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Acceptance criteria for quality and densification control of reclaimed sandfill

Critères d'acceptation pour le contrôle de la qualité et de la densification de remblais hydrauliques de sable

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ABSTRACT
In selecting an acceptance criterion for quality and densification control of reclaimed sandfill for foundation purposes, one often faces a great difficulty. To overcome the problem, one needs to investigate the suitability of using penetration resistance such as the cone resistance for deriving suitable acceptance criteria based on settlement considerations. One usually has to make reference to observations of settlement of real or model foundations such as that in the plate load test. From a study conducted on a typical reclaimed sandfill in Singapore, acceptance criteria suitable for fill quality and densification control have been developed and presented graphically for two different types of reclaimed sandfill for potential applications in practice.

RÉSUMÉ
En sélectionnant un critère d'acceptation pour le contrôle de la qualité et de la densification de remblais hydrauliques en sable pour des fondations, on fait souvent face à d'importantes difficultés. Pour surmonter ce problème, on a besoin d'investiguer la possibilité d'utiliser la résistance à la pénétration telle que la résistance au cône pour en déduire des critères d'acceptation pour le tassement. Pour cela, on fait généralement référence à des observations de tassements de fondations réelles ou modèles comme des plaques de chargement. A partir d'une étude faite sur un remblai hydraulique en sable à Singapour, des critères d'acceptation pour le contrôle de la qualité et de la densification ont été développés et présentés graphiquement pour deux types de remblais hydrauliques.

1 INTRODUCTION

In the development of reclaimed sites covered by sandfill, it is required that the sandfill is capable of providing an adequately support of surface loading. Reclaimed sandfill are usually loose and/or highly heterogeneous and densification may be required in order to reduce future settlements. Penetration tests, such as the cone penetration test (CPT), are often used to characterize the sand. The difficulty of choosing a penetration resistance based acceptance criterion in fill quality or densification control usually lies on the specification of the minimum resistance.

Current practice of fill quality or densification control for sand fill is highly empirical, relying on past experience of similar developments. As the performance of most structures supported by shallow foundations in sand is usually governed by settlements, compressibility is often the dominant parameter in design. One therefore needs to study the relationship between penetration resistance and soil compressibility and to select an acceptance criterion based on the penetration resistance and its relevance to settlement for fill quality and densification control.

Jamiolkowski et al. (1988) investigated the use of cone penetration resistance (q_c) for quality control of granular fill and indicated that any control measure based solely on relative density would be inadequate. An acceptance criterion needs to be established based on soil responses or soil parameters that are directly related to the prediction of foundation performances.

Welsh (1986) in investigating different ground modification techniques in granular soils recommended that a q_c value of between 8 and 15 MPa be specified for any mechanically densified ground. An effective acceptance criterion, however, would require more specific values of penetration resistance.

The objective of this paper is to explore the suitability of q_c as a measure of sand compressibility and a parameter for developing an acceptance criterion for fill quality and densification control of reclaimed sandfill. One can make reference to the observed settlements of standard plates in the plate load tests a selected reclaimed site. By adopting an established analysis framework, such as Schmertmann (1970) and Schmertmann et al. (1978), suitable q_c-based acceptance criteria can be developed and presented in the form of design charts for potential applications in various types of reclaimed sandfill.

2 SANDFILL AT CHANGI EAST RECALAMTION SITE

The Changi East reclamation site, located next to the Changi Airport in Singapore, was constructed for future expansion of the airport and other facilities. In the land reclamation that involved the creation of 1500 ha of land, huge sandfill was placed over the existing seabed underlain by Singapore marine clay primarily by hydraulic pumping method.

In the investigation, sandfill that was used as surcharge for the preloading of the underlying clay along the proposed runway was selected. The sandfill was placed part by sub-aerial hydraulic pumping and part by direct truck dumping. The step-by-step removal of surcharge after the underlying foundation clay had been adequately preloaded provided an opportunity for the investigation of the compressibility of sand and load-settlement response of footing at various levels.

The sand used in the reclamation work was of marine origin and was relatively clean. The specific gravity of the sand solids was 2.66 and the grain particles were sub-angular. The characteristic particle size D_{60} was around 0.5mm, and the coefficient of uniformity C_u was between 2 and 6 for the sand, which was classified as SP. The carbonate content fluctuated between 4 and 16 % and the moisture content averaged around 8% in the sandfill.

The sandfill was stratified and varied across the 20m x 40m site, because two distinct placement methods were used at the three test locations, namely Lot 1, Lot 2, and Lot3. In the first two lots, the sand was placed by the sub-aerial hydraulic pumping method, and it was generally medium dense to very dense with the relative density (RD) ranging from 53 to 100%. In Lot 3, the sand was formed by direct dumping from trucks and the typical RD is between 30 and 40%, except the surface layer which had been compacted by traffic.

3 INVESTIGATION PROGRAM AND METHODS

The soil investigation included mainly cone penetration tests (CPTs) and plate load tests (PLTs) in the three test lots each measured 4 m x 4 m in area. CPTs were carried out from the top of the surcharge to determine the penetration resistance profiles. The compressibility of sand was assessed by plate load tests performed at 2 m intervals after each stage of surcharge removal. In-situ density tests were also carried out at localities around the PLTs to evaluate the relative density of the sand.

The plate load test involved jacking a circular steel plate of 0.1 m in thickness and 0.5 m in diameter against a kentledge and measured the resulting settlement. Three dial gauges erected near the rim of the rigid plate were used to monitor the plate settlements 30 min. after the load application, and the average of the three measurements was taken as the plate settlement. The load increment was selected at 125 kPa for the tests in the medium to very dense sand in Lots 1 and 2 and at between 25 and 50 kPa in the primarily loose sand in Lot 3. An unload-reload loop was incorporated in the initial stage of the test prior to the sequential loading that continued until the plate failed by bearing or reached a settlement of around 40 mm. The unload-reload minimized the disturbance effect from test preparation and provided data for calculating the reload modulus.

CPTs were carried out at the site following the standard test procedure specified in ASTM D3441-94 using an electrical cone penetrometer with an apex angle of 60°, a cross-sectional area of 10 cm^2, and a friction sleeve area of 150 cm^2. The cone was advanced hydraulically at a recommended rate of 20 mm/s using a CPT rig. Both the cone point resistance (q_c) and the unit shaft friction (f_s) were measured at a depth interval of 50 mm.

4 RESULTS OF INVESTIGATIONS

Figure 1 shows a typical load-settlement curve from a plate load test carried out in Lot 1. As the PLTs were performed after the removal of surcharge in stages, the Young's modulus calculated from the reload part of pressure-settlement curve represents the soil modulus E in recompression, or E_{oc}, The corresponding range of vertical strain was generally 0.1% to 0.5%, similar to the strain level in sand presented beneath conventional foundations under serviceability state, which is considered to be typically 0.1% in overconsolidated sand and 0.25% in normally consolidated sand (Baldi et al. 1988). Similar to the maximum shear modulus G_o, E_{oc} is relatively unaffected by stress history of the sand. E_{oc} is, therefore, selected as the reference elastic soil modulus for the subsequent verification of the distributions of vertical strain influence factor proposed by Schmertmann et al. (1978) in settlement analysis.

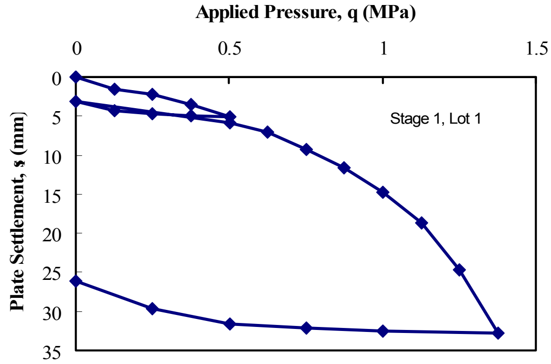

Figure 1. Typical load-settlement curve from PLT

Table 1 summarizes the values of reference modulus E_{oc} deduced from the PLTs. The modulus values are found to be generally higher for the hydraulically placed sand in Lot 1 and Lot 2, compared to those for the direct dumped sand in Lot 3.

Table 1: Reload modulus from PLTs

Test Sequence	Elevation (m)	E_{oc} (MPa)		
		Lot 1	Lot 2	Lot 3
Stage 1	11.2~12.2	51.29	62.97	60.60
Stage 2	10.80	77.27	33.45	17.69
Stage 3	8.80	54.95	62.75	27.82
Stage 4	6.6~6.8	62.97	53.95	15.94
Stage 5	5.50	60.05	64.10	20.50

Figure 2 shows the q_c profiles from the CPTs. The underlying sand present below the mean sea level (about EL +3 m), which was placed hydraulically underwater, has a q_c value of typically around 6 to 8 MPa. For the surcharge sandfill in Lots 1 and 2, which was placed hydraulically above water, the q_c value is much greater than 10 MPa, with high values exceeding 20 MPa. In contrast, for the sandfill that was placed by direct dumping in Lot 3, the q_c value is typically 2 to 3 MPa, much lower than that at the corresponding levels in Lot 1 and Lot 2, except for the thin platform layers at the top of each placement lift that had been subject to traffic compaction.

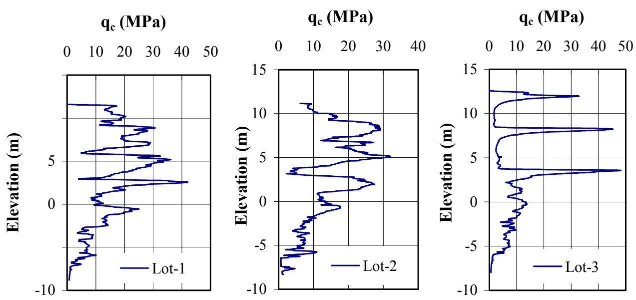

Figure 2. CPT results from Changi East reclamation site

5 VERIFICATION OF SCHMERTMANN'S METHOD

Among the various methods for settlement analysis of footings in sand, Schmertmann's method (Schmertmann, 1970; Schmertmann et al., 1978) is by far the most common. The proposed equation for the calculation of settlement (s) is as follows:

$$s = C_D C_c \Delta q \sum_{0}^{2B\sim 4B} \left(\frac{I_z}{E}\right) h \qquad (1)$$

where C_D is the depth correction factor, C_c is the creep factor, Δq is the net increase in pressure at foundation level, B is the foundation width or diameter, h is the thickness of each layers, and I_z is strain influence factor. The distribution of I_z is as shown in Fig. 3. Schmertmann et al. (1978) recommended that the field modulus E be estimated from q_c based on the following correlation factors: $\alpha = E/q_c$ of 2.5 for square footings and 3.5 for the plane strain condition.

Marangos (1995) studied Schemertmann's method and suggested that the maximum I_z, or I_{zp}, be modified to account for the density and the stress level effect in consideration that Schmertamnn's method could lead to very unsafe predictions of settlements particularly in loose sand.

In order to verify the suitability of Schmertmann's proposed normalized strain distributions, a simplified elastic analysis was carried out by assuming that the soil is a homogeneous, isotropic, and linearly elastic, and the plate is rigid. FEM is adopted to simulate the stress-strain behavior of the soil under vertical loading. The applied pressure q, Poisson's ratio ν, and the shape factor of footing L/B (L = length; B = width) were considered in the analysis. The value of I_z was found to be relatively unaffected by the variation of q, although a slight change is observed with the variation of ν. The value of I_z at ground surface I_{zo} is around 0.2 to 0.3, and it does not change significantly with increases in q and L/B ratio. Fig. 4 illustrates how the normalized strain distribution varies with the L/B ratio. The maximum value of I_z occurs consistently at the depth of about 0.5B and I_{zp} is typically between 0.4 and 0.5. The maximum influence depth z_m increases gradually with increasing L/B and and the increase practically ceases as L/B ≥ 10, when z_m reaches around 5B.

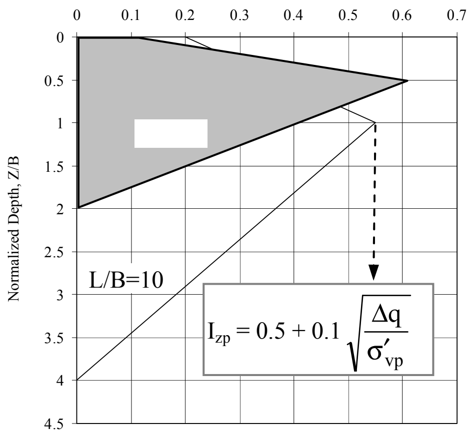

Figure 3. Assumed distributions of I_z (Schertmann et al. 1978)

Although the simplified analysis shows that the distribution of normalized strain does not vary significantly as the applied pressure q increases, it is known that the stress-strain relation is generally nonlinear and the soil modulus decreases with increasing stress level in the soil. One therefore needs to adopt a variable I_{zp}, as was proposed in Schmertmann et al. (1978), to account for the nonlinearity. Adopting a variable I_{zp} is also necessary in consideration of the difficulty in determining and adopting a varying average modulus in the settlement analysis of shallow foundations. The I_{zp} equation proposed by Schmertmann et al. (1978), as shown in Fig. 3, however, warrants a careful verification with field settlement records.

To provide a verification of Schmertmann's proposed I_{zp}, the following generalized equation is considered:

$$I_{zp} = 0.5 + n\left(\frac{\Delta q}{\sigma'_{vp}}\right)^m \qquad (2)$$

where σ'_{vp} is the effective original vertical stress at the depth where maximum I_z occurs, and m and n are curve fitting parameters, which were selected as 0.5 and 0.1, respectively, by Schmertmann et al. (1978).

By comparing pressure-settlement curves predicted based on Eq. (2) using E_{oc} back-calculated from the PLT with the measured curve for one of tests at the site, the use of Schmertmann's I_{zp} was found to produce a poor match. On the other hand, by varying both m and n in the equation and using a z_m of 5B, a close match is possible. Interestingly, for m = 0.5, as adopted by Schmertmann et al. (1978), a reasonable match can be achieved, with a matching n value equal to 0.031 in this case.

Further analyses were carried out based on all PLT and CPT data collected at the site. The matching value of n was found to fluctuate typically between 0.018 and 0.047 and the average is around 0.04 for the medium to very dense sand with RD ≥ 50% in Lots 1 and 2. These matching n values are consistently lower than that the value of 0.1 selected by Schmertmann et al. (1978), which was meant for normally consolidated sand. The overconsolidated nature of the hydraulically reclaimed sand is believed to have a profound effect. For the loose sand present primarily at Lot 3, the matching n was found to range widely from 0.27 to 0.39, with a typical value of around 0.3. The much larger variation and higher values of n are associated with the random and highly compressible nature of the direct dumped loose fill. These results are consistent with the observations of Marangos (1995).

It is suggested that appropriate modifications be made on Schmertamnn's distributions of normalized axial strain as follows: (a) taking I_{zo} = 0.2; (b) calculating I_{zp} from Eq. (2) using n = 0.04 for OC or medium to very dense sand with RD ≥ 50% and n = 0.3 for loose to medium sand with RD < 50%; and (c) selecting z_m = 2.5 (1+ log(L/B)).

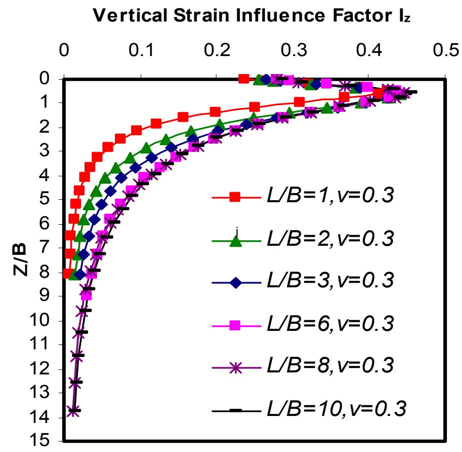

Figure 4. Effect of L/B on normalized vertical strain

6 ESTIMATING ELASTIC MODULUS FROM CPT

Using the present proposed framework of analysis, the elastic modulus E_s that prevails in the field can be back-calculated from the results of PLTs and compared with the corresponding q_c value. The E_s/q_c ratio generally ranges from 2.2 to 4.8 and averages around 4 for the medium dense to very dense sand with RD ≥ 50% in Lots 1 and 2. For the loose to medium dense sand with RD < 50%, primarily in Lot 3, the ratio varies widely and bears no specific relation with the relative density. Interestingly, the back-calculated E_s/q_c ratio is comparable to the corresponding E_{oc}/q_c ratio.

Note that for shallow foundations on sand, the vertical strain would probably fall within the range of 0.1% to 0.25 % (Baldi et al. 1988) or subject to a upper strain limit of around 0.1% under the normal working load (Burland, 1989). This range of strain is similar to that experienced by the sand in recompression in the present investigation. The E_s/q_c ratio of 4 deduced from PLTs can therefore be taken as representative of the field modulus coefficient (α), or the E/q_c ratio, and used in the proposed modified Schmertmann's framework of analysis for predicting settlement of square foundations on as-compacted hydraulically placed sand.

As recompacted sandfill is usually dynamically densified and any prestressing effect from the placement would be destroyed, a lower E/q_c ratio, such as a α = 2.5 (Schmertamnn et al., 1978), might be more appropriate for such sandfill. However, appropriate adjustment should be made if the sand subsequently becomes overconsolidated as a result of static preloading based on, for example, Lambrechts and Leonards (1978) to account for prestressing.

7 DEVELOPMENT OF ACCEPTANCE CRITERIA

In developing suitable acceptance criteria, Schmertmann's proposed distributions of normalized vertical strain were used, with both n = 0.04 considered for hydraulically filled and both n = 0.1 and n = 0.3 considered for loose or normally consolidated sand. A modulus coefficient α of 4 was chosen for hydraulically filled sand, which is usually overconsolidated, for the axis-symmetrical (L/B=1) condition, and 4.0 x (3.5/2.5), or 5.6, for the plain strain (L/B=10) condition. For normally consolidated or dynamically densified sand, α =2.5 and α =3.5, respectively, were chosen for axis-symmetrical and plain strain conditions, following the proposals of Schmertmann et al. (1978). One must recognize that, at most reclaimed sites, the sand is usually heterogeneous and any blanket specification of minimum value of q_c for fill quality and densification control is impractical. Flexibility should be allowed for accepting some localized low q_c values in thin layers within the sandfill. An equivalent q_c value, or q^*_c, calculated based on weight-averaging of individual q_c values in various soil layers using the selected distribution of normalized vertical strain such as those proposed earlier in the paper, is therefore recommended.

Figure 5 shows a chart developed based on the proposed acceptance criteria for a commonly accepted allowable settlement of 25 mm for shallow foundations resting on hydraulically reclaimed sand fill for (a) L/B =1 and (b) L/B =10 cases. Fig. 6 shows a similar chart for square foundations resting on dynamically densified or normally consolidated sand fill, with n = 0.1 for less conservative and n = 0.3 for more conservative applications. These charts can be potentially used in the development of reclaimed sites that are underlain by reclaimed sandfill in order to ensure a satisfactory performance of shallow foundations.

As an example, we assume that a 3m x 3m footing is to be built on the surface of a quartz-rich sandfill. The net vertical pressure is expected to be 300 kPa, and the settlement shall be restricted to 25 mm. Taking C_c= C_D=1 and α of 4 for hydraulically placed sandfill and 2.5 for dynamically densified sandfill, the required q^*_c value would be 7.7 MPa and 14.7 MPa, respectively based on Figs. 5 and 6. The required q^*_c value for dynamically densified fill is much higher at 26.0 MPa if n = 0.3 is selected for a more conservative application.

One should take note that densification by dynamic means imposes a positive effect on the compressibility of sand primarily because of an increase in the relative density; but there is also a negative effect from the damage to the prestressed soil fabric. A high q^*_c value is required in order to compensate for the negative effect. If the sand is statically preloaded using a surcharge fill, conventional settlement requirements can be more easily met due to the induced positive prestress effect.

The proposed criteria are reasonable and simpler to use if one compares with the relationship proposed by Jamiolkowski (1988) which requires the information on RD and the overconsolidation ratio OCR, as well as K_o, the coefficient of earth pressure at rest.

8 CONCLUSIONS

A comprehensive field investigation program comprising primarily plate load tests and cone penetration tests was carried out to assess the compressibility of the sandfill at Changi East reclamation site in Singapore. The key findings from the investigation are as follows:

(1) Calibration of settlements predicted using Schmertmann's framework of analysis with settlement observations from plates resting on reclaimed sand indicates that Schmertmann's proposed distributions of normalized vertical strain can be modified to provide an improved pressure-settlement curve.

(2) The average elastic modulus that prevails in the field, as back-calculated from settlement observations made on the model foundations or plates using modified distributions of normalized vertical strain, indicates a E_s/q_c ratio of 4 for plates resting on the hydraulically reclaimed sand in Changi East; but the corresponding ratio varies widely for plates resting on direct dumped sand at the same site.

(3) Acceptance criteria for fill quality and densification control have been developed to facilitate the selection of the required equivalent q_c value in both hydraulically placed sandfill and dynamically densified sandfill for a specified settlement limit by adopting a modified Schmertmann's analysis method.

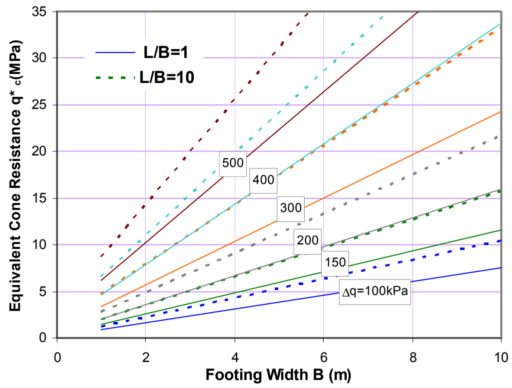

Figure 5. Acceptance criteria for hydraulically placed sandfill

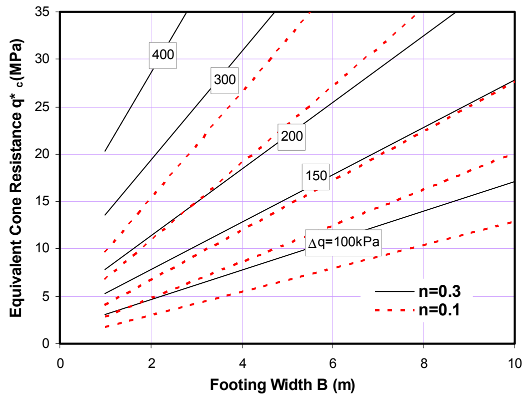

Figure 6. Acceptance criteria for dynamically densified sandfill

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