Undrained shear strength gain with consolidation at soft and sensitive soil sites

Undrained le gain de force de cisailles avec la consolidation au sol doux et sensible place

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ABSTRACT

Staged construction and ground improvement are two strategies adopted in constructing high embankments at sites underlain by soft and sensitive soils. Usually the gain in shear strengths are estimated from poorly documented local experience database and an elaborate scheme of construction monitoring is implemented to ensure the pace of construction does not lead to remolding and progressive failure. A procedure based on the well known SHANSEP approach has been proposed herein for estimation of undrained shear strength as a function of embankment height and rate of construction. To illustrate its use, the procedure was utilized in predicting the embankment response at three soft soil sites. One of these embankments performed satisfactorily while the others failed during construction. Use of the undrained shear strengths in limit equilibrium slope stability assessments was found to provide results which were in qualitative agreement with the observed embankment behavior.

RÉSUMÉ

La construction montés et amélioration de sol sont deux stratégies adoptées dans construire les hauts remblais aux sites sous-tendus par les sols doux et sensibles. D'ordinaire le gain dans les forces de cisailles est estimé de la base de données d'expérience locale mal documentée et un arrangement soigné d'interception de construction est appliqué pour garantir le rhythme de construction ne mene pas à remouler et à l'échec progressif. Une procédure fondée sur l'approche de SHANSEP renommée a été proposée en ceci pour le jugement d'undrained force de cisailles comme une fonction de hauteur de remblai et le taux de construction. Pour illustrer son usage, la procédure a été utilisée dans prédire la réponse de remblai à trois sites de sol doux. Un de ces remblais exécutés satisfaisamment pendant que les autres a échoué pendant la construction. L'usage des undrained forces de cisailles dans les évaluations de stabilité de pente d'équilibre de limite ont été trouvées pour fournir des résultats qui étaient dans l'accord qualitatif avec le comportement de remblai observé.

Keywords: Undrained shear strength, consolidation, staged construction, marine clay.

1 INTRODUCTION

In staged construction embankments on soft soil sites the rate of fill placement is controlled by the rate of increase of the undrained shear strength of the underlying soil layers as a result of consolidation. A slow enough construction rate is maintained so as to preclude instability at any stage of construction. If the shear strength gain is not appropriately accounted for in the design, the design is unlikely to be economical or even feasible. On the other hand, if the rate of consolidation-related strength gain of the foundation soils is overestimated, instability could be triggered during construction. An elaborate numerical can be used to account for drainage during construction and the consequent strengthening of soils (e.g., Chung et al. 2006, Indraratna et al., 1992), in a routine project undertaking such an exercise is difficult. Usually the gain in shear strength is estimated from poorly documented local experience database and an elaborate scheme of construction monitoring is implemented to ensure the pace of construction does not lead to remolding and progressive failure. A simple framework for assessing the gain in undrained shear strength due to staged construction and ongoing consolidation is therefore preferable.

The objective of this study is to develop a simple method that accounts for consolidation-related strengthening of soils for assessing the stability of earth structures founded at soft soil sites. Towards this, published case histories reporting performance of embankments constructed over soft fine grained foundation soils have been used. The results of the stability analyses using estimates of undrained shear strength from the proposed procedure have been compared against observed embankment performance at three sites underlain by soft clayey soils of marine origin to validate the approach.

2 PROCEDURE OUTLINE

For estimating the undrained shear strength after the development of a certain amount of settlement a fictitious surcharge that would give an ultimate settlement equal to the observed settlement is back figured first assuming the fictitious surcharge to have an identical footprint as that of the preload fill. From the result the stress increments within the layers of foundation soils are estimated assuming the 2 (vertical) on 1 (horizontal) load-dispersion or using the elastic solution, e.g., the Newmark's influence chart. The apparent over consolidation ratio is then computed for foundation soil layers using

$$OCR = \left(\sigma'_v + \Delta\sigma'_v\right)/\sigma'_v \tag{1}$$

where OCR is the apparent overconsolidaiton ratio, σ'_v is the original effective vertical stress and $\Delta\sigma'_v$ is the effective vertical stress increment within a certain soil layer due to the fictitious surcharge.

The undrained shear strength can now be estimated following the SHANSEP (Ladd et al., 1977) approach in which

$$s_u/\sigma'_v = S \times OCR^m \tag{2}$$

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where s_u is the undrained shear strength, and SHANSEP parameters S and m are obtained from test data as explained later. The ratio, s_u/σ'_v, is often referred to as the undrained shear strength ratio. For estimating s_u values of OCR obtained from Equation 1 are used if these values exceed those obtained from consolidation test. Otherwise, the estimates from laboratory consolidation tests are used.

A similar approach is used for shear strength forecasting. For instance, the effective vertical stress at the center of layer prior to embankment construction, σ'_v, and the vertical effective stress increment due to embankment construction, $\Delta\sigma'_v$, are obtained at the centre of the soil layers. The settlement at a certain instant of time is then forecasted after fill placement can be estimated from the Terzaghi one dimensional consolidation theory using the hydraulic conductivity obtained from borehole permeability testing or laboratory tests on undisturbed samples and the compression index obtained from laboratory tests after applying the corrections to account for sampling disturbance. The fictitious surcharge can then back figured for which the ultimate settlement is equal to the forecasted settlement. The OCR and the undrained shear strength can then be estimated proceeding as explained earlier.

It should be noted that the proposed procedure relies on the SHANSEP framework. The framework is considered to be inapplicable for sensitive fine grained soils (Jamiolkowski et al., 1985). However, for the soils considered in this research characterized with sensitivities of up to 8, SHANSEP framework was found to perform satisfactorily.

3 UNDRAINED SHEAR STRENGTH RATIO

For obtaining parameters S and m (Equation 2), the s_u/σ'_v values obtained from field vane shear testing or laboratory UU testing of undisturbed or high quality soil samples are plotted against the corresponding overconsolidation ratios obtained from one dimensional incrementally loaded consolidation testing of undisturbed or high quality soil samples conducted in the laboratory. The laboratory consolidation test data are corrected for sampling disturbance following the Casagrande-Schmertmann approach (Casagrande, 1936; Schmertmann, 1953). The data are fitted to the relationship of Equation 2 to obtain parameters S and m.

The parameters for a number of soil types obtained in this manner are listed in Table 1. Relationships representing the first six soil types were developed by Ladd et al. (1977). The data pertaining to the remaining soil types obtained from the sources listed in Table 1 were fitted to the SHANSEP relationship (Equation 2) by these writers. The data are presented in Figure 1.

Table 1. Soil parameters of various clays

Site	S, m	Goodness of fit (r^2)	Source
Marine Organic clay	0.29, 0.81	–	Ladd et al. 1977
Bangkok clay	0.28, 0.76	–	Ladd et al. 1977
Atchafalya clay	0.24, 0.79	–	Ladd et al. 1977
AGS CH clay	0.27, 0.70	–	Ladd et al. 1977
Boston Blue clay	0.22, 0.70	–	Ladd et al. 1977
Connecticut valley varved clay	0.17, 0.71	–	Ladd et al. 1977
West Breakwater	0.21, 0.96	0.84	Chung et al. 2006
East Breakwater	0.20, 1.08	0.97	Chung et al. 2006
Muar	0.20, 0.80	0.95	Indraratna et al. 1992
Suvarnabhumi	0.15, 1.04	0.95	Bergado et al. 2001
k18 (CL/CH)	0.34, 0.62	0.62	Roy and Singh 2008
k26 (CL/CH)	0.15, 1.08	0.84	Roy and Singh 2008

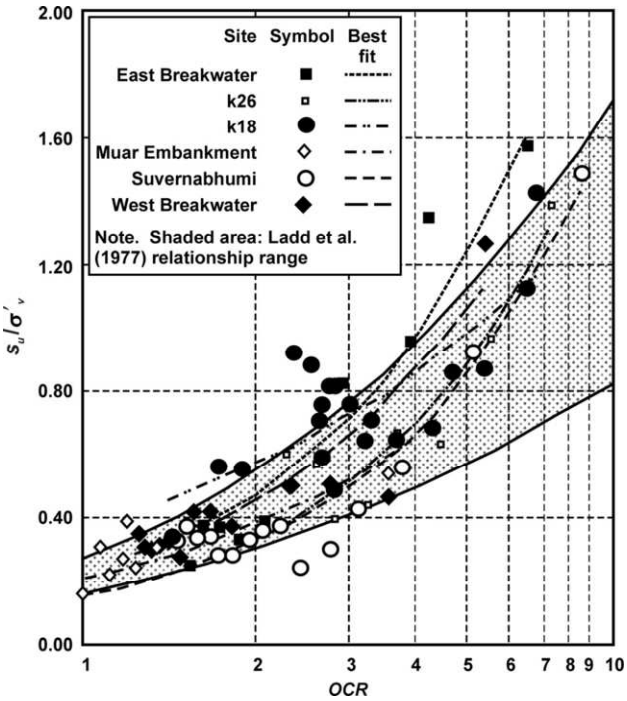

Figure 1. s_u/σ'_v – OCR relationships

Using such s_u/σ'_v – OCR relationships and the overconsolidation ratio estimated for a given stage of embankment construction obtained according to the procedure outlined in the preceding section, the undrained shear strength ratio for that stage of embankment construction is estimated.

4 VALIDATION

The proposed procedure was utilized in predicting the embankment response at three soft soil sites. The details of the exercise are provided in following subsections.

4.1 *East and West Breakwater Structures*

Two breakwater structures were constructed in the new Busan port in South Korea (Chung et al. 2006). A normally consolidated clay deposit, 35 to 55 m in thickness, underlies the structures. For the breakwater sites, the consistency of the clay varied from very soft to firm as the depth increased and was separated from an irregular thickness of hard clay by a thin sand layer of maximum thickness 4 m. The total thickness of soft and firm clay layers underneath the failed portion of the west breakwater was 36.5 m. The corresponding thickness at the east breakwater site was 27.5 m. The depth of the sea bed was about 9 m for the west breakwater site and 11.4 m for the east breakwater site with respect to the lowest low water level. The upper layer of the deposit was excavated and replaced with sand in order to increase the stability of the rock fill structure (Figure 2). The original design of breakwaters indicated a limit equilibrium factor of safety of 1.32. The east breakwater was successfully constructed based on the original design, while a portion of west breakwater failed during the final stages of construction. The total thickness of soft and firm clay layers at the failed part of the west breakwater was 36.5 m. The sensitivity estimated from cone penetration test data following to Robertson and Campanella (1986) is between 2 and 6 for west breakwater site. The corresponding estimates for the east breakwater site are between 3 and 8.

Chung et al. (2006) analyzed the structures using elaborate finite element models accounting for the consolidation related increase in the undrained shear strength during construction. They also present the results of limit equilibrium stability

assessments of these structures. However, in reassessing limit equilibrium stability of the breakwater structures the consolidation-related increase in undrained shear strength was not accounted for. Based on the results, use of unconservative input parameters by the original designers was identified as the possible cause of failure of the west breakwater structure.

Using the estimated SANSHAP parameters listed in Table 1 and the construction sequence it was estimated that the effect of the staged construction is equivalent to that of instantaneously imposed surcharge pressures of 86 kPa and 115 kPa, respectively, for west and east breakwaters placed over an identical footprint as those of the rock fill structures. Accordingly, the undrained shear strengths beneath the rockfill structures were computed (Tables 2 and 3). These shear strength estimates were used for the soil underneath the embankment. For the soils outside the breakwater footprints, no consolidation-related shear strength gain was considered.

Table 2. Estimation of shear strength gain for west breakwater

Depth	σ	σ'	$\Delta\sigma'$	*OCR*		*OCR*	s_u
m	kPa	kPa	kPa	Eq 1	Lab		kPa
1	15.6	5.8	81.9	15.15	1.99	15.15	20.55
3	46.8	17.4	80.2	5.62	1.21	5.62	24.55
5	78.0	29.0	78.2	3.70	1.63	3.70	15.50
7	109.2	40.5	76.1	2.88	1.45	2.88	20.55
9	140.4	52.1	73.9	2.42	1.11	2.42	26.00
11	171.6	63.7	71.7	2.12	2.48	2.48	31.90
13	205.8	78.2	69.4	1.89	1.08	1.89	29.20
15	240.0	92.9	67.1	1.72	1.43	1.72	38.90
17	274.3	107.5	64.9	1.60	1.19	1.60	44.90
19	308.6	122.2	62.7	1.51	1.14	1.51	33.75
21	342.9	136.9	60.6	1.44	1.05	1.44	44.64
23	377.2	151.6	58.6	1.39	1.16	1.39	47.56
25	411.5	166.3	56.6	1.34	1.10	1.34	49.75
27	445.9	181.0	54.7	1.30	1.04	1.30	55.67
29	480.2	195.8	53.0	1.27	1.07	1.27	68.64

Table 3. Estimation of shear strength gain for east breakwater

Depth	σ	σ'	$\Delta\sigma'$	*OCR*		*OCR*	s_u
m	kPa	kPa	kPa	Eq 1	Lab		kPa
1	15.6	5.8	104.3	19.02	1.43	19.02	25.00
3	46.8	17.4	101.6	6.85	1.03	6.85	27.34
5	80.7	31.6	98.7	4.12	1.25	4.12	30.30
7	114.8	46.1	95.8	3.08	1.15	3.08	38.00
9	149.0	60.8	92.9	2.53	1.06	2.53	34.61
11	183.3	75.4	89.9	2.19	2.03	2.19	29.34
13	217.7	90.1	87.1	1.97	1.15	1.97	30.00
15	252.0	104.9	84.3	1.80	1.20	1.80	38.90
17	286.4	119.6	81.5	1.68	1.01	1.68	44.90
19	320.7	134.3	78.9	1.59	1.12	1.59	33.75

Based on these results, two analytical models shown in Figures 2 and 3 were set up for stability analyses. Limit equilibrium stability was assessed using software package XSTABL Version 5.1 (Interactive Software Designs, Inc. 1994) according to the modified Bishop approach. The results of the stability analyses, also presented in Figures 2 and 3, indicate the likelihood of instability at the west breakwater site and

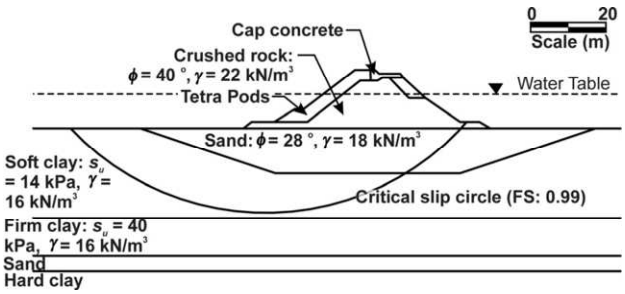

Figure 2. West Breakwater

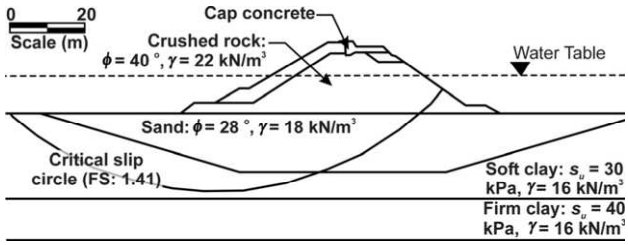

Figure 3. East Breakwater

satisfactory performance of east breakwater. These results are therefore consistent with observed performance.

4.2 *Muar Embankment*

A test embankment constructed over the 10 to 20 m thick soft marine clay deposits with undrained shear strengths as low as 8 kPa and water contents as high as 100 % for the construction of the Malaysian North South Expressway across the Muar plain in Malaysia (Indraratna et al. 1992). The embankment height was increased until the initiation of a deep seated circular slip. At the time of failure the embankment was 5.5 m in height. A tension crack was observed, which penetrated the entire thickness of the embankment. However, it is unclear whether the crack was developed during the failure process or it was a pre existing feature. There was a 2-m thick weathered crust at this site above a 16.5-m thick layer of soft silty clay. The deeper layer could be divided into an upper very soft sub layer and a lower soft sub layer. Underneath the lower clay layer was a 0.3 to 0.5-m thick layer of peat underlain in turn by stiff sandy clay. The clayey succession was found to terminate over a dense sand layer at about 22.5 m below ground level. The plasticity index was found to range between 40% and 50%. The minimum value of undrained shear strength as measured by field vane shear testing was about 8 kPa occurring at a depth of about 3 m. Thereafter the undrained shear strength increased approximately linearly with depth. The ground water table was found to be at 0.5 m below the ground surface. Field vane shear test data from the site indicate that the sensitivity of foundation soils is between 2 and 5.

Using the parameters listed in Table 1 and the construction sequence it was estimated that the effect of the staged construction is equivalent to that of an instantaneously imposed surcharge pressure of 35 kPa placed over an identical footprint as those of the rock fill structures. Accordingly, the undrained shear strengths beneath the embankment was computed (Table 4). These shear strength estimates were used for the soil underneath the embankment. For the soils outside the breakwater footprints, no consolidation-related shear strength gain was considered.

Table 4. Estimation of shear strength gain for Muar embankment

Depth	σ	σ'	$\Delta\sigma'$	*OCR*		*OCR*	s_u
m	kPa	kPa	kPa	Eq 1	Lab		kPa
1	16.5	14.5	23.6	2.62	7.57	7.57	22
3	48.0	26.4	19.3	1.73	3.79	3.79	15
5	79.2	38.0	14.3	1.38	1.05	1.38	9.8
7	110.3	49.4	10.4	1.21	1.01	1.21	11.5
9	141.3	60.9	7.6	1.13	0.99	1.13	13.4
11	172.3	72.3	5.7	1.08	0.83	1.08	15.4
13	203.4	83.7	4.3	1.05	0.72	1.05	17.4
15	234.4	95.1	3.3	1.03	0.63	1.03	19.5
17	265.4	106.5	2.6	1.02	0.56	1.02	21.7
19	297.4	118.8	2.1	1.02	0.51	1.02	24.1

Based on these results, an analytical model shown in Figure 4 was set up for stability analyses. The results of the stability analyses based on the modified Bishop approach as incorporated in XSTABL, also presented in Figures 4, indicate that at the stage when the instability was triggered the computed

factor of safety against deep seated circular slip was 0.95. Stability analyses also indicate that the embankment was expected to lose stability once its height reached 5.1 m. Inclusion of a vertical tension crack bisecting the entire thickness of embankment lead to an estimate of 4.6 m for the maximum height of a stable configuration. The stability of the embankment is therefore slightly underestimated.

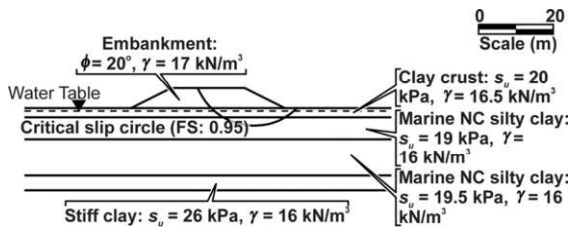

Figure 4. Muar embankment

Interestingly, Indraratna et al. (1992) present two attempts of estimation of limit equilibrium stability of the embankment by others. In one of these assessments, which included the vertical tension crack cutting across the entire thickness of the embankment, the maximum height before loss of stability was estimated as 3.8 m. The other attempt, which ignored the presence of the tension crack, gave the maximum stable embankment height to be 5.0 m. The result obtained from the procedure proposed in this study therefore appears to be in slightly better agreement with observations than those obtained earlier.

5 CONCLUSIONS

A procedure has been proposed in the paper for assessing the increase in the undrained shear strength as a function of consolidated related gain in undrained shear strength with an intent to provide a simple tool for scheduling construction of embankments at sites underlain by soft fine grained soils. The procedure is based on the well known SHANSEP approach. The procedure is validated using three well documented embankment performance case histories from Southeast Asia. These sites are underlain by soft marine clays of sensitivities of up to 8. Two of the earth structures used in validation failed during construction while the other continued to perform satisfactorily. The results obtained from the use of the proposed procedure appear to be in reasonable agreement with the observed performance of these earth structures.

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