A CASE STUDY OF THE BEHAVIOUR OF A VERTICAL SEAWALL

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ABSTRACT

Reclamation is a major construction activity in Singapore, and this often entails the construction of a vertical seawall to contain the reclamation. In Singapore a frequently used technique is the use of L-Blocks. However, there are concerns about the movement and tilt of such relatively light blocks. An ongoing reclamation in the northeast of Singapore makes use of nearly 5.6 km of such wall. As a requirement, prior to commencement of construction, a finite element study was carried out to analyze the possible movement and tilting of the blocks as well as the long-term settlement of the seabed. Instrumentation clusters were then installed to provide valuable data to validate the accuracy of the design and also allow lessons to be learned from the difference between prediction and measurement. This paper is a report of the comparison between prediction and performance that was carried out at this site. As is the case in many actual constructions projects, the effect of certain activities unanticipated in the design caused the measured value to differ significantly from prediction. When these are factored out, it is evident that the finite element analysis can produce reasonable trend, though not the magnitude. However, the results thus far suggest that two simple steps can greatly improve performance, namely a more accurate soil profiling and accounting for the higher stiffness of stiffer strata due to non-linearity at small strain.

Key words: case history, finite element method, land reclamation, seawall, settlement (IGC: E2/H7)

INTRODUCTION

A major reclamation is currently being carried out in the northeastern part of Singapore. One of the requirements in this project is the construction of a long stretch of vertical seawall to contain the reclamation. This entails the construction of over 5.6 km of a vertical seawall using L-shape concrete blocks. Such seawalls have been used previously in a number of projects in Singapore. An important consideration in the design of this wall is the movement of the wall during and after construction as these are relatively light-weight walls.

As part of the design, a finite element study was required for the primary purpose of evaluating the movement and tilt of the L-Block as well as the consolidation settlement. The relevant authorities also used this study as the basis for site control; that is, if the field performance deviated significantly from the predicted values, the contractor would have to provide justifications for the work to continue. It was therefore important that this type of study was able to produce reasonable results. This approach is increasingly common in Singapore and

has been adopted by a number of regulatory agencies. Thus, it is important that such an approach be also evaluated with hindsight, that is, in comparison with actual field performance. Only through such an evaluation can adequate experience be developed to ensure reasonable design in the future for similar construction.

As this is meant for practicing engineers, it is also important that the numerical programs used in the evaluation are commercially available and that engineers can appreciate the soil models used. Just as important is the fact that the relevant soil parameters for a model chosen must be able to be obtained from site investigations that are typically carried out for such construction works. In other words, customized non-commercial programs with sophisticated soil models that are in the domain of researchers are not evaluated in this study. The contribution such fundamental developments can make is not ignored, but is considered to be a separate issue from the perspective of this paper.

In this study, SAGE-CRISP, a commercial finite element program is used. However, the results of this study are equally applicable to other similar programs. In the

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study, only two types of soil model are used. These are the modified CAM-Clay and the elastic-plastic model with failure criterion according to the Mohr-Coulomb model. It is recognized that these models are not the most advanced and have certain inherent weaknesses. However, in Singapore, from a practicing engineer's point of view, these two models represent 'the state-of-the-art' at the moment. Until more sophisticated models become prevalent and better understood by practicing engineers, these two models will continue to be the main tools for actual design purpose. Hence, it is important to evaluate these as they represent the norm.

Data from two instrumentation clusters, denoted as ACL-1 and ACL-3 were used for this evaluation study. To test the effectiveness of the lessons learnt from the back analyses, of the two clusters, ACL1 will be back analyzed in details first, and the lessons learnt will then be used for the back analysis of ACL3. For back analysis, there are many factors that could be considered. For this paper, the primary focus is to evaluate the importance of two commonly encountered issues, namely, the soil stratification and the stiffness used in analysis, in particular the stiff stratum.

SITE CONDITION

The arrangement of the instrumentation clusters relative to the retaining system is shown in Fig. 1. There is one monitoring cluster in ACL1, whereas for ACL3, there are two clusters, named ACL3a and ACL3b. ACL1 and ACL3a are placed 30 m behind the vertical seawall while ACL3b is placed another 15 m behind ACL3a, and is designed to provide back-up data as instrumentation clusters are often damaged in reclamation work where sand is pumped hydraulically. The data provided include readings from extensometers, inclinometers and piezometers that are located at the monitoring cluster while a set-

tlement marker and a tilt-plate are placed on top of the L-Block. The instruments for ACL1 were installed by 25 January 1999 and the most recent readings reported here were until 27 November 1999. The monitoring for ACL3 started on 26 April 1999 and the latest data reported were up to 4 Dec. 1999.

SOIL PROFILE

Around ACL1, the soil profile used is shown in Fig. 2, and under L-Block 30 is a layer of marine clay approximately 11 m thick, this being a member of a formation known locally as Kallang Formation (Chong et al., 1998). Below this is a 5 m thick silty clay, often referred to as a fluvial clay, and typically at this site, has an SPT value increasing from about 6 to 12. Below this is a completely weathered granite, which has an SPT value starting from 20 and increasing to more than 100 very rapidly. As there

Fig. 2. Soil profile at ACL 1 along revetment line

Fig. 1. Layout of instrumentation cluster and instrumented L-Block at (a) ACL1 and (b) ACL3

Fig. 3. Soil profile at ACL 3 along revetment line: (a) Horizontal dimensions in mm and vertical dimensions in m CD

were no boreholes perpendicular to the wall, the soil profile described above is used in the design stage assuming only vertical stratification, that is the soil profile is assumed to be uniform horizontally.

Around ACL3, the soil profile adduced from boreholes and CPT results is shown in Fig. 3. For this site, the seabed level is more even, with an average elevation of -3 m CD (Chart Datum). The top 5 m is an organic clay, and below this is approximately 4 m of marine clay ending at an elevation of -12.4 m CD. Then, there is a layer of sandy soil about 8 m thick with the very stiff layer starting at -20.3 m CD, the latter two are assessed to be members of the Old Alluvium (Pitts, 1983). Similar to ACL1, the information from this single borehole is used during the design stage for the finite element analysis.

SOIL PROPERTIES AT DESIGN STAGE

An immediate, and probably the most important but also the most difficult problem in any finite element study in geotechnical engineering is the choice of soil models and the selection of relevant soil parameters. As the present study arises out of an actual design analysis carried out for a construction project, the soil investigation was conducted in accordance with typical requirements of the industry in Singapore. In the present case, a number of boreholes were sunk, only along the alignment of the vertical seawall. Standard penetration tests (SPT) were conducted in all the boreholes; this is a practice adopted in most soil investigations in Singapore. Undisturbed sampling was conducted at selected boreholes, especially those containing soft clay. For these samples, the usual oedometer, and undrained and unconsolidated (UU) tests were conducted prior to award of contract. After award, a limited number of CK_0UC , CK_0UE and Mikasa's type direct shear tests were also conducted; these tests are not typical in Singapore and were conducted as a result of a special request. At some locations, these tests were complemented using cone penetration tests. As these are typical tests, and done by commercial soil investigation companies, the soil parameters derived are thus partly based on the soil tests and partly based on local knowledge of these soils. Eventually, during the design, the models and associated soil parameters listed under Run 1 in Table 1 were chosen for the analysis. At the design stage, a primary consideration in the selection is that these should represent safe parameters, and not necessarily the most accurate parameters. Hence the parameters chosen are generally on the conservative side.

The Modified CAM-Clay model (Roscoe and Burland, 1968) is chosen for the marine clay for two reasons. First,

Table 1. Properties used for FEM analysis—ACL1

Soil	Run 1		Run 2		Run 3	
	κ	λ	κ	λ	κ	λ
Upper MC	0.1	0.5	0.1	0.5	0.1	0.5
	k (cm/s)	E (kPa)	k (cm/s)	E (kPa)	k (cm/s)	E (kPa)
Silty sand	—	—	1.00E-06	6000	1.00E-06	3000
Clayey silt	—	—	1.00E-06	8000	1.00E-06	8000
Fluvial clay 1	1.00E-06	5000	1.00E-06	5000	1.00E-06	6000
Fluvial clay 2	1.00E-06	10000	1.00E-06	10000	1.00E-06	15000
Fluvial clay 3	1.00E-06	10000	1.00E-06	10000	1.00E-06	18000
Fluvial clay 4	1.00E-06	10000	1.00E-06	10000	1.00E-06	30000
Completely weathered granite	1.00E-06	18000	1.00E-06	18000	1.00E-06	100000
Fill sand (Top)	1.00E-04	17000	1.00E-04	17000	1.00E-04	17000
Fill sand (Bottom)	1.00E-04	17000	1.00E-04	17000	1.00E-04	17000
Sand key	1.00E-04	17000	1.00E-04	17000	1.00E-04	17000

this is one of the simplest elasto-plastic models, and more importantly, the various parameters needed can be selected based on routine site investigations. Nowadays, this model is used more frequently in routine design in Singapore where finite element analysis is carried out. Secondly, this in-situ marine clay is lightly overconsolidated (Chong et al., 1998), and thus expected to consolidate significantly during the reclamation. Thus having an accurate compression behaviour is important. This made the elastic-plastic model, the other popular choice in Singapore for practical engineering analysis unsuitable. The Upper Marine Clay of Singapore is a relatively young, lightly structured and only mildly anisotropic with shear strength values from UC and UE tests nearly similar (Chong et al., 1998). Thus it is expected that the CAM-Clay model will be able to model the deformation behavior reasonably. The CAM-Clay parameters were obtained directly from the soil tests. λ and κ were obtained directly from 1-D oedometer tests using the following relations, namely $\lambda = C_c/2.3$ and $\kappa = 1.5(C_s/2.3)$.

For the other soils, an elastic-plastic model is used, assuming a Mohr-Coulomb failure criterion. For most of these soils, it is expected that the yield criterion is not a major factor, but the estimation of the stiffness is. A parametric study on this was carried out and the above observation was confirmed. For the fluvial clay, an intermediate layer below the Upper Marine Clay, typically Young's modulus is estimated from the SPT value using an empirical formula of 800–1000N. In the present study, for the design analysis, the value chosen was approximately 800–900N. A similar value was used for the completely weathered granite, though this is recognized to be conservative.

CONSTRUCTION PROCESS

The sand key is constructed by first dredging and trimming the trench and then by direct dumping of the sand and the only compaction effort comes from the tamping carried out to compact the stones; granites, which are on top of the sand where the L-Block will be placed. This compaction effort is not expected to be significant and hence the sand is expected to be fairly loose.

For this reclamation, an aspect of concern is the movement of the vertical wall as a result of the sand filling operation, in particular the dynamic effect from the water carrying the sand during the hydraulic placement. In the project, the sand fill was placed progressively in a direction parallel to the wall, and some distance from it. The main purpose for this is to reduce the lateral movement of the L-Block. This method was adopted based on information gathered on other similar projects in Singapore where there were L-block movements when sand filling works were carried out in the direction of shore to sea.

Trailer-hopper suction dredges were used in the hydraulic filling operations. A branch from the discharge pipe was placed about 25 m behind the L-blocks parallel to the revetment line. The sand was deposited from be-

Table 2. Construction sequence

Stage	Time (month)	Activity
1	1.5	Dredging and sand fill placement
2	1.25	Inspection and survey
3	1.5	Placement of stone and granite in sand key
4	0.4	L-Block installation
5	0.5	Placement of back-fill stone
6	0.5	Sand-fill to 1.0 m
7	0.75	Sand-fill to 5.33 m at a distance 20 m away
8	0.75	Installation of coping concrete
9	0.75	Sand-fill of the gap
10	1.5	Surcharge imposed
11	360	Long term consolidation

hind the L-blocks and progressed towards the shore. This method helps to minimize the lateral movement of the L-blocks and prevent any circular slip failure during reclamation. Considering the soil condition at this site, it would have been impossible to avoid slip failure, if the filling works had been carried out from shore towards the sea with a bulldozer moving the sand forward.

FINITE ELEMENT STUDY

The time dependent behaviour of the L-Block retaining structure and its supporting ground was studied using the program, SAGE-CRISP, which incorporates a fully coupled consolidation analysis based on Biot's formulation (Britto and Gunn, 1987). SAGE-CRISP is the commercial version of an original program known as CRISP (CRITICAL State Program) and is based on the finite element method. This program can simulate the construction of the sand key, placement of L-Block and the sequence of fill placement. CRISP is a well-known program and has been used to study problems of excavation and earth retaining structures, particularly those involving time-dependent behaviors (Powrie and Li, 1991; Bolton et al., 1989; Lee et al., 1993).

In the finite element analysis during design stage, the construction sequence proposed by the contractor as shown in Table 2 was used. In the study, the finite element mesh used must be adequate for simulating the entire construction process. CRISP allows the flexibility of removing and adding of elements, a feature that is particularly useful in this study to simulate the dredging of clay, dumping of sand, placing of L-Block and finally the filling process. For the design analysis conducted, the mesh used simulates a cross section as shown in Fig. 4.

TYPE 'A' RETAINING WALL REVEITMENT WITH INSTRUMENTATION CLUSTER ACL-1

(a) Horizontal dimensions in mm and vertical dimensions in mCD

Fig. 4. (a) Cross section of retaining wall, (b) Corresponding finite element mesh used

COMPARISON OF COMPUTED AND MEASURED RESULTS FOR ACL1

Run 1—Design Analysis—ACL1

As a first comparison, the original design predictions were compared against the measured values. This run is denoted as Run 1 in all the figures to be shown. Figure 5 shows the predicted vertical settlement of the L-Block for this run versus measured values. The figure indicates that the actual settlement of the L-Block is significantly larger than the predicted value. In particular, it can be observed

that the L-Block settled very significantly, and deviates from the measured values, during the stage where back-fill stones are placed just behind the wall. This placement is by dropping from grabbers large stones of a size of up to 100 mm. One likely reason is that the sand placed in this area is very thick, more than 10 m actually. This sand is placed by direct dumping, and thus may result in an inconsistent relative density, prevalent when a thick layer is formed. Since it is very difficult to demonstrate the dynamic effect of backfill stone placement, this omission has resulted in the difference between analysis and ac-

Fig. 5. Settlement of marker on L-Block 30

Fig. 7. Tilt of L-Block 30 (adjusted)

Fig. 6. Settlement (adjusted) on L-Block No 30

tual measurements. Fortunately, the vertical wall was designed with a coping concrete to be added to the top of the L-Block in the final stage, thus allowing for some adjustment, provided this settlement resulted in a tilt that was acceptable. In the present case, the maximum tilt recorded was 1:130, and this was within the acceptable limit of 1:100.

As the finite element study conducted cannot consider this dynamic effect, the predicted results were clearly off. For a fairer comparison, this settlement due to the dynamic effect of the hydraulic fill was removed from the results. The new comparison for the vertical settlement of the L-Block is now shown in Fig. 6. A similar adjustment is done for the tilt of the L-Block and shown in Fig. 7. The predicted settlement is now closer to the measured value, especially in the early stages. However, the prediction in the later stages is still smaller than the measured value. Further, the predicted settlement of the L-Block indicates some tilting landwards resulting in an upward tilt whereas the measured settlement does not show this at all. The predicted tilt, on the other hand, agrees better with the measured data, both indicating landwards tilt. Clearly, the finite element study has room for improvement.

To evaluate how to improve the analysis, especially the

Table 3. Compression of different layers of soils at ACL1

Run No.	Run 1	Run 2	Run 3	Actual
Depth (m CD)	Settlement at each level (mm)			
-4.8	-112	-194	-101	-123
-7.8	-63	-76	-25	-41
-9.8	-53	-51	-16	-3
-11.3	-35	-46	-13	-3
-13.3	-31	-40	-9	-4
-15.8	-25	-34	-7	-3
-18.8	-19	-27	-5	-1
Run No.	Run 1	Run 2	Run 3	Actual
Layer (m)	Compression of each layer (mm)			
-4.80 to -7.80	49	118	76	82
-7.80 to -9.80	10	25	9	38
-9.80 to -11.30	18	5	3	0
-11.30 to -13.30	4	6	4	-1
-13.30 to -15.80	6	6	2	1
-15.80 to -18.80	6	7	2	2

soil parameters selected, it is essential to evaluate the settlement of different soil layers, some distance away from the L-Block. This is to avoid complications arising from the placement of the L-Block and the stone placement operations. The instrumentation cluster, ACL1, was placed 30 m from the L-Block and was designed to provide this information through the extensometers attached to an installed inclinometer. The compression of different layers is summarized in Table 3 while Figs. 8 to 9 show the settlement at two levels, namely at -4.8 m CD and -11.8 m CD while Fig. 10 shows a comparison of the lateral displacement with measured inclinometer results at the end of reclamation; the location of the in-

Fig. 8. Settlement at -4.80 m CD Fig. 10. Lateral displacement (end of reclamation to $+6$ m CD) Fig. 9. Settlement at -11.30 m CD Fig. 11. Pore pressure at -6.4 m CD

clinometer is in Fig. 4.

Two glaring aspects can be observed from this comparison. First is the gross over-prediction of the settlement of the stiff base layer (below a depth of 18.8 m). The finite element analysis suggested a settlement of 19 mm whereas in reality the movement is only 1 mm. This trend is also observed in Fig. 10 where the predicted lateral displacements in the stiffer soils are consistently much larger than the measured values at all stages. In reality, the inclinometer indicates that in the stiff soil, practically no lateral displacement can be detected. On the other hand, the predicted compression of the top marine clay layer (-4.8 m to -9.8 m) is too small, yielding a value of 59 mm whereas the actual compression was 120 mm. As was mentioned in the introduction, this was an actual design using values derived from routine soil investigation reports. This comparison therefore indicates that in the design run, RUN1, the soil parameters chosen have under-predicted the compression of the marine clay layer and over-predicted the compression of the stiff completely weathered granite.

However, for the settlement at the top, the errors from different layers partly compensate each other, producing a predicted settlement of 112 mm at a depth of -4.8 m, which is close to the measured value of 123 mm. Such

coincidental agreement due to compensating errors is not uncommon in geotechnical engineering, but does not give any room for complacency requiring instead to be well understood. It also points to the importance of examining a wide array of parameters to obtain an objective assessment in a back analysis of field performance.

The pore pressure readings obtained from one location at the instrumentation cluster ACL1 are shown in Fig. 11. This indicates that in the numerical analysis, very little of the excess pore pressure is dissipated compared to the measured values, which suggests a much faster rate of dissipation. This could be due to a lower permeability used or because the thickness of the marine clay layer used in the analysis at the location of ACL1 is smaller than the actual thickness. As the properties of the marine clay are fairly well established (Chong et al., 1998), it is more likely that the difference in predicted and observed values for the compression of the marine clay layer is due to a difference in the thickness of the marine clay layer used in the analysis rather than using a set of incorrect soil properties.

Fig. 12. New soil profile along L-Block 30 and ACL 1

Run 2—ACL1

With additional information from the installation record of the extensometer for ACL1, the soil profile from L-Block 30 to ACL1 was interpolated and the new soil profile, shown in Fig. 12, shows that the marine clay layer at ACL1 is 7 m thick, thinner than the 11 m assumed originally. Thus instead of the uniform profile as in the case of Run 1, Run 2 was conducted using this new soil profile with two new soil types added in, but the rest of the soil parameters are kept the same. These properties are summarized in Table 1 under Run 2.

The newly computed results for Layers 1 and 2, as shown in Table 3 under Run 2, now match better the measured results. Even though the soil properties were not altered in this analysis, the computed compression of Layer 1 in Run 2 showed a much larger value than that obtained in Run 1, and was also larger than the actual value. This is possibly due to a thinner layer of marine clay, which means a faster rate of dissipation of excess pore water pressure and thus a higher degree of consolidation in the time frame considered in this study. This is consistent with the computed pore pressures at -6.40 m CD shown in Fig. 11, and a similar trend is observed at other locations. The predicted pore pressures are also much closer to the actual results compared to Run 1, an indication that this was an overlooked issue in the original design, an issue which could be easily redeemed with just a few more points in the site investigation program.

However, the overall settlement trend shown in Figs. 8 to 9 still shows significant deviation from the measured value due to significantly larger predicted settlement in the stiff layer as well as in the top marine clay layer. In spite of the changes in the soil profile, the predicted behaviour of the lateral displacements as shown in Fig. 10 shows negligible improvement. The difference is almost entirely in the over-prediction in the stiffer layers. Similarly, from Figs. 7 and 8, the changes made in Run 2 have very little impact on the tilt and settlement of the L-Block. This last observation is not unexpected as the soil

profile under the L-Block itself is still the same as in the previous run.

Run 3—ACL1

In the previous two runs, the computed settlements are larger than the measured data below -11.3 m CD. As an elastic-plastic model is used to model each of these layers, one adjustment is to increase the Young's Modulus, E of the soils. This adjustment can be easily rationalized since in the very stiff stratum, it is likely that the strain experienced is small. With advances in understanding of the non-linearity of the modulus at small strain (Burland, 1989), it is recognized that the modulus at small strain is likely to be much larger than values based on conventional testing or correlation. Therefore, in Run 3, the Young's Modulus, E , of the residual soil is increased by 1.2 times for the upper layer, and increases to 3 times for the lowest layer. For the very stiff completely weathered granite, the E value is increased to about 5.5 times. It must be stated that these formations are not well studied, and to the authors' knowledge, no published literature is available on the expected G_{max} of these soils. Thus, the values chosen here, to some degree represents the authors' best guesses. Also, from Table 3, it was observed that the computed settlement from -7.80 m to -9.80 m CD was smaller than the measured value. Hence, it was decided to reduce the value of E of Silty Sand. These changes are shown in Table 1 as Run 3.

With these adjustments, the computed settlements now show a better fit compared to the previous two runs. From Table 3, the compression of the marine clay (Layer 1) was reduced from the previous computed value of 118 mm in Run 2 to the present compression of 76 mm, which is much closer to the measured result of 82 mm. This improvement is achieved by adjusting only the underlying layers and without adjusting the properties for marine clay. However, in spite of a lower value for E , the computed compression for Layer 2 has become smaller than Run 2, which was already smaller than the actual value.

For the stiffer soils, with the increase in E , the computed results now match better the measured readings compared to Run 2. Figures 8 to 9 clearly show this significant improvement. The computed vertical settlements show signs of stabilizing to steady values, indicating that the excess pore pressures in these layers should be near to complete dissipation. As the permeability of the soil was not altered, the pore pressures did not differ significantly from Run 2, and similar to Run 2, the computed pore pressures at PP1 and PP2 have almost dissipated to the expected long-term value, as shown in Fig. 11.

With a stiffer bearing layer, the settlement of the L-Block shown in Fig. 6 is, as expected, smaller than that from Run 2, the decrease in settlement being mainly due to the stiffer values used in the soils beneath the sand key. The computed tilt is also correspondingly smaller than that predicted in Run 2, but compared to the measured value, it shows a surprisingly good fit, as shown in Fig. 7. In reality, the settlement at one point of the L-Block is

due to the settlement of the underlying soil as well as the tilt of the L-Block both in the perpendicular direction and along the wall. As the settlement is measured at only one point, it is difficult to differentiate these contributions from the measured data. However, just for evaluation purposes, the predicted settlement at the bottom corner at the seawards edge of the L-Block is used for comparison, and this is indicated as Run 3(bot) in Fig. 6. This settlement is less likely to be influenced by the tilt of the L-Block. A surprisingly good fit is obtained and more importantly, the significant upwards tilt of the L-Block is no longer present. Clearly, the complicated rotation of the L-Block has made its prediction more complicated than anticipated.

Summary of Lessons Learnt from ACL1

From ACL1, it was found that the dynamic effect due to stone backfilling and sand backfilling to +4 m CD has a significant effect on the measured results that could not be modelled. Therefore, it is likely that any future design work under similar site conditions will also require the dynamic effect of the above-mentioned activities to be considered. In this case, the settlement due to this effect accounts for more than 50% of the total settlement (before adjustment in subsequent comparison). This shows the importance of adequate compaction of the sand key. If the sand has been compacted sufficiently, the sudden settlement under the dynamic effect would have been very much reduced. In addition, due to the non-linearity of stiffness at small strain, the stiffness of the stiffer soils, namely the fluvial clay and the completely weathered granite can be increased to several times the original design value, which were based on conventional testing and thus are more correct for larger strain.

Furthermore, during design, when an over-simplified soil profile was used in the analysis, the prediction was found to be far from the measurements even after adjustments for any unanticipated activities were made. An improved soil profile through interpolation using information from the installation records at the monitoring cluster and the nearby borehole's drilling log gave a significantly better agreement. This shows the importance of doing adequate soil investigation.

From the parametric studies on the various soil properties that are not reported in this paper, it was found that the variation of parameters relating to the strength, namely c' , ϕ' , M and e_{cs} , did not have much effect on the results of prediction as most parts of the soils are not at their critical state. In addition, the range of values for the unit weight of soil is rather limited and it is unlikely that the actual unit weight will be very different from the values used in the analysis.

Clearly many more changes can be made to fine-tune the results until they match with the measured values. These changes can then serve as "micro-lessons" to be learnt. As there are so many factors, not all well accounted for, there could therefore be many different sets of "micro-lessons" to be learnt. This is not the focus of the paper. Instead the paper wishes to concentrate on the use

of some simple models in a commercially available finite element program and focuses on two broad lessons, or "macro-lessons" that can be learnt. These two are mainly the effect of getting a correct soil profile and also the importance of accounting for non-linear effect, albeit in an approximate way, in the analysis.

COMPARISON OF COMPUTED AND MEASURED RESULTS FOR ACL3

Run 1—ACL3

This is the design run and the material properties in this run are shown in Table 4. The comparison of predicted results with measured values is summarised in Table 5. One typical settlement curve from ACL3a is shown in Fig. 13 and one from ACL3b is shown in Fig. 14. The pore pressures at one location at ACL3a and one location at ACL3b are shown in Figs. 15 and 16 respectively while the lateral displacement at one point in time is shown in Fig. 17. The settlement and the tilt of the L-Block 362 are shown in Figs. 18 and 19 respectively.

At ACL3b, the predicted settlements, as shown in Fig. 14 and in Table 5, far exceed the measured results. Table 5 shows that the settlement calculated at the end of the analysis is approximately 8 to 10 times larger than the measured results at the locations of the four extensometers. However at ACL3a, Fig. 13 and Table 5 show that the computed settlements agree much better with the measured results. These two observations taken together suggest that the soil profile is probably not uniform as assumed. This is supported by the installation record of the extensometer at ACL3b, which shows that the soils found at this location are generally sandy in nature with only a thin layer of silty clay, whereas for ACL3a and the L-Block locations, the soils are mainly clayey until an ele-

Table 4. Properties used for FEM analysis—ACL3

Soil	Run 1		Run 2	
	κ	λ	κ	λ
Cam-clay model				
Upper organic clay	0.09	0.46	0.09	0.46
Upper marine clay	0.1	0.5	0.1	0.5
Mohr-Coulomb	k (m/s)	E (kPa)	k (cm/s)	E (kPa)
Old alluvium	1.00E-06	10000	1.00E-06	10000
Old alluvium	1.00E-06	12000	1.00E-06	12000
Old alluvium	1.00E-06	14000	1.00E-06	14000
Old alluvium (SPT > 50)	1.00E-08	18000	1.00E-06	80000
Fill sand (Top)	1.00E-04	17000	1.00E-04	17000
Fill sand (Bottom)	1.00E-04	17000	1.00E-04	17000
Sand key	1.00E-04	17000	1.00E-04	17000
Loose silty sand	—	—	1.00E-06	10000
Dense sand	—	—	1.00E-06	30000
Soft silty clay	—	—	1.00E-07	2000

Table 5. Compression of different soil layers at ACL3

ACL3a				ACL3b			
Run No.	Run 1	Run 2	Actual	Run No.	Run 1	Run 2	Actual
Depth (m CD)	(mm)			Depth (m CD)	(mm)		
-5.86	-217	-276	-270	-4.71	-350	-199	-49
-8.46	-80	-128	-154	-8.2	-69	-40	-17
-10.66	-41	-93	-83	-11	-50	-27	-15
-12.86	-26	-17	-3	-15.4	-25	-19	-7
Layer (m)	(mm)			Layer (m)	(mm)		
-5.86 to 8.46	137	148	116	-4.71 to 8.2	281	159	32
-8.46 to -10.66	39	35	71	-8.2 to -11	19	13	2
-10.66 to -12.86	15	76	80	-11 to -15.4	25	8	8

Fig. 13. Settlement at -5.86 m CD at ACL3a

Fig. 15. Pore pressure at -7.1 m CD at ACL3a

Fig. 14. Settlement at -8.20 m CD at ACL3b

vation of about -12.4 m CD where silty sand is encountered. Therefore, it is probable that the actual soil profile is fairly uniform from the L-Block to ACL3a but changes rather rapidly from ACL3a to ACL3b. The soils

at ACL3b are therefore likely to be stiffer in strength and this accounts for the much smaller settlement observed.

The computed values for the pore pressure at ACL3b, as shown in Fig. 16, match reasonably the measured data initially. However, after 27 September 1999, the measured pore pressure values started to increase suddenly. From site records, it was established that during that period, both the piezometers at ACL3b were damaged and subsequently reinstalled in mid-October. On the other hand, the predicted pore pressures at ACL3a do not match the measured values as well, the prediction suggesting a high pressure built-up whereas the measurement indicates a rather small increase. Therefore, these two results suggest that the rate of dissipation of the excess pore pressures was not correct in the analysis. Adjustment of the soil properties is required to increase the dissipation rate to match the field behaviour.

The settlement and tilt of the L-Block shown in Figs. 18 and 19 respectively paint a more confusing picture. The predicted settlement, even after adjustment, is still significantly smaller than observed. More importantly, Fig. 19 shows that the analysis predicted a landwards tilt

Fig. 16. Pore pressure at -6.5 m CD at ACL3b

Fig. 17. Lateral displacement on 4 Dec. 1999

Fig. 18. Settlement (adjusted) at L-Block 362

whereas the measured data indicates a seaward tilt, resulting in a significant upward movement of the L-Block. The behaviour of the L-Block, seemingly simple, is actually more complicated than that for the compression be-

Fig. 19. Tilt at L-Block 362 in ACL3 location

haviour of the various soil layers. This is suspected to be due to the non-uniformity in the sand key. During the L-Block installation and stone backfilling, the landwards portion of the sand-key is subjected to some compaction and greater overburden. Hence it becomes stiffer. Thus, when the reclamation took place, the less stiff seawards side will not be as stiff and will settle more than the landwards section. This difference in the properties of the sand-key is not accounted for in the analysis as the effect of such tamping is not possible to quantify at this stage. So in the analysis, the sand-key is assumed to be uniform.

Run 2—ACL3

It can be seen that the predicted results based on a run executed at design deviate significantly from the measured readings in most instances and especially at ACL3b for the amount of compression in the various soil layers. Therefore, the lessons learnt from the analysis of ACL1 were now applied to the analysis of Section ACL3 and this formed the basis of Run 2. In this run, two main changes were made. First, the stiffness of the bearing layer, the Old Alluvium, is increased to reduce the amount of predicted settlement and lateral displacement in the stiffer soils. The permeability of this layer is also reduced to 1×10^{-4} cm/s to reduce the heaving in the settlement; this is a result of a micro parametric study that was conducted. Also, an updated soil profile using installation records for ACL3a and ACL3b was used in the new analysis. The changes made to the soil properties and the parameters of the new soil layers are all given in Table 4.

A better prediction for the settlement at ACL3a can now be observed with the predictions now much closer to the actual measurement in terms of both magnitude and trend, but the predicted rate of settlement is still too high as compared to the measured value, as can be seen from Fig. 13. As the permeability of the lowest Old Alluvium layer was reduced, the computed pore pressures dissipate a little faster compared to the previous run, as can be seen Fig. 15. However, since the soil profile at ACL3a was not altered and no change was made to the soil properties except for the layer mentioned above, the difference is not large.

With the new soil profile, the predicted settlements at ACL3b, shown in Fig. 14 and Table 5, also show an improved agreement with the measured values. This is particularly true for the compression observed in the individual soil layers. This improvement can also be seen in the lateral displacement shown in Fig. 17, where the agreement in the stiff layer is now significantly improved. However, in spite of the significant improvement, and a much more consistent trend and magnitude as compared to the results measured by the 4 extensometers, the computed settlements at the end of the analysis are still much larger than the measured data.

For the L-block, the predicted and measured tilts are in the opposite direction, as shown in Fig. 19, and actually Run 2 becomes worse. The reason for this was given earlier: in actual fact, the sand on the landwards side is stiffer than that in the seawards side resulting in a tilt seawards. This also results in a significantly smaller settlement than measured, as can be observed from Fig. 18. As in Run 3 for ACL1, the predicted settlement at the corner of the seawards edge of the L-Block is also shown in Fig. 18 as Run 2 (bot). Again, a surprisingly good fit can be obtained. This means that the non-uniformity in the sand key needs to be better evaluated to obtain a better prediction of the settlement at the top of the L-Block, which is sensitive to tilt in all directions.

Overall Assessment of ACL3

Generally, with highly simplified soil profile and poor assessment of the stiffness properties when an elastic-plastic model is used for the stiff soil layer, the design prediction is significantly out as compared to the measured values. However, when the lessons learnt from ACL1 are applied, the analysis has been able to capture the behaviour of the different soil layers much better. Though the original objective of the analysis was to assess the behaviour of the vertical seawall, the matching of the tilt and settlement at the top of the wall indicates a poor agreement, and the main reason is suspected to lie in the non-uniformity of the sand. By using the settlement at the bottom corner of the seawards edge, just as in the case study for ACL1, a surprisingly good fit was obtained.

DISCUSSION AND CONCLUSION

The present study offered an opportunity to evaluate an actual design calculation against measured field performance. The ability to perform good design analysis in a large project cannot be over-emphasized, given the serious cost implications. In most practical cases, such design is carried out using commercially available finite element programs and the adoption of simple models, whose parameters can be obtained or assessed from routine soil investigations carried out for such construction projects. The use of finite element analyses based on properly coupled-analysis as a routine design tool is a very recent development in Singapore and thus the adoption of the elastic-plastic model using the Mohr-Coulomb failure criterion or the simplified Cam-Clay

model would still represent the present "state-of-the-art".

This study is focused on the evaluation of a design using such an approach and the simple changes one can make to significantly improve the design analysis. In back analyses of an actual construction, a number of factors will contribute to make any attempt to ensure good matching very difficult to appreciate and also run the risk of using the wrong parameters to correct an observed difference. These factors are mainly due to imperfect knowledge of the soil profile, actual construction process and simulating them, and difficulties in establishing correct soil parameters. In this paper, only two macro issues are addressed, namely the soil profile used and the use of higher stiffness to account for non-linearity in the stiffer soils. Such an attempt is unlikely to produce perfect matching, but rather to provide simple guides for practising engineers to significantly improve their design analysis.

A number of observations can be made from the comparison carried out in this study:

1. As is usual in construction projects, certain unanticipated processes often cause major deviation from design. In this case, the most significant movement of the seawall occurs during the placement of the back-fill stones on a thick sand key where some inconsistencies of density are likely to be observed. Without the relevant experience, such events cannot be anticipated in the design analysis, and as expected the agreement with observed behaviour is poor. This also strongly suggests the need for such experience to be shared to ensure greater awareness; one way is for journals and conferences to report more actual case studies.
2. The finite element analyses conducted were nevertheless able to produce somewhat reasonable predictions of the settlement and compression of the soils layer. However, the prediction of the movement and tilt of the vertical seawall is much poorer due mainly to the non-uniformity of the sand key, an aspect not modelled here due to imperfect knowledge. What is surprising is that the predicted settlement at the bottom corner of the seawards edge agrees well with the measured settlement at the top of the wall.
3. Two factors can make a significant contribution to improve markedly the analyses, namely the use of a more accurate soil profile and the use of higher stiffness for stiffer soil layers. The latter is a pseudo attempt to cater for the higher stiffness associated with a stiff soil as a result of a much smaller strain induced. This pseudo attempt is important at this stage as the use of an elastic-plastic model using a Mohr-Coulomb failure criterion, without accounting for non-linearity of stiffness at small strain still represents the "state-of-the-art", at least for the case of Singapore.
4. In the present analysis, there is still significant room for improvement by using appropriate adjusted soil parameters. However, with so little information

available and so many uncertainties, such micro-refining will result in many possible sets of adjustment. The obvious challenge is the ability to know the right combination during design and not in back analysis.

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