

WORKING PLATFORMS TO BRE OR NOT TO BRE IS THE QUESTION

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ABSTRACT

The Building Research Establishment (BRE) produced a practice guide for working platforms for tracked plant, which has become a “standard” in the industry in the absence of any other widely published simple design method. The BRE design method does not apply for thick platforms or for soft subgrades, but continues to be used in those applications in the absence of an alternative document. A case study is discussed which applies the BRE in such a situation, but then compares with alternative methods to assess the required working platform. Additionally a stochastic approach is used with the BRE method, given its sensitivity to the material strength parameters input, to provide a risk understanding rather than a factor of safety approach which does not define the risk explicitly. The derivation of the parameter inputs is discussed to show how assumed values for preliminary design and measured values produce different design platform thickness. Given the consequences of a failure, a construction control proof roll testing was used with deflection criteria. The derivation of this criterion is presented.

1 INTRODUCTION

Construction plant for piling and ground improvements often need a working platform to ensure the stability of the plant. The eccentricity of the loading when these rigs are in operation creates high applied pressures on the ground surface. The high centre of gravity for such equipment presents a hazard for overturning if the ground does not provide sufficient support and needs a working surface that will not settle excessively. The working platform is then required to provide that support. The thickness of the platform is dependent on (a) the rig type, (b) the underlying soil type and strength and (c) the working platform material.

BRE 470 (2004) provides design procedures on working platforms for tracked plants. The approach is based on a punching failure mechanism. In the BRE method no allowance is made for stress distribution with depth. Typically the method provides a working platform much thicker than commonly used successfully in Australian practice. However, the BRE approach is the nearest to a “standard” procedure. One cannot therefore disregard this approach, although accepting it is likely to be conservative. There are significant consequences from under design of a working platform.

Corke and Gannon (2010) showed that the design process set out in BRE 470 is sensitive to the angle of friction of the granular platform material and the shear strength of the sub-grade. Typically, an increase in the angle of friction of 5° for the platform material reduces the calculated required thickness of the platform by about 20%. If the strength of the sub-grade is doubled, the calculated platform thickness is approximately halved. Yet standard practice would not typically involve measuring the friction angle.

This case study is based at the Port of Brisbane (POB) where the installation of wick drains at a reclamation site required various wicking rigs to be operated. The site has a dredged sand over a very soft clay. Two limitations stated in BRE 470 (2004) and relevant to this site are:

- The punching shear failure mechanism is not applicable where the subgrade has virtually no strength. For a cohesive subgrade, the routine design calculations are not appropriate for very soft clay with $c_u < 20$ kPa. This site was 5 kPa subgrade strength and
- With large platform thicknesses, punching shear is unlikely to be the critical failure mechanism and the failure may be largely within the platform material. The routine design calculation method based on punching shear is not appropriate where $(D/W) > 1.5$. W is the width of the loaded area and D is the platform thickness. $D/W = 2.8$ to 1.9 for the preliminary estimate of thickness and various equipment to be used at this site.

Thus the BRE is not applicable, but was applied in the preliminary design in the absence of an alternative procedure.

Three different theoretical approaches were used given the deviation from this “standard”. For the theory to be applicable, there must also be confidence in both the material parameters and thickness specified used in the analysis. These were verified by proof rolling. But proof rolling by itself without defining acceptable criteria is also not sufficient and that criteria had to be developed analytically as well from results of limited *in situ* testing of the existing sand fill.

The working platform had already been constructed. Mud waves formed during construction in pushing the fill on the very soft surface resulted in a non - uniform thickness of the working platform. The proof rolling therefore also served a key function of defining the thickness over a large area.

2 APPROACH

2.1 GENERAL

The natural ground was Holocene clay with a preliminary design undrained strength of 5 kPa adopted within the top 6 metres, and with an increase in strength with depth. Using the BRE approach and a factor of safety of 1.6 for the standing / travelling load condition, a platform design thickness of 2.5 m is appropriate for the heaviest equipment load. The constructed platform consisted of dredged spoil and, due to the method of placement with mud waves, there was some uncertainty as to the platform thickness over this large site (12.7 ha).

Dredged mud was initially placed on the soft clay over several years. The isolated paddock area was left to allow the water from the dredging to decant. The reclaimed paddock was then filled with dredged sand pushed by truck and bulldozer on top of the dredged mud. During the sand placement, localised bearing capacity failure occurred in the dredged mud layer. This 'mud wave' was approximately 1m to 2 m in height, but part of the mud wave was removed and replaced with sand using an excavator, however a large portion of the mud wave was still present in the paddock. The thickness of the sand cap was highly variable across the whole paddock.

The next stage of construction involved wick installations with wicking equipment operating at this area over 9 months. The variables were the water table depth (assumed to be 0.5 m below the surface), the thickness of the placed fill (2.5 m on average but varies locally), and the geotechnical properties of the working platform. The sensitivity of the various parameters needed to be examined to assess the associated risk.

Additionally as the BRE 470 is not directly applicable to this site (based on subgrade strength and D/W ratios), other methods were required to be used to provide confidence that a suitable platform thickness had been adopted. Three analytical methods were used to assess the required platform thickness:

- The BRE 470 approach for a working platform, but with a probability analysis to understand the risk rather than a factor of safety approach,
- A method based on acceptable movements using Finite Element modelling in Plaxis 2D,
- A method based on acceptable stress using Finite Element modelling in Plaxis 2D.

For this paper, the emphasis is on alternative approaches to the BRE approach, as that calculation method states that it is not applicable. However, in this case study, various other project issues are also discussed. Figure 1 provides an overview of the multiple issues considered during this project.

Thus there were shortcomings in both the BRE methodology and input data. Yet equipment had already been mobilised to the site on a platform of uncertain thickness and underlying properties.

2.2 TRACK PRESSURE LOADS

The Federation of Piling Specialists (2005) sets out the basic procedure for calculating the track bearing pressures for a crane or piling rig for use in the working platform design process set out in BRE470. The track bearing pressures calculated by the appropriate method for use in the BRE design method are commonly much higher than given by a simple calculation of the total rig weight divided by the total track area. Using the weights of the various rig components and the eccentricity of the component from the centre of rotation, the overturning moment can be calculated. This needs to be done for the range of operations that will be carried out, e.g. standing, travelling, handling, penetrating, extracting, with all possible jib or mast orientations considered. Table 1 provides the specification of the three wicking rigs to be used at this site.

The preliminary analysis was based on the heaviest rig (81.1 t). Subsequent rigs were later introduced, and although of a lighter load, this equipment provided higher pressures due to the reduced track width. Thus one must be vigilant to not assume the heaviest equipment will exert the design pressure.

Conversely one must also not assume that the highest design pressure is from the critical equipment. The effect of the higher track pressure may be offset by less track width zone of influence.

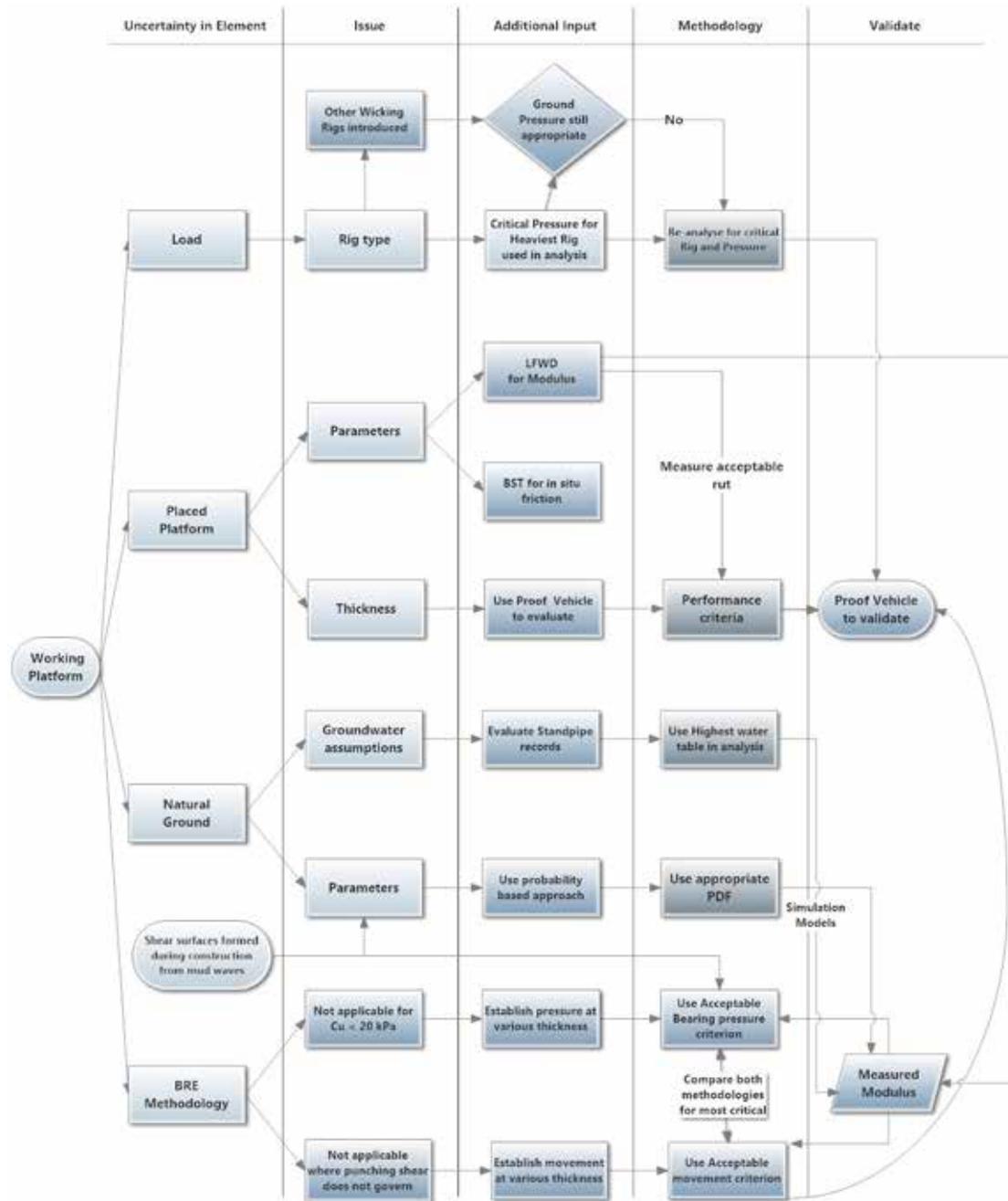


Figure 1: Summary of issues considered for this case study.

Table 1: Equipment for installation of wick drains.

Equipment Type	Weight (t)	Track width (m)	Length (m)	Critical Track Pressure (kPa)
Komatsu excavator	59.1	0.9	4.25	130
CAT excavator	45.4	0.9	4.5	167
Komatsu excavator	81.1	1.31	4.8	97

2.3 GEOTECHNICAL PARAMETERS

2.3.1 *In situ* testing

The initial estimate of required platform thickness was based on assumed “static” soil parameters and assumed ground water conditions: GWL 0.5 m below ground level, friction angle (33 °), Elastic Modulus of platform (25 MPa), and clay subgrade strength of 5 kPa. The working platform parameters were assumed from the soil description.

Given that punching shear was not likely to be the governing mechanism and no tests had been a carried out on that sand cap layer (already in place), the following tests were carried out *in situ*:

- 27 No. Light Falling Weight Deflectometer (LFWD) Tests with a 300 mm plate for modulus assessment,
- 8 No. Dynamic Cone Penetrometer (DCP) Tests,
- 4 No. Borehole Shear Tester (BST) Tests for the friction angle.

The LFWD is similar to a plate load test, but able to achieve a wider coverage of the site, and without requiring the use of heavy reaction field equipment. This equipment would be constrained to a shallow influence zone, but the DCPs to a greater depth provides a depth calibration for that test. The angle of internal friction for the sand layer was derived from the BST test data. The results of the 4 BST tests show a reasonably consistent result (Figure 2).

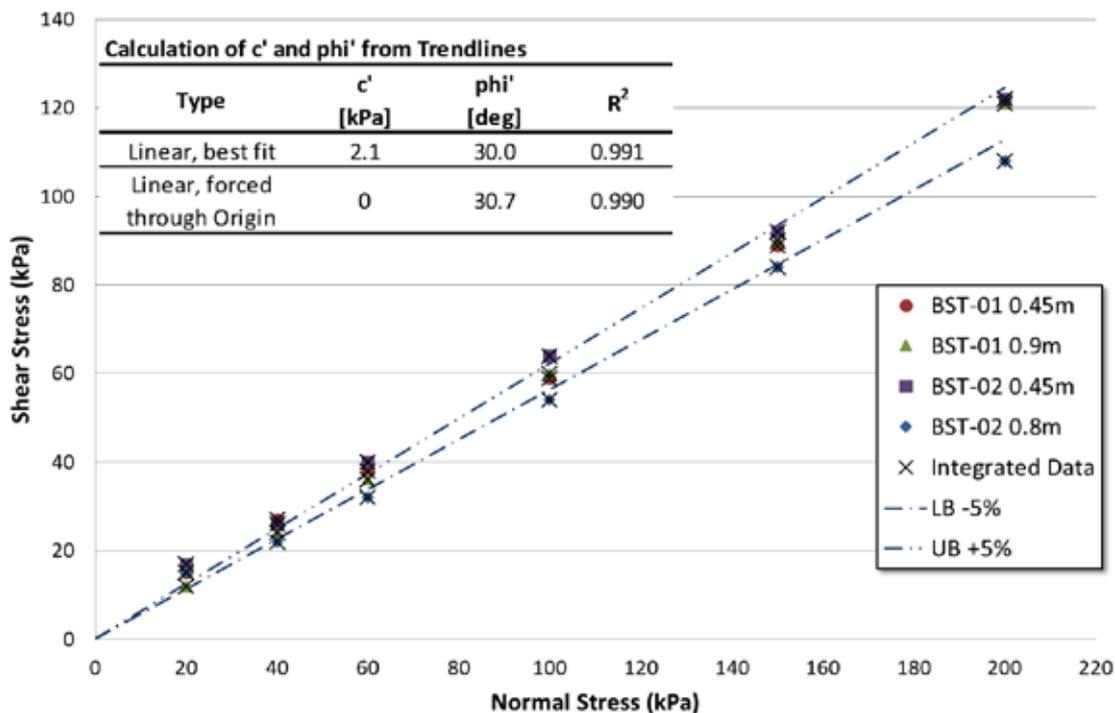


Figure 2: Angle of friction summary from BST data.

The adopted design value is $\phi' = 31^\circ$, and used in the BRE calculations only. The sensitivity of these calculations for a lower bound angle of $\phi' = 30^\circ$ was also checked.

2.3.2 Elastic Modulus of Sand

LFWD tests were carried out on proof rolled (2 passes) and non-proof rolled (0 passes) areas to establish the design value for the Elastic Modulus (E') of the sand. The resulting data shows a distinct influence of the proof rolling on the Elastic Moduli, with both higher values (20% increase), and less variation. There were some minor differences between the 2 areas tested, as well as whether the test was conducted in the tracks of the rock filled or water truck proof rolling equipment. However, for analysis purposes the results were combined.

The LFWD data had 2 distinct purposes:

- 1) To provide a reliable modulus for the placed sand – on the non - proof rolled area. This is for design purposes.
- 2) To provide a reliable modulus value for the proof rolled area to calibrate with the measured rut depth. This is for construction validation prior to the wicking equipment traversing a particular area.

The latter is discussed in later sections of this paper. For design purpose the lower characteristic value of modulus is used. For construction validation, the median / mean value of modulus is adopted.

Table 2 summarises the statistical variation of the LFWD Modulus. It was found that the deflections and thus the Moduli, were influenced by the number of proof rolling passes. The 10% value of the Modulus was used in the calculations of the working platform thickness, while the median value was used for the proof rolling assessment.

Table 2: Summary of LFWD tests and statistical inferences.

Values	No. of values	LFWD Modulus (MPa) at the probability value						
		5%	10%	25%	Median	Mean	SD	COV
Proof rolled	13	29.4	30.4	33.2	38	38	5.5	14%
Away from Proof rolled	11	20.9	22.5	25.9	30.6	32	8.5	27%

SD – Standard Deviation; COV - Coefficient of Variation.

2.3.3 Adopted Parameters in Analysis

Table 3 summarises the adopted geotechnical design parameters, which are slightly different from the initial estimates for the preliminary design provided in section 2.3.1. A distribution function was applied to the key parameters as described in section 2.3.4. The groundwater (Section 2.3.5) was also not used as 0.5 m below the surface.

Table 3: Adopted Geotechnical Design Parameters

Soil Type	$\gamma/\gamma_{\text{sat}}$ [kN/m ³]	c_u [kPa]	ϕ' [°]	ν [-]	E [MPa] with (No.) of proof roll passes
Sand Cap	19/9	-	31	0.35	30.4 (2); 26.5 (1); 22.5 (0)
Clay/Silty Clay	14/4	5	-	0.45	1

For the clay the E_u / c_u ratio of 200 was adopted.

2.3.4 Cohesion of Underlying Clay - a simulation approach

The design value for the undrained strength parameter was 5 kPa based on CPT tests carried out 9 months before placement of the wicking equipment. A design value can vary from a lower bound value, a cautious estimate, or a typical value based on “experience”. However, the strength of the material directly below the capping sand was measured with a minimum of 2 kPa to a maximum of 18 kPa. Selection of a “One” design value is based on a cautious estimate of the parameter and is often subjective. The use of statistics provides both transparency and a medium to model the effects of the variation. Thus while 5 kPa was used in the preliminary design, this approach herein, uses the full range of results to assess the risk.

These statistical considerations are shown on the PERT probability distribution function (PDF) in Figure 3, which was input in the subsequent simulation models. While one can select various distribution functions, the principal author has found the normal distribution is generally not appropriate when analysing natural soil and rock data (Look, 2015). Additionally in defining a PDF for a normal distribution in the @Risk software would require the mean, standard deviation and a “static value” to be input. This latter value is the “design value” and is 5 kPa in this instance.

The PERT PDF requires the minimum, most likely, maximum and the static value to be specified. This is based on inputting the highest, lowest values and most likely value. The shape of the PDF is then calculated by the “most likely value” in the simulation model. Thus the PERT PDF is used as a pragmatic and readily understandable distribution with all input parameters known, and is considered superior to the simple Triangular distribution, as the shape of the curve is more realistic.

A Latin Hypercube sampling is then used in the simulation as this stratifies the input probability distribution as compared to Monte Carlo Sampling. The procedure is described in the Palisade @Risk manual. Simulations then establish the PDF shown. (Note for the calculation of design thickness the Monte Carlo sampling was used). Similarly, the PDFs of other parameters (unit weight and friction angle) have been derived. However these have less spread with a low coefficient of variation.

Additionally as there as a time lag between the placed fill and the wicking equipment being used there would be an expected increase in the strength of the clay – but this was not taken into account in the absence of any further testing

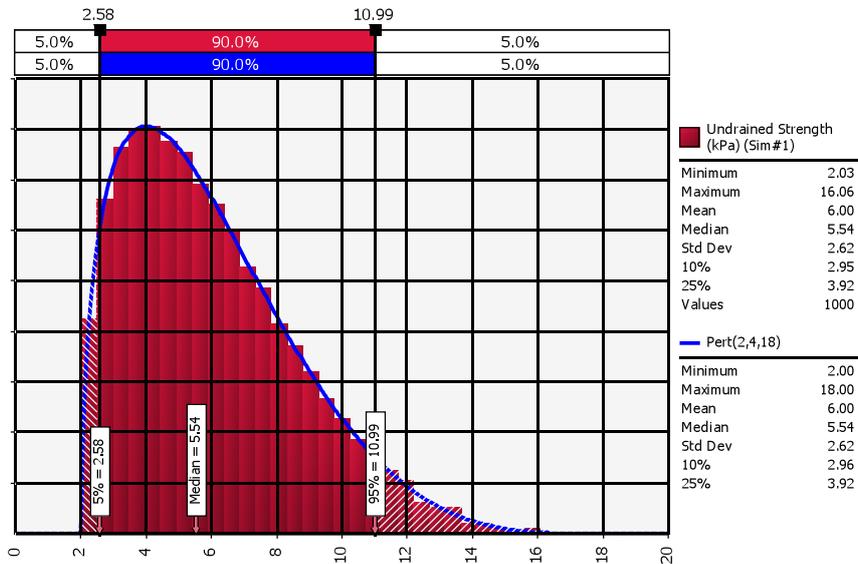


Figure 3. Undrained strength simulation Profile with a PERT PDF

2.3.5 Ground Water Table

Measurements of the ground water table at 4 No. standpipes were obtained. A statistical analysis on the ground water level measurements was carried out. Table 4 summarises the values associated with the best fit PDF of these measurements. There seem to be distinct areas of water level 1) Location CSP113: GWL is 3.1 m below ground level; other locations: GWL is 1.9 m below ground level. Thus the preliminary conservative design assumption of 0.5 m below the surface could be superseded.

Table 4: Water Level Variation.

PDF	Depth (m) below surface for standpipes			
	CSP 112	CSP 113	CSP 114	CSP 115
5%	2.2	3.1	1.9	2
25%	2.3	3.2	2.1	2.1
Median	2.3	3.2	2.2	2.1

3 ANALYSIS

3.1 BRE 470 METHODOLOGY

The deterministic analysis based on the BRE 470 Method was carried out with the 2 ground water conditions identified in Table 4. The required minimum thickness sand caps of 2.3 m and 2.2 m were calculated for the high and low water tables respectively. A sensitivity analysis for a reduced angle of internal friction of 30° resulted in a thickness increase of less than 0.1 m.

However this is a deterministic approach and a load factor does not provide a risk assessment given the many variables involved. The tornado graph of Figure 4 provides the output calculations of Monte Carlo simulations of platform thickness calculated by the BRE method and using the PDF of Figure 3 as input and the spread of results for friction angle and unit weight. The Tornado graph displays a ranking of the input distributions which impact the output. Inputs that have the largest impact on the distribution of the output will have the longest bars in the graph. The highest and lowest statistic value is plotted as each bar ranges above and below the baseline. The baseline represents the overall statistics calculated using all iterations in the simulation model.

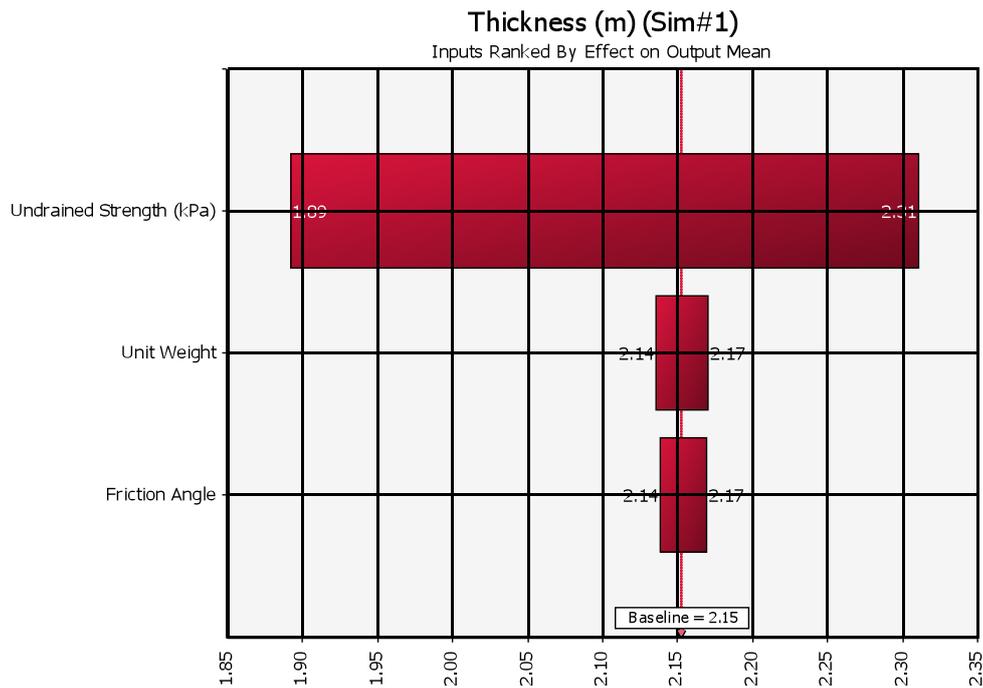


Figure 4. Sensitivity check of platform thickness showing 5% to 95% risk spread

This approach shows the requirement for a 2.3 m platform thickness (traditional BRE calculation) has less than a 5% probability of being exceeded with 2.15 m representing the baseline requirement. The cohesion has the most effect on the required platform thickness. The construction thickness was expected to be 2.5 metres but this varied due to the mud waves produced. This simulation provided a tool for risk assessment using the BRE approach directly, but does not address the methodology issue, that the BRE method is not technically applicable for such soft material.

3.2 ACCEPTABLE DISPLACEMENT CRITERION

The General Shear Failure (Ultimate Bearing Capacity) has a peak value at failure. With a punching failure (BRE simplified mechanism for working platforms), there is no peak value, and failure is defined as when the settlement / applied pressure becomes the largest and practically constant. A mainly elastic response is expected prior to that point.

A Finite Element analysis to calculate the surface displacement under the critical rig loading was carried out using the modelling parameters shown in Table 3 and Table 4. The model is plain strain and soil layers are modelled as linear elastic material. Analyses of both proof rolled areas and non-proof rolled areas were carried out with a sensitivity analysis on the effect of ground water. The LFD results (Table 2) provided confidence on the modulus parameters. The observed displacement for the known load of the proof vehicle proof could then be used to calibrate the numerical model. The load was then changed to the wicking equipment. The resulting displacements using critical wicking crane and loading are presented in Figure 5.

Adopting a factor of safety of 2.0 on displacement (Liu *et al.*, 2008), a deflection of 50 mm or less is deemed acceptable with a 100 mm criterion. However observations of proof roll at this site were of ruts that were deeper than 150 mm in areas that were known to be 2.5 metres thick. The proof vehicle was loaded to provide an equivalent stress as the wicking equipment. Based on this field observation and a FS = 2.0, then 75 mm is considered appropriate at this site. The required minimum platform thickness of 2.1 m for non - rolled areas was determined using this approach.

No difference in required thickness was found between models with a ground water table of 1.9 m and 3.3 m below ground level. The effective unit weight of the platform is used (similar to the BRE method). However the BRE method does not take into account the stress reduction at depth. Even with the BRE method, the depth of water influenced only 0.1 m of platform thickness.

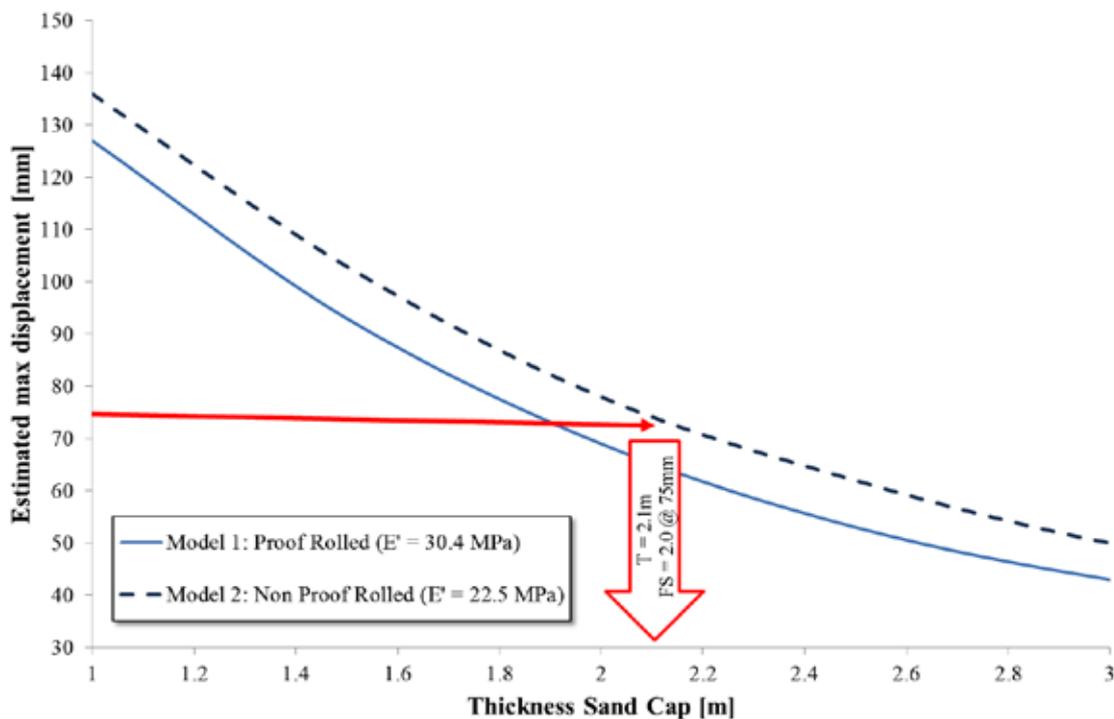


Figure 5. Layer Thickness Sand Cap vs Displacement at surface level under Rig Loading.

3.3 ALLOWABLE STRESS CRITERION

Using the same Plaxis 2D model as discussed above, the calculated effective pressures were also determined at the interface between the sand and the clay layer for a range of thicknesses of the sand layer.

In this method, the effective pressure must not exceed the ultimate bearing pressure using the standard bearing capacity equation for an undrained condition with the $N_c = 5.14$ bearing capacity factor. An appropriate factor of safety then applies. However, if a shear stress applies simultaneously, then $N_c = 3.14$ for a stress ratio of 1 (Jewell, 1982) and this represents a lower bound condition. The presence of mud waves during construction would induce this reduced bearing capacity.

Comparing the allowable bearing capacity with vertical stress, then a 2.1 m minimum thickness was calculated using this lower bound approach.

3.4 ANALYSIS SUMMARY

The analyses show

- 1) Using the traditional BRE Method – 2.3 m thickness is required.
- 2) Using a BRE simulation approach – the 2.3 m thickness requirement represents less than 5% chance of being exceeded.
- 3) Using a displacement criterion – 2.1 m thickness applies.
- 4) Using a stress based criterion – 2.1 m thickness applies.

For this case study, the displacement and stress based approach provides similar results of 2.1 m thickness. However, the principal author has found that this is not always the case in other working platform designs.

4 PROOF ROLLING DEFLECTIONS

The above analysis represents various analytical strategies for risk assessment and confidence on the required platform thickness for the various methods. However observations during construction are usually considered prudent for risk mitigation. Proof rolling was used as a method for demonstrating capacity, but it had limitations in not providing “proof loading” to the full depth of influence of the wicking equipment.

An approach which adopts a stress ratio to obtain a “Factor of Safety” does not account for the influence zone i.e. the heavy pressure proof roll / track pressure ratio is not reliable due to the reduced depth of influence. The track pressure has a larger length / width ratio as compared to proof roll tyres, and hence the latter has a lesser influence zone. Figures 6 and 7 compares the proof vehicle vs the wicking equipment.



Figure 6. Proof vehicle – dump truck with rock fill.



Figure 7. Wicking Equipment with tracks provides different zones of influence to proof vehicle.

Additionally proof rolling by itself is meaningless unless quantified in terms of both proof roll vehicle and expected deflection for the ground condition. For example, a no tolerable deflection criteria (no visible limit is actually less than 3 mm) is only applicable for high CBR materials and does not apply to low CBR material such as this site, where a deflection criterion then applies.

The observed test deflections were used to calibrate the analysis to provide meaningful criteria for proof rolling acceptance. Two proof roll vehicles with similar wheel configurations were used:

- 40 t dump truck with 35,000 L water tank
- 29.1 t dump truck with 32.4 t of rock fill

Table 5 presents the results of the deflections observed on site (Figure 8). The proof trucks had a slight difference in loading but similar zone of influence based on tyre sizes and axle configuration. The results show that, while both wheel tracks are similar after 1 pass, the rock fill truck produced a greater deflection after 2 passes.

Table 5: Measured Deflections during proof rolling.

Vehicle	Pass	Rut Depth (mm)					
		Mean	SD	COV	10%	Median	90%
Rock Fill	1	93	15	16%	71	93	114
Rock Fill	2	133	15	11%	111	133	154
Water Truck	1	90	14	16%	71	90	109
Water Truck	2	106	20	19%	91	100	128



Figure 8. Proof Rolling rut depth measurements.

The Plaxis 2D Finite Elements models were then used to establish an indicative relationship between observed deflections during proof rolling and the estimated platform thickness. In establishing a platform with a likelihood of failure of less than 1%, the 10 percentile modulus value was used with the 5% likelihood water level (Refer Section 3.2). However, for proof rolling as actual deflections have been measured, the most likely modulus (median to mean) was adopted. The results are summarised in Figure 9. This shows:

- For a placed thickness of 2.5 m then 120 mm deflection (proof rolled) is expected. However, due to uncertainty in that placed depth, and the analysis previously discussed then
- For a fill platform thickness of 2.1 m (proof rolled) a 133 mm is expected.

Overall these values suggest that the proof roll may not always extend to the full depth of the platform (varies but to the theoretical placed 2.5 m depth). However it provides a reasonable basis for assessment of the acceptability of the ground prior to placing the more critical wicking rig equipment. The various procedures suggest that a 2.1 m thickness is required, hence the minimum criteria is being satisfied although the full thickness depth would be unknown. The simulation model (Figure 4) had showed had 1.9 m thickness with 95% risk is clearly unacceptable. The underlying very soft clay layer is then within the influence zone of the equipment stresses with added displacement occurring. This would be approximately 150 mm deflection in Figure 9 which compares favourably with the 154 mm on site measurement at 90%.

Using these 2 criteria of a typical 2.5 m and 2.1 m platform thickness as likely in place and minimum acceptable, respectively, then the one pass proof roll criteria becomes:

- ≤ 120 mm proof roll rut is an acceptable value as the placed platform thickness has been satisfied,
- 130 mm proof roll rut suggests the possibility of a reduction in platform thickness to 2.1 m but still an acceptable value,
- > 160 mm proof roll for 2 passes measured suggests no wicking rigs should proceed on this area, because the sand cap has insufficient depth.

The rock filled truck is preferred as a proof roll test based on its more uniform loading / deflection as compared to the water truck.

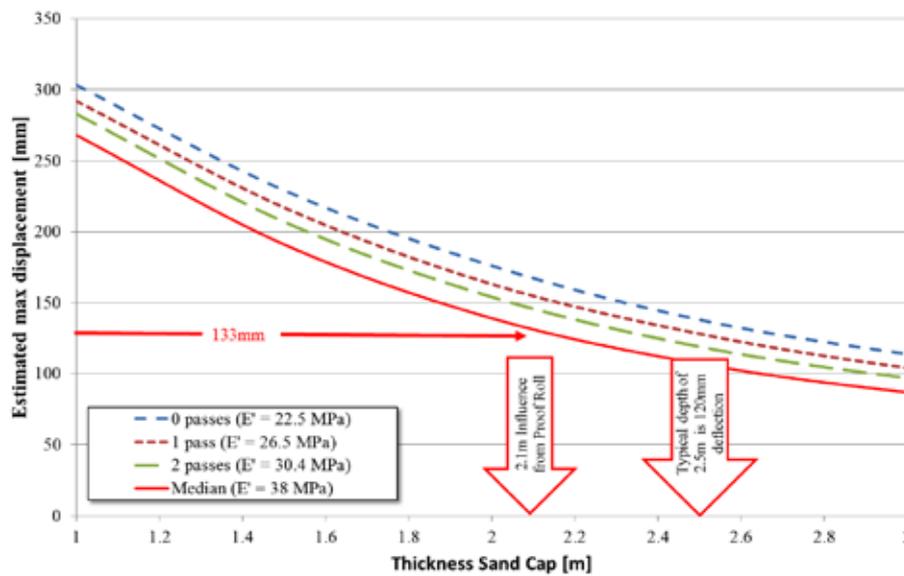


Figure 9. Platform thickness vs displacement for Proof Rolling.

5 CONCLUSIONS

This case study examined the multiple issues associated with a working platform which required a 2.5 m thickness from preliminary design. However the constructed thickness was uncertain due to the mud waves formed. A methodology to assess the actual thickness was required. This had to be factored for the limitations of the BRE method for this application and the uncertainty of the parameters used in the assessment.

The BRE approach is not applicable with low strengths and with significant platform thickness. Yet it represents the “reference” design to be used. A risk based approach was adopted, which showed a 2.3 m requirement had a less than 5% chance of being under designed using the BRE approach. Stress based and deflection criteria were also used. These other approaches show a 2.1 m minimum thickness was appropriate for the given critical wicking rig loads.

Proof rolling is an acceptable method for demonstrating capacity, but an acceptance criterion needs to be applied to be meaningful. A 120 mm deflection criterion was found to be acceptable, with no wicking equipment allowed where an observed proof rolling deflection of 160 mm or above occurred, based on both LFWD modulus measurements input into a finite element analysis and observed deflections. The proof rolling depth of influence is nominally less than the rig equipment influence zone. Thus the proof roll approach while providing improved confidence, should still be viewed as having some uncertainties.

6 ACKNOWLEDGEMENTS

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