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Settlement Evaluation for Shallow Foundation on Artificial Island made by Granular Material

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ABSTRACT: In the shallow foundations supported by granular material, the allowable bearing capacity of soil is generally determined by the shear stress and elastic settlement of soil calculated using empirical formulas presented in the literature and is a major design factor in evaluating the stability of the foundation. The allowable settlement of plant industry structure is recommended to be around 25mm because the facilities are consisting of very sensitive equipment with the strict displacement criteria and connected by pipes or cables each other. Therefore, to secure the structure stability and perform the economical construction work, it's important to be correctly evaluated the allowable bearing capacity of soil determined by the settlement evaluation. In this study, it's reviewed the soil parameters of empirical formulas for settlement estimation used in granular material, and the evaluation way to increase the reliability of each empirical formula on artificial island made by the compacted granular material through comparison with the measured settlement results in 95 actual footings.

KEYWORDS: Allowable Bearing Capacity of Soil, Allowable Settlement, Shallow Foundation, Empirical Formula, Artificial Island, Granular Material.

1 INTRODUCTION

In the stability analysis of shallow foundation to support the buildings and the plant facilities, the settlement evaluation is one of the priority topics. The settlement of foundation is generally estimated by the conventional formulas considering two major components the elastic settlement (S_e) and the consolidation settlement (S_c). However, since the consolidation settlement occurs in high compressible soil layer like the saturated soft clay, it's not component that needs consideration for the shallow foundation installed on the granular soil with the influence zone of stress distribution.

The site of this study is the artificial island of LNG import project, which is located at Al Zour, Kuwait City, and have been prepared by the reclamation work as followings.

Excavate/dredge and remove completely upper soft layer, caprock and sabkha soil up to top of medium dense layer.

Fill with the dredged granular soil up to reclamation level and improved with vibro compaction.

Total thickness of reclamation was about 18m.

Figure 1. LNG Import Project Site.

Although the filling layer was consisted of well-graded and compacted granular materials, because of the rearrangement and crushing between the particles, it could indicate creep behavior with time-dependent compression phenomenon similar with the secondary consolidation in clays.

Therefore, after completion of the site preparation work, the settlement of the reclaimed island was measured at 12 clusters using vary of the monitoring instruments (settlement marker, magnetic extensometer, deep datum etc,.) and evaluated the subsidence trend for long-term settlement.

Since the settlement survey of the shallow foundations had been performed during different period accordance with the construction sequence, the monitored creep settlement within the measurement duration is excluded for the evaluation of accurate elastic settlement.

2 SITE SOIL CONDITIONS AND MONITORING RESULTS

2.1 *Geology Conditions of Site*

The geology of Kuwait is strongly influenced by developments in the Arabian Gulf and lies on the eastern edge of the deep sedimentary basin that forms the Arabian Peninsula.

Geologically, the area is upper Miocene–lower Pleistocene age (approximately 2 to 10 million years) and belongs to the Dibdibah Formation of the Kuwait Group which outcrops in large extent. The Dibdibah formation consists of siliceous sands and gravels with intercalations of clayey/silty sand and clay.

The Kuwait Group was deposited in the late Tertiary/older Pleistocene and is usually pre-consolidated to different degrees. The compactness of the Kuwait Group Sand varies significantly but is generally regarded as a highly competent formation consisting of medium dense to very dense sandy, slightly calcareous, uncemented to slightly cemented material with excellent strength and deformation properties. The fines content of the Kuwait Group Sand can be quite significant and normally increases with depth and compactness.

The Dammam Formation underlies the Kuwait Group with reportedly no exposure at the ground surface. It is Upper Eocene to Middle Miocene in age (approximately 38 to 42 million years), lithologically comprising inter-bedded marine marls, limestone and clays. The thickness of this formation varies from 30 to 101 m (Al Sulaimi & Al Ruwaih, et al., 2004).

Before site reclamation, a number of geotechnical and geophysical surveys were conducted to verify the site geology conditions for the original ground. The survey encountered light grey to brown, very soft, highly compressible, clay of thickness range 0.3 m to 1.2 m covering the seabed. Also, this layer is overlying a thin very weak to strong cap rock layer of about 0.7 m to 1.5 m thickness and below the cap rock is a very loose silty sand layer of average thickness 4.0 m overlying medium dense to very dense sand layer with silt which continues to the depth of 64 m. The deepest layer encountered at 64 to 67 m during investigations is light grey to light brown, weak to strong, highly to slightly weathered soft rock comprising alternate beds of sandstone and conglomerate. It's shown as below figure.

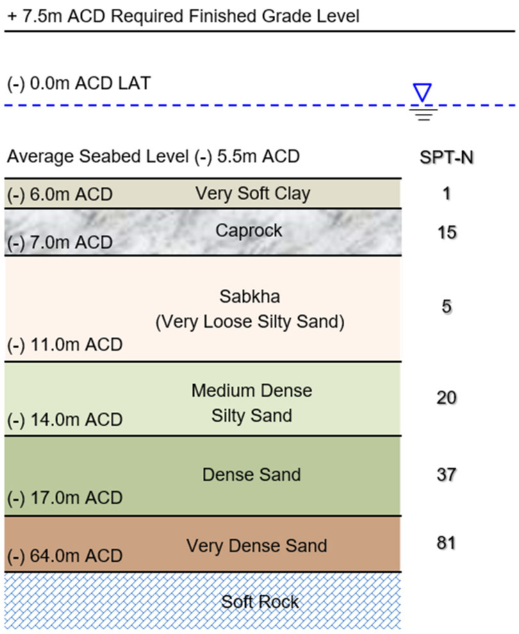


Figure 2. Typical Soil Profile of LNG Import Project Site.

By comparison with the regional geology and assessing the characteristics and composition of the sand it is evaluated that the upper part of the strata encountered (5.5 m to 64 m) is upper Dibdibah Formation of the Kuwait Group and the deep layer of soft rock appears to be the part of the lower Dibdibah Formation of the same group.

The seabed bathymetry with the project area superimposed is presented on Figure 3. The seabed level at the Reclamation Area varies from about -5.0 m ACD (Admiralty Chart Datum) at the western side to -8.5 m ACD on the eastern side.

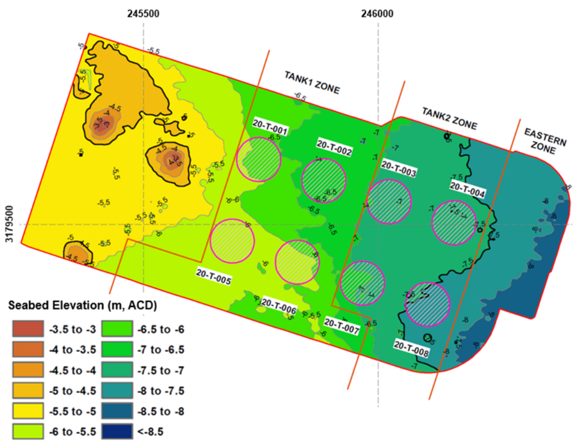


Figure 3. Results of Bathymetry Survey at LNG Reclamation Area

2.2 *Site Reclamation and Improvement*

Based on geotechnical investigation results, the clayey soil overlying the Caprock layer was found all over the reclamation area and was removed the clayey soil to avoid unacceptable settlement. Further, the sabkha soil below the Caprock was classified as unsuitable material, and it may lead to intolerable differential settlements. Therefore, the problematic soils and Caprock was dredged and disposed before the reclamation work of the project site. The reclaimed land was surrounded to the north, east and south sides by the shore protection rock revetment structure and connected with the onshore area at its west side. The offshore reclaimed land was completed as the earth fill structure using dredged material from the nearby approved borrow area. The reclamation materials placed up to final ground level comply with the requirements as shown table.

Table 1. Requirements of Reclamation Fill Material

Properties	Value	Unit
Maximum Particle Size	125	mm
Max. Passing 75-micron Sieve	20	%
Max. Passing 2-micron Sieve	2	%
Max. Plasticity Index	10	%
Minimum Soaked CBR	15	%

For the reclamation fill layer, Vibro-Compaction was proposed to compact loose granular soils and to achieve a uniform level of densification. The method is used to increase bearing capacity, reduce foundation settlements, and reduce seismic subsidence and liquefaction potential. This technique drives a vibrating probe into the filling ground, which generates lateral vibratory forces to rearrange particles into a dense state. It's suitable for densifying deep deposit or fills with cohesionless geo-material up to 20% fines but less than 2~3% of clay particles. The detailed plan for Vibro-Compaction such as spacing, grid arrangement was decided depending on the type of mobilized equipment and the trial test result as below:

- Grid Pattern: Triangular Pattern
- Prove Spacing: 3.5 m
- Improvement Depth: 18.0m~20.0m from F.G.L+7.5m
- Compaction Criteria: 95% of Maximum Dry Density

After ground improvement, the verification test (CPTs) was conducted at 3 points every 50 m grid to confirm that the ground was improved successfully satisfying designed level.

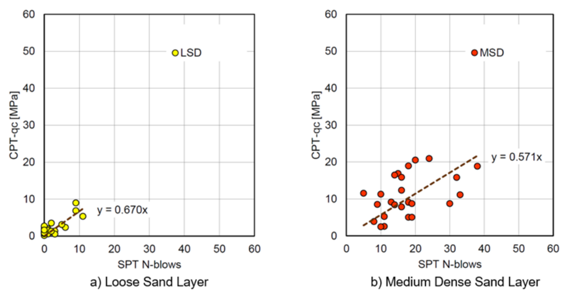
2.3 *Ground Profiles of Artificial Island*

Additional geotechnical investigation was conducted to provide the accurate engineering judgement for subsoil characteristics of the reclaimed area as followings:

- Borehole with Standard Penetration Test: 55 nos
- Piezo Cone Penetration Test: 16 nos
- Downhole Seismic Test: 7 nos
- Laboratory Tests

The ground conditions as disclosed by the field and laboratory tests consist of granular material from existing ground surface to the terminal depths of the boreholes and it's classified as silty sand (SM), poorly/well graded sand with silt (SP-SM/SW-SM).

In order to find the relationship between the SPT N-blows and CPT tip resistance (qc) for each soil units, both SPT-N blows and CPT-qc are plotted in one graph and evaluated as shown Figure 4.



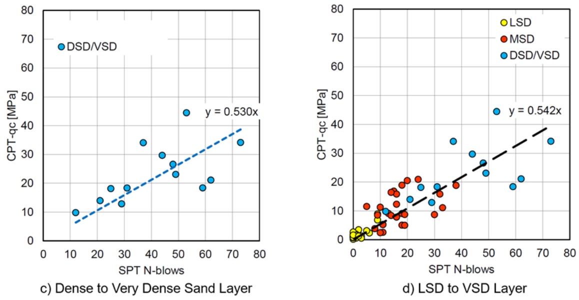


Figure 4. SPT(N-blows) vs CPT(qc) correlation

Based on the correlation graphs, each soil layer shows a different trend with slightly scattering of data with MSD and DSD/VSD units show close similarity. To generalize the correlation between SPT and CPT for the full depth, data from all units are combined as shown in (d). The trend line shown in (d) depicts a comparable fitting trend for the full depths of all CPTs. The SPT-N and CPT-qc results are presented together, adopting the depicted correlation in Figure 4(d) equal to qc ≈ 0.55N.

In this study, the settlement monitoring was performed in the shallow footings of process area that can be observed the displacement until the completion of construction and the representative soil profile is shown as below.

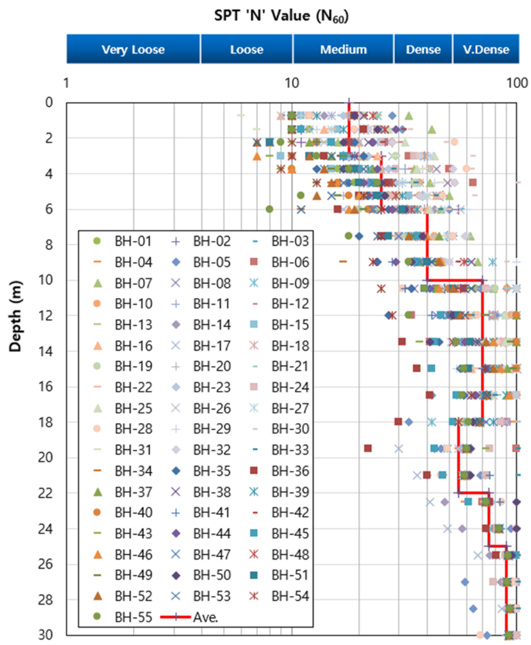


Figure 5. Site Soil Profiles from Finished Grade Level +7.5m.

Considering the variable soil profiles of each borehole, the settlement calculation was estimated base of the closest field test results to each monitored footing.

3 SETTLEMENT CALCULATION BASED ON EMPIRICAL FORMULA

3.1 *Procedure of Settlement Measurement*

The permanent benchmarks were installed at the project site for construction work and used as base of the monitoring work.

The settlement monitoring points were selected the foundations that can be checked as the visual inspection from benchmarks, and the measurement devices (Angle Stud & Name Plate, non-corrosive material) were fixed at the concrete pedestal/column of foundation as shown in Figure 6.

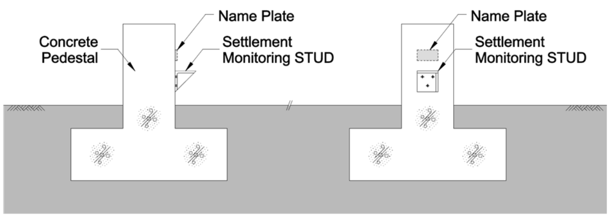


Figure 6. Monitoring Survey Points for Faoundations

It's connected the angle stud at each shallow foundation after the construction of the footing and the settlement monitoring was conducted until the completion of structure installation.

In this research, since the actual loading was accumulated over a long period of time during construction stage, the creep settlement of the artificial island was estimated through the ground monitoring of the reclaimed layer and excluded from the measured total settlement at each foundation. The long-term settlement of the subsoil focused on the creep settlement of the reclamation fill considering the homogeneous profile: Medium Dense Sand, Dense Sand, and Very Dense Sand. The settlement measuring devices were installed as part of the instrumentation program presented in Table 2.

Table 2. Program of Instruments Installation

Instrumentation	Time of Installation	Location
Deep Datum	On completion of filling and compaction	Grid 300m
Rod Settlement Gauge	On completion of filling and compaction	Grid 250m
Settlement Marker	On completion of filling and compaction	Grid 250m
Magnetic Extensometers	On completion of filling and compaction	Grid 300m
Standpipe Piezometer	On completion of filling and compaction	Grid 300m

Total creep settlement ranged from 7 mm to 23 mm after the instrument installation. During the settlement monitoring period of shallow foundation, the creep of 1 mm to 3 mm was measured to have occurred as below table.

Table 3. Summary of Creep Settlement for Reclaimed Site

Point No.	Creep Se. (mm)		**Point No.**	Creep Se. (mm)	
	Total	Mo. Period		Total	Mo. Period
P-1	12.0	3.0	**P-7**	16.0	2.0
P-2	9.0	1.0	**P-8**	23.0	2.0
P-3	13.0	2.0	**P-9**	13.0	2.0
P-4	7.0	2.0	**P-10**	9.0	1.0
P-5	22.0	2.0	**P-11**	12.0	2.0
P-6	23.0	3.0	**P-12**	9.0	1.0

3.2 *Elastic Settlement Calculation for Each Foundation*

Regarding to the settlement analysis of soil, many studies have been proceeding since the 1930's and proposed the various formulas to accurately predict the amount of settlement. The methods of the elastic settlement calculation for the granular soil can be divided into three general categories, methods based on theoretical relationship derived from the elastic theory of soil, the empirical formulas using in-situ test results, the semi-empirical formulas based on a combination of field test and some theoretical studies.

The settlement estimation of this study was conducted using the empirical formulas and compared to the observed settlement results. The outlines of these methods are given in the followings.

3.2.1 *Terzaghi and Peck's Method*

Terzaghi and Peck (1948, 1967) proposed a correlation for the allowable bearing capacity, standard penetration blow number (N_{60}), and the foundation width (B) corresponding to a 25 mm settlement based on the observation given by the plate load test. This correlation can be approximated by the relation as below equation.

$$S_e(mm) = C_W C_D \frac{3q'}{N_{60}}\left(\frac{B}{B+0.3}\right)^2 \quad (1)$$

Where, q' : Net bearing pressure in kN/m^2
C_W : Ground water table correction = 1.0
C_D : Correction for embedment depth = $1-(D_f/4B)$
D_f : Embedment depth of the foundation
N_{60} : Average value up to depth 4B from B.O.F

3.2.2 *Meyerhof's Method*

Meyerhof proposed relationships for the elastic settlement of foundations on granular soil like Eq. (1) in 1956 comparing with the predicted and observed settlements of eight structures. The revised relationships expressed as below.
For $B \leq 1.22m$

$$S_e(mm) = C_W C_D \frac{1.25q'}{N_{60}} \quad (2)$$

For $B > 1.22m$

$$S_e(mm) = C_W C_D \frac{2q'}{N_{60}}\left(\frac{B}{B+0.3}\right)^2 \quad (3)$$

Where, C_W : Ground water table correction = 1.0
C_D : Correction for embedment depth = $1-(D_f/4B)$
N_{60} : Average value up to depth 4B from B.O.F

3.2.3 *Peck and Bazaraa Method*

Peck and Bazaraa (1969) recognized that the original Terzaghi and Peck method was overly conservative and revised Eq. (1) to the following formula.

$$S_e(mm) = C_W C_D \frac{2q'}{(N_1)_{60}}\left(\frac{B}{B+0.3}\right)^2 \quad (4)$$

Where,

$$C_W = \frac{\sigma_0 \ at\ 0.5B\ below\ the\ footing\ bottom}{\sigma'_0 \ at\ 0.5B\ below\ the\ footing\ bottom}$$

$$C_D = 1.0 - 0.4\left(\frac{\gamma D_f}{q'}\right)^{0.5}$$

$$(N_1)_{60} = \frac{4N_{60}}{1+0.04\sigma'_0} \quad (for\ \sigma'_0 \leq 75kPa)$$

$$(N_1)_{60} = \frac{4N_{60}}{3.25+0.01\sigma'_0} \quad (for\ \sigma'_0 > 75kPa)$$

σ_0 : Total overburden pressure
σ'_0 : Effective overburden pressure
r : Unit weight of soil
N_{60} : Average value up to depth 4B from B.O.F

3.2.4 *Method of Burland and Burbidge*

Burland and Burbidge (1985) proposed the empirical formulas to estimate the elastic settlement of sandy soil using the field standard penetration number N_{60}. The method for normally consolidated soil was suggested as follow.

$$S_e(mm) = q' \cdot B^{0.7} \cdot I_c \cdot \left[\frac{1.25\left(\frac{L}{B}\right)}{0.25+\left(\frac{L}{B}\right)}\right]^2 \quad (5)$$

Where, I_c : Compressibility = $1.71 / N_{60(a)}^{1.4}$
$N_{60(a)}$: $1.25N_{60}$, $15+0.5(N_{60}-15)$ if $N_{60} > 15$
L : Length of the foundation
N_{60} : Average value up to depth Z_1 from B.O.F
Z_1 : Influence Depth = $B^{0.75}$

3.3 *Results of Settlement Measurement*

Settlement measurement was monitored at 95 footings with the different sizes and the recorded settlements were indicated as Figure 7 & 8. Since the shallow foundations were designed and constructed according to the project specification with 25mm allowable settlement criteria, the elastic settlement of all footings was measured to occur under 25mm.

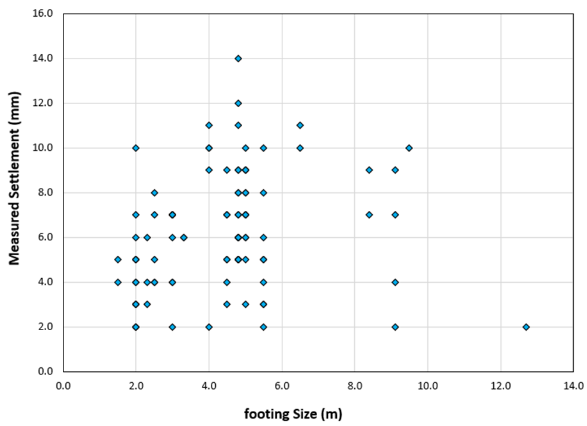


Figure 7. Measured Settlement Results vs. Footing Size

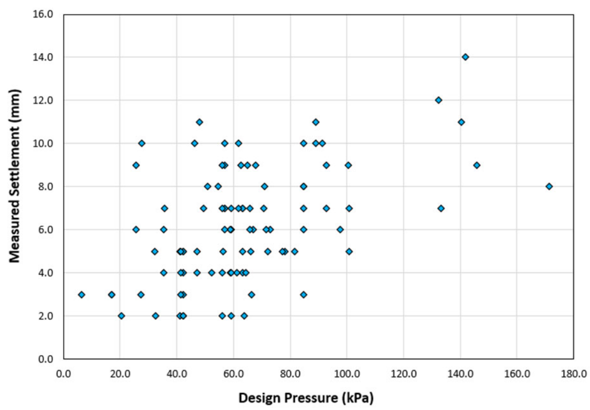


Figure 8. Measured Settlement Results vs. Design Pressure

The focus of this study is to evaluate four available methods for predicting the elastic settlement of the shallow foundations through comparison between the calculation results with the measured values from field monitoring.

Based on the soil profile and the monitoring results, the assessment and comparison of these 4 methods are provided in the following chapter.

4 REVIEW AND ANALYSIS FOR SETTLEMENT ESTIMATION

4.1 *Settlement Calculation Results*

According to the empirical formulas, the predicted settlement of each foundation is calculated, and the representative design parameters of the calculation are considered as following table.

Table 4. Design Parameters for Each Method

Method	Influence Depth	Design N-value
Terzaghi and Peck	4B	44.5
Meyerhof	4B	44.5
Peck and Bazaraa	4B	69.0
Burland & Burbidge	$B^{0.75}$	19.7

Based on these conditions, the estimated settlement from different methods is compared to the measured settlement obtained in field monitoring as shown in Figure 9.

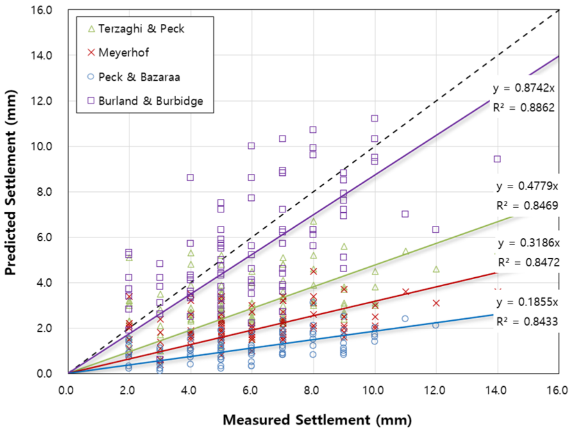


Figure 9. Plot of Measured versus Predicted Settlement based on Empirical Methods

- Terzaghi and Peck : $y = 0.478x, R^2 = 0.847$
- Meyerhof : $y = 0.319x, R^2 = 0.847$
- Peck & Bazaraa : $y = 0.185x, R^2 = 0.843$
- Burland & Burbidge : $y = 0.874x, R^2 = 0.886$

In the calculation results of the conventional method, almost of the predicted settlements are estimated as lower values than the measured settlement results except for Burland & Burbidge's method.

4.2 *Evaluation and Analysis for Settlement Calculation*

Generally, these empirical formulas have been considered as one of the conservative methods to be overestimated the elastic settlement rather than the realistic values. However, in this analysis, the estimated results of these methods are forecasted around 20 to 80% of the measured settlement, and it could generate some problems to ensure the stability of structures.

Therefore, there are analyzed the reliability of each empirical formula on the artificial island through the studying and reviewing for the design parameters of these solutions.

4.2.1 *Relationship for Influence Depth*

Regarding to the stiffness of soil, even though the project site is artificial island prepared by reclamation and vibro-compaction improvement, in-situ SPT-N blows were indicated high values like over-consolidated sand layer. So, in 3 methods using the influence depth 4B, the design N-values are suggested very strong soil layer as much as very dense sand.

To evaluate the relationship of the calculated settlement depending on the influence depth, the elastic settlement of soil recalculated for the design N-values of 3 cases, influence depth 1B, 2B and 3B. The calculation results are followings and Burland and Burbidge's method is calculated considering the influence depth $B^{0.75}$.

Table 5. Comparison of Measured and Predicted Settlements as per Influence Depth

Method	Settlement Ratio as per Influence Depth			
	1B, $B^{0.75}$	2B	3B	4B
Terzaghi and Peck	0.88	0.65	0.53	0.48
Meyerhof	0.58	0.43	0.35	0.32
Peck and Bazaraa	0.33	0.25	0.20	0.18
Burland and Burbidge	0.87	-	-	-

These results show the comparison of the observed settlements of shallow foundations and the settlements predicted by Eq. (1), (3) & (4). In Terzaghi and Peck's method with the influence depth 1B, the predicted settlements are estimated closely to the measured settlements, but other cases may be evaluated as the inappropriate analysis. Specifically, the results calculated by Peck and Bazaraa's method were less than 50% of the measured settlement values.

Considering the influence depth of the foundations, these results would be relied on the creep settlement to occur in the reclaimed sand filling layer during construction period for the equipment installation. Therefore, in next chapter, the predicted settlements are evaluated considering the creep settlement of filling layer as per time.

4.2.2 *Relationship for Creep Settlement*

The creep settlement of the artificial island was estimated through the ground monitoring and reflected on the settlement calculation. However, additional creep settlement could occur at each footing due to the unpractical conditions like structure loading and construction activities.

Schmertmann method (1970) and Burland and Burbidge method (1985) are the most distinguished methods which can be used for estimating the long-term creep settlement (C_2) of cohesionless soils. Schmertmann method applies the following time factor to account the creep settlement:

$$C_2(mm) = 1.2 + 0.2\log t \text{ (t : time, in year)} \tag{6}$$

Burland and Burbidge (1985) noted time-dependent settlement of footings and suggested a multiplication factor (f_t) given by $f_t=1+R_3+R_t\log(t/3)$. Where, R_3 takes into consideration the time -dependent settlement during the first three years of loadings, and the last component accounts for the time-dependent settlement that takes place after the first three years at a lower rate. Suggested values for R_3 and R_t are 0.3 and 0.2, respectively.

For the reclaimed project site, the construction phase was performed for 2 years after the ground improvement work. The additional creep settlements that occurred during this stage are calculated as follows:

$$C_2(mm) = 1.2 + 0.2\,log\, t = 1.26 \tag{7}$$

$$f_t(mm) = 1 + 0.3 + 0.2\,log\, t/3 = 1.26 \tag{8}$$

Two methods conclude almost the same values: that the creep settlement will be 26% of the elastic settlement. The predicted settlements are evaluated as shown in Figure 10. Additionally, 3 methods using an influence depth of 4B conducted additional calculations for influence depths of 1B, 2B, and 3B to analyze the relationship of the estimated settlement.

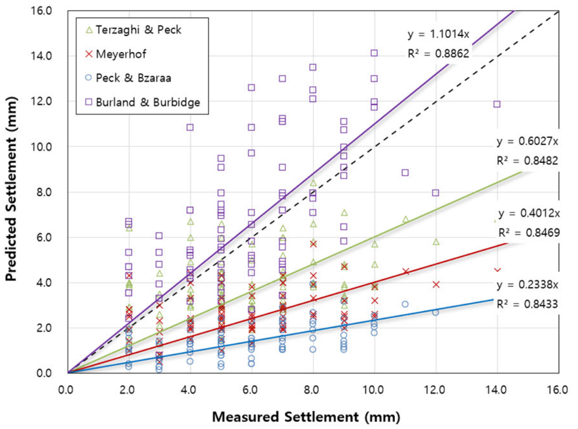


Figure 10. Plot of Measured versus Predicted Settlement based on Empirical Methods and Additonal Creep Settlement

- Terzaghi and Peck : $y = 0.603x, R^2 = 0.848$
- Meyerhof : $y = 0.401x, R^2 = 0.847$
- Peck & Bazaraa : $y = 0.234x, R^2 = 0.843$
- Burland & Burbidge : $y = 1.101x, R^2 = 0.886$

Table 6. Comparison of Measured and Predicted Settlement as per Influence Depth and Additional Creep Settlement

Method	Settlement Ratio as per Influence Depth			
	1B, $B^{0.75}$	2B	3B	4B
Terzaghi and Peck	1.11	0.82	0.67	0.60
Meyerhof	0.73	0.55	0.45	0.40
Peck and Bazaraa	0.42	0.32	0.26	0.23
Burland and Burbidge	1.10	-	-	-

5 CONCLUSION

In this study, 4 existing analytical methods for estimating the elastic settlement of shallow foundation were evaluated for their performance and accuracy comparing with the measured settlements at the actual 95 footings. Based on the results of the analysis, the following general conclusions can be provided.

1. Since the settlement of the shallow foundation has been measured for 2 years due to the construction sequence, the creep settlement is considered to evaluate the predicted elastic settlement.
2. For the 3 methods considering the influence depth 4B, Terzaghi and Peck's relationship provides more accurate results than other methods comparing with the observed settlement in the field.
3. However, the settlement ratio is around 60% and the predicted settlements are evaluated to underestimate than the measured settlements.
4. Burland and Burbidge's solution provides more reasonable estimations of the elastic settlement than those obtained from the solution of Terzaghi, Meyerhof and Peck.
5. In this analysis, the influence depth is considered as the main parameter to predict the elastic settlement of the shallow foundations, and the estimated settlements are indicated the similar results with the monitoring results when the influence depth is under 1B.

Consequently, Burland and Burbidge's method is evaluated as more accurate approach for calculating the elastic settlement of shallow foundation in the artificial island made by reclamation and vibro-compaction. However, the scatter plots are widely distributed in the graph, and the calculated settlements are predicted to be underestimated than the measured settlements unlike the general outcome in other studies.

Therefore, even those empirical solutions that showed effectiveness in this study should be applied cautiously for the foundation design. For the reclaimed fill ground, in scenarios considering more complex or variable ground conditions, and to accurately account for the settlement characteristics of multiple soil layers, it would be required the numerical modeling or semi-empirical methods.

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