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Behavior of a Breakwater on Soft Sediments - Failure and Success

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SYNOPSIS: The observed responses during the staged construction of an unreinforced section and a reinforced section of a breakwater founded on soft sediments are presented. Both sections were instrumented and the unreinforced section approached instability at the end of Stage I construction. A series of Class 'C' predictions were made with three different analytical methods. It was particularly difficult to predict the impending instability of the unreinforced section. A fully coupled finite element analysis, with close modelling of the construction sequence and using a strain softening model for the foundation clay, was needed to predict all the observations.

INTRODUCTION

An instrumented breakwater was constructed at a site called Red Bay near China's largest special economic zone, Shenzhen during the period March 1984 to September 1985. The breakwater has a length of 464m, a height of 7.5m and a crest width of 8m. The foundation level is 5m below mean sea level. A general view of the project is presented in Fig. 1.

The stratification of the site consists of a layer of highly compressible silty clay of approximately 7m thick, followed by a layer of soft sandy silty clay (Fig. 2). At a depth of about 10m, a layer of stiff clay is encountered. The results of in-situ vane shear tests and cone penetration testing indicate that the two layers of soft clay are very uniform across the site. No stiff soil crust was detected. Extensive site investigation and laboratory testing were conducted on the two layers of soft clay, referred to as the foundation clay in this paper, prior to the commencement of this project.

The original design of the breakwater is shown in Fig. 2a. The top two to four meter of soft clay was first excavated and a sand blanket was laid prior to the placement of the crushed rocks. In the design of the embankment, the concept of 'staged construction' was used. The embankment was designed to be constructed to 5.5m high in stage I. Then after six months waiting period to allow dissipation of excessive pore water pressure, the breakwater was to be raised to the full height of 7.5m, referred to as stage II. However, due to frequent collapse of the embankment even during stage I, the construction had to be suspended. The design was then subsequently changed to that of a reinforced embankment with a layer of woven geosynthetics laid in the sand blanket (Fig. 2b). The breakwater with the reinforced design was successfully completed in September, 1985.

The objectives of this paper are: i) to establish explanations for the collapse of the unreinforced embankment constructed to the original design, and ii) to investigated the capability of a number of analytical methods in predicting *all* the observed responses of the breakwater.

INSTRUMENTATION

The pore water pressures in the foundation were monitored with hydraulic piezometers at the depth of 5m, 6m and 7m below the original foundation surface along the centerline of the embankment. The vertical displacements along the centerline of the embankment were measured using settlement plates. The horizontal movements near the toe of the embankment were measured with inclinometers. No instrument was mounted for the measurement of reinforcement strain or tension. The locations of the monitoring points are shown in Figs. 2a-b. In the reduction of excessive pore water pressures data, corrections were made to allow for the effect of tidal fluctuation.

OBSERVED BEHAVIOUR DURING CONSTRUCTION

The observed performances of two typical sections, Sect 0+100 and Sect 0+370 are presented below. All the instrumentations in these two sections survived. Section 0+100 is representative of the original (unreinforced) design whereas Sect 0+370 is representative of the modified (reinforced) design.

Construction of Sect 0+100, of original design, started on 8th April, 1984. It was raised in 40 days to the height of 5.5m as Stage I. Immediately after completion of stage I, the settlement at the foundation surface was 740 mm and the horizontal displacement was 250 mm. The observed settlement and horizontal displacements were increasing at an alarming rate. Such a behaviour was similar to

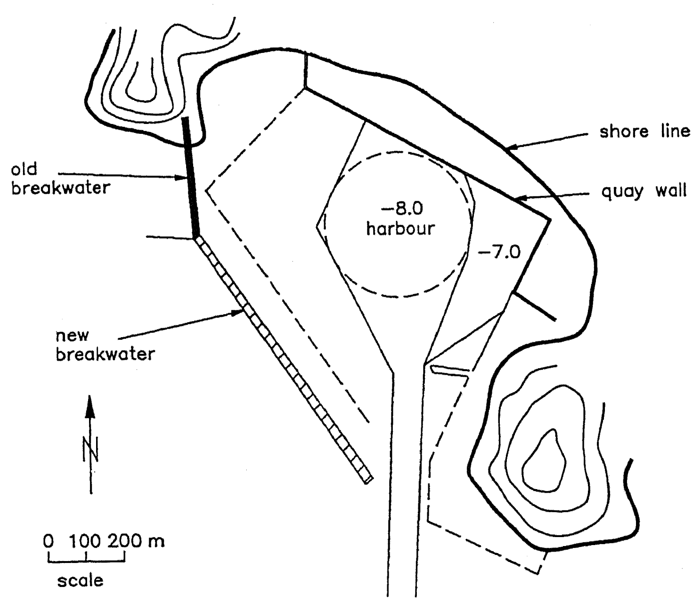


Figure 1 General Layout of Breakwater

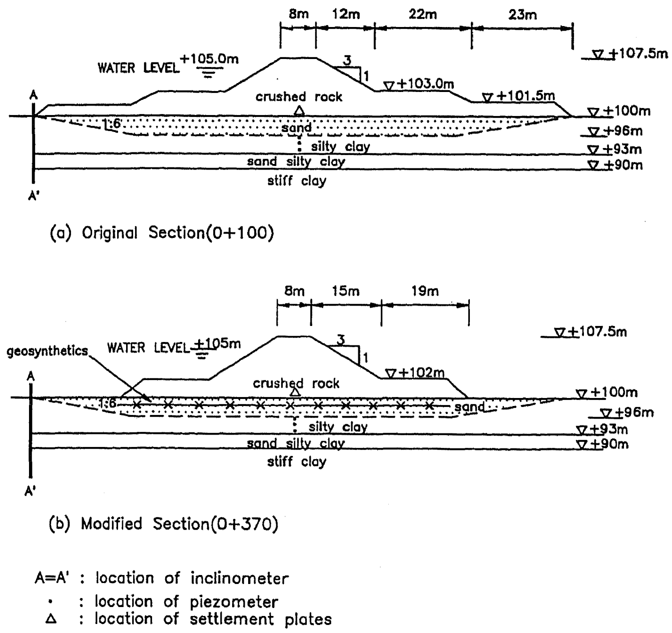


Figure 2 Design of Breakwater

other sections of same design that suffered local collapses during construction. Hence Sect 0+100 was considered as approaching an unstable condition despite collapse had not yet occurred. Instead of a waiting period of six months, a waiting period of 14 months was adopted. Stage II construction commenced from 20th June, 1985 and was completed in 40 days. The settlement at the foundation surface was about 1500 mm and the horizontal displacement was about 500 mm at Stage II completion. Based on the observed performance of Sect 0+100 and the frequent collapse of other sections of similar design, one can conclude that the original unreinforced design is on the verge of instability at completion of Stage I.

Construction of Sect 0+370, a reinforced design, commenced on 4th September, 1984. Stage I construction, i.e. raising the breakwater to a height of 5.5m, was completed in 45 days. The settlement at the end of Stage I was only 250 mm. Only six months waiting period was adopted before the commencement of stage II construction. The settlement of the foundation surface at completion of Stage II was about 760 mm. No collapse was encountered during the construction of this section or other sections of similar design. It can be concluded that the reinforced design is stable.

MATERIAL PROPERTIES

a) Soil:

The parameters for the three clay layers are (λ, κ, M) for the Cam-Clay model, permeability, void ratio and unit weight. These parameters were obtained directly from laboratory testing prior to the construction of the embankment and reproduced as Table I.

The sand blanket is a medium grained sand with a friction angle of 35°. The crushed rocks for constructing the embankment was observed to have a angle of repose of 40°. The hyperbolic nonlinear elastic model developed by Duncan and Chang (1970) was used to describe the stress strain responses of both the sand fill and the crushed rock. The parameters as reported by Shi (1988) are presented in Table II.

b) Geosynthetic Reinforcement:

The reinforcement was a woven geosynthetics. The reinforcement was tested in air. Strain rates of 2.4mm/min and 9.6mm/min were

Table I: Parameters for Foundation Soils

Parameter	Symbol	Material		
		Silty Clay	Sandy Clay	Stiff Clay
Virgin compression index	λ	0.42	0.294	0.126 *
Recompression Index	k	0.012	0.017	0.020
Permeability (10^{-9} m/s)	k_y	0.27	1.6	0.46
Slope of critical state line	M_o	0.88	0.96	1.04
Initial void ratio	e_o	2.44	1.64	1.09
Unit Weight	γ	1.50	1.76	1.81
Liquid limit (%)	W_L	53	-	-
Plastic limit (%)	W_p	28	-	-
Plasticity index (%)	pl	25	-	-

* Likely to be affected by sample disturbance

used to check the rate effect, which was found to be small (Yue et al, 1986). The ultimate tensile strength was 38.9kN/m at 26.86% strain. The load extension relationship was evidently non-linear. At the breakage of the reinforcement, it was noted that the value of $\partial T/\partial \epsilon_a$ was not equal to zero, where T is the reinforcement tension and ϵ_a is the axial strain. These characteristics were modelled by truncating the modified hyperbolic equation (Prevost and Keane, 1990) with a tensile strength (T_u) of 39kN/m and a strength ratio, T_0/T_u, of 1.1, where T_0 is the hypothetical strength obtained by extending the stress strain curve to a state of $\partial T/\partial \epsilon_a = 0$. An initial stiffness ($E_o$) of 269kN/m was used. Thie modelling of tensile test data is illustrated in Fig. 3. The detailed equations are contained in Li (1992).

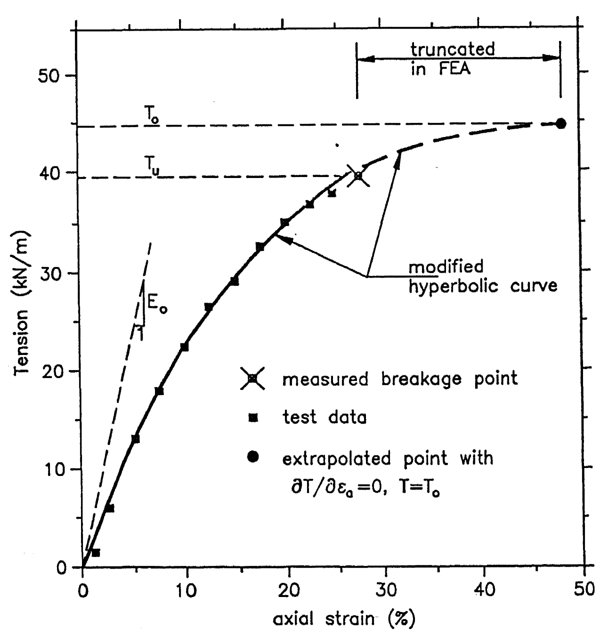


Figure 3 Load Extension Relationship of Geosynthetic

Table II: Parameters for Sand Fill and Crushed Rocks

Parameter	Sand Fill	Crushed Rock
Cohesion (kPa)	0	0
Friction angle (deg)	35	40
Modulus number	500	1050
Modulus exponent	0.64	0.5
Failure ratio	0.80	0.78
Unit weight (kN/m³)	20	2.2
Poisson's ratio	0.3	0.3

ANALYTICAL STUDIES

Attempts were made to 'predict' the observed behaviour of Sect 0+100 (unreinforced) and Sect 0+370 (reinforced) by three different methods of analysis. These analyses are:

i) limit equilibrium analysis,
ii) strain hardening FEA (Finite Element Analysis), and
iii) strain softening FEA.

Limit Equilibrium Analysis:

The slip surface used in the limit equilibrium analysis consists of a circle through the foundation clay and a log-spiral through the embankment. The log-spiral has to be described by an equation conforming to plasticity requirements. This type of slip surface was first proposed by Leshchinsky (1987) for reinforced embankment on soft clay. The advantages of such a slip surface were discussed in Leshchinsky (1987) and Lo & Xu (1992). The critical slip surface, i.e., one with the lowest factor of safety, was determined by a numerical optimization technique as detailed in Lo & Xu (1992). The undrained cohesion used in the limit equilibrium calculation was taken as the value prior to the construction. To maintain consistency with the finite element analysis, it was derived from the effective stress parameters using the modified Cam-Clay model. The reinforcement tension at failure was taken as, optimistically, the tensile strength.

Strain Hardening Finite Element Analysis:

Fully coupled analyses were conducted to allow close modelling of pore water pressure dissipation during construction. The flow of pore fluid is described by Darcy's equation whereas the deformation of the soil skeleton is described incrementally by an elasto-plastic matrix. The finite element formulation and numerical scheme are given in Li (1992). The stiff clay was replaced by an impermeable rigid bottom boundary. Both the sand blanket and the embankment fill were modelled as fully drained. Due to the presence of the sand blanket, the foundation surface was modelled as permeable and hence no pore water pressure could be generated. The modified Cam-Clay model was used for the foundation clay whereas the Duncan and Chang (1970) non-linear elastic model was used for the sand fill and embankment material. Since no horizontal permeability data was available, the ratio of horizontal permeability to vertical permeability of the foundation clays was assumed to be 2. Parametric studies by Li (1992) indicated that the value of k_x/k_y only has a slight influence on the behaviour *during construction*. The ability of this type of analysis in predicting the responses of stable embankments was validated by comparison with centrifugal testing results (Li 1992).

One important issue in modelling the embankment construction to failure is the criterion used to determine, from the computed results, embankment instability. Detailed discussions on the 'numerical identification' of embankment failure are contained in Li (1992). In this paper, an embankment is considered to have failed if further increase in *net height* with additional fill is not possible. This criterion proposed by Rowe (1987) is considered as most objective and appropriate. However, updating of mesh co-ordinates is essential.

Strain Softening Finite Element Analysis:

The formulation is identical to strain hardening analysis except a strain softening model is used for the foundation clay. A Cam-Clay type elliptical yield surface was still used but the slope of the critical state line, M, reduced with post-peak shearing. This process can be expressed as:

$$M = M_o(1 - \xi\Delta) \tag{1}$$

$$\xi = 1 - exp[-\rho(\epsilon^p - \epsilon^p_{peak})^2] \leq 1 \tag{2}$$

where M_o denotes the value before onset of strain softening, subscript 'peak' denote peak value, ϵ^p is the plastic shear strain. Two more soil parameters, Δ to described the maximum amount of softening, and ρ to describe the relative rate of strain softening are needed. Although Eqn (2) requires infinite shear strain to achieve a state of complete strain softening, i.e., $\xi = 1$, only a post-peak shear strain in the range of 20% to 30% is needed to achieve a ξ value in excess of 0.95 for a wide range of reasonable ρ values. The contraction of the elliptical yield surface during strain softening is described by:

$$p'_c = p'_{c0} exp(\frac{\epsilon^p_v}{x} - \xi\Delta) \tag{3}$$

where $x = (\lambda - \kappa)/(1 + e_0)$, p'_c is the intersection of the current yield surface with the p' axes, p'_{c0} is the intersection of the initial yield surface with the p' axes, and e_0 is the initial void ratio. The above equations, ensure continuity at ϵ^p_{peak} when the model changes from strain hardening to strain softening. The evolution of yield surfaces is illustrated in Fig. 4. To avoid possible numerical problems in the vicinity of ϵ^p_{peak} and ambiguity in the definition of loading and unloading, a strain space formulation was used for both the hardening and softening analysis. However, the original site investigation program does not provide data for the determination of ρ and Δ. Hence $\rho = 40$ was assumed in the analyses and the effect of Δ in the range of 0.1 to 0.5 on the predictions was studied. Furthermore, a finer mesh was needed as discussed in Li (1992).

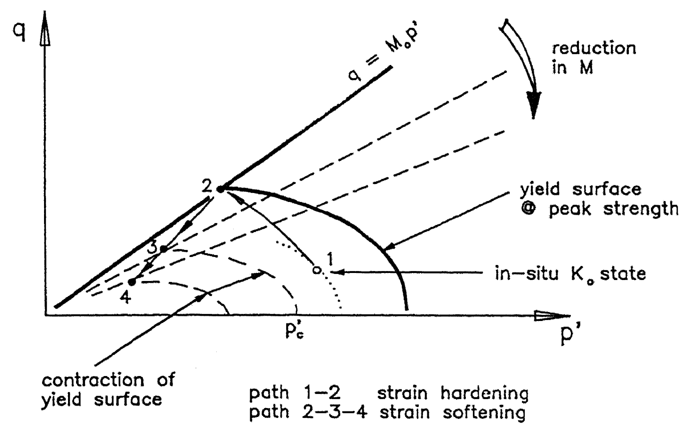


Figure 4 Evolution of Yield Surfaces

COMPUTED AND MEASURED RESULTS

Original Unreinforced Design at Sect 0+100:

The limit equilibrium analysis gave a factor of safety of 1.22 for Stage I. This value is supposedly conservative due to partial drainage, hence increase in undrained shear strength, during construction. This implies that the embankment has to be stable at the completion of stage I, which contradicts the observed instability. Many factors may be involved in this discrepancy. One possibility is the inherent shortcomings of any limit equilibrium analysis. Hence the 'factor of safety' of the breakwater was calculated by FEA using factored strength parameters. The factored strength parameters used in the analysis were $tan\phi'/F$, M/F and T_u/F, where F denotes Factor of Safety. Thus for a given embankment height, the 'factor of safety' is the value of F that reduces the strength parameters so that failure is just predicted by the FEA. This procedure gave F=1.3 for Stage I in a strain hardening FEA. This value was larger than the factor of safety calculated from the limit equilibrium stability analysis. This is expected because the coupled FEA takes into account the partial drainage, hence the enhancement of the undrained shear strength, during construction. Again, the finite element analysis based on the strain-hardening Cam-Clay model fails to predict the impending instability of the embankment at the completion of Stage I.

The observed development of foundation settlement and horizontal displacement near the toe during Stage I construction are compared with predictions in Fig. 5. Initially the discrepancies between observation and prediction by strain hardening FEA are small. However, the discrepancies increase with the embankment height. At the completion of Stage I, only 450 mm of foundation settlement was predicted but the observed settlement was 740 mm and the trend indicated an increasing rate of settlement.

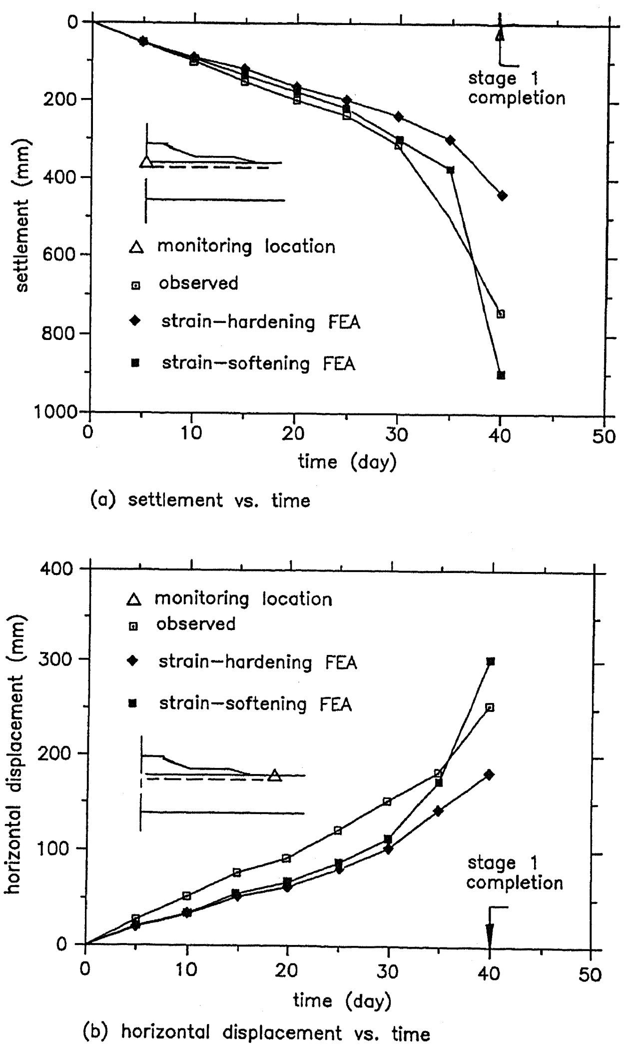


Figure 5 Displacements @ Sect 0 + 100 for Stage 1

The inevitable conclusion is that strain hardening FEA fails to adequately predict the behaviour of the embankment constructed to impending instability, despite reasonable prediction of movement can be achieved when the embankment height is significantly below the failure height. A study of the strain field in the foundation clay indicated high shear strain, in the order of 14%, as Stage I completion was approached. At such a strain value, strain softening may have a significant effect on the embankment performance. Furthermore, any strain softening will further increase the magnitude of shear strain. Hence a series of strain softening FEA were conducted to study the effects of strain softening. It was found that if $\Delta = 0.3$ was assumed, the strain softening FEA indicated impending instability at the completion of Stage I.

Hence, the development of displacements during Stage I construction calculated using $\Delta = 0.3$ are compared to the observed values in Fig. 5. Good agreement is achieved for both settlement and horizontal displacement during stage I construction. Analysis was also conducted to simulate the waiting period of 14 months followed by Stage II construction for Sect 0+100. Excellent agreement is achieved for the development of foundation settlement (Fig. 6) and generation of excess pore water pressure (Fig. 7). A settlement of 1560 mm was predicted. This predicted settlement is very close to the observed value of 1500 mm. The excess pore water pressure at 5 m below foundation surface was predicted to be 49 kPa, only 6% difference from the observed value of 52 kPa. The horizontal displacement profiles near the toe of the embankment immediately after completion of Stage I and II are presented in Fig. 8. In general the strain hardening FEA consistently under-estimated the horizontal displacements whereas the strain softening FEA consistently over-estimated the displacements. The strain softening FEA, however, gave better predictions. Immediately after completion of Stage I, the maximum horizontal displacement was 300 mm from observation, 230 mm from strain hardening FEA and 350 mm from strain softening FEA. The maximum horizontal displacement at Stage II completion was 560 mm from observation, 460 mm from strain hardening FEA and 580 mm from strain softening FEA.

Modified Reinforced Design at Sect 0+370:

The limit equilibrium analysis gave factors of safety of 1.25 and 1.12 at the completion of Stage I and Stage II respectively. Hence the observed stability of the embankment is predicted although a factor of safety of 1.12 may be considered as marginal to ensure stability. The factors of safety computed using factored parameters with strain hardening FEA are 1.32 and 1.20 for Stage I and Stage

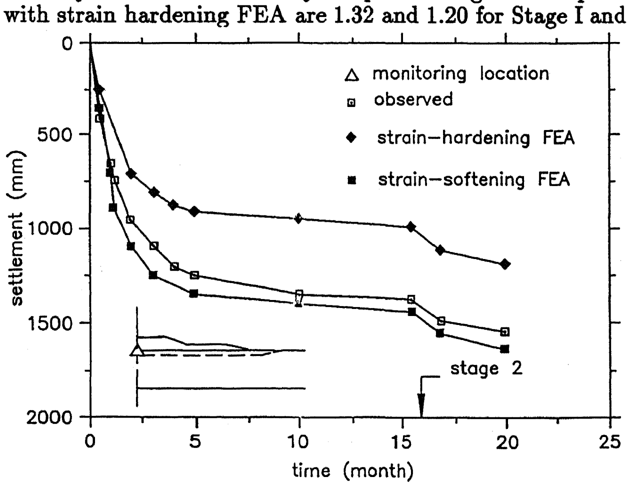


Figure 6 Foundation Settlement V_s Time @ Sect 0 + 100

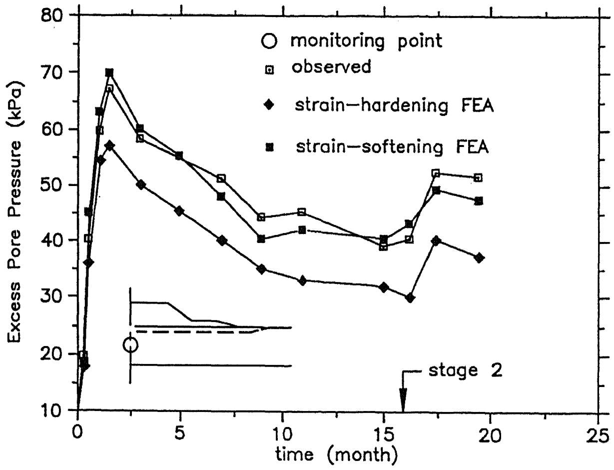


Figure 7 Excess Pore Water Pressure V_s Time @ Section 0 + 100

II respectively. Hence the observed stability is considered as well predicted. The observed and predicted settlements at foundation surface during Stage I construction are compared in Fig. 9. A comparison between the observed and predicted lateral movement profiles are presented in Fig. 10. In general the observed trend is well predicted by the strain hardening FEA. The discrepancy between observation and prediction by strain hardening FEA is small in the early stage of the construction when the embankment had a high safety margin. However the discrepancies increase with raising of the embankment height, i.e., reduction in safety margin. For example at the end of Stage II, the observed maximum lateral displacement was 410 mm but only 270 mm was predicted by the strain hardening FEA. Indeed the strain hardening FEA consistently under-estimated displacements.

Strain softening FEA was conducted using $\Delta = 0.3$ and $\rho = 0.3$, i.e., identical to those used for the unreinforced section. Good agreement between observation and prediction is achieved for both vertical and lateral displacements (see Figs. 9-10). For example, the maximum lateral displacement at completion of Stage II was 450 mm, i.e., only 11% difference from the observed value of 410 mm. The generation of excess pore pressures at a depth of 5m (Fig. 11) is also well predicted by the strain softening FEA. However, the strain hardening analysis consistently under-estimated the excess pore water pressure. The strain softening FEA was also superior to the strain hardening FEA in predicting settlement for the whole construction period. The settlement immediately after the completion of Stage II was 760 mm from measurements, 810 mm from strain softening FEA, but only 530 mm from strain hardening FEA.

DISCUSSIONS AND CONCLUSIONS

A case study of a breakwater on soft sediments is presented in this paper. The breakwater consists of two portions: an unreinforced portion and a reinforced portion. The unreinforced portion is on the verge of instability at completion of Stage I. The reinforced portion has a low but adequate safety margin at completion of Stage I and Stage II. The findings from this study can be summarized below.

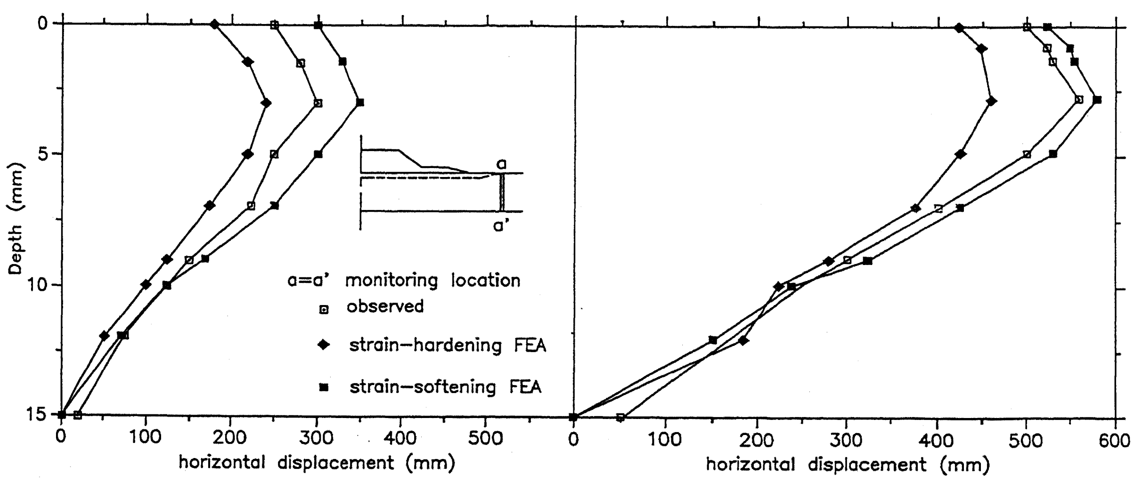
i) Limit equilibrium analysis may not always be able to predict accurately the stability, or the lack of it, of an embankment on soft clay.

ii) A coupled FEA using a Cam-Clay (strain hardening) model can reasonably predicts the response of the reinforced design at Sect 0+370 during construction. However, the strain hardening FEA consistently under-predicts the movements and the discrepancies increase with the embankment height.

iii) The strain hardening FEA gives fair prediction of the responses of the unreinforced embankment at Sect 0+100 when the embankment height is low. However, it fails to predict the impending instability of the unreinforced design at Sect 0+100 at the completion of Stage I. Indeed, the strain hardening FEA predicts movements significantly lower than those manifested during the last 10 days of Stage I construction.

iv) The strain-softening FEA using $\rho = 40$ and $\Delta = 0.3$ gives good predictions of *all* the observations of both the unreinforced and reinforced sections.

It needs to be recognised that the predictions made in this paper are only Class 'C' predictions. However, nearly all the input parameters are measured by laboratory tests that do not require subjective interpretation and completed prior to this study. The only important input parameters that needed to be assumed are ρ and Δ. These parameters can be considered as obtained by back-analysing, with a strain softening FEA, the impending instability of the unreinforced embankment at completion of Stage I. These two back-analysed parameters enables satisfactory prediction of *all* other observations using a strain softening FEA. Hence, the role of strain softening is a plausible expanation for the collapse of the unreinforced embankment constructed to the original design. Furthermore, this case study illustrates the difficulties in generalising the predictive reliability of a given analysis. It is also interesting to note that the factors of safety of the unreinforced and reinforced sections are very close at completion of Stage I. However, the unreinforced section is on the verge of instability whereas the reinforced design is proved to be stable. This may implied the geosynthetic reinforcement 'suppresses' progressive failure. It is also noted that



(a) End of Stage 1.

(b) End of Stage 2.

Figure 8 Horizontal Displacement Profiles @ Sect 0 + 100

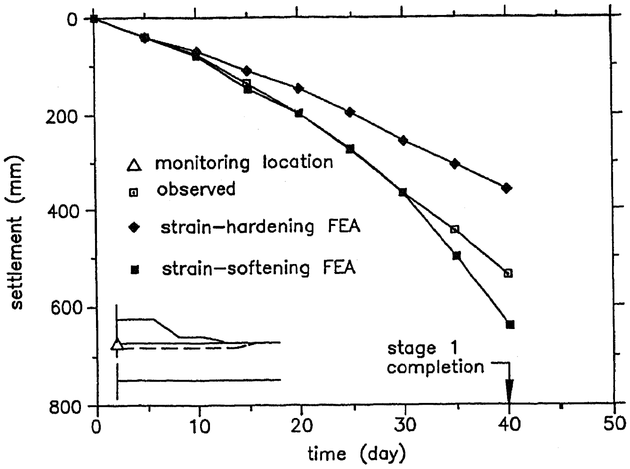


Figure 9 Foundation Settlement @ Sect 0 + 370 for Stage 1

this breakwater, even the portion with a reinforced design, has a low safety margin. Hence it appears from this study that it may be easier to predict the responses of a stable system with a high safety margin.

ACKNOWLEDGMENT

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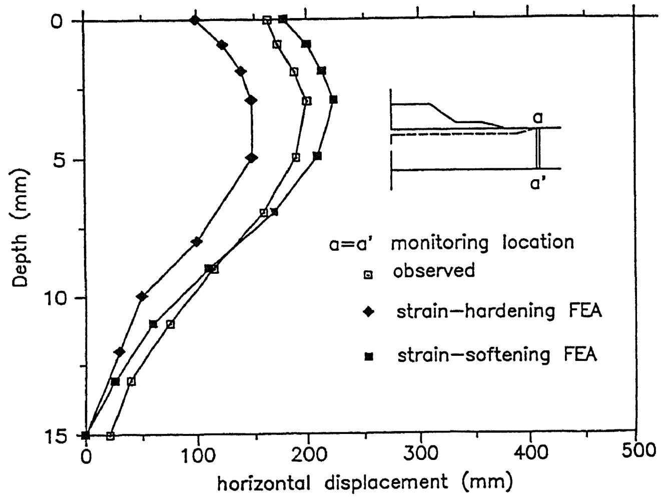
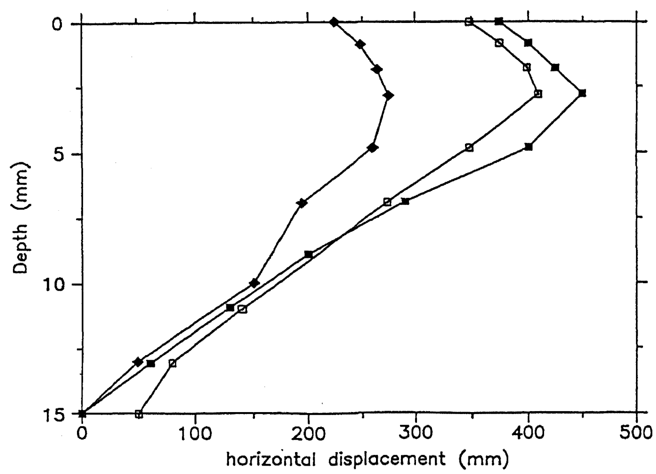


(a) End of Stage 1.



(b) End of Stage 2.

Figure 10 Horizontal Displacement Prifles @ Sect 0 + 370

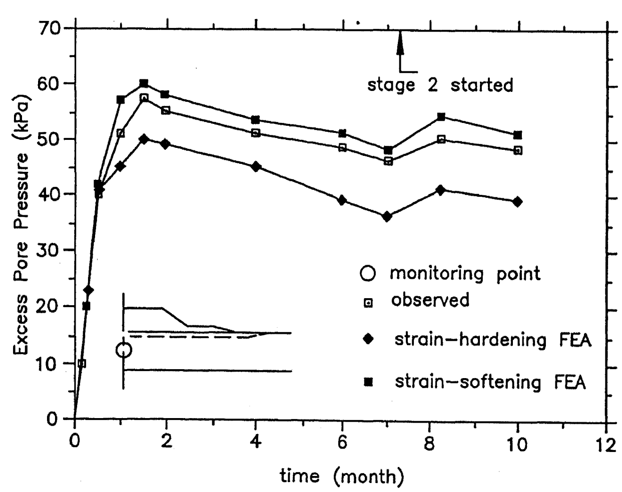


Figure 11 Excess Pore Water Pressure V_s Time @ Sect 0 + 370