Land reclamation & soil improvement works for a coal-fired power plant in Malaysia

Yoon C. Lam i), Dennis Ganendra ii) and Krishna Prasad iii)

i) Technical Director, Minconsult Sdn Bhd, Lot 6, Jalan 51A/223, 46100 Petaling Jaya, Selangor, Malaysia.
ii) Director, Minconsult Sdn Bhd, Lot 6, Jalan 51A/223, 46100 Petaling Jaya, Selangor, Malaysia.
iii) Manager, Minconsult Sdn Bhd, Lot 6, Jalan 51A/223, 46100 Petaling Jaya, Selangor, Malaysia.

ABSTRACT

The project site is underlain by superficial deposits of recent to sub-recent age alluvium with the upper soil stratum comprising very soft to soft marine clays (Halocene clay) of up to about 25m thick. The reclaimed area on swampy coastal land was pre-loaded with hydraulic sand fill and aided with pre-fabricated vertical drains (PVD) to accelerate the consolidation settlement. This paper presents the design considerations including the important factors for realistic prediction of consolidation settlement and fill embankment slope stability. Emphasis is placed on the monitoring data and their comparison with the design predictions.

Keywords: reclamation, improvement, consolidation, settlement, back-analysis, instrumentation

1 INTRODUCTION

Three coal-fired generating units with an aggregate net capacity of 2100 MW were proposed on a fast-track schedule. The project site is located on low-lying swampy coastal land near the southern tip of west coast Peninsular Malaysia as shown in Figure 1 & 2.

The project site is underlain by superficial deposits of recent to sub-recent age alluvium. The upper soil stratum comprises very soft to soft marine clays (Halocene clay) of about 25m thick. A comprehensive programme of ground investigation and laboratory tests were carried out to obtain soil parameters for detailed engineering design of the proposed reclamation works. The adopted design is based on well-established empirical relationships for soft ground engineering and advanced finite element modeling. The site was extensively instrumented and monitored using settlement plates, surface markers, inclinometers, piezometers and vane shear tests for stability analysis.

The Observation Method was implemented for the reclamation construction. Monitoring data were continuously reviewed and compared with the initial theoretical predictions to confirm the validity of the adopted parameters and assumptions used in the design calculations and advanced numerical analyses.

This paper presents the design considerations including the important factors for realistic prediction of consolidation settlement and fill slope stability. Emphasis is placed on the instrument monitoring data and their comparisons with the design predictions. This paper will serve as an informative case history for land reclamation construction over soft marine clay.

Fig. 1. Key Plan of Project Site

Fig. 2. Location of Project Site beside a major shipping channel.
1.1 Technical Requirements
The scope of works under the contract requires EPC contractor to carry out detailed engineering design and implementation of soil improvement works to attain the specified key performance objectives for settlement and bearing capacity for respective areas before the final Acceptance dates. Tight construction schedule and key geotechnical performance objectives were imposed. This include the achievement of target settlements equivalent to at least 90% of primary consolidation settlement at a load attributable to the finished platform level plus 14m high coal stacks to be stocked piled at the proposed storage yard. The contract required that the specified performance criteria including differential settlement and bearing capacity criteria should be achieved within the agreed final acceptance dates.

2 CHARACTERIZATION OF PROJECT SITE

2.1 The Site
The reclamation site is located at Tg. Bin, Johor. It is adjacent to a major navigation channel for Port of Tanjung Pelepas (PTP) and overlooking Singapore to the south. The coastal area is characterized by mangrove swamp and mud flats. The site is at an average ground level of +0.75m above MLSD. The total area of reclamation is approximately 70 hectares and for functional and contractual purposes, it was sub-divided into four areas, namely the Power island, coal storage yard, switch-yard and construction lay-down areas. An indicative footprint of the land reclamation works is shown in Figure 3.

![Footprint of reclamation](image_url)

2.2 Geology
The site is underlain by superficial deposits of recent to sub-recent age alluvium (Geological Survey of Malaysia, 1973). This Holocene deposit is predominantly very soft to soft marine clays (with occasional inter-beds of sand lenses) approximately 22m to 25m thick at the coal yard. SPT N-value within the marine clay is generally 0 blows/30cm at the top upper stratum of up to 20m. In-situ penetration vane shear tests carried out within the soft marine clay layer at the coal storage yard indicates undrained in-situ and remoulded shear strength, varying increasingly with depth from 4 to 53 kPa and 2 to 20 kPa, respectively. The plot of undrained shear strength versus depth is presented in Fig. 4. The proposed trend-line for undrained shear strength versus depth adopted for design is also indicated in the said graph (\(C_u/\sigma'_u=0.3\)). The sensitivity of the clay ranges from 2 to 7, with an average value of approximately 3.

The depth to bedrock varies from 23m to greater than 38m below the prevailing ground levels. Overlying the bedrocks are weathered soil zones and overburden layers. The zone of weathered soil material is expected to vary from 12m to 28m below the ground level. Generally, the thickness of medium stiff to very stiff soil material varies between 2m to 10m and is further underlain by hard to very hard soil stratum. It can be noted that the superficial deposits of marine clay with occasional inter-bedded sand lenses extends throughout the project site, up to a depth of about 28m below the existing ground level.

Considering the low-lying coastal site, the prevailing ground water table is generally high; varying from above ground surface to approximately 1m to 3m beneath the existing ground surface.

2.3 Subsoil Conditions
The subsoil condition throughout the site is generally quite consistent, with two distinct subsoil strata. The upper soil stratum is the alluvial deposits consisting predominantly very soft to soft marine clays (with occasional inter-beds of sand lenses) approximately 22m to 25m thick at the coal yard. SPT N-value within the marine clay is generally 0 blows/30cm at the top upper stratum of up to 20m. In-situ penetration vane shear tests carried out within the soft marine clay layer at the coal storage yard indicates undrained in-situ and remoulded shear strength, varying increasingly with depth from 4 to 53 kPa and 2 to 20 kPa, respectively. The plot of undrained shear strength versus depth is presented in Fig. 4. The proposed trend-line for undrained shear strength versus depth adopted for design is also indicated in the said graph (\(C_u/\sigma'_u=0.3\)). The sensitivity of the clay ranges from 2 to 7, with an average value of approximately 3.

The subsoil condition throughout the site is generally quite consistent, with two distinct subsoil strata. The upper soil stratum is the alluvial deposits consisting predominantly very soft to soft marine clays (with occasional inter-beds of sand lenses) approximately 22m to 25m thick at the coal yard. SPT N-value within the marine clay is generally 0 blows/30cm at the top upper stratum of up to 20m. In-situ penetration vane shear tests carried out within the soft marine clay layer at the coal storage yard indicates undrained in-situ and remoulded shear strength, varying increasingly with depth from 4 to 53 kPa and 2 to 20 kPa, respectively. The plot of undrained shear strength versus depth is presented in Fig. 4. The proposed trend-line for undrained shear strength versus depth adopted for design is also indicated in the said graph (\(C_u/\sigma'_u=0.3\)). The sensitivity of the clay ranges from 2 to 7, with an average value of approximately 3.
The marine clay is classified as having high to extremely high plasticity (CV-CE) as per the British Soil Classification System (BS5930). A plot of Atterberg limits on the plasticity chart is given in Fig 5. Although the general distinction between Clay and Silt is often taken to be the A-line on the plasticity chart, with Clays plotting above and Silts below; however, the reliability of the A-line in this regard is poor especially for borderline cases. The variations of subsoil engineering parameters (e.g. moisture content, Atterberg limits, compression index, coefficient of consolidation, pre-consolidation pressure, etc.) with depth are shown in Fig 6 and 7. Over-consolidation ratios derived from 1-D oedometer tests are shown in Fig. 8.

Immediately underlying the marine clay is the sedimentary residual soil of the Bukit Resam/Gunung Pulai formation. The thickness of residual soils ranges from 2m to 10m and generally comprise of firm to stiff clayey silt/silty clay with SPT-N values ranging from 5 to greater than 50 blows/30cm. At deeper levels, highly to moderately weathered form of the parent rocks (SPT-N >100 blows/30 cm) namely, siltstones, sandstones and volcanic rocks are encountered.
The available CPTU data confirm the subsoil stratification as generalised from the borehole results. In particular, the soft marine clay strata can be easily identified from the deep sounding profiles based on the very low cone resistance, q, and values of friction ratio. at the coal storage yard is plotted in Fig. 9. It can be deduced that the thickness of the highly compressible marine clay varies from 22m to 27m. This compressible subsoil layer is expected to consolidate significantly under the fill embankment and surcharge load. The geotechnical aspects are addressed in the detailed design for proposed soil improvement works at the Coal Storage yard.

![Fig. 9. Typical Subsoil Profile from CPTU](image)

3 PERFORMANCE

The most stringent requirements are imposed on the works for the proposed 20 hectares coal storage yard. This includes the need to achieve a target settlement equivalent to at least 90% of primary consolidation settlement at a load attributable to the finished platform level (RL +4.0m MLSD) plus the 14m high coal stacks. Considering a specific gravity of coal at 0.95 kN/m³, the minimum equivalent sand fill thickness at full surcharge load required for the works is 15.4 m. In addition, it is specified that the differential settlements along the center-line of the 500 m long corridor of stacker reclaimers shall not exceed 125 mm over 50 m distance at 12 months after hand-over the agreed date. For economic consideration, there is also a challenge to limit the total volume of hydraulic sandfill to be imported. This involved sub-dividing the coal-yard into 3 zones to allow the roll-over of surcharge fill while having to meet the tight construction schedule.

The specified method for confirming compliance is the Asoaka construction (Asoaka, 1978) using data from 20 settlement plates installed at approximately a hundred metres in grid spacing.

3.1 Estimation of Total Settlement

The estimated total primary settlement of 5.25 m is computed for a surcharge load equivalent to 15.4 m thick sand fill, overlying the highly compressible clay stratum of about 24.5 m. The adopted parameters are shown in Table 1.

![Table 1. Parameters for estimation of Primary Settlement](image)

For over-consolidated soils, the primary settlement is computed as follows:

\[
\sigma'_{vo} + \Delta \sigma < \sigma'_c:
S_{ult} = \left(\frac{\sigma'_c}{1+e_0}\right) \times H \times \log \left(\frac{\sigma'_v + \Delta \sigma}{\sigma'_c}\right)
\]  

(1)

\[
\sigma'_{vo} < \sigma'_c < \sigma'_{vo} + \Delta \sigma:
S_{ult} = \left(\frac{\sigma'_c}{1+e_0}\right) \times H \times \log \left(\frac{\sigma'_v}{\sigma'_vo}\right) + \left(\frac{\sigma'_v}{1+e_0}\right) \times H \times \log \left(\frac{\sigma'_v + \Delta \sigma}{\sigma'_v}\right)
\]  

(2)

where:

- \(S_{ult}\) = ultimate primary settlement
- \(\sigma'_{vo}\) = initial vertical stress
- \(\sigma'_c\) = pre-consolidation pressure
- \(H\) = thickness of compressible layer
- \(\Delta \sigma\) = increase in vertical stress on the clay layer.

4 SEQUENCE OF CONSTRUCTION

The stability of the fill embankment is of paramount concern in view of the prevailing very soft ground. The sequence of staged construction was determined based on the need to ensure a minimum factor safety of 1.10 against slope instability and the short time period available for construction, commencing from the placement of first hydraulic sand fill layer to the removal of excess surcharge to designed finished platform level. PVDs were installed at triangular grid spacing of 1.0 m c/c and to an average termination depth of about 27m, from the top of the first lift.

The typical sequence of staged construction is presented in Table 2. The rest period in between each lift is necessary to allow a sufficient gain in shear strength of the marine clay before the placement of the next lift. The fill slope geometry, sequence of staged construction and subsoil parameters are numerically modeled using the finite element software, PLAXIS, Professional Version 8.2.
Table 2. Sequence of Staged Construction

<table>
<thead>
<tr>
<th>Lift No.</th>
<th>Cummulative Thickness (m)</th>
<th>Construction Sequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.0</td>
<td>Placement of hydraulic sand fill Installation of PVDs Rest Period</td>
</tr>
<tr>
<td>2</td>
<td>5.5</td>
<td>Placement of hydraulic sand fill Rest Period</td>
</tr>
<tr>
<td>3</td>
<td>8.0</td>
<td>Placement of hydraulic sand fill Rest Period</td>
</tr>
<tr>
<td>4</td>
<td>10.5</td>
<td>Placement of dry sand fill Rest Period</td>
</tr>
<tr>
<td>5</td>
<td>13.0</td>
<td>Placement of dry sand fill Rest Period</td>
</tr>
<tr>
<td>6</td>
<td>15.4</td>
<td>Placement of dry sand fill Rest Period</td>
</tr>
<tr>
<td>#</td>
<td>RL+4.0</td>
<td>Removal of excess surcharge fill</td>
</tr>
</tbody>
</table>

Note: # Removal of surcharge load to the finished platform level shall depend on settlement monitoring results and degree of consolidation achieved.

5 NUMERICAL MODELLING

The adopted mesh for numerical analyses using Plaxis is shown in Fig 10. The analyses were carried out using a plane strain model with 6-node elements for a soft clay thickness of 24.5m. The Plaxis function, “updated mesh and pore water pressure”, is switched on to take into account the large deformation of the mesh. The “drain” function in Plaxis is utilised to model the PVD. On the vertical face of PVD, the excess pore water pressure (epwp) is automatically set to zero. The left and right boundaries of the mesh were specified as “closed consolidation boundary”. A roller displacement boundary condition was specified at the left and right boundaries of the mesh. At the bottom of mesh, displacement boundary condition is fixed in both horizontal and vertical directions. Ground water level is specified at the top of marine clay layer.

The input parameters adopted for the sand fill, soft marine clay and sandy silt stratum are summarised in Table 3. The predicted total primary settlement is 5.17m and the results are presented in Fig 11. It is in good agreement with the estimated 5.24m as obtained hand calculations with equations (1) & (2). The estimated settlement upon removal of surcharge at Day 281 is 4.70 m, which is equivalent to a degree of consolidation of about 91%, a required in the Contract.

![Mesh of numerical modelling](image1.png)

![Predicted Settlement at top of Clay Layer](image2.png)

6 STABILITY OF FILL EMBANKMENT

Prior to the PLAXIS analysis, the stability of proposed embankment at each stage of filling were analysed using SLOPE/W. The initial undrained shear strength, $c_u$ of the soft clay (before placement of sand fill) is taken as per Fig. 4. Some gain in shear strength of the soft marine clay is expected due to the consolidation process as the thickness of sand fill increases. A conservative estimation for the gain in shear strength is made at each relevant section of subsoil underneath the fill embankment in relation to the installed PVD and geometry of the slopes as determined using the following equation:

$$\Delta c_u = c_u / \sigma' \times U \Delta \sigma$$  (3)

where
- $c_u / \sigma' = 0.3$ (see Fig.4)
- $U$ = predicted degree of consolidation at each Lift
- $\Delta \sigma$ = surcharge load (kPa), taking into account the buoyant unit weight of fill that settles below the groundwater table.

The rest period in between each sand fill lift is adjusted to ensure a minimum safety factor of 1.10 is obtained at all times. Morgenstern and Price limit equilibrium method is used in the stability analyses computation.
Prior to the placement of each subsequent lift of sand fill, a review of geotechnical instrumentation results and in-situ field tests were carried out to verify the gain in shear strength of the underlying soft clay. This is to ensure that the fill slopes are stable at all times, especially immediately after placement of a new lift.

7 SETTLEMENT-TIME BEHAVIOUR

For monitoring purposes, the site is divided into approximately 100m x 100m square panels with a settlement plate installed at its center. The settlement-time behaviour of the ground were closely monitored throughout the construction period and a typical settlement-time plot is presented in Fig. 12. As per contract requirements, Asaoka’s method is used to predict ultimate settlement and hence, the degree of consolidation achieved prior to the removal of excess surcharge fill.

Back-analysis for the coefficient of horizontal consolidation (c_h) were carried out from the slope of best-fit line of Asaoka’s plot (β1) using Barron’s (1948) theory of radial consolidation expressed by:

\[ \beta_1 = \exp \left( \frac{-8c_w \Delta t}{d_e^2 F(n)} \right) \]  

where:
- \( \Delta t \) = time increment adopted in Asaoka’s plot
- \( d_e \) = equivalent soil cylinder diameter
- \( d_e = 1.05 \times \text{PVD spacing installed on triangular grid,} \)
- \( F(n) \approx \ln(n) - 0.75 \), where \( n = d_e/d_w \)
- \( d_w \) = equivalent PVD diameter = \( 2(a+b)/\pi \)
- \( a \) and \( b \) are width (100 mm) and thickness (4 mm) of PVD drain, respectively.

It must be noted that the \( F(n) \) function in equation (4) does not specifically account for smear effect caused by the PVD installation and the back-analysed \( c_h \) is therefore a lumped value which also reflects the smear effect embodied in the slope (\( \beta_1 \)) of the best-fit Asaoka line. The back-analysed \( c_h \) values range from 1.38 to 2.55 compared to the adopted design value of 1.0.

Back-calculations were also carried out to estimate the compression ratio of the underlying soft clay using equation (2) and assuming that \( c_r = 5c_h \). It was found that the back-analysed average compression ratio (\( c_r/1+c_r \)) ranges from 0.26 to 0.32, which are generally within the adopted values in the design as shown in Fig. 7.

8 CONCLUSIONS

Significant advances have been made in recent decades on the understanding of soft clay behaviour although it remained a challenge having to rapidly accelerate the construction works for proposed Power Plants under such ground conditions. In the quest to rapidly surcharge the fill embankment, it is very important to ensure its stability at all times with the appropriate prediction on the gain of shear strength for the soft clay under surcharge loads.

ACKNOWLEDGEMENTS

The data for this work has been collected by IMW Dredging Sdn Bhd who is the appointed EPC contractor. It would not have been possible to prepare this paper without the valuable source data. The authors wish to thank Malakoff Berhad and IMWD for permission to publish this paper and bring this important project to a wider audience. The support of Rentak Jitu Project Management Sdn Bhd and Zelan Construction is acknowledged.

REFERENCES