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Design for Settlement of Pile Groups by the Unified Design Method. A Case History

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ABSTRACT   A series of tall apartment buildings was planned to be built on reclaimed ground over a thick deltaic deposit near-shore deposit outside City of Pusan in South Korea. The soil profile consisted of approximately 30 to 50 m of soft clay, silt, and sand on sandy gravel extending to bedrock at about 100 m depth. The deep foundation system normally used in Korea consists of steel pipe piles driven to significant toe bearing in dense soils or on bedrock. Because of the anticipated significant costs of this solution, a more economical alternative foundation system was essential, and the alternative of the PHC pile, a pretensioned spun high strength concrete pile, was proposed. To evaluate the feasibility of a PHC pile alternative, a comprehensive test programme was carried out, encompassing dynamic tests, long-term monitoring of negative skin friction, laboratory pilot tests, static loading tests on instrumented test piles, and settlement analysis. The analysis of the test data required study of strain effects from hydration and swelling of concrete, and of development of residual load before static testing, as well as of distribution of resistance along the pile due to applied load, and development of negative skin friction from settling soil and resulting drag force. Reliable estimation of pile group settlement was a key issue. Five methods for calculation of pile group settlement were compared and applied to the actual foundation layouts, of which one, the Unified Design Method, could include the effect of ongoing consolidating of the soft, compressible clay layer, interaction of adjacent foundations, and factual distribution of pile shaft resistance.

1. INTRODUCTION

Very thick deposits cover the Nakdong River estuary delta located west of the city of Pusan, South Korea. The soil profile consists of soft clay, sand, and gravel layers on bedrock sometimes found as deep as 100 m. Two areas, called Shinho (SH) and Myeongji (MJ), were reclaimed in the early 1990s by placing fill to raise the land above flood level. The areas were vacant due to large construction costs of deep foundations. Since Year 2000, space has become increasingly limited in the city and development of the two areas was commenced comprising tall apartment buildings to house approximately 80,000 people.

Figure 1 shows the sites for the residential complexes to be developed at the MJ and SH sites. The sites are located in the southwest part of Nakdong River estuary and are close to the ocean shore. Total area is 0.84 and 0.24 km^2 for the MJ and SH sites, respectively.

Fig. 1 Location map (Google Maps)

In areas of deep soft and compressible deposits, piled foundations in Korea are mostly designed with steel pipe piles driven to bearing in dense soils or on bedrock. This means that foundations become very costly when these piles are very long, such as the 70 to 80 m lengths thought necessary at the MJ and SH sites.

As an alternative pile, the Pretensioned spun High strength Concrete (PHC) pile, a cylindrical concrete pile, was considered. The in-place cost of a 600 mm diameter PHC pile in Korea is between a quarter to a third o the cost of a similar diameter and length steel pipe pile. Because the PHC pile would be unable to withstand the hard driving needed for reaching the deep, very dense soil layers or bedrock, a PHC alternative pile would have to mobilize the required capacity by combining shaft resistance with toe resistance in dense sand found at about 35 to 50 m depth. If so, the shorter length would make it an even more attractive alternative; the PHC would reduce total costs by at least US$300 million.

The foundation design had to take into account the ongoing consolidation settlement at the site and the subsequent negative skin friction affecting the piles. For example, seven months wait between the driving of the test pile and the static loading test in 2006, about 50 mm of settlement of the surface of the reclaimed site was observed. Before 2006, the customary design concept in South Korea was to subtract the drag force from the bearing capacity of a pile before determining the allowable load by dividing the balance with a factor of safety and, then, to determine the maximum allowed working load by subtracting the drag force from the so determined allowable load. This concept significantly reduces the working load for the long piles, radically exacerbating foundation costs. Such design requirement would put the feasibility of the subject project in doubt.

However, the drag force was expected to be large also for the shorter pile, which required checking adequacy of the structural strength of the PHC pile, and, because downdrag of the piled foundation also was an issue, it was decided to explore the applicability to the project of the Unified Design Method (Fellenius 1984, 1988, 1991, 2004, 2009). The Unified Method emphasizes design for settlement of a piled foundation and recognizes that pile capacity is not affected by presence of a drag force and that it is, therefore, neither correct to subtract the drag force from the pile capacity nor to reduce the allowable load with any amount of the drag force. The method correlates the interaction between forces, load distribution, and settlement to the design, as well as considers the axial structural strength of the pile.

Three design issues for the PHC pile were recognized to affect the project.

1. Drivability of the pile

2. Magnitude of the drag force and the maximum load in the pile in relation to the pile structural strength

3. As the piles were expected to find adequate capacity in less dense layers and rely on shaft resistance for much of the capacity, foundation settlement could become a deciding aspect

To provide insight to the issues, a comprehensive test programme was instigated that included dynamic monitoring of pile driving, static loading tests, and long-term monitoring of instrumented piles.

Results from static loading tests and associated studies at the site have been published by Fellenius et al. (2009) and Kim et al. (2011). This paper briefly discusses the overall process from design to construction of the project, summarizes the design method applied to the foundation design, and presents the settlement analysis.

2. SITE CONDITIONS
2.1 General conditions

Soil deposits along two for sections, Sections A-A and B-B, of the MJ and SH sites are shown in Figure 2. For both sites, the soil profile consists of a 5 to 8 m thick fill of silty sand placed about ten years ago over an approximately 10 m thick layer of loose silty sand with interbedded layers of fine-grained soil, followed by a 20 to 35 m thick compressible layer of soft silty clay deposited on dense silty sand. The latter layer contains zones of silty clay. Placing the sand fill initiated a still ongoing consolidation process with excess pore pressure of about 20 to 30 kPa remaining in the silty clay at the time of construction of the buildings. The groundwater table lies about 1.5 m below the finished fill surface. Detailed descriptions of the soil deposits are given by Chung et al. (2002; 2005; 2007).
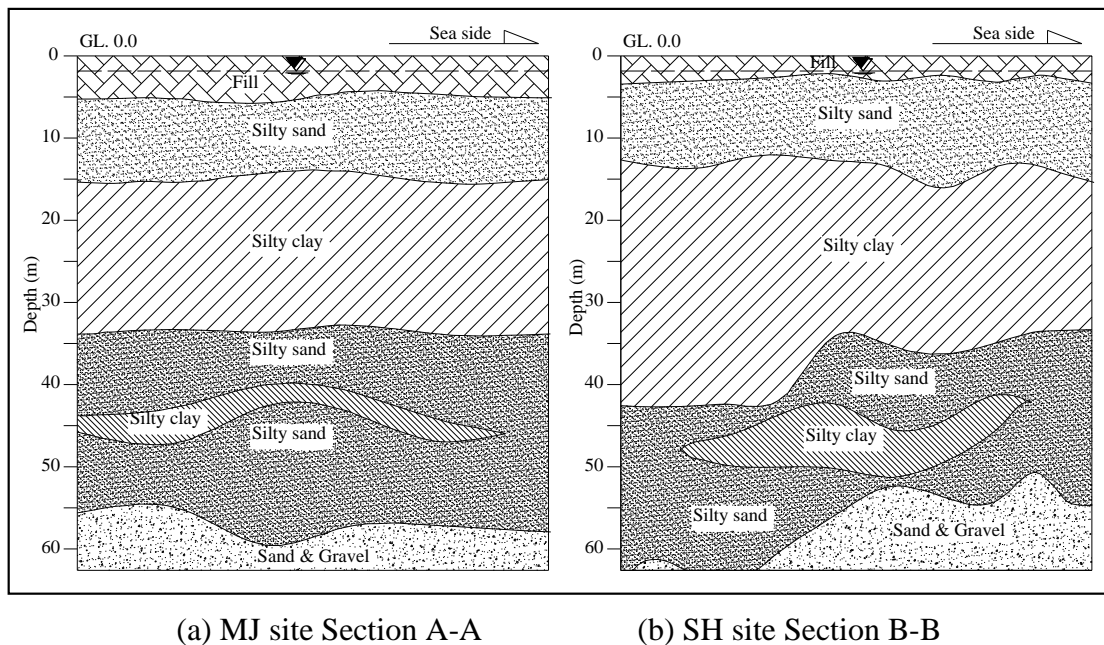

(a) MJ site Section A-A   (b) SH site Section B-B

Fig. 2 Generalized soil profiles of MJ and SH sites

2.2 Geotechnical Parameters

Figure 3 shows representative soil profiles of the MJ and SH sites. The soil parameters were determined from CPTU soundings, field vane tests, and laboratory tests.

Geotechnical properties varied significantly in both vertical and horizontal directions. To account for such variations, areas with similar CPT profiles were grouped and the sites were zoned into several sectors. Figure 4 shows a typical example of the zoning in the C block of the MJ site. At least two, sometimes five, CPTU soundings were performed in each building sector.
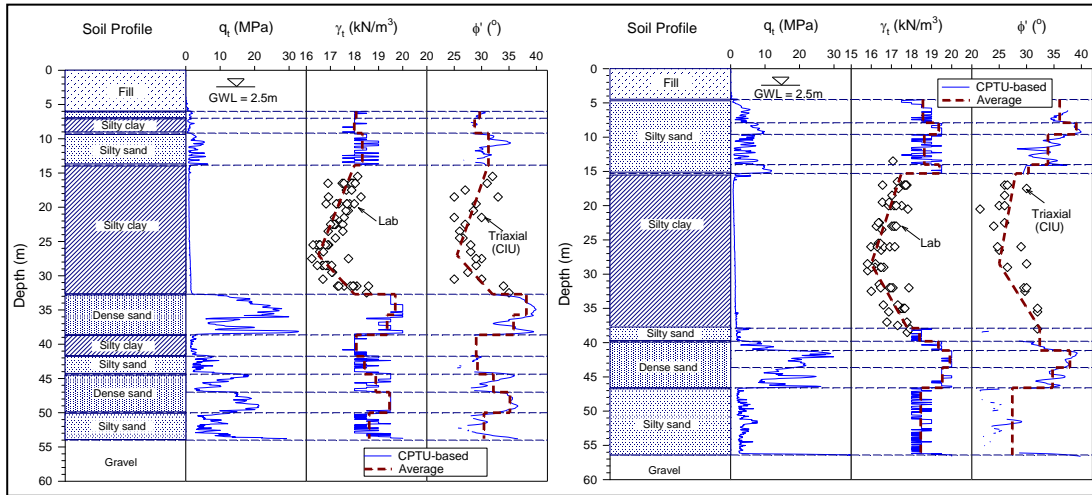
(a) MJ site　　　　　　(b) SH site

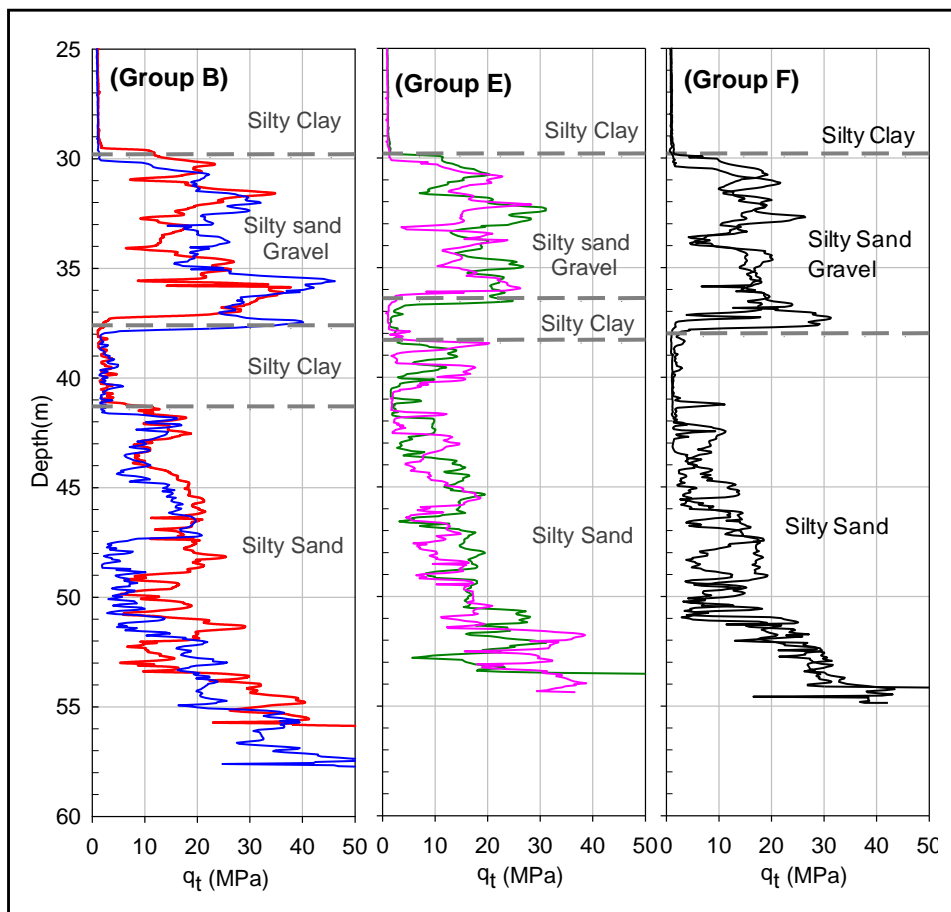**Fig. 3　Typical CPT profiles with soil properties at MJ and SH sites**

Fig. 4　Example of zoning in the C block of MJ site

3.   CONSIDERATION FOR THE PILED FOUNDATIONS

Design of the superstructure and foundations for the MJ site commenced in early 2006. The site was divided into four blocks based on building size; each block containing 35 to 41 buildings of a specified number of stories. Block A was to have of 15-storey buildings, Blocks B and C were to have 5 to 10-storey buildings, and Block D 10-storey buildings. Most buildings incorporated two basement floors that were connected to below-ground car parks in between the buildings. The buildings were to be supported on piled foundations, while the car parks were to be on floating foundations. An expansion joint was constructed where the two different structures connected. The footprint shapes of foundations varied. Figure 5 shows a section of Block C of the MJ site, including the soil sectors of similar soil profile. A similar series of buildings was planned at the SH site.
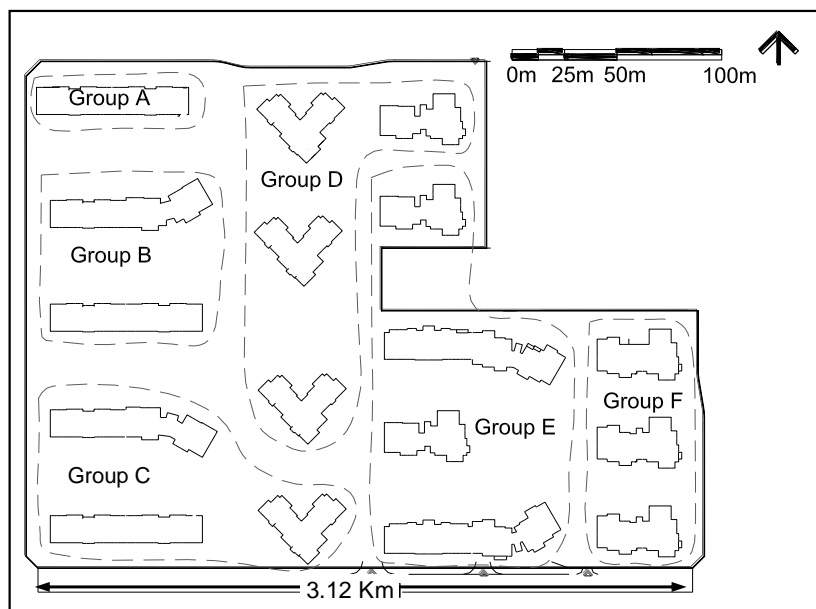

Fig. 5 Layout of Block C buildings in MJ site

A critical issue was the determination of allowable axial working load. The Korean Foundation Engineering Manual (Korean Geotechnical Society 2003) stipulates that the drag force be subtracted from the full-length pile capacity before determining the allowable load (by division of the difference with a factor of safety; a working-stress approach), and that the maximum working load be the so-determined allowable load minus the drag force. A similar approach is indicated by Eurocode 7 (1997) with commentary and examples by Frank et al. (2004). The difference is that the Eurocode (a LRFD code employing limit states approach) disregards the capacity in the negative skin friction zone, i.e., the drag force is subtracted from the pile capacity before determining the factored pile capacity. The US Load and Resistance Factor Specification (AASHTO 2010) borrows this approach from the Eurocode. In contrast, many other codes apply the principles of the Unified Method, for example, the Canadian Foundation Engineering Manual, CFEM (2006), the Canadian Highway

Design Code (Canadian Standard Council, 2006), the Australian Piling Standard (1995), Hong Kong Geo (2006), the US Federal Highway Administration Guidelines (Hannigan et al. 2006), and others.

Considering the thickness of the settling deposit, the approach by the Korean Foundation Engineering Manual (Korean Geotechnical Society 2003) would result in a working load smaller than half the working load normally considered for the long steel pile alternative (where the piles would have significant toe bearing) and leave practically no room for working load for the PHC alternative (where the piles would derive a large portion of the capacity from shaft resistance).

4. TEST PROGRAMME

Table 1 presents the test programme intended to evaluate pile resistance, drivability, and suitability of the PHC piles.

Table 1 Test programme

Test	Extent	Scope
(1) Initial PDA	1 test pile per block	Dynamic testing at initial driving and restrike with CAPWAP analysis
(2) Monitoring	1 test pile at each of MJ and SH sites	Monitoring of strain development over 200 days
(3) Laboratory	2 PHC pieces	Analysis of strain development due to temperature and swelling
(4) Static test	1 test pile at each of MJ and SH sites	Static Loading tests: bidirectional-cell test followed by head-down test
(5) PDA	2 to 3 construction piles per building	Drivability in each block to check for effect of variation of soil layering
(6) Static testing	1 construction pile per building	Proof testing — bidirectional-cell tests

1. First Pile Driving Analyzer (PDA) test occasion was to evaluate the drivability of PHC piles.

2. For two instrumented piles at the MJ and SH sites, the build-up of drag force due to negative skin friction was monitored during six to eight months before static loading tests.

3. During the long-term monitoring of negative skin friction, strain readings indicated development of tensile strains in the pile and rapid strain variation during hydration process of the grouted central void. To provide insight in the observed strains, a laboratory investigation was instigated on two 2.0 m long pieces of the PHC pile. For details on the instrumentation, testing methods, and results of the laboratory test, see Fellenius et al. (2009).

4. The static loading test at the MJ site was a conventional head-down test. At the SH site, the results of a first performed conventional head-down static loading test were inconclusive because the pile failed structurally before the shaft resistance along the lower length of the pile and the toe resistance had been mobilized. The testing method was then amended to include an initial bidirectional-cell test with a cell placed at the pile toe. The bidirectional-cell test mobilized the toe response and some shaft resistance. A following head-down test with the cell free-draining was then used to mobilize shaft resistance along the full length on the now, as it were, a shaft-bearing pile — the toe resistance having been removed by the cell test. For details on the instrumentation, testing methods, and results of the static loading tests, see Kim et al. (2011).

5. PDA tests were performed on two to three piles per building to check the drivability. CAPWAP toe resistances at end-of-initial-driving (EOID) were correlated to CPT-based toe resistance relations so that CPTU soundings could be used to assist in determining expected toe resistance for the building piles.

6. Finally, after the completion of the pile driving phase, a static loading head-down proof-test test was performed on one pile from each building to twice the working load.

The results according to Points 1 through 4 have been published by Fellenius et al. (2009) and Kim et al. (2011). and showed that the PHC pile was well suitable for the project. Moreover, because of the large drag forces expected to develop, a design according to the Korean Foundation Engineering Manual (Korean Geotechnical Society 2003) would have incorrectly indicated that the piles had no margin for load from the structure. Indeed, long steel piles per the original design solution would also have had to be very thick wall piles or be filled with reinforced concrete in order improve the structural strength so that the maximum axial load in the pile due to dead load and drag force could be accepted.

Points 5 and 6 are integral to the design effort, but will not be discussed in this paper, which concentrates on the settlement aspect of the project. To the authors' disenchantment, a planned monitoring phase encompassing measuring settlement of a few buildings and surrounding grounds was called off as a consequence of the recession impacting the financing of the project. However, visual observation of the site and buildings until the time of writing has shown the buildings to be in good conditions with no distress and to exhibit no signs of any differential settlement of concern for the buildings.

5. ANALYSIS OF SETTLEMENTS
5.1 Methods for Settlement Calculation

The usual range of acceptable maximum total settlement for frame buildings in Korea follows recommendations by the Korean Society of Architectural Engineers (2004) and ranges from 100 mm to 150 mm for total settlement and 20 mm to 60 mm for

differential settlement. Holtz (1991) suggests a total settlement range of 50 mm to 100 mm and differential settlement of 1:500 for frame buildings. For the subject project, the acceptable limits of total and differential settlement were set to 100 mm and 1:500.

Several methods for estimating the settlement of pile groups exist, ranging from simple empirical approaches through sophisticated nonlinear finite element analyses. Methods useful for design and construction of buildings are usually based on the concept of equivalent raft (a raft foundation with the same footprint as the building foundation), which was proposed by Terzaghi and Peck (1948): the settlement of a piled raft can be calculated as the settlement for an "equivalent raft" placed at the lower third point of the average pile length and loaded by a raft stress equal to the total load on the piles divided by the footprint. Settlement for that equivalent raft was calculated by spreading the stress outward and below the equivalent raft using the 2(V):1(H) distribution. Often, the Terzaghi-Peck method is applied also to foundations on a series of individual piles or on several small groups of piles distributed over a building footprint. As a part of the study for the Nakdong River estuary project, a comparative study was carried out to compare five common equivalent raft methods used in practice.

5.1.1 Terzaghi-Peck Method

As mentioned, Terzaghi and Peck (1948) proposed that the settlement of the piled foundation be calculated as settlement of an equivalent raft assumed located at one third of the pile length above the pile toe ("the lower third point") with the stress distributed into the soil at a slope of 2(V):1(H). An elastic modulus, E_s, was applied to determine strain due to the applied stress, and settlement was calculated as the sum of the accumulated strains.

To include influence of depth for calculation of settlement of raft foundations, Fox (1948) proposed a depth factor (settlement ratio), F_D, defined as the ratio of the mean vertical displacement of the embedded foundation to that of a similar foundation placed on the ground surface. The Terzaghi-Peck equivalent depth approach with the depth factor applied is expressed in Eq. 1.

$$S_{Raft} = F_D \sum_{i=1}^{n} \frac{\Delta q_i}{E_i} h_i \qquad (1)$$

where S_{Raft} = raft settlement
       Δq_i = effective stress increase in i^{th} layer by the 2(V):1(H) method
       h_i = thickness of ith layer
       E_i = elastic modulus of ith layer
       F_D = depth factor proposed by Fox (1948).

Axial compression of the piles for the length above the equivalent raft was not included in the Terzaghi-Peck approach.

5.1.2 Meyerhof Method

Meyerhof (1976) proposed to model the piled foundation as a piled raft by means of Eq. 2, which accounts for pile length, and raft width. The depth to the equivalent raft was not specified by Meyerhof (1976).

$$S_{Raft} = \frac{qBI_d}{E_i}$$ (2)

where S_{Raft} = raft settlement
q = average net stress applied to raft
B = width of raft
E_i = average elastic modulus in the influence zone calculated as $2q_c$
q_c = cone stress from a CPT sounding (not adjusted for pore pressure)
I_d = influence factor for pile length determined by Eq. 3

$$I_d = 1 - \frac{D}{8B} \geq 0.5$$ (3)

where D = depth to the underside of the pile cap (i.e., depth to base of the equivalent raft)

Axial compression of the piles for the length above the equivalent raft was not included in the Meyerhof approach.

5.1.3 Schmertmann Method

Schmertmann (1970) and Schmertmann et al. (1978) proposed a method to estimate the settlement of footings on granular soils using a strain influence approach, as expressed in Eq. 4. The method presumes a rigid raft. Tomlinson and Woodward (2008) extended the method to the calculation of settlement of a piled foundation by incorporating the equivalent raft concept, with the raft placed at the lower third depth or below toward the pile toe depending on conditions of the soils.

$$S_{Raft} = C_1 C_2 q \sum_{i=1}^{n} \frac{I_{zi} h_i}{E_i}$$ (4)

where S_{Raft} = raft settlement
C_1 = embedment correction
C_2 = creep factor
q = average net stress applied to raft
I_{zi} = vertical strain influence factor
h_i = thickness of ith layer
E_i = elastic modulus of the ith layer

The embedment correction factor, C_1, is determined from Eq. 5.

$$C_1 = 1 - 0.5\left(\frac{\sigma'_{v0}}{q}\right) \geq 0.5 \tag{5}$$

where C_1 = embedment correction factor
        σ'_{v0} = effective overburden stress at the raft depth
        q = net applied stress applied by the raft

The creep factor adjusts for the effect of the secondary compression as expressed in Eq. 6.

$$C_2 = 1 + 0.2\log_{10}\left(\frac{t}{0.1}\right) \tag{6}$$

where t = time since load application (years)

No contribution due to pile compression for length above the equivalent raft was proposed.

5.1.4 Poulos Method

In calculating the settlement of the piled foundation, Poulos (1993) applied the equivalent raft concept and proposed the strain influence method expressed in Eq. 7. The equivalent raft level was adopted from that proposed by Tomlinson (1986) and Tomlinson and Woodward (2008).

$$S_{Raft} = F_D q \sum_{i=1}^{n}\left(\frac{I_{zi}}{E_i}\right)h_i \tag{7}$$

where S_{Raft} = raft settlement
        F_D = depth ratio proposed by Fox (1948) for Terzaghi-Peck method
        q = net stress applied to the raft
        I_{zi}, = vertical strain influence factor
        E_i = elastic modulus of the ith sub-layer
        h_i = thickness

Poulos (1993) provided typical curves of strain factors for three typical foundation footprint shapes, two rectangular and one strip footing. The method to derive the curves of strain influence factors was not explicitly described in the paper. However, the curves correspond to that calculated from Boussinesq stress distribution under the center of a uniformly loaded rectangular area placed on surface of an elastic half space (Nguyen 2008, Nguyen et al. 2010). That is, Eq. 7 can be transformed, as expressed in Eq. 8, which is identical to Eq. 1, but for that the methods of stress-distribution are different.

$$S_{Raft} = F_D \sum_{i=1}^{n} \frac{\Delta q_i}{E_i} h_i \tag{8}$$

where S_{Raft} = raft settlement
Δq_i = effective stress increase in i^{th} layer by the Boussinesq distribution
h_i= thickness of ith layer
E_i = elastic modulus of ith layer
F_D = depth ratio proposed by Fox (1948)

The axial compression of the piles for the length above the equivalent raft due to the applied load is calculated as shortening of the pile as a free-standing column.

5.1.5 Fellenius Method

Fellenius (1991; 2009) recommended that the first step in analyzing settlement of a piled foundation be determining load-transfer movement developing when the piles are loaded by the structure. Thereafter, the distribution of soil settlement is calculated, as caused by change of effective stress distribution under the building footprint. In the latter, all additional changes of effective stress from external loads and pore pressure changes are included, not just the sustained (dead) load on the piles. The pile settlement is governed by the settlement developing at the neutral plane. The procedure is illustrated in Figure 6 and is carried out in four interrelated steps.

1. Normally, the applied sustained (dead) load is smaller than the total shaft resistance; only a very small portion, if any, will have reached the pile toe at the end of the construction. Therefore, the total load-transfer movement appearing at the pile head is mostly a result of 'elastic' shortening of the pile for the applied load, which reduces with depth due to shaft resistance. The load-transfer pile-toe movement will be correspondingly small. Moreover, the portion reaching the pile toe is usually smaller than the residual toe load, which will further reduce the pile-toe load-transfer movement.

2. The main amount of settlement occurring with time is a combination of the effect of the sustained (dead) load and change of effective stress due to all additional changes of effective stress from loads on adjacent foundations, fills, change of groundwater table and pore pressure distribution, unloading due to excavations, etc. The calculation of the settlement caused by the change of effective stress is assumed for an equivalent raft located at the depth of the neutral plane determined in Step 1.

3. In determining the soil settlement, the soil compressibility must include the stiffening effect of the "pile-reinforced" soil. The settlement calculation can be according to conventional calculations for change of effective stress, as well as more sophisticated methods. Because the "soil reinforcement effect"

usually results in that only very small settlement develops between the neutral plane and the pile toe level, the soil settlement can alternatively be calculated from a raft placed at the pile toe level — a conservative approach that avoids having to repeat the calculations with different depths to the raft, as the iteration calculation proceeds. It also makes calculation Steps 1 and 2 independent of each other.

4. The settlement of pile head is the soil settlement calculated for the equivalent raft (the downdrag) at the neutral plane plus the 'elastic' shortening of the pile for the increase of axial load, which includes the effect of the drag force, acting above the neutral plane.

The complete analysis includes assessing the potential affect of build-up of residual load and the load-transfer pile toe movement due to the applied axial loads. The increase of effective stress in the soils below the pile toe depth is usually only appreciable for large pile groups. Therefore, the settlement at the neutral plane is decided by conditions other than the load on the piles, such as groundwater table lowering, on-going consolidation due to fills and loads due to adjacent buildings, as well as unloading due to excavations.
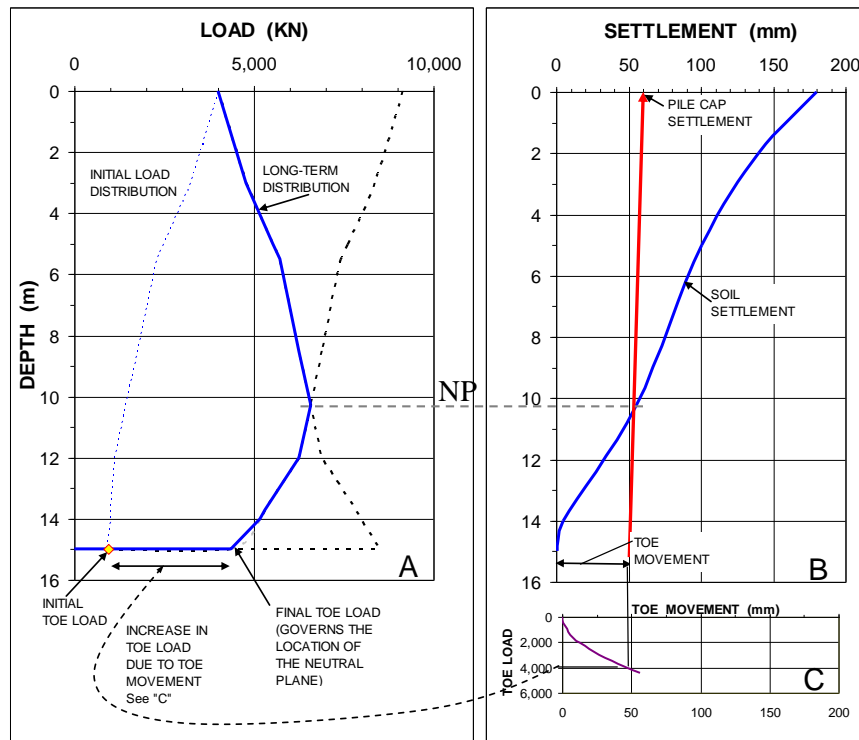

Fig. 6 Results of a typical trial -and-error approach to match pile toe load, neutral plane location, and pile toe movement due to downdrag (Fellenius 2009). (Values of load and depth shown do not pertain to the current project).

The method allows for calculation of settlement using the compressibility response best suited to the particular soil layers, e.g., elastic modulus, C_c-e_0, or Janbu approach (Janbu 1963, Fellenius 2009). The stress distribution can be calculated using Boussinesq, Westergaard, or 2(V):1(H) distribution. The 2:1 method is restricted to the case of settlement from stress from the raft only.

5.2 Comparison of the Methods to Case Histories

The methods listed in the Section 5.1 were applied to a back-analysis of six well-documented case studies of large piled foundations. A brief compilation of case data is provided in Table 2. Cases 1 and 2 are approximately flexible foundations, whereas Cases 3 to 6 are foundations of intermediate rigidity. Although they do not account for the foundations of intermediate rigidity, the calculations were corrected for raft (pile cap) stiffness. The procedures used are described in Nguyen et al. (2010).

Table 2. Brief summary of data for the six case studies

Case	Soil Profile	E_s (MPa)	B (m)	L (m)	d (m)	# (--)	b (mm)	c/c (m)	D (m)	S_{meas} (mm)
1	till, sand	100	17.4	60.4	variable	179	500-600	2.62	4.9	32
2	sand	10-90	24.3	33.5	flexible	132	410	2.72	7.6	85
3	clay, sand	20-200	16.5	27.6	4.0	48	760-910	3.60	18.6	30
4	sand, clay	40-50	26.9	26.9	4.3	281	500	1.71	25.0	51
5	clay, sand	5-30	34.3	85.1	1.2	697	520	2.13	13.4	190
6	sand	50-70	17.5	51	0.5	264	510	1.96	13.5	18

Notes: [1]DeJong and Harris (1971); [2]Koerner and Partos (1974); [3]Hooper and Wood (1977); [4]Borsetto et al. (1991); [5]Goossens and VanImpe (1991); [6]Tejchman et al. (2001)

E_s = elastic modulus range; B = foundation width; L = foundation length;
d = pile cap thickness; # = number of piles; b = pile diameter;
c/c = average center-to-center spacing between piles;
D = average pile embedment depth; S_{meas} = measured settlement.

Each soil layer was assigned an elastic modulus, E_s, as either taken from the case paper or estimated from the information in the paper for the pertinent soil. The same values were applied to all five methods of calculation. The ranges shown in Table 2 represent the variations of soil layers at the sites.

The results of the calculations are shown as ratio of calculated to measured settlement in Figure 7. All methods produced values close to the measured except for the deviation of the Meyerhof method for the last two cases. The Meyerhof method was suggested for small pile groups in a homogenous sand deposit, while the Schmertmann method was developed based on model footings on sand, so they are not particularly suitable for the six cases, which soil profiles are inhomogeneous and stratified.

While the Schmertmann method was proposed for rigid rafts, the Terzaghi-Peck 2:1-method and the Meyerhof methods were proposed for flexible rafts. The

Meyerhof method is based on a conventional elastic formula with a correction factor for embedment effect (I_d). The Poulos method was derived from elastic theory of a flexible area and applies to flexible rafts.

The calculations using the Fellenius method assumed Boussinesq stress distribution and were made with the UniSettle program (Goudreault and Fellenius 1996), which can be directed to any specific location on the raft, as well as outside the raft footprint. For Cases 1 through 4 and Case 6, the calculations were made for the characteristic point, which is where, theoretically, flexible and stiff rafts have the same settlement. (CFEM 2006, Fellenius 2009). The characteristic point lies about 0.37 B and 0.37 L away from the raft center (B and L are raft width and raft length, respectively). For Case 5, the measured settlement was obtained at a benchmark located at the mid-point of the raft side. The Fellenius method calculations were made for this location on the raft.
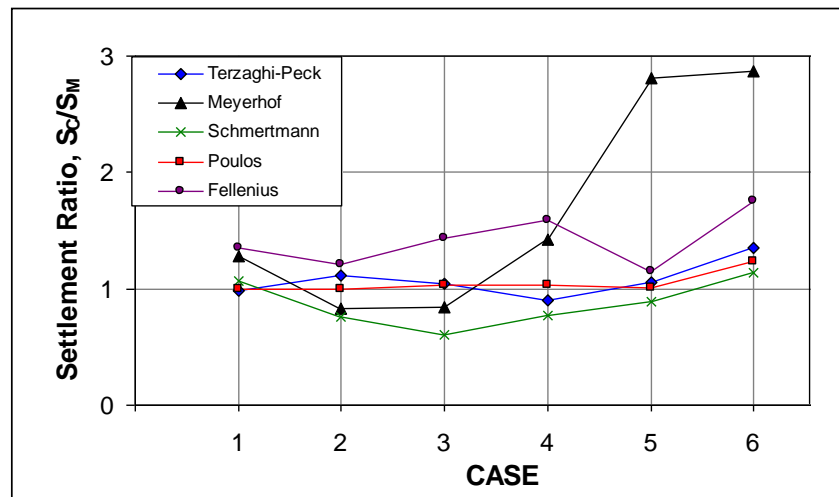

Fig. 7 Ratio between calculated and measured settlement for the six cases

For Case 5, settlements were measured at three benchmarks located along the side of the raft and one benchmark at the corner of the raft. When adjusting the assumed compressibility parameters (Table 2) of the case so that the settlement calculated at the mid-side benchmark agreed with the measured value (190 mm), the calculation showed that the values calculated for the other benchmarks, including the corner (110 mm), also agreed with the measured values. This means that the raft, despite being 1.2 m thick, was flexible rather than stiff and that the raft center probably settled 270 mm, approximately 40 % more than at the mid-point of the raft side (Fellenius 2011).

5.3   Calculation Example from the MJ Site

Building footprint and relative locations, pile lengths, and soil profile varied somewhat across the sites. A typical case, presented in the following, consists of a building foundation footprint approximated to two rectangular areas, denoted Block 1

and Block 2, with a small connecting area, as indicated in Figure 8. The pile cap was 1.15 m thick with underside placed 6.0 m below ground surface (two basement floors). The piles were driven into the middle dense sand layer with the average toe depth at 35 m. According to the design, each pile (of a total of 90 piles) was subjected to 2,000 kN dead load, Q_d, and 500 kN live load, Q_{live}. The total dead load corresponds to a stress of 263 kPa uniformly distributed across the footprint for both building blocks.
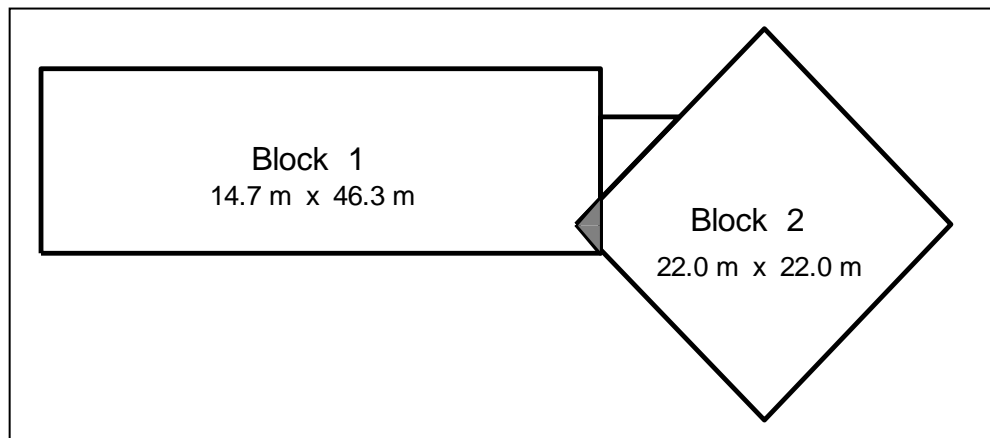

Fig. 8 Plan view of the building footprint

Based on the results of the instrumented long-term observations and the short-term static loading tests (Fellenius et al. 2009, Kim et al. 2011), the distribution of axial load in the pile is estimated to be as shown in Fig. 9, as calculated by the UniPile program (Goudreault and Fellenius 1998). The neutral plane is located in the silty clay, at about 33 m depth, about 2 m above the dense to very dense sand layer. Calculations of the location of the neutral plane from the indicated pile toe load, about 3,600 kN and a pile toe penetration, about 3 mm, agreed with toe load-movement curves determined in bidirectional-cell tests at the site.

Figure 10 shows cone stress distribution and soil profile at the example building location along with the basic parameters for settlement calculation evaluated from the CPTU data and laboratory trixial tests. The friction angle distribution was determined from the CPTU soundings using procedures proposed by Robertson and Campanella (1983). The Janbu modulus number values were determined using Massarsch CPTU method (Massarsch 1994, Massarsch and Fellenius 2002). Four CPT-based methods were used to evaluate the elastic modulus values of the sand layers: Lunne and Christophersen (1983), Eslaamizaad and Robertson (1996), Schmertmann (1978), and Canadian Foundation Engineering Manual (2006). For a linearly elastic soil, which has a j-exponent of unity, the Janbu modulus number and the E_s-modulus (MPa) are proportional at a ratio of 10. Thus, the E_s-value of 60 MPa shown in the gravelly sand correlates to a modulus number, m, of 600, rather than the value of 350 indicated in Figure 10. The latter modulus number is considered somewhat on the low side.

Table 3 shows parameters for settlement analysis compiled from both CPTU soundings and laboratory tests. Modulus number (m) and stress exponent (j) were used in the calculation according to the Fellenius method with input to the UniSettle program (Goudreault and Fellenius 1996), while the elastic modulus (E_s) was used for the Terzaghi-Peck and Poulos methods.
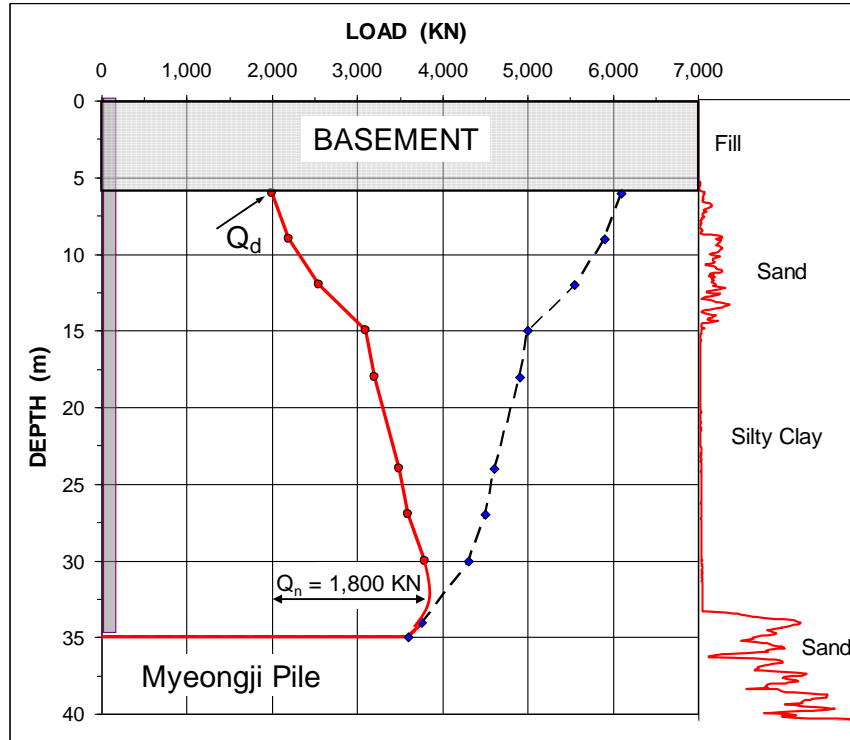

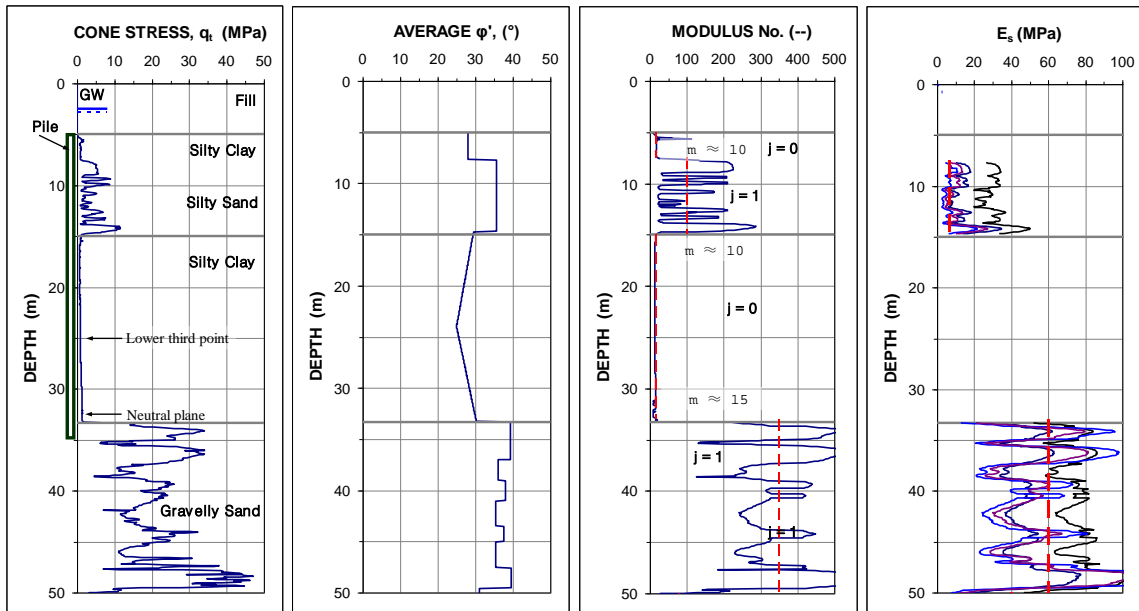**Fig. 9 Load transfer and neutral plane location**

Fig. 10 Soil parameters determined from CPTU sounding

Table 3 Compiled design parameters for settlement analysis

Layer	Depth Range (m)	Density kg/m^3	Modulus Number, m (--)	Stress Exponent (--)	Elastic Modulus (MPa)
Fill	0 - 5	1,850	20	1	
Silty Clay	5 - 8	1,600	10	0	
Silty Sand	8 - 15	1,900	100	1	10
Silty Clay	15 - 33	1,700	10 - 15[*]	0	
Gravelly Sand	33 - 50	2,100	600	1	60

[*] Range from upper to lower boundary

Building settlement was calculated for the example using the Terzaghi-Peck, Poulos, and Fellenius methods. Of these, the first two do not include the effect of "outside" concerns, such as the new fill, adjacent buildings, non-hydrostatic pore pressure distribution, etc., but such influences are included in the third method.

Calculations were made for equivalent rafts placed at the lower third point (Terzaghi-Peck and Poulos methods) and at the neutral plane (Fellenius method) and the results are compiled in Table 4. The calculations for the Terzaghi-Peck and Poulos methods applied E_s-modulus in all layers, whereas the calculations for the Fellenius methods applied Janbu modulus numbers, (m) and stress exponents (j) and included the reinforcing effect on the soil from the piles within the pile length below the raft by proportioning the pile and soil compressibilities to the respective areas of pile and soil. The reinforcing effect resulted in compressibility of E=700 MPa and m= 7,000, essentially entailing an "incompressible" condition for that zone.

For the Terzaghi-Peck and Poulos methods, pile 'elastic' shortening above the raft was computed for the pile as a free-standing column loaded by the dead load, while for the Fellenius method, shortening was calculated corresponding to the load distribution (dead load and drag force) shown in Figure 9.

Table 4 Calculated settlements

Method	Block 1			Block 2		
	Raft (mm)	'Elastic" (mm)	Total (mm)	Raft (mm)	'Elastic" (mm)	Total (mm)
Terzaghi-Peck	43	6	49	40	6	46
Poulos	45	6	51	47	6	53
Fellenius	33	15	48	24	15	39

In contrast to the Terzaghi-Peck and Poulos methods, the Fellenius method can be used to reflect development over time. For the MJ example, about half the load is estimated to have been placed on the piles (and on the equivalent raft, therefore)

before any particular concern for settlement would arise. In the coarse-grained soil below the neutral plane, settlement for the building load thereafter applied to the piles would occur almost immediately. Therefore, only about half of the total settlement due to the load on the raft will be of relevance for the settlement assessment of the structure. The long-term settlement of the buildings are essentially a function of the long-term ongoing settlement of the reclaimed area that over the first year or so after construction would develop the drag force in the piles, which would result in pile shortening and manifested as settlement (downdrag) of the piles. Maximum settlement over time would therefore be about no more than "half-an-inch", which is a small portion of the maximum accepted value (Section 5.1).

6. Tensile Behavior of Piles due to Excavation

To avoid working in water, only a partial excavation, 2.0 m, of the building footprint area was made before pile driving was commenced. Excavation to full depth, 6.0 m, was made after completed pile driving. Due to the unloading of the soil, heave was expected to develop, which could, possibly, induce excessive tension in long piles. The maximum allowable tension is 10 MPa, which corresponds to a strain of about 300 $\mu\varepsilon$.

At the MJ site, a strain-gage instrumented test pile was installed and monitored during two months after the driving, while the excavation was open. The instrumentation was installed in the central void of the pile in the same manner as the instrumentation of the test pile described by Kim et al. (2011).

Figure 11 shows the strains measured during 60 days of monitoring until the construction started to place load to the piles. The zero reference is the strain values taken immediately after concreting the central void in the pile. That is, strains (expected to have been in compression) due to the first few days of set-up after initial driving are not included. The strain response during the hydration temperature change is similar to that measured for the test piles reported by Kim et al. (2011). During the excavation from 2 m depth to full depth (6 m) following the pile driving, the monitored pile elongated corresponding to a strain of up to 50 $\mu\varepsilon$. An additional lengthening corresponding to 20 $\mu\varepsilon$ occurred when the excavation near the test pile was completed. These tensile strains are very much smaller than the limit allowed.

Unfortunately, no measurements of soil heave were taken. The values, while measurable, are expected to have been small. A calculation of the heave, i.e., swelling of the soil, due to unloading, made applying the reloading modulus of the soil layers, indicates that the bottom of the excavation might have heaved about 2 mm. Such small relative movement will not be able to mobilize the full shaft shear values. Uneven excavation (one-sided) introduced temporary bending into the monitored pile.
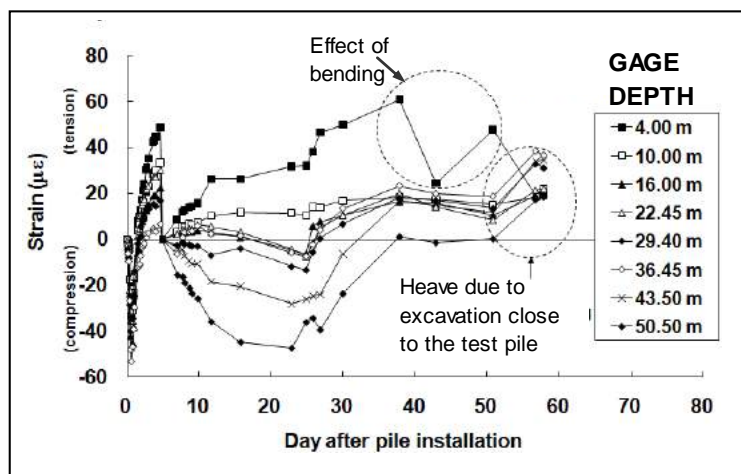
Fig. 11 Strains measured during excavation for basement

7. SUMMARY AND CONCLUSIONS

The results of the full-scale tests at the site showed that the ongoing consolidation of the reclaimed area would develop negative skin friction along the piles and accumulate large drag force.

The test programme was successful in verifying that the Unified Design Method was suitable for the project and large savings of foundation costs were realized.

The combination of an bidirectional-cell test and a head-down test in which the bidirectional-cell test was performed first and left open and free-draining during the subsequent head-down test. This approach provided data both from a full pile toe response and from mobilization of ultimate shaft resistance along the full length of the strain-gage instrumented pile, necessary for determining long-term load distribution, as well as settlement analysis.

Five methods of settlement analysis were applied to six case histories of settlement measured for pile raft foundations. An equivalent-raft approach was adopted for all analyses. The methods provided results approximately agreeing with measured values with one method deviating for two of the cases.

Three of the methods, the 1948 Terzaghi-Peck equivalent raft method, a method proposed by Poulos (1993), and the Unified Method by Fellenius (1991; 2009) were applied to demonstrate a typical settlement calculation for building at the site. The methods gave approximately similar values of calculated total settlement. However, the Fellenius method includes the effect of external loads, participating in the settlement process, i.e., loads other than the raft loads. It recognizes that settlement of flexible rafts differs between different locations over the building footprint. For a rigid raft, the method allows for calculation of settlement at the characteristic point, which is where settlement for a flexible and rigid raft is considered equal — i.e.,

independent of the degree of flexibility or rigidity. It also allows for assessment of settlement that can develop during and after completed construction.
Observations of the tension build-up in a test pile during excavation of 4 m of soil for building basement showed tension strain of up to 50 $\mu\varepsilon$, well below damage level.

Visual observation of the site and buildings shows the buildings to be in good conditions with no distress and no signs of any differential settlement of concern for the buildings.

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