Plaxis Model Parameters for Mangrove Mud, Finucane Island, Western Australia

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ABSTRACT

The development of BHP Billiton's Western Iron Ore Stockyard over soft mangrove clays at Finucane Island, near Port Hedland, Western Australia, in 2003-2004, included ground improvement works and a monitored staged loading approach, during commissioning of the 19m high iron ore stockpiles. The iron ore stockpiles are placed adjacent to settlement sensitive stacker and reclaimer machines. The ground improvement works included wick drains at 1.2m spacing in a triangular pattern and 3 layers of high strength woven geotextiles. The performance of the ground improvement works was monitored through measurements of settlements, lateral movements and pore pressures. Monitoring data collected during the staged loading of the Western Stockyard was recently used to re-calibrate appropriate soil parameters for modelling of the mangrove mud in the Plaxis® finite element geotechnical modelling program. Back-analysed parameters were then used with a greater level of confidence for design of ground improvement works for additional iron ore stockyards, over the same geological strata, on Finucane Island.

1 INTRODUCTION

The development of the BHP Billiton's Western Stockyard at Finucane Island, Western Australia, in 2003-2004 included a monitored, staged loading approach for commissioning of the iron ore stockyard over soft mangrove mud. Monitoring data, including settlements, lateral movements and soil pore pressures, was used to assess the degree of consolidation and strength increase in order to confirm appropriate times when it was safe to increase the height of the stockpiles. The monitoring data collected during the staged loading was recently used to re-calibrate the soil parameters used for the original design of the Western Stockyard, and thus provide a greater level of confidence for design of future works over the same geological strata on Finucane Island.

2 SITE CONDITIONS

2.1 Site Geology

During the last ice age, when sea level was considerably lower than its present level, Finucane Island was considerably more rugged, with valleys and limestone ridges. Sand was present in the valleys. The limestone and sand were underlain by dense clayey sands, locally referred to as the Red Beds. When the sea level rose during recent times, mangroves grew in the low lying valleys and contributed to the deposition of soft blue grey, organic clays of varying thickness. This highly plastic estuarine clay deposit is known as Mangrove mud. Since then, moving sand dunes and also hydraulically placed dredged sandy fill have covered over the mangrove mud with some 4.5m or more of sands and made the site relatively flat. A schematic of the geology at the Western Stockyard is shown on Figure 1.

2.2 Stockyard Design

The original analysis and design of the Western Stockyards was discussed by Fuge (2004). The iron ore stockpiles at the Western Stockyard generally have a base width of just over 60m with side slopes of 1:1.5 (V:H). The stockpiles were designed to have a total vertical height of 19m, as shown on Figure 1 and exert about 500kPa bearing pressure on the ground. Due to the presence of nominally 5.3m of soft Mangrove mud at a shallow depth, the stockpiles were designed to have a staged commissioning, with intermediate heights of 10m and 15m, to allow consolidation and strength increase of the mangrove mud to occur before the stockpiles were filled to their design height of 19m. If the stockpiles were initially loaded to 19m without prior consolidation, by staged loading, then it was predicted that the mangrove mud would undergo a bearing capacity failure.

To complete the staged loading within 12 months, wick drains were installed beneath the stockpiles at a spacing of 1.2m, in a triangular grid, to increase the rate of pore pressure dissipation and subsequent strength gain. Three layers of high strength woven geotextile were also installed so that the number of loading stages could be kept to a minimum, whilst maintaining adequate safety factor against shear failure and limiting lateral deflections of the stacker and reclaimer rails that are located on either side of the stockpiles.

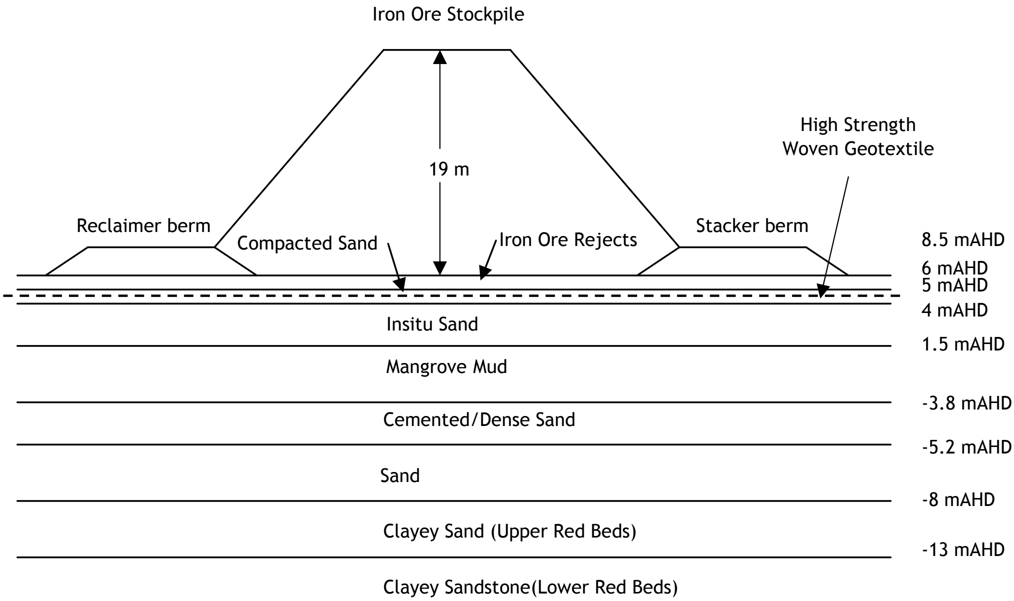

Figure 1 Schematic cross section through stockpile

2.3 Stockpile Loading Profile

Stockpile 1 (stockyard area K9) had a length of 100m and extended from Chainage 920m to 1020m. Commissioning of Stockpile 1 involved many unloads/reloads and partial loadings in the vicinity of Chainage 1000m. Figure 2 shows the recorded loading history in terms of ore tonnage. This loading profile was simplified for the back analysis, and the loading profile adopted for the analysis in terms of stockpile height is also shown in Figure 2. The iron ore density was estimated to be about 2.0t/m^3. Pore pressure monitoring data was used to confirm the loading stages where the stockpile height was increased from 10m to 15m and then finally to 19m.

2.4 Monitoring Data

As part of the stockpile monitoring system, a total of 4 vertical inclinometers, 8 vibrating wire push in Piezometers and 12 vibrating wire settlement cells were installed. Conventional survey data was also collected from 13 ground monitoring points. However, many of the monitoring devices did not perform as would have been expected, possibly due to the extremely high daily temperatures experienced at Finucane Island. Review of the data indicated that Piezometer P7 and Settlement Cells S2 and S11 appear to have provided the most reliable instrument data in addition to the conventional survey data. Piezometer P7 and Settlement Cell S2 were installed at Chainage 1000m, below Stockpile No.1. Note that the vibrating wire piezometers were installed to measure pore pressures in the mangrove mud layer.

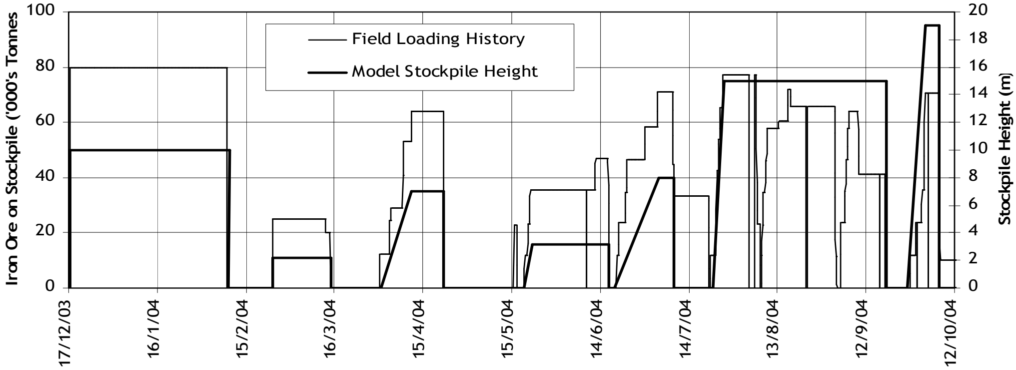

Figure 2 Loading Profile

3 BACK ANALYSIS

3.1 Geotechnical Model

Back analysis was performed using the PLAXIS® Finite Element package. The PLAXIS finite element mesh comprised 15 node plane strain triangular elements, which were used to represent the Stockyard and the foundation soils. The back analysis concentrated on the parameters of the 5.3m thick, soft mangrove mud layer, since this layer would have contributed the majority of the consolidation settlements.

PLAXIS includes various constitutive soil models. In this case the Mohr-Coulomb, Hardening Soil and Soft Soil Creep models were trialled for the Mangrove mud layer. The rest of the strata layers were modelled using Hardening Soil model. The three PLAXIS models trialled adopt effective stress parameters for the mangrove mud (i.e. a friction angle and little or no cohesion) and model undrained loading by restricting the rate of pore pressure dissipation according to the permeability and drainage path of the soil.

3.2 Soil Parameters for Mangrove Mud

Laboratory tests on mangrove mud indicated Atterberg Limits comprising Liquid Limit 28% to 74%, Plasticity Index 13% to 31% and natural moisture content 26% to 43%. The undrained shear strength was estimated to be 30kPa to 45kPa, based on Electric Friction Cone Penetrometer Test results.

The soil parameters adopted for the mangrove mud layer, in the back analysis, to obtain good comparison to the observed responses, are provided in Table 1. Soil parameters for the other layers were generally based on the original design parameters and not modified in the back analysis.

Table 1 - Soil Parameters for Mangrove Mud Used in the Back Analysis

Cohesion (c', kPa)	Friction Angle (ϕ', degrees)	Young's Modulus (E', MPa)	Poisson's Ratio ν'	Unit Weight (γ, kN/m^3)
0	20	4	0.35	17.0

The coefficient of volume compressibility (m_v) measured from oedometer tests on samples vary between 0.12 and 0.25 m^2/MN, over a stress range of 100 to 200kPa. The Young's modulus (E') may be estimated from the m_v value, using the following relationship:

$$E' = \frac{(1+\nu')(1-2\nu')}{(1-\nu')m_v} \tag{1}$$

The estimated E' from oedometer tests vary between 2.5MPa to 5.2MPa and straddles the back analysed E' value of 4MPa.

The soil parameters used in the Soft Soil Creep model are provided in Table 2. Consolidation tests carried out on samples obtained from the Western Stockyard were used as the basis for selecting the soft soil creep model parameters. The compression index (C_c) and swell index (C_s) from the laboratory tests (C_c = 0.6 and C_s = 0.06), were then modified to obtain good comparison with observed values. The creep parameter, C_α, was assumed based on the recommendations of Baudin et. al.(1999). It should be noted that the creep parameter, C_α used in Plaxis is related to the strain based creep parameter, $C_{\alpha\varepsilon}$ which is typically reported on consolidation tests according to the following equation:

$$C_\alpha = C_{\alpha\varepsilon}\,(1 + e) \tag{2}$$

Where e is the void ratio.

Table 2 - Soft Soil Creep Parameters Used in the Back Analysis

Compression Index, C_c	Swell Index, C_s	Creep Parameter, C_α	Initial Void Ratio, e_0	Friction Angle, ϕ' (degrees)
0.48	0.045	0.045	1.574	20

4 BACK ANALYSIS RESULTS

4.1 Settlements

The back-estimated settlements using different soil models are provided in Figure 3 along with the measured settlement from Settlement Cell S11. The Hardening Soil model and the Soft Soil Creep model have shown good agreement with the measured data. Although the Mohr-Coulomb model predicted the initial settlement better, it over-predicted both the settlement and the swelling on each subsequent loading and unloading cycle. The over prediction of swelling was more severe and so the model under predicted the total settlements.

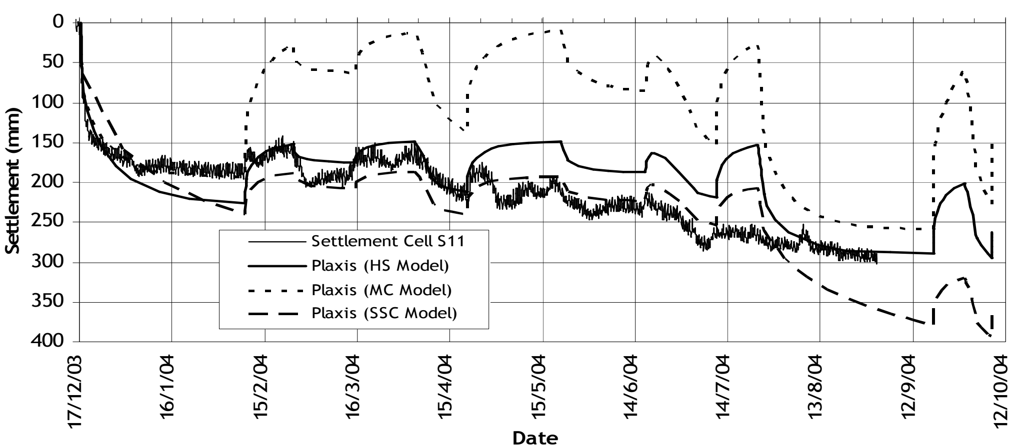

Figure 3 Vertical settlements beneath stockpile

4.2 Excess Pore Pressure

The predicted excess pore pressures in the Mangrove mud, using different soil models, are provided in Figure 4, along with the measured pore pressure data from Piezometer P7. All the soil models have shown reasonably good agreement with the measured data. The rate of consolidation predicted in PLAXIS is sensitive to the soil permeability. Note that Stockpile 1 is underlain by wick drains in a 1.2m triangular grid. Since it is very difficult to model the individual wick drains in PLAXIS, an equivalent coefficient of permeability was used to represent the effect of wick drains. A coefficient of permeability k of $5x10^{-4}$m/day (vertical and horizontal) for the combined wick drains and soil modelled the dissipation of excess pore pressure reasonably well.

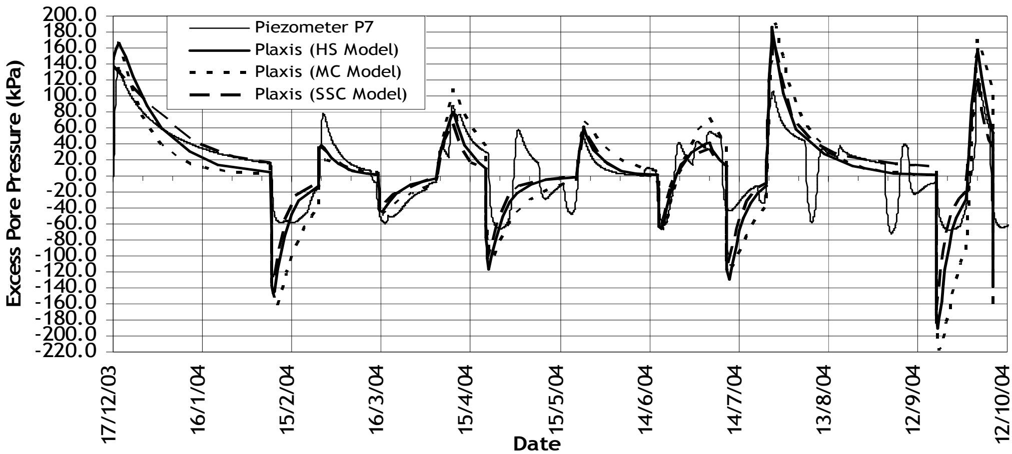

Figure 4 Excess pore pressures beneath stockpile

The field permeability of the mud prior to wick drain installation was estimated to be of the order of $4.3x10^{-6}$m/day based on laboratory oedometer tests, and $5x10^{-6}$m/day based on piezocone dissipation tests. The back analysis therefore indicates that the installation of wick drains appears to have increased the equivalent permeability of the combined soil and wick drains by about two orders of magnitude. The effectiveness of wick drains can be assessed by computing a coefficient of consolidation from the measured pore pressure dissipation data, for comparison with the laboratory measured values or the values used in the design. The average degree of consolidation for radial drainage to wick drains can be expressed as:

$$U_r = 1 - \exp\frac{-8T_r}{\alpha} \qquad \text{(Hausmann, 1990)} \qquad (3)$$

Where,

$$T_r = {c_h t}/{D^2} \text{ = time factor for radial consolidation;} \qquad (4)$$

c_h = coefficient of horizontal or radial consolidation;
t = time elapsed since application of the load;
D = equivalent diameter of cylinder of soil around drain = 1.06s for triangular pattern;
s = spacing of drains;

$$\alpha = \frac{n^2 \ln n}{n^2 - 1} - \frac{3n^2 - 1}{4n^2} \qquad (5)$$

n = D/d; and
d = drain diameter (or equivalent diameter for strip drains).

The measured pore pressure data indicates about 90% dissipation was achieved in 54 days which corresponds to a c_h value of approximately 6.4m^2/year. This compares very well with the c_h value of about 5m^2/year obtained from piezocone dissipation tests conducted at the Western Stockyard.

Using the method suggested by Baligh and Levadoux (1980) and based on the back calculated c_h value of 6.4m^2/year, and the average overburden pressure due to the stockpiled ore, the k_h value for the mangrove mud was then back calculated to be nominally $4x10^{-6}$m/day. This estimated k_h value compares well with the previously mentioned k values estimated from the laboratory oedometer tests ($4.3x10^{-6}$ m/day) and field piezocone dissipation tests ($5x10^{-6}$ m/day). Although it has not been possible to distinguish between vertical and horizontal permeability in the Plaxis back analysis, some difference between vertical and horizontal permeability in the field is likely.

4.3 Horizontal Movements

The back-estimated horizontal movement using different soil models are provided in Figure 5 at the Reclaimer (Inclinometer 1) position. The initial design prediction of lateral movement at the Reclaimer was 150mm and the measured value during the commissioning period at the Inclinometer 1 location was 40mm.

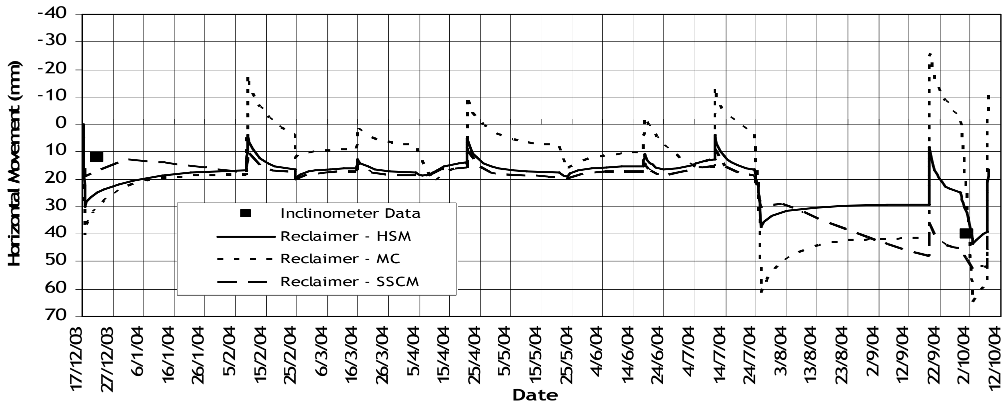

Figure 5 Horizontal movements at ground surface near reclaimer

5 CONCLUSIONS

The following conclusions may be drawn from the back analysis of the data:

- The PLAXIS Hardening Soil model and Soft Soil Creep models are more appropriate for modelling the Mangrove mud layer than the traditional Mohr Coulomb model which tends to over predict rebound behaviour on each unloading cycle.
- The wick drains were effective at increasing the rate of consolidation and strength increase. Plaxis modelling indicates the effective vertical permeability of the combined layer of mangrove mud and wick drains was nominally 2 orders of magnitude higher than the permeability of the unimproved mangrove mud.
- Soil Stiffness (E), Coefficient of consolidation and permeability values back analysed from the monitoring results compare well with the values estimated from the piezocone dissipation tests and the laboratory consolidation tests.
- Consolidation parameters C_C and C_S for the mangrove mud were back analysed using the Plaxis Soft Soil Creep model to be 20% to 25% lower than the values measured in laboratory consolidation tests. The laboratory consolidation test values would have given a conservative prediction of settlement in this case.

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