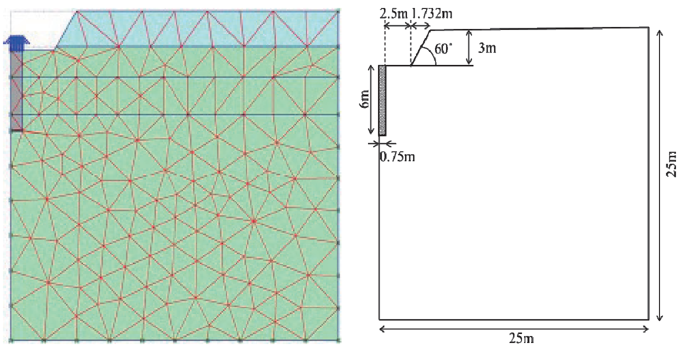
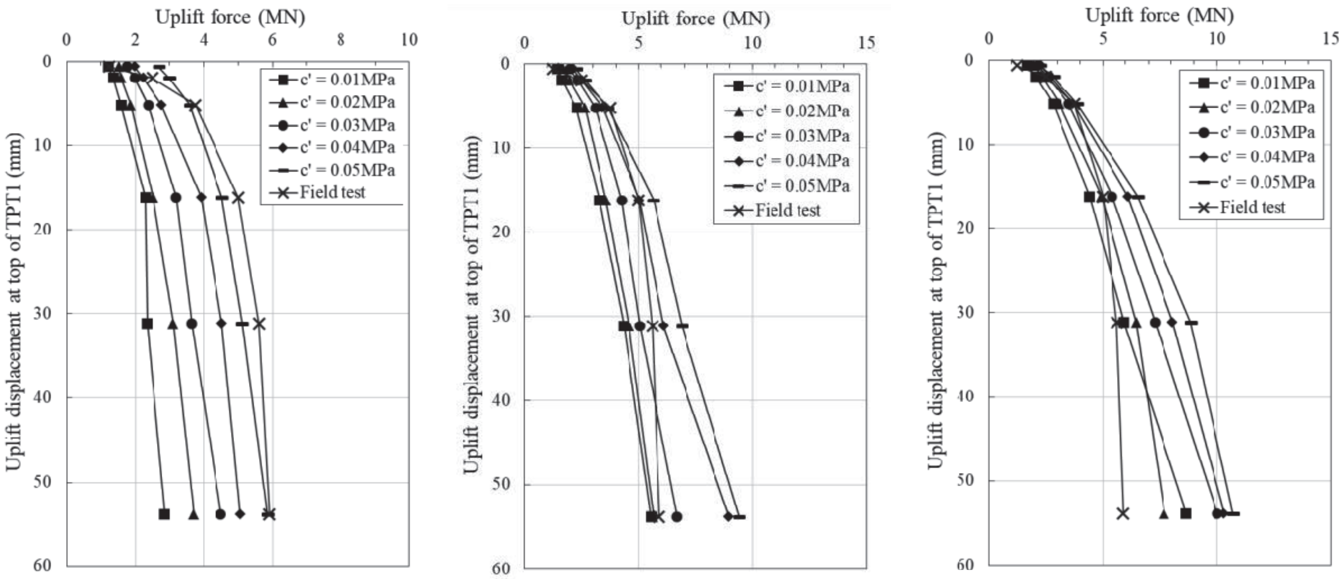


Fig. 15 PLAXIS analysis domain for the test pile

gravel and hand-dug caisson, are listed in Table 12. A series of (c', ϕ') parameters, covering c' from 0.01 MPa to 0.05 MPa and ϕ' from 30° wide to 55°, were adopted in the numerical simulations. The uplift capacities from numerical simulations with different (c', ϕ') values are presented in Table 13. As indicated in Table 13, it appears that (c'= 0.05 MPa, ϕ' = 45°) has an approximate fit of the uplift test result.

The simulated load-displacement curves for ϕ' = 45°, 50°, 55° are also presented in Fig. 16 for reference. None of these simulated curves has a close fit of the field test curve, which is perhaps the drawback of adopting a simple Mohr-Coulomb model together with a low Young's modulus of 200 MPa. Additional numerical simulations were carried out using Young's modulus varying from 125 MPa to 300 MPa in conjunction with (c' = 0.05 MPa, ϕ' = 45°), and the results are shown in Fig. 17. Only slight improvement of the simulation results is observed if the Young's modulus is increased from 125 MPa to 300 MPa. It is speculated that a simple Mohr-Coulomb model is not fully capable in simulating the load-displacement behavior of a hand-dug caisson under tension in Taichung gravel.

The above results were numerical simulations at a primitive level, and this pair of parameters (c' = 0.05 MPa, ϕ' = 45°) are yet to be regarded as the actual strength parameters for this specific site unless more detailed studies are carried out. However, it has the implication that Taichung gravel may have an apparent cohesion together with a high effective friction angle. Though field experience indicated that Taichung gravel might have a friction angle higher than 45°, care must be exercised in selecting the appropriate design parameter as ϕ' > 45° generally implies that this material is perhaps behaving as certain type of rock from a theoretical point of view.



(a) ϕ' = 45° (b) ϕ' = 50° (c) ϕ' = 55°

Fig. 16 Numerical simulation results of the load-displacement curve with different friction angle and cohesion

Table 12 Input parameters for soil strata and caisson

Parameters	Topsoil	Gravel	Caisson
Unit weight, γ_t (MN/m^3)	0.018	0.021	0.024
Young's modulus, E (MPa)	20	200	20270
Shear modulus, G (MPa)	7.69	769	8660
Poisson's ratio, μ	0.3	0.3	0.17

Table 13 Uplift capacities from numerical simulations for different cohesions and friction angles

	c' = 0.01 MPa	c' = 0.02 MPa	c' = 0.03 MPa	c' = 0.04 MPa	c' = 0.05 MPa
ϕ' = 30°	0.68	0.99	1.3	1.53	1.92
ϕ' = 35°	0.79	1.05	1.50	2.21	2.86
ϕ' = 40°	1.64	1.75	3.03	3.62	4.16
ϕ' = 45°	2.86	3.7	4.50	5.06	5.85
ϕ' = 50°	5.54	5.65	6.67	8.96	9.42
ϕ' = 55°	8.65	7.69	10.01	10.32	10.72

Unit of uplift capacity: MN.

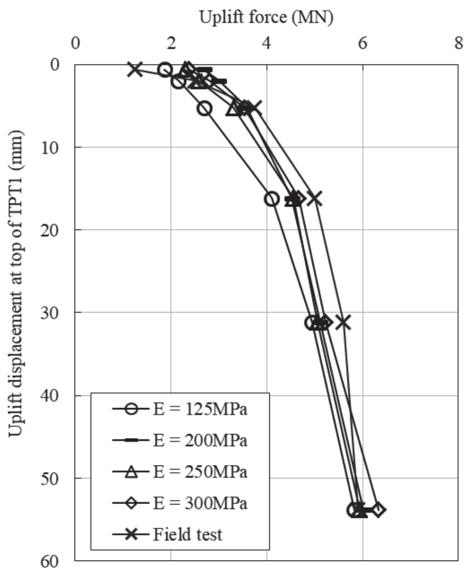


Fig. 17 Numerical simulation results of the load-displacement curve with different Young's modulus

10. POSSIBLE FAILURE MODE

When approaching the ultimate test load of 5.9 MPa, surface cracks near the test pile were observed. At the end of test when loading was fully removed, a residual upward pile top displacement of around 100 mm was recorded and surface cracks were visible as shown in Fig. 18. The surface cracks induced by the upward movement of test piles were marked by red paint that clearly identified the distribution, length and direction of cracks. It appears that these surface cracks were in a radial pattern and extended about 2 m away from the test pile, implying that the side resistance of this hand-dug caisson was in a cylindrical shear mode. On the other hand, the depth to diameter ratio of this test pile is 4 (6/1.5 = 4), which is considered as a short pile that may exhibit cone breakout failure pattern when subjected to uplift load (Kulhawy 1985). However, a cone breakout failure was not observed in this case. Detailed numerical simulation might yield more information regarding the actual failure pattern about this pile, but this issue is not within the scope of this study.

Fig. 18 Ground surface cracks around the test pile

11. β VALUES

It is postulated by many researchers that the side friction (f_s) of piles in gravelly soils is a function of effective stress in the form:

$$f_s = \beta\, \sigma'_z \tag{1}$$

where σ'_z = vertical effective stress in soil at depth z; β is a function of depth; and z = depth below ground surface. As proposed by Rollins *et. al.* (2005), the β value for gravel (> 50% gravel size and SPT N > 25) can be estimated using the following equation:

$$\beta = 3.4 \times {}^{(-0.085z)} \qquad 0.25 \le \beta \le 3.0 \tag{2}$$

where e = natural base (2.718). Plotting the side friction data shown in Tables 7, 10, and 11 on the β versus depth figure, it is obvious that the data points for uplift test piles in Taichung gravel are well on the right-hand side of Rollins' curve (Fig. 19). A revised curve in the form of $\beta = 6.0 \times e^{(-0.085z)}$ can be plotted, which may serve as a rough estimate of β value for Taichung gravel. The physical meaning of this new curve is Taichung gravel may perhaps provide more side friction than the average value of data base gravels presented by Rollins (2005). Noting that the SPT N value of Taichung gravel well exceeds 100 and the gravel/cobble content is over 80% as revealed by sieve analysis, there is little surprise that Taichung gravel tends to have a higher β value.

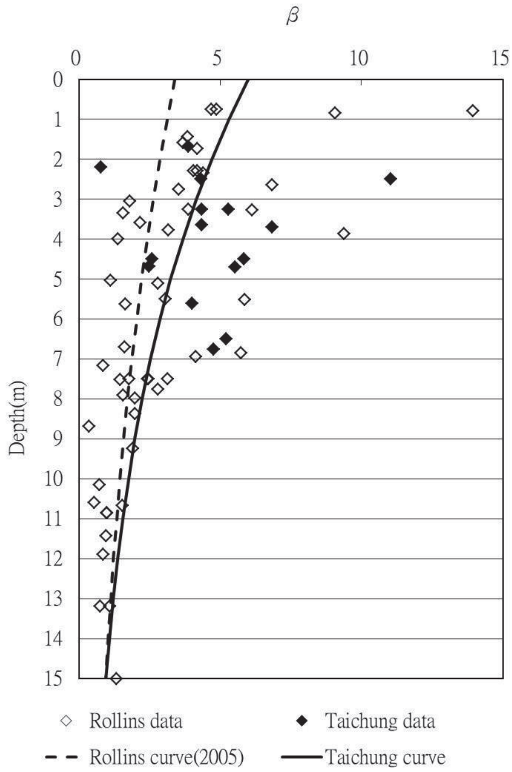


Fig. 19 Comparison of β values

12. DISCUSSIONS

Though there is only one set of uplift test, this test pile was well instrumented and closely observed for the entire construction stage, therefore the result may be of good reference value for future in-depth study. Other subjects covered in this paper, including numerical simulations and the β values of Taichung gravel, are admittedly an over stretching of the lone test result. The interpretations regarding numerical simulations and β value derivation are inconclusive in nature, but can at least provide the design engineers a preliminary guide when confronted with the excavation or foundation design in Taichung gravel.

One issue worth mentioning is the length of tension piles addressed in this paper, either constructed by hand-dug or full-cased drilled technique, is at least 6 m in all cases. Though empirical, 6 m is regarded as a minimal depth required to provide adequate uplift capacity in Taichung gravel and is a design rule observed by foundation designers and construction crew. Any pile less than 6 m in length may be confronted with the risk of being pulled out of ground well under the design load, perhaps under a failure mechanism vastly different from the one observed for this hand-dug caisson. Needing a maneuver diameter of at least 1.5 m for one single construction crew to work inside the hand-dug caisson, the depth to diameter ratio (D/B ratio) of a 6 m deep caisson is 4. In some instances, 1.8 m diameter caissons are constructed to accommodate for the occurrence of large boulders, and the D/B ratio drops to an even smaller 3.3. For tension piles with such low D/B ratios to perform well in Taichung gravel is truly surprising, perhaps the shear strength of Taichung gravel is much higher than expected and its contribution alone on pile capacity outweighs all other factors.

As for the practical use of β value, it is a straight forward calculation for the design engineer to estimate the side friction if both β and σ'_z are readily available. However, for deep excavation projects with tension piles installed underneath the base slab of basement, the use of β method is in jeopardy. Since the weight of basement is supposedly less than buoyance, the σ'_z experienced by the tension piles are negative numbers, that makes the use of β method very confusing. Though it is believed that these tension piles are still well confined by gravel and are able to provide adequate uplift resistance even the σ'_z values are negative, there is currently no guideline on how β method can be used under such circumstances. It is acknowledged that the stress field underneath the base slab is complex, the ground first experiences an unloading process during basement excavation, followed by a reloading process resulted from the construction of basement and superstructure. Ground water level also fluctuates significantly for this type of high-rise building construction. The ground water level is initially lowered to well below the final excavation depth to allow for the basement excavation and construction of the hand-dug caissons, then the deep wells are progressively switched off and ground water gradually returns to its normal level before the completion of superstructure. All these construction activities make the estimation on the variation of stress field extremely difficult, and as a result, the designers have low confidence level on the magnitude of vertical stress or confining stress acting on the tension piles when these piles are finally in service. Some design engineers even argued that the full weight of ground water acting above the top elevation of tension piles should be used in calculating the σ'_z value instead of the weight of basement, making σ'_z always a positive number and β method applicable under such circumstance. Though this appears to be a convincing argument, a consensus is yet to be reached among design engineers to calculate σ'_z in such a manner. Amid all the confusions regarding β method, it is suggested that the measured side friction from field test be used for design, while all the uncertainties on stress level acting on the tension piles be hopefully covered by the factor of safety at this moment.

13. CONCLUSIONS

This paper presents the uplift loading test result of a hand-dug caisson installed within the very competent Taichung gravel. The hand-dug caisson is 1.5 m in diameter and 6 m in depth. It reached its ultimate uplift capacity at a load level of 5.9 MN. The measured upward displacement at pile top is 53.8 mm when the applied load approached 5.9 MN. The ultimate side frictions deduced from the readings of rebar strain gages installed within the caisson vary with depth and range from 0.12 MPa to 0.5 MPa. At the caisson's ultimate capacity of 5.9 MN, the average side friction is about 0.19 MPa, which is about 25% higher than the maximum value of side friction stipulated in local design code.

Numerical simulations were also carried out to assess the possible shear strength parameters of Taichung gravel. Uplift load-displacement curve was back analyzed using a series of shear strength parameters to investigate which pair of c' and ϕ' values best fit the test curve. Though primitive, these numerical simulations indicated that Taichung gravel perhaps have a high friction angle of at least 45° and an apparent cohesion of around 0.05 MPa.

Comparing the uplift test results between hand-dug caissons and full-cased drilled shafts, it appears that pile construction techniques have little effect on the development of ultimate side friction. There is no discernable difference between the test results of these two construction techniques, implying that full-cased drilled shaft may also produce rough shaft surface comparable to that of a hand-dug caisson.

With a gravel content more than 80% and SPT N values higher than 100, the β value of Taichung gravel is about 75% higher than that of the average gravel reported by Rollins (2005). For construction projects with deep basement and high ground water level, the use of β method in estimating the uplift capacity of pile may be difficult as the effective vertical stress σ'_z can be of negative value. It is suggested that tension pile loading tests be always carried out and the measured side friction be used as the design value.

In summary, Taichung gravel is a competent material that provides high bearing capacity for foundation of high-rise buildings. However at this moment, the probable shear strength parameters of Taichung gravel remain unclear, and the design of piles rely more on experience rather than on theory. The test data presented in this paper provide the design engineer with direct and valuable information on the behavior of tension piles, but this is just one case history, and it is advised that advanced and systematic studies on the behavior of tension piles embedded within Taichung gravel be carried out in the near future to establish a robust design approach.

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DATA AVAILABILITY

This study does not generate new data and/or new computer codes.

CONFLICT OF INTEREST STATEMENT

The authors state that there are no financial interests or personal relationships that might influence the work reported in this paper.

NOTATIONS

c' Effective cohesion (MPa)

E Young's modulus (MPa)
f_s Side friction (MPa)
G Shear modulus (MPa)
N Blow counts of standard penetration test
S_u undrained shear strength (MPa)
z DEPTH below ground surface (m)
β Factor to estimate side friction
μ Poisson's ratio
ϕ' Effective friction angle (°)
Γ_t Total unit weight of soil (MN/m^3)
γ_w Total unit weight of water (MN/m^3)
σ'_z Effective overburden pressure (MPa)

REFERENCES

Chen, C. and Hsieh, H.S. (2001). "Interpretation of uplift test results in Taichung gravel layer." *Sino-Geotechnics*, **84**, 19-28 (in Chinese).

Chu, B.L., Lee, T.M., and Chien, C.Y. (1995). "A non-linear finite element analysis of a deep excavation in gravel." *International Symposium on Underground Construction in Gravel Formations,* Taipei, 3/39-3/50 (in Chinese).

Lin, P.S., Yang, L.W., and Juang, C.H. (1998). "Subgrade reaction and load-settlement characteristics of gravelly cobble deposits by plate-load tests." *Canadian Geotechnical Journal*, **35**(5), 801-810. https://doi.org/10.1139/t98-044

Ministry of Interior (2001). *Design Code for Foundation of Buildings* (in Chinese).

Kulhawy, F.H. (1985). "Drained uplift capacity of drilled shafts." *Proc., 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, **3**, 1549-1552.

PLAXIS (2013). *PLAXIS 3D Computer Software*. Delft, Netherlands, PLAXIS.

Rollins, K.M., Clayton, R.J., Mikesell, R.C., and Blaise, B.C. (2005). "Drilled shaft side friction in gravelly soils." *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **131**(8), 987-1003. https://doi.org/10.1061/(ASCE)1090-0241(2005)131:8(987)