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# Bearing Capacity of Hollowed Shallow Foundation on Cikarang Expansive Clays – An Alternative Foundation for Light Weight Structures

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**ABSTRACT**

An 800 mm x 800 mm dimension shallow foundation is proposed as an alternative to bear light-weight structure. Unlike the conventional shallow foundation which is installed in-situ, the proposed foundation system is precast concrete and predesigned with void within inner part of the foundation body to accommodate the predicted heave as it is installed on expansive soil layer. Full-scale load test was conducted to study its bearing capacity performance. The site is located in Cikarang – a town in West Java, Indonesia – that is widely known as expansive clay deposit. The field test shows allowable bearing capacity of the proposed hollowed foundation reached 16.4 ton despite smaller contact area due to void. In addition, a finite element model is used for comparison and to furthermore estimate soil stiffness parameter. Through this study, it is suggested that the proposed foundation is considered feasible as an alternative to conventional footing foundation to bear light-weight structure.

## 1. Introduction

Jakarta, the capital city of Indonesia, has been recently under the busy and crowded human activities. The development has been expanding to the peripheral cities and regencies, known as Jabodetabek, stands for Jakarta-Bogor-Depok-Tangerang-Bekasi – a megapolitan that connects and supports the larger area. The vast developing economy of megapolitan leads to constructions of many civil structures (high rise buildings), industrial facilities, (factories and warehouses), as well as residential clusters. These cities have become a satellite area to support of Jakarta in many aspects, for years (Fig. 1).

The high demand level of expansion has been proven by dense construction activity of many infrastructures initiated by the Government. Jabodebek LRT, for example – a light metro system that connects the capital to the satellite cities around it – has been in near-end of the construction phases. This is also motivated the construction of 'Japek Layang' or Jakarta-Cikampek Elevated Toll Road (now is officially named Sheikh Mohammed Bin Zayed or MBZ Toll Road) – a newly built elevated highway road constructed above the existing one, which is also a part of Trans-Java Toll Road. The currently on-going construction is a High-Speed Rail (HSR) that provide a new railway system that lies from Jakarta to Bandung, a capital city of West Java Province.

Cikarang, an integrated residential-commercial-industrial town located in Bekasi regency, is also targeted for the vast development as it lies as close as ± 30 kilometer from Jakarta. The development is not focusing on industrial or commercial purposes, but also many residential complexes to support the workforce mobilization. Unfortunately, the proposed area has been becoming concern in civil works for years as it has been remarkably infamous for its 'problematic soil' deposit in West Java known as 'expansive clays'. Therefore, due to massive constructions in this area, geotechnical problem related to expansive soils significantly becomes concern.

Many cases are frequently found in this area related to expansive soil problems. In the USA – based on Department of Housing of Urban Development – the estimated cost in foundation damage due to expansive soil problem might reach $ 9,000,000,000 due to expansive problem itself, varying widely from infrastructures, buildings, and landslides induced damages (Witherspoon, 2006). Damages occurred on structures are often due to uplift (heave) or excessive external force during swelling phase, or loss of bearing capacity due to shrinkage process; Both originate from the unique characteristics of expansive clays. Remedy is always possible but comes with high cost, which might not always be feasible investment for light structures (e.g. one or two stories residential building). Therefore, an engineering invention should be considered for a preventive action.

Fig. 1. Cases of civil structure damages induced by expansive soil

**2. Main Geotechnical Issues**

There are mainly four class of clay minerals: kaolinite, illite, chlorite, and smectite. These clay minerals contribute to the plastic behaviour of clays. This makes clay has high plasticity index as it has wide span of liquid to plastic limit. This means clays tend to bond larger amount of water rather than another fine-grained soil like silts. Additionally, particle size also plays role to this cause, known as specific surface; For example, sands that is known having very little tendency to absorb water as it also has very low plasticity to none.

An expansive clay is a more specific type of clay that mostly contains high content of montmorillonite – a subclass in smectite mineral that has the weakest ion bonding compared to the others (Fig. 2). Among many minerals that form clays, montmorillonite is easily separated by water particle and has capability to attract water by absorbing it into soil skeleton and thus increasing its volume. On the contrary, the loss of moisture makes the soil decrease in volume and undergo shrinkage. This behaviour makes expansive clays basically susceptible to volume change induced by water content.

Tourtelot (1973) and Chen (1975) in the recent study (PT. GEC, 2017a) indicates that montmorillonite is formed by at least two origin materials; The main material is weathered rock material comes from highlands, eroded by river stream and deposited in the coastal area. Within thousands to millions of years, the material sedimentation becomes shales accumulated in sea basin; the other material comes as volcanic ash from past eruptions that settled in the lowlands for long period. Due to combination of poor drainage and sedimentation process, volcanic ash and shale form montmorillonite.

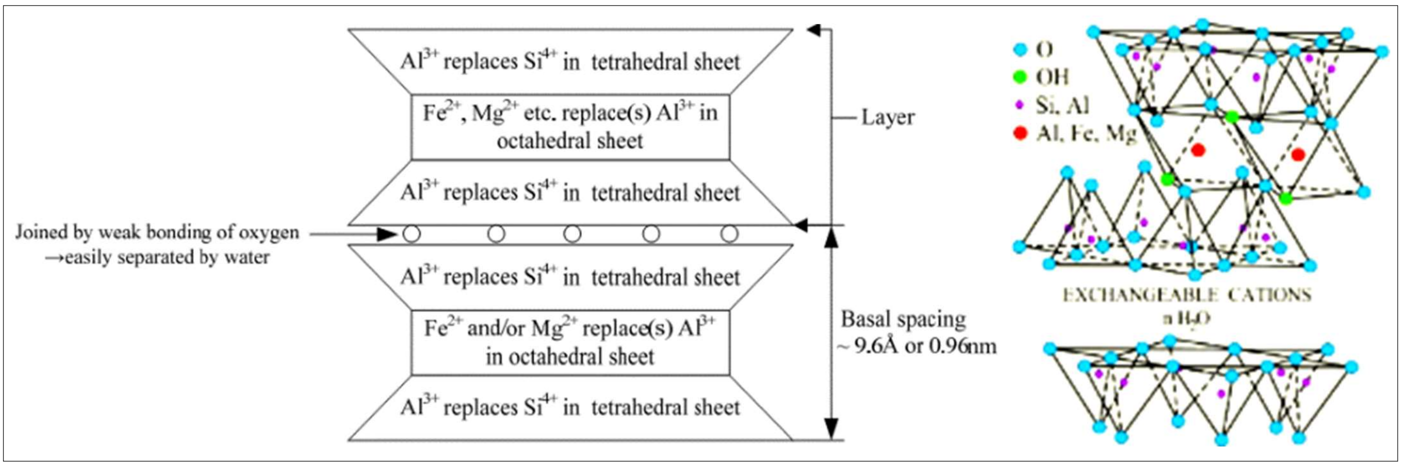



Fig 2. Interlayer bonding in smectite mineral causing clay to swell (Lucian, 2008)


Fig 3. Study location on Google Earth imagery

Geotechnical problem emerged in expansive clays are mostly circling around the two topics: (1) swelling potential of the soil, and (2) strength degradation of the initially-unsaturated soil due to change of moisture content. These two problems may interfere with the performance of foundation, either deep foundations (bored or precast piles), or shallow foundations (footings or slab-on-grounds).

The outcome of this research is to propose an alternative to the conventional shallow foundation for the light weight structure constructed on a near-surface soil layer that is susceptible to swelling. A typical light structure for one-story residential house in Indonesia weighs 10 – 15 ton per column as based on PPIUG (PPIUG, 1983) and updated later in SNI (SNI, 2020), the Indonesian national standard for loading criteria in buildings. This study is to confirm whether the proposed hollowed foundation is feasible in bearing capacity aspect.

## 3. Soil Characteristics

A brief desk study has been performed to determine geological condition of the locus study. In order to determine engineering characteristics, geotechnical exploration includes of in-situ measurement, soil sampling, and laboratory testing has also been conducted.

### *3.1. Geographic location*

The study locus is geographically located in Cikarang, a town in West Java, Indonesia - located approximately 40 kilometers from the capital city. This is an area designated mostly for factories, warehouses, and industrial facilities. The study area is an empty lot without any construction or operational activities, as shown in Fig 3.

### *3.2. Geological condition*

Based on desk study on Systematic Geological Map of Indonesia – Karawang Quadrangle (Achdan and Sudana, 1992), it is known the study area lies on the coverage between the tertiary layer of Cihoe formation (Tpc), and the quaternary layers of conglomeratic and tuffaceous siltstone (Qay) to sandstone (Qoa) unit. The unit consists of mostly tuffs, and either tuffaceous clays or silts. Cihoe formation is susceptible to swell and shrinkage potential. The geographic map of the study locus is presented in Fig 4

### *3.3. Geotechnical properties*

Prior to the work, nearby study had also been introduced to identify the soil behavior (Winata, 2012). The most recent data acquisition program [8] consists of in-situ investigation by core drilling, standard penetration test (Fig 5), and piezo-cone penetration test (Fig 6). Undisturbed samples were collected from field at representative depths (3 meter and 15 meter), taken to laboratory for determining soil parameters. According to the recent data (PT. GEC, 2017b), it is confirmed that study locus has thick deposit of fine grains (silts and clays). As per visual observation of core samples, soil material found at near surface (0-3 m depth) had been oxidized, distinguished by its color change and predicted to have strength degradation – compared to the similar material at greater depth. Through laboratory testing tabulated in Table 2, the clay material found in the study area is known as potentially expansive, indicated by high plasticity and fine content.

The ground water table was encountered approximately at 2-3 m depth below the local ground surface. This is determined through visual observation from the predrilled hole, compared to pore water pressure development (u2) from piezo-cone test (CPTu). However, ground water found inside borehole during the field activity was higher and apparently seemed to be affected by the water used for drilling process. In terms of gravimetric moisture content, there is little difference between water content of soil sample taken at near surface (3 m) to the greater depth (15 m), shown in Table 2.

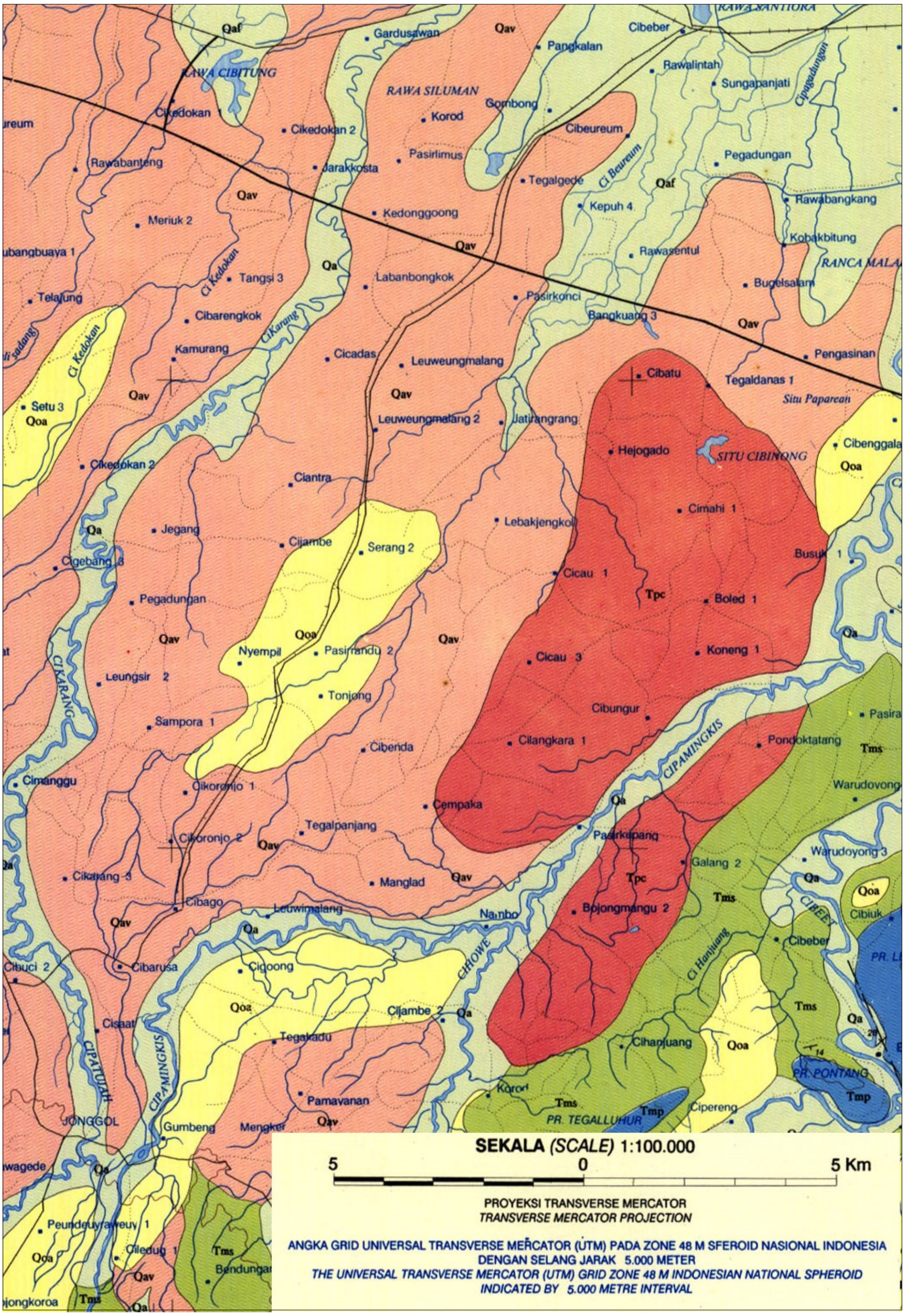



Fig. 4. Locus study on the Karawang Quadrangle geology map

### *3.4. Foundation model*

In principle, swelling pressure that develops during saturation will act as external load working on the footing. This results an extra uplift force and manifests as heave. If this pressure is greater than the dead weight of the structure in any particular column, deformation on structural body will occur. In the proposed hollowed precast foundation (SENARAI OUTPUT, 2017), the concept of stress release is implemented in order to minimize the impact of swelling pressure. At the time saturation begins, heave is allowed to occur only within the designed void, hence minimizing the swelling pressure acting on the footing area; The footing will still take external load but with reduced magnitude as most of heave and swelling force will occur in the void area.

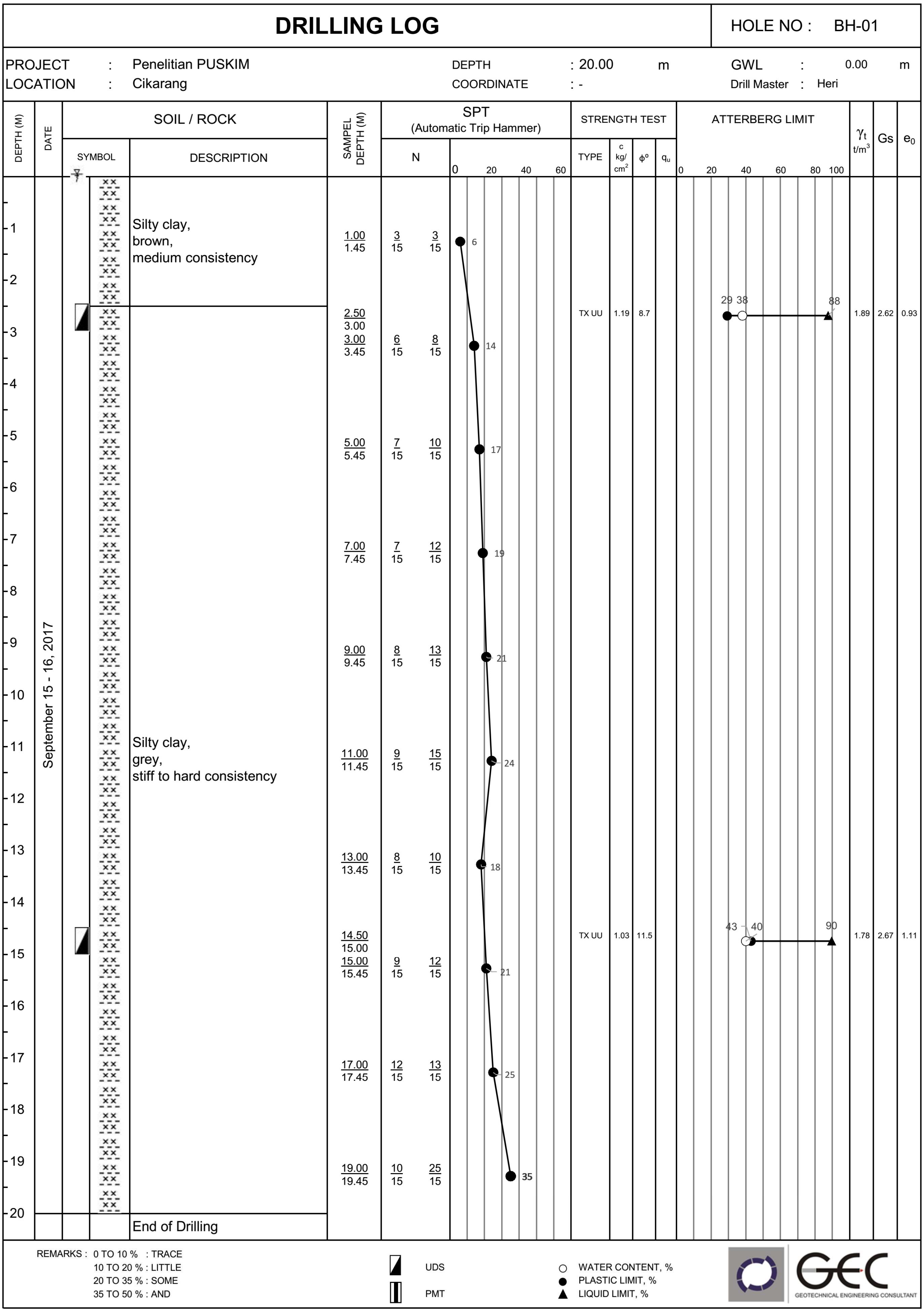



Fig. 5. Drilling log taken in study site (Sudjatmiko, 2017)

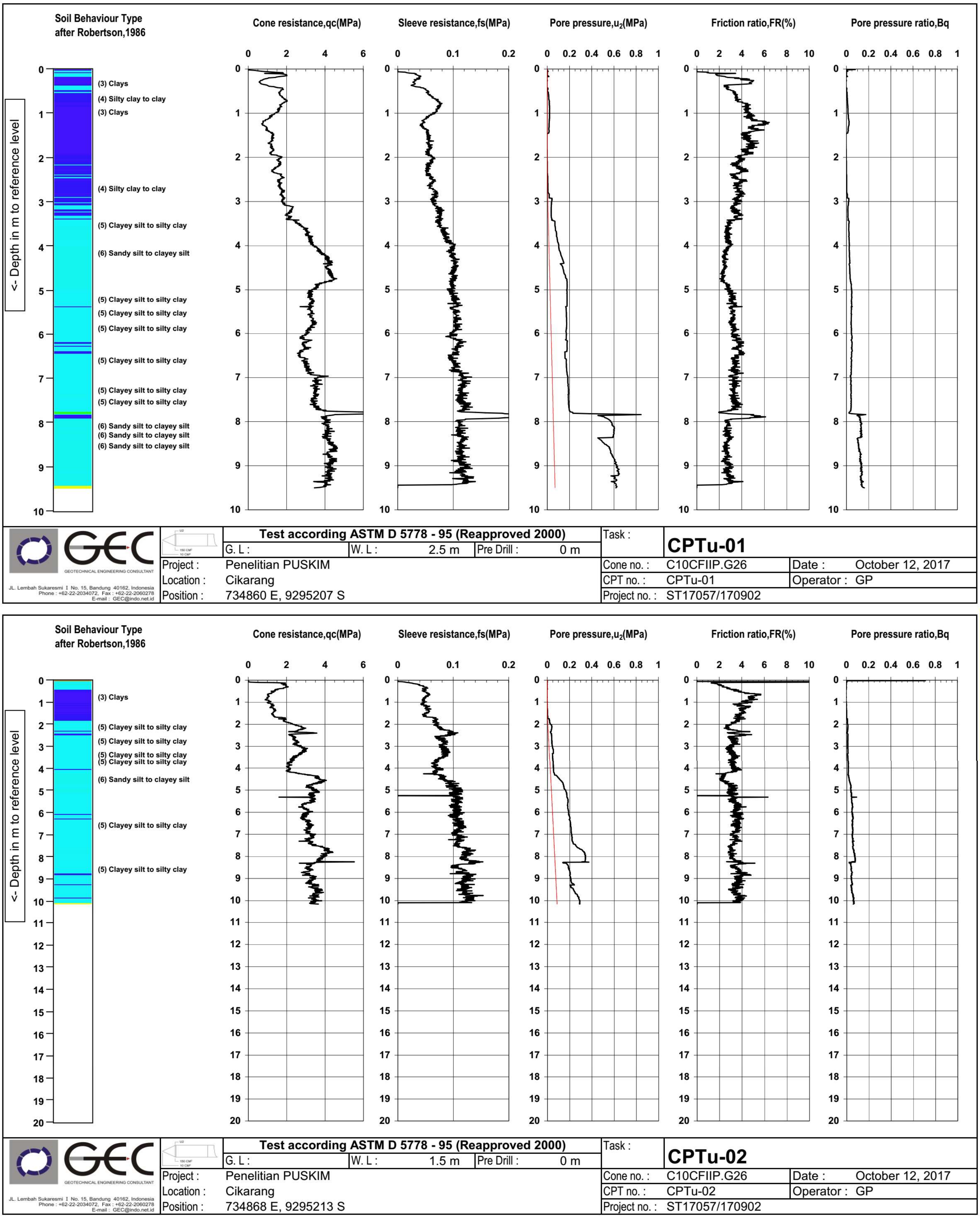



Fig. 6. Piezocone (CPTu) records taken in study site (Sudjatmiko, 2017)

The footing is precast concrete and has gross rectangular area of 80 cm x 80 cm (6400 cm$^2$) with void at its center. The void itself is also rectangular with dimension 40 mm x 40 mm (1600 cm$^2$), making the net area of footing equals to 4800 cm$^2$. An approximate self-dead weight of the concrete body is 1 ton.

### 3.5. Bearing capacity of shallow foundation

The general formula to determine bearing capacity of rectangular shallow foundation, known as Terzaghi's method [11], as follow:

$$q_u = 1.3\, c.N_c + q.\left(N_q - 1\right) + 0.4\, \gamma.B.N_\gamma \quad (1)$$

where $q_u$ is the ultimate bearing capacity of shallow foundation, $c$ is the soil cohesion, $q$ is the overburden stress at the depth of footing, $B$ is the effective width of footing (or the shortest direction), and $\gamma$ is the effective unit weight of soil layer below the footing.

Meanwhile $N_c$, $N_q$, and $N\gamma$ are the bearing capacity factors, defined as function of internal friction angle, which can be determined by the following equations (Terzaghi, 1943):

$$N_q = e^{\pi \tan\phi}.\tan^2\left(45 + \frac{\phi}{2}\right) \quad (2)$$

$$q_u = 1.3\, c.N_c + q.\left(N_q - 1\right) + 0.4\, \gamma.B.N_\gamma \quad (3)$$

$$q_u = 1.3\, c.N_c + q.\left(N_q - 1\right) + 0.4\, \gamma.B.N_\gamma \quad (4)$$

In particular, for loading condition in cohesive soil, it is critical to use undrained shear strength ($S_u$) as cohesion variable ($c$) in equation yukarıda, while the internal friction angle ($\phi$) is 0°. Using the $\phi$ = 0°, the value of $N_c$ is 5.7 while $N_q$ and $N\gamma$ are both zero for this analytical approach.

Geotechnical input parameter for determining bearing capacity through the analytical approach above is derived from the recent soil investigation. Terzaghi and Peck (1967) suggests a widely-used empirical correlation to determine the value of undrained shear strength of cohesive soil based on SPT blow count (BSN, 2017), which is idealized as follows:

$$S_u = 5 \text{ to } 10\, N \quad (5)$$

In many cases, the constant value of 10 from the above equation can be used for most clay with high plasticity behaviour, while the lower bound is typically used for lower plasticity silt. Therefore, for the high plasticity soil found in this study area – which $N$-value lies around 6 blow/ft as shown in (Fig. 5). The $S_u$ is taken 60 kPa in average.

Table 1. Soil layer identified in study area (Sudjatmiko, 2017)

| Depth (m) | Soil Description |
|---|---|
| 0-3 | Light brown-colored, firm to stiff silty clay with N-SPT value 6-14, or with cone resistance less than 2 MPa |
| 3-18 | Grey-colored, stiff silty clay with N-SPT value 14-25, or with cone resistance between 2 to 4 MPa |
| 18-20 | Grey-colored, hard silty clay with N-SPT value reaching 35 |

Table 2. Index properties parameter obtained from laboratory test (Sudjatmiko, 2017)

| Soil Parameter | Value range (%) |
|---|---|
| Natural moisture content, $w_n$ | 38-40 |
| Liquid limit, *LL* | 88-90 |
| Plastic limit, *PL* | 29-43 |
| Plasticity index, *PI* | 47-59 |
| Specific gravity, *Gs* | 2.66-2.72 |
| Fine content (silt and clay) | 97-100 |

Using this parameter, the computed ultimate bearing capacity ($q_u$) = 445 kPa (21.3 ton). However, the Indonesian national standard (SNI) for geotechnical design requirements suggests a common practice to use safety factor value (FS) = 3 for shallow foundation or footing (PT. GE, 2017) – and thus making the allowable bearing capacity ($q_{all}$) = 148 kPa (7.1 ton), equals half of the designed working load on a single column.

### 3.6. Full-scale loading test program

To verify this analytical approach, the full-scale load test was executed (PT. GEC, 2017a; PT. GEC, 2017a; Terzaghi and Peck, 1967). It was also to decide whether the soil is in actual capable to bear the designed load of 15 tons. An incremental loading was conducted until one of these three termination criteria have been met: (1) 200% of design load, (2) 50 mm of settlement or (3) visible failure criteria identified.

Prior to the testing, the soil was excavated to -1 meter from its original elevation. The foundation specimen was then placed on the bottom of excavation, and backfilled with the similar local material from the pit. As reaction system, a dump truck filled with local material – weighed approximately ± 30 ton – was placed and connected to the loading system to provide sufficient counterweight. Schematic pictures shown in Figs. 9 and 10 present the load test configuration and placement of instrumentations. Fig. 11 documentary photographs might give additional information during the setting process.

#### 3.6.1. Instrumentations

Three type of main reading instruments are installed to record the main output used for interpretation:

- Pressure gauge connected to the hydraulic hand pump

system-to measure the applied load on the foundation body
- Two dial gauges connected to the reference beams – to measure vertical displacement (settlement) of foundation body during each corresponding load increment
- Modified crack meter ruler functioned to measure heaving of soil surface at the center void.

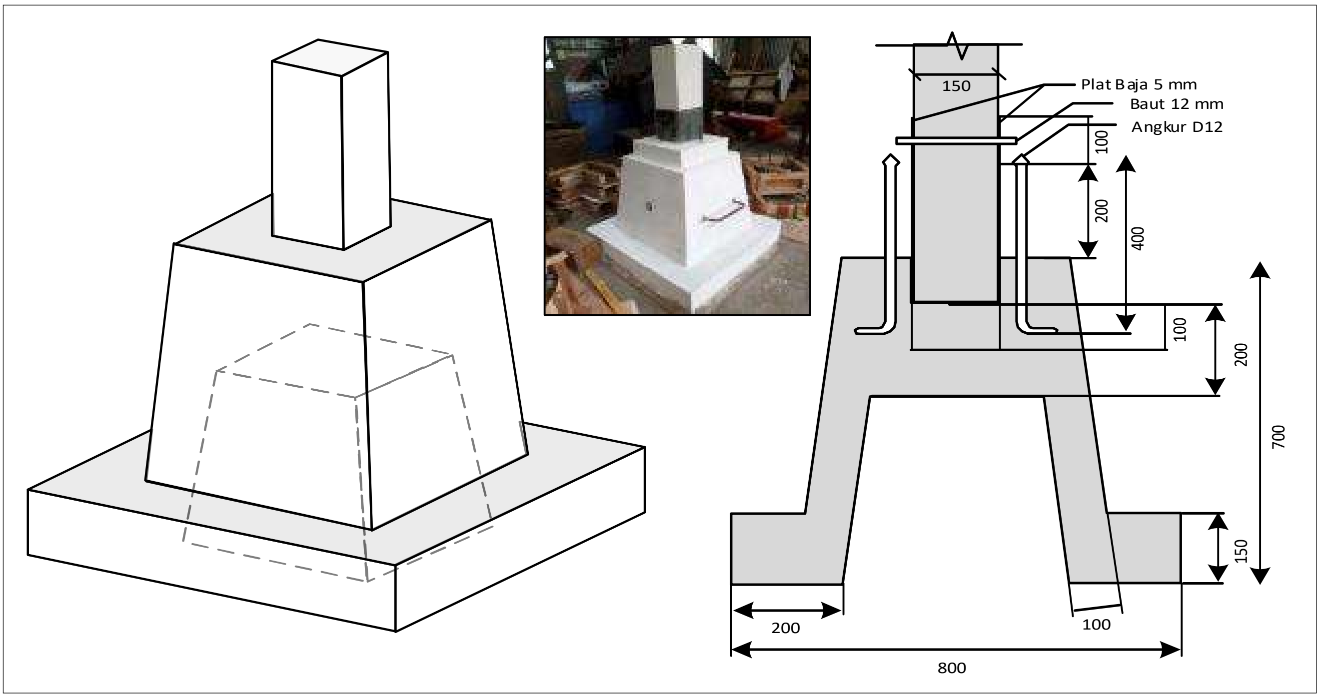



Fig. 7. Precast hollowed shallow foundation proposed in this study

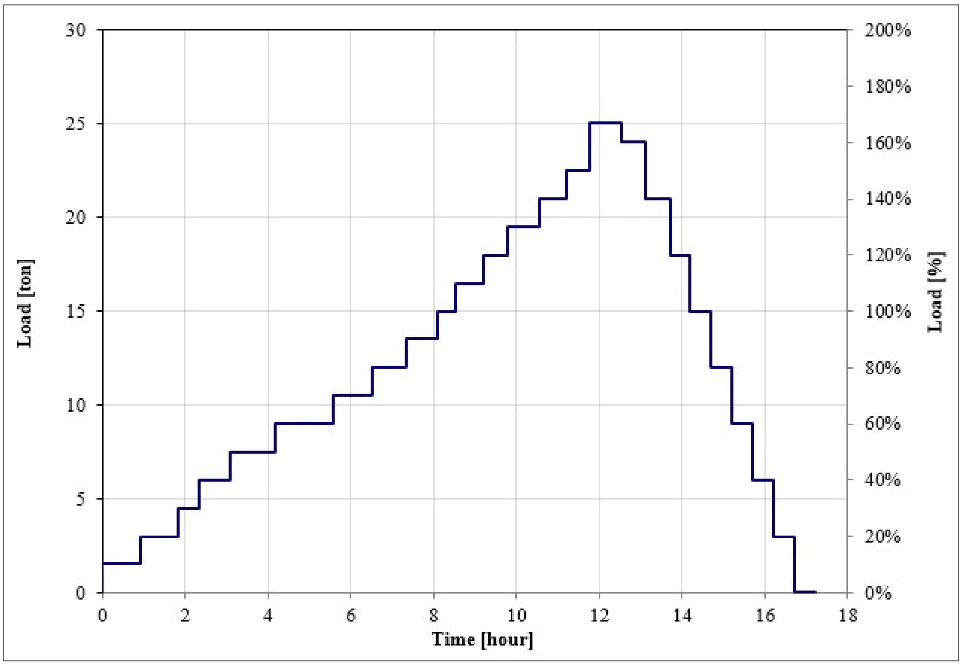



Fig. 8. Loading schedule of static load test

*3.6.2. Load test execution*

The test was done and terminated at 167% of the designed load, or equals to 25 tons; at this point, 50 mm settlement criteria was already met. Interpreting the load vs settlement curve, the plastic condition has been visible and valid for bearing capacity analysis and can be done without extrapolation. Test was halt temporarily due to readjustment of system configuration at load 15 ton. After six hours, the test was then resumed from the last loading cycle (Fig. 12).

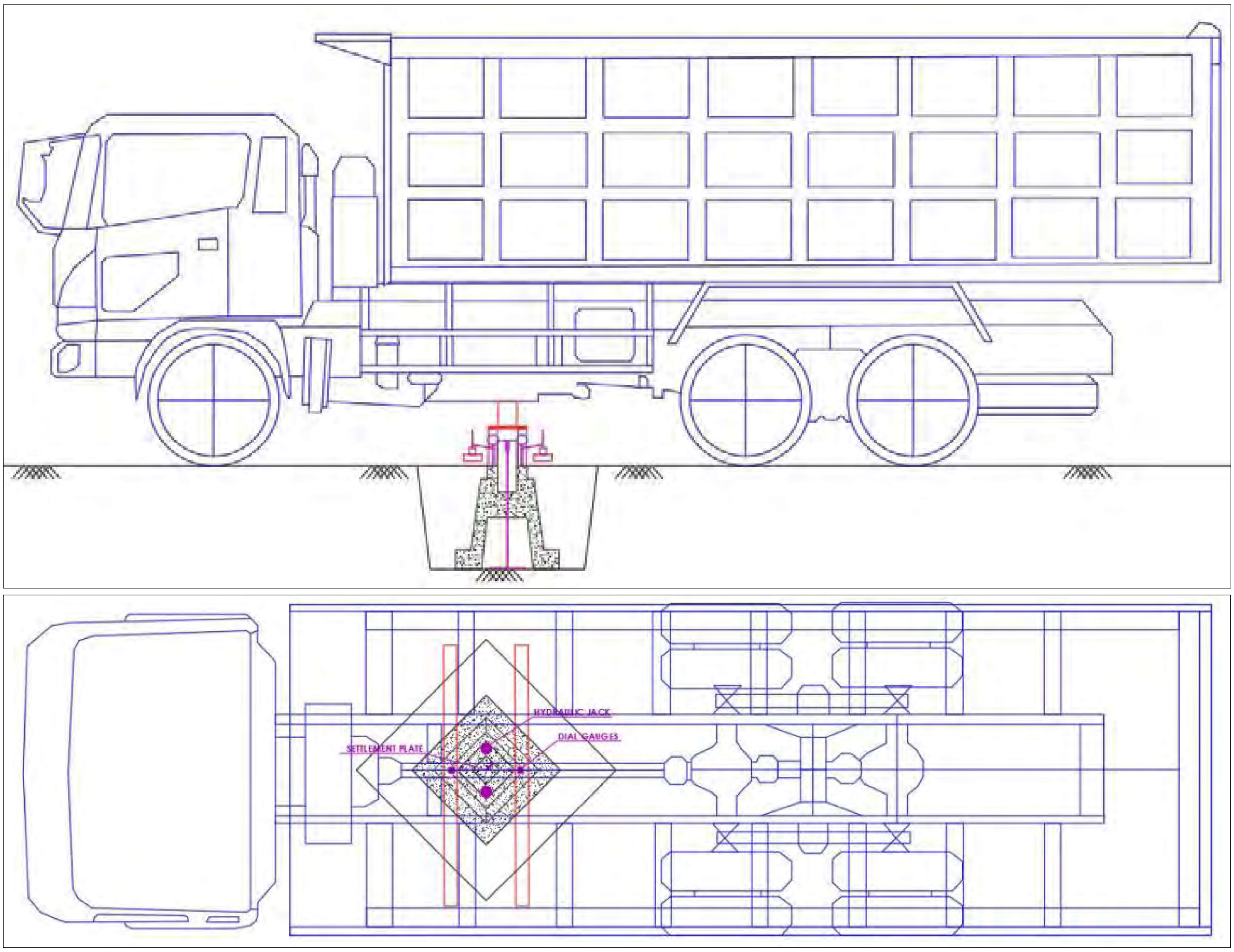



Fig. 9. Load test configuration in side view (upper) and top view (lower)

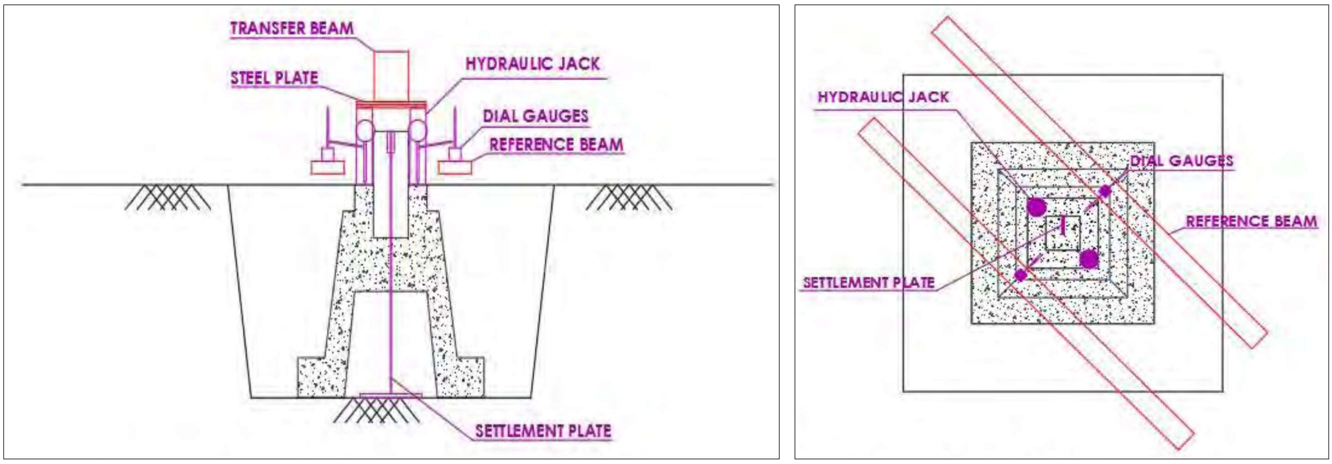



Fig. 10. Installment of instrumentation in cross-sectional view (left) and plane view (right)

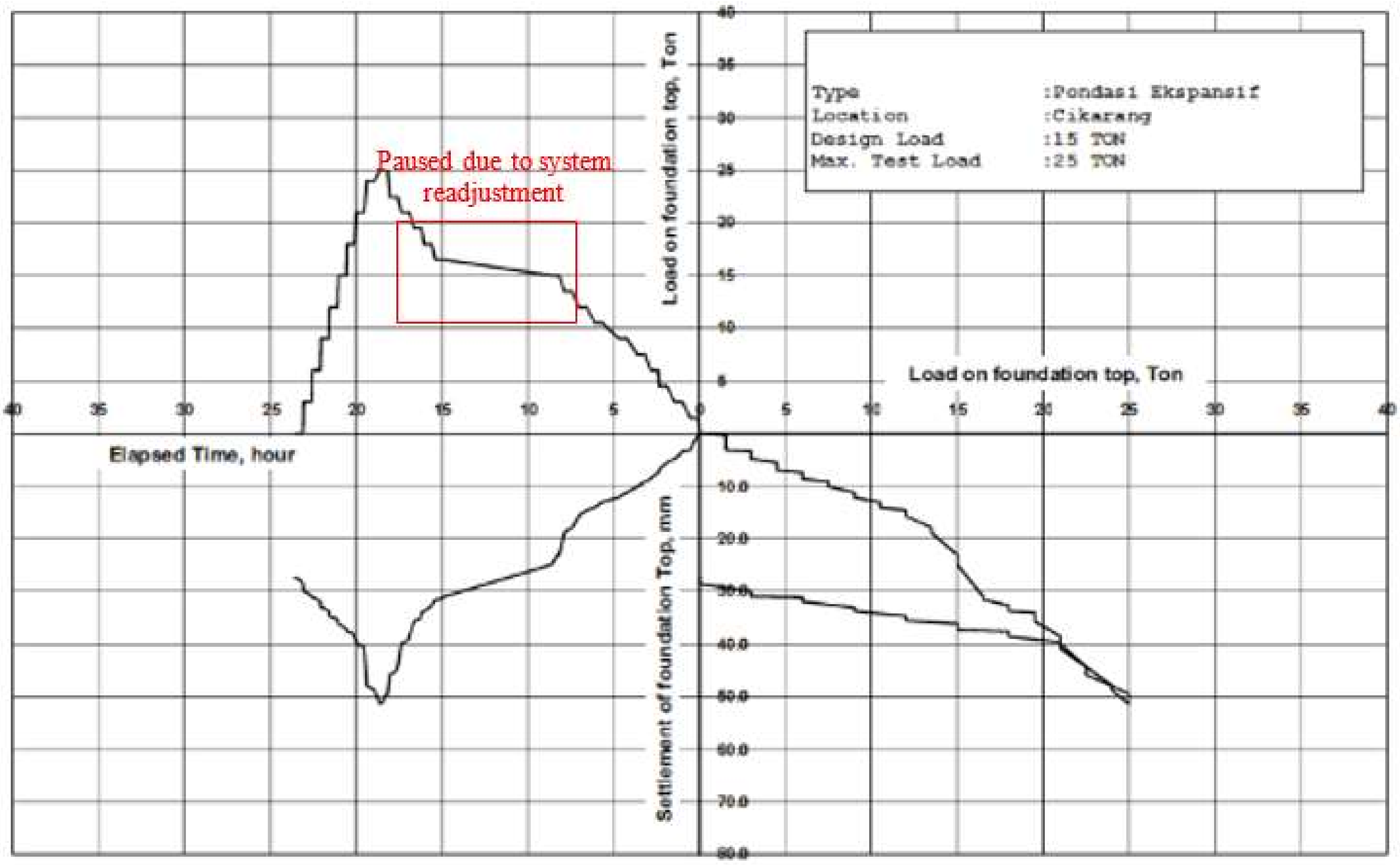
*3.6.3. Data Interpretation*

Bearing capacity interpreted by Mazurkiewicz's method results the ultimate bearing capacity of 28.7 ton (Fig. 13). Chin's method yields 35.8 ton, giving higher value of ultimate capacity (Fig. 14). Through the interpreted data shown in Figs. 13 and 14, these two results make an average of 32.3 ton. As based on the SNI (PT. GE, 2017), the safety factor (FS) = 2 shall be used for the estimated value taken from load test approaches. Therefore, the allowable capacity according to load test is 16.4 ton.


Fig. 11. Process of conducting full-scale load test: a) initial excavation, b) lifting foundation model, c) placing foundation on excavation base, d) backfilling the pit, e) setting up the reaction system, f) configuring main beam as frame, g) setting dial gauges and beams and h) load test execution

Table 3. Settlement test reading on full scale load test

| Axial load applied | | Settlement |
|---|---|---|
| (%) | (ton) | (mm) |
| 0% | 0.0 | 0.00 |
| 10% | 1.5 | 3.15 |
| 20% | 3.0 | 4.85 |
| 30% | 4.5 | 6.95 |
| 40% | 6.0 | 8.60 |
| 50% | 7.5 | 10.05 |
| 60% | 9.0 | 12.25 |
| 70% | 10.5 | 14.10 |
| 80% | 12.0 | 15.75 |
| 90% | 13.5 | 18.80 |
| 100% | 15.0 | 24.50 |
| 110% | 16.5 | 31.50 |
| 120% | 18.0 | 33.78 |
| 130% | 19.5 | 35.75 |
| 140% | 21.0 | 39.98 |
| 150% | 22.5 | 45.90 |
| 167% | 25.0 | 51.40 |
| 160% | 24.0 | 47.95 |
| 140% | 21.0 | 39.68 |
| 120% | 18.0 | 37.63 |
| 100% | 15.0 | 36.18 |
| 80% | 12.0 | 34.70 |
| 60% | 9.0 | 33.10 |
| 40% | 6.0 | 31.10 |
| 20% | 3.0 | 29.96 |
| 0% | 0.0 | 27.40 |




Fig, 12. Result of static load test on hollowed shallow foundation

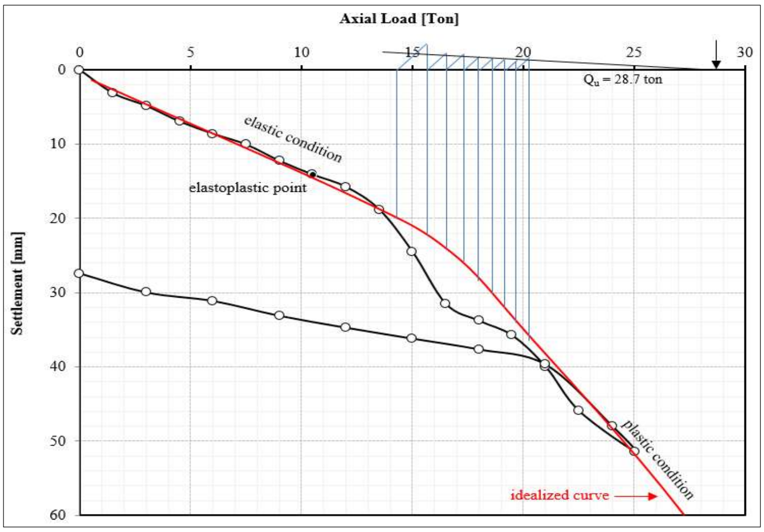



Fig. 13. Bearing capacity interpretation based on Mazurkiewicz's method

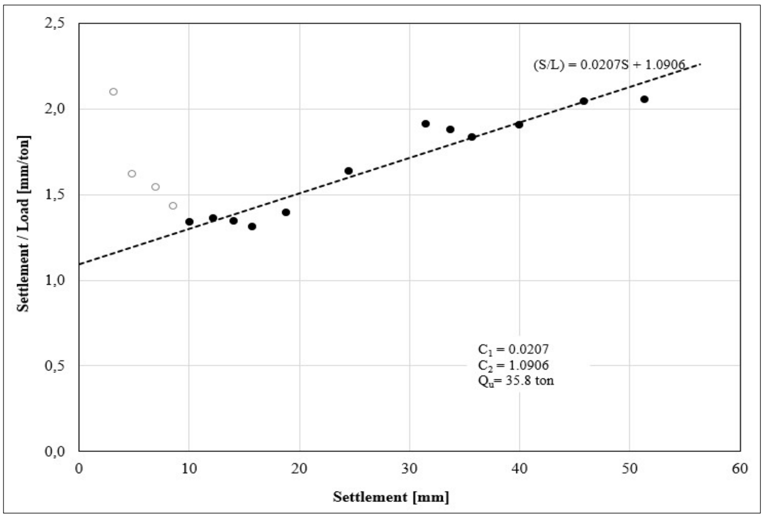



Fig. 14. Bearing capacity interpretation based on Chin's method

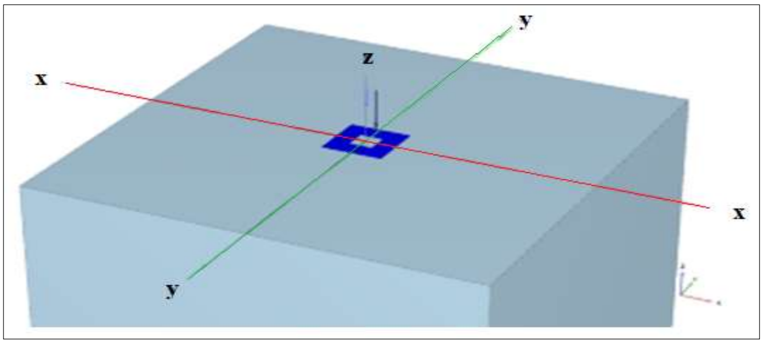



Fig. 15. Full scale foundation model in FEM

Table 4. Suggested input parameter obtained through iteration process

| Parameter | Value | Unit |
|---|---|---|
| Unit weight, γ | 18.9 | $kN/m^3$ |
| Initial void ratio, $e_{init}$ | 0.93 | - |
| Secant modulus at 50% strength, $E_{50}$ | 3936 | kPa |
| Unload-reload modulus, $E_{ur}^{ref}$ | 10000 | kPa |

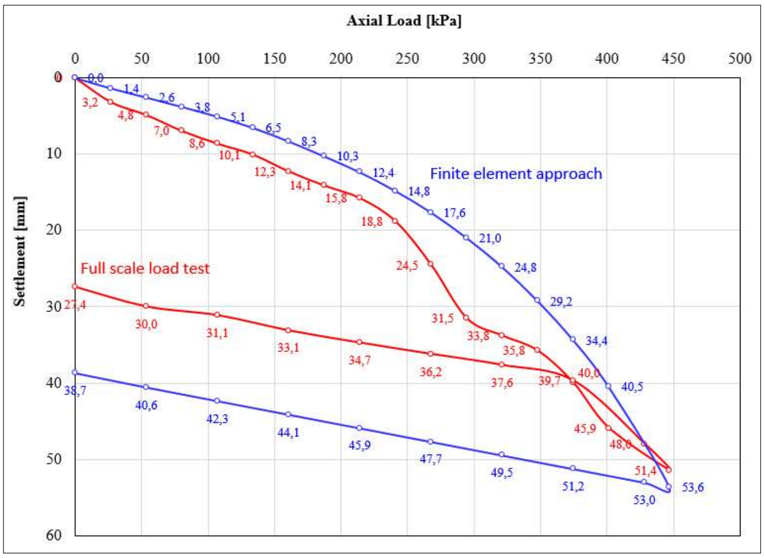



Fig. 16. Load versus settlement curve between full scale load test and FEM

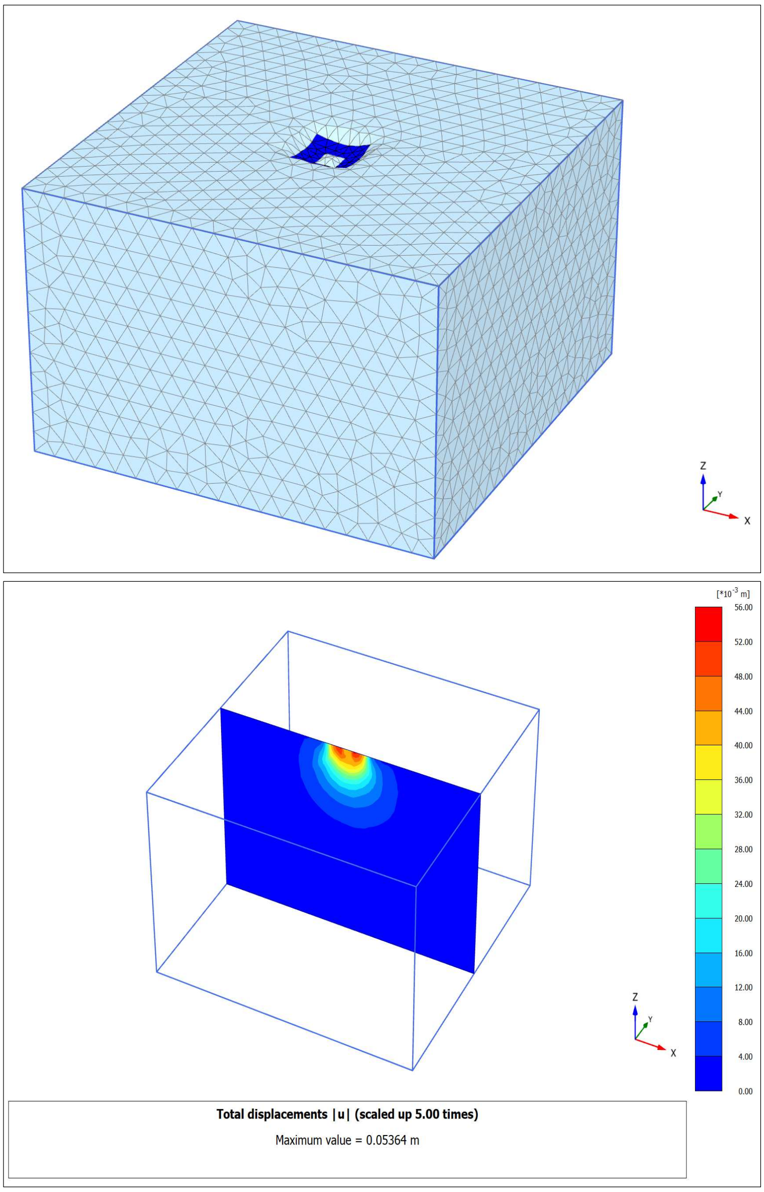



Fig. 17. Outcome of FEM approach showing deformed soil elements and deformation contour

### *3.7. Finite element modelling (FEM)*

A three-dimensional FEM is built to determine the most possible estimation of soil stiffness parameter (in this case is elastic modulus) (Fig. 15-17). The model is used to analyse the soil deformation below the installed foundation system. The output is then used through back-analysis attempt for comparison to the actual full-scaled load test.

There are two main moduli of soil considered for the approach: The E50, which is defined as the 'secant modulus' at 50% of its peak strength; and the Eur or the 'unload-reload' modulus. These two parameters are determined through several iterations, and yield the fittest value as tabulated in below. Meanwhile, a hyperbolic soil model is used to define the Cikarang soil.

## 4. Conclusions and Recommendations

This study points out several conclusions:

- The conventional bearing capacity formula proposed by Terzaghi method tends to underestimate the bearing capacity in Cikarang expansive clay. This is possibly due to unsaturated behaviour of the local soil during the time of testing, and therefore needs further research to study these soil strength characteristics due to partially-saturated condition.

- Through full-scaled load testing to determine the factual bearing capacity, it is concluded that the hollowed shallow foundation system proposed might bear an allowable load of 16.4 ton, higher than the requirement (estimated 15 tons per single column). Therefore, the proposed hollowed foundation might be a potential alternative for light-weight structure.

- Deformation behaviour of Cikarang soil under certain load can be modelled using FEM approach. The attempt in this study focuses to develop the pattern of vertical deformation (i.e. settlement) due to applied axial load. Two calibration variables are used in the approach: (1) settlement occurred at maximum design load; and (2) the gradient of curve for both loading and unloading condition which might represent the stiffness parameter E50 and Eur, respectively. Despite slight deviation between the FEM analysis and the full-scale load test, load-versus-deformation curve in Fig 16 shows similar pattern between these two approaches.

## Acknowledgement

This study is contained within the joint research initiated by Parahyangan Catholic University and Research Institute for Housing and Human Settlements (Puskim) – Ministry of Public Works and Housing of Indonesia (PUPR). The site where field work was executed is under authorisation of PT. Lippo Cikarang in West Java, Indonesia. Geotechnical investigation on early study was conducted by PT. Geotechnical Engineering Consultant, while the full-scaled load test was performed by PT. Geotech Efathama under the supervision by the joint team.

## Nomenclatures

| | |
|---|---|
| *B* | Effective (shortest) width of foundation |
| *C1* | Slope of regression line in Chin's method |
| *C2* | Intercept of regression line to ordinate in Chin's method |
| *C* | Cohesion of soil |
| *FS* | Factor of safety |
| $N_c$ | Terzaghi's bearing capacity factor for soil cohesion variable |
| $N_q$ | Terzaghi's bearing capacity factor for depth variable |
| $N_\gamma$ | Terzaghi's bearing capacity factor for soil unit weight variable |
| $Q_u$ | Ultimate bearing capacity of foundation (in force unit) |
| $Q_{all}$ | Allowable bearing capacity of foundation (in force unit) |
| $q_{all}$ | Allowable unit bearing capacity of foundation (in pressure unit) |
| $q_u$ | Ultimate unit bearing capacity of foundation (in pressure unit) |

## Greek Symbols

| | |
|---|---|
| $\gamma$ | Unit weight of soil |
| $\phi$ | Internal friction angle of soil |

## Abbreviations

| | |
|---|---|
| *FEM* | Finite Element Method |
| *Jabodebek* | Geographical acronym for metropolis consisting: Jakarta, Bogor, Depok, and Bekasi |
| *LRT* | Light Rail Transit |
| *PPIUG* | Peraturan Pembebanan Indonesia untuk Gedung, the Indonesian standard for loading criteria in buildings |
| *PT* | Perseroan Terbatas, an Indonesian abbreviation for 'limited company' |
| *PUPR* | Pekerjaan Umum dan Perumahan Rakyat, the Indonesian ministry specialized in public works and housing |
| *Puskim* | Pusat Litbang Perumahan dan Pemukiman, the Indonesian agency under PUPR that specialized for research and development of housing and human settlement |
| *SNI* | Standard Nasional Indonesia, the Indonesian standard used for geotechnical design requirement |

## Appendix

### *Research Program*

As shown in Fig. 18 below, the research started with desk study to determine whether expansive soil is potentially present in the study locus. The desk study is mainly focused on geological and geotechnical data. Based on this study;

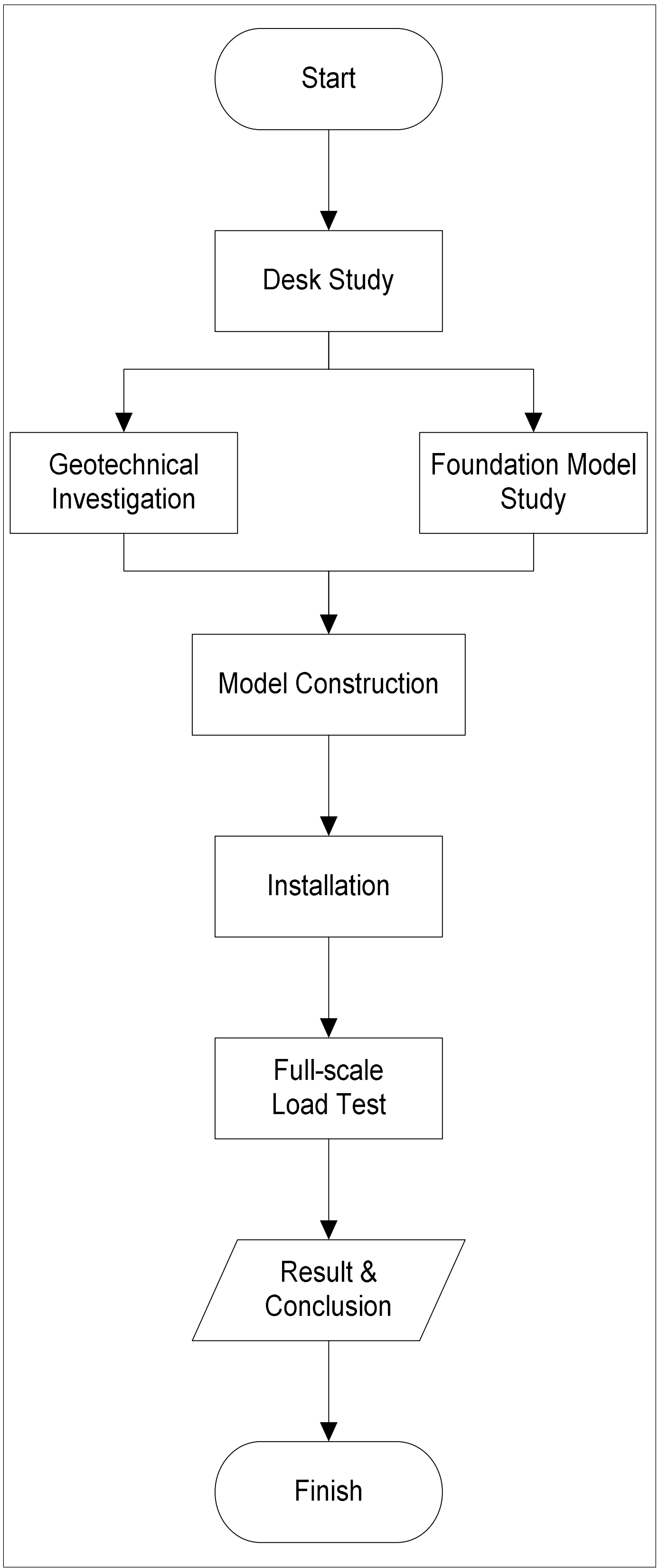



Fig. 1. Flowchart of the research

**Note**

- Desk study includes secondary data collection in geological and geotechnical study,
- Geotechnical investigation consists of soil sampling, in-situ measurement, and series of laboratory testing
- Full-scale quick load test done with non-cyclic loading
- Main result is interpretation of bearing capacity based on Mazurkiewicz's and Chin's method

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