SETTLEMENT ANALYSIS OF COMPACTED GRANULAR FILL

CALCUL DES TASSEMENTS DE REMBLAIS GRANULAIRE COMPACTE

K. Rainer Massarsch

Geo Engineering AB and Department of Soil and Rock Mechanics
Royal Institute of Technology, Stockholm, Sweden

SYNOPSIS: A concept for the calculation of settlements in granular fill is presented, based on cone penetration tests with friction sleeve measurements. The tangent modulus is determined from a normalised cone resistance, which is corrected for the effect of the mean effective stress. The normalised cone penetration value reflects the influence of the relative density and thus the compressibility of the soil. An empirical correlation is proposed to determine from the normalised cone penetration resistance the modulus value m, which is required to calculate the tangent modulus M_t. The overconsolidation effect, which is caused by soil densification, can be taken into account in a rational way, based on friction sleeve measurements. The calculation of settlements according to the tangent modulus method is demonstrated by a case history, where a hydraulic fill was compacted by deep vibratory compaction. After soil densification, the cone penetration resistance as well as the sleeve friction increased by a factor of 2 - 8. The sleeve friction increased more in the fine-grained soil layer, while the cone penetration resistance was largest in the coarse-grained soil layer. A comparison of calculated settlements shows the importance of lateral stress increase after soil compaction for the settlement calculations.

INTRODUCTION

In many countries, the availability of land has become an important aspect of industrial and social development. Placement of hydraulic fill is a convenient method of creating new land, e.g. for infra-structure projects. Most of the fill material is marine-dredged sand, as material from land borrow areas is generally expensive and can have severe environmental effects. The placement method of hydraulic fill has great influence on its geotechnical properties. The prediction of settlements of compacted hydraulic fill is an important aspect of land reclamation design, as it can have significant technical and economical consequences for a project. In the present paper a concept for the estimation of stress-induced settlements of granular soils is presented, which is based on cone penetration tests (CPT) with friction sleeve measurements. When the smallest width of the loaded area exceeds about the thickness of the compressible soil layer, the constrained modulus M should be used. The load deformation relationship of soils is generally non-linear and this effect must be taken into account. Janbu (1963) has shown that the tangent modulus M_t for a wide range of soils can be described by the following empirical relation

$$M_t = m\,\sigma_r\,(\sigma'/\sigma_r)^{(1-j)} \qquad (1)$$

where m is a dimensionless modulus number, σ_r is an arbitrarily chosen reference stress (100 kPa), σ' is the effective stress and j is the stress exponent. The relative compression (vertical strain) ε of a normally consolidated soil layer, resulting from an increase of the effective stress $(\sigma_0' + \Delta\sigma')$ can then be calculated from

$$\varepsilon = [\,\{(\sigma_0' + \Delta\sigma')/\sigma_r\}^j - \{\sigma_0'/\sigma_r\}^j\,]\,/\,(m \cdot j) \qquad (2)$$

where σ_0' is the initial effective overburden stress and $\Delta\sigma'$ is the increase of effective stress. For cohesive soils, values of the modulus number m and the stress exponent j can be determined by conventional laboratory tests. However, this is usually not possible for granular soils.

The Canadian Foundation Engineering Manual (1985) proposes empirically determined values for the stress exponent j, and the modulus number m, Table 2.

Table 2. Stress Exponent j and the Modulus Number m, Canadian Foundation Engineering Manual (1985)

Soil Type	Stress Exponent, j	Modulus Number, m
Gravel	0,5	40 - 400
Dense Sand	0,5	250 - 400
Compact Sand	0,5	150 - 250
Loose Sand	0,5	100 - 150
Dense Silt	0,5	80 - 200
Compact Silt	0,5	60 - 80
Loose Silt	0,5	40 - 60
Hard Silty Clay	0	20- 60

DETERMINATION OF TANGENT MODULUS FROM CPT

The CPT is widely used for the investigation of granular soil deposits and several authors have proposed empirical correlations between the cone penetration resistance and the tangent modulus, e.g. Jamiolkowski et al. (1988), Leonards and Frost (1988), Robertson and Campanella (1986). Results from cone penetration tests are affected mainly by two parameters, the density of the soil and the level of effective horizontal stress, Jamiolkowski et al. (1988). Attempts have been made to correlate directly the tangent modulus M_t to the cone penetration resistance, q_c, but with limited success, Jamiolkowski et al. (1988).

Because of the greater confinement pressure, caused by the increased overburden pressure, cone penetration values at increasing depth may indicate larger relative densities than actually exist. Peck et al. (1974) proposed an empirical method to take into account the effect of the effective overburden pressure for settlement estimates, based on the

Standard Penetration Test (SPT). Peck proposed the following relationship to normalise the SPT N-value with respect to an arbitrarily chosen reference stress of $1 t/ft^2$ (approximately 107 kPa),

$$C_N = 0{,}77 \log_{10} (20/p') \tag{3}$$

where C_N is the stress correction factor and p' is the effective vertical overburden pressure. Based on extensive laboratory tests published by Marcuson and Bieganousky (1976), Seed (1976) proposed a similar correction factor for the liquefaction analysis of loos saturated sands,

$$C_N = 1 - 1{,}25 \cdot \log (\sigma_v'/\sigma_1') \tag{4}$$

where σ_v' is the effective overburden pressure (in t/ft^2) and σ_1' is a reference stress ($1\ t/ft^2$). Jamiolkowski et al. (1988) have reviewed extensive CPT tests, performed in calibration chambers, and found that for normally consolidated and overconsolidated granular soils, the cone penetration resistance q_c can be correlated to the relative density DR and the mean effective stress σ_m according to the following relationship,

$$q_c = C_o (\sigma_m')^{C_1} \cdot e^{(C_2 \cdot DR)} \tag{5}$$

where C_0 = 205, C_1 = 0,51 and C_3 = 2,93 are empirically determined constants. Based on equation (5), it is possible to establish a similar correction factor C_M for cone penetration tests, similar to the correction factor C_N for SPT,

$$C_M = (100 / \sigma_m')^{C_1} \tag{6}$$

In Fig. 1, a comparison is made between the stress correction factors of equations (3), (4) and (6). The agreement is surprisingly good between the extensive, recent calibration chamber tests for CPT:s, the empirical correlation by Peck et al. (1976) and the SPT laboratory tests by Marcuson and Bieganousky (1976).

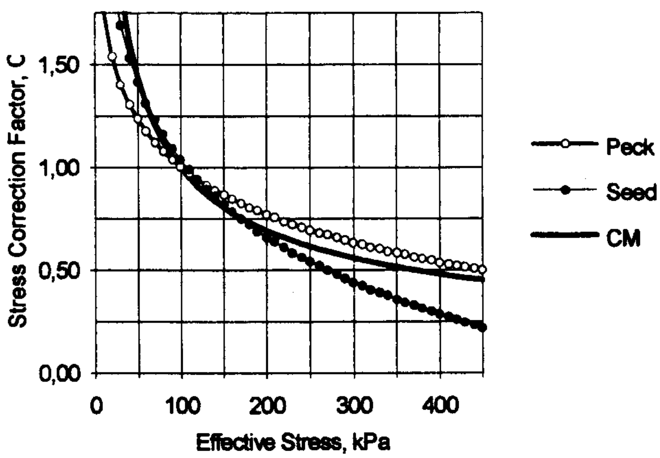

Fig. 1. Comparison of stress correction factor C_N and C_M

The simple relationship from equation (6) appears to be valid for a wide range of normally and overconsolidated soils. However, it is important to take into account the effect of the lateral effective stress, by using the mean effective stress σ_m' rather than the vertical effective stress, as proposed by Peck et al. (1974) and Seed (1976). For most practical applications, it is sufficient to use a value of C_1= 0,5. The measured cone penetration resistance, q_c can be normalised with respect to the reference stress of 100 kPa.

$$q_{co} = q_c \cdot C_M \tag{7}$$

where q_{co} is the normalised cone penetration value. The correction of the cone penetration resistance, q_c, with respect to the mean effective stress requires the estimation of the coefficient of later earth pressure, K_o. In normally consolidated soils (i.e. in a hydraulic fill prior to compaction), the horizontal effective stress σ'_{hA} can be estimated with sufficient accuracy form the relationship

$$\sigma'_{hA} = K_o \sigma'_{vA} = (1 - \sin \phi) \sigma'_{vA} \tag{8}$$

where ϕ is the effective friction angle. Leonards and Frost (1988) have proposed an interesting approach to assess the lateral effective stress for natural soil deposits, based on CPT and Dilatometer tests.

The variation of the normalised cone penetration resistance q_{co} reflects the relative density, which in turn is related to the compressibility of the soil. A correlation has been performed between the m-values from Table 2 and the normalised cone penetration resistance q_{co}, which yields the following simple empirical relationship

$$m = a \cdot (q_{co} / \sigma_r)^{0,5} \tag{9}$$

where a is a modulus factor, which depends on soil type, q_{co} is the normalised cone penetration resistance (in kPa) and σ_r is the reference stress (100 kPa). Typical values for the empirical parameter a are given in Table 3 for different soil types.

Table 3. Modulus factor a, for granular soils

Soil Type	Modulus Parameter a
Silt, loose	6
Silt, compact	10
Silt, dense	12
Sand, loose	16
Sand, compact	18
Sand, dense	22
Gravel, loose	15
Gravel, dense	25

Good agreement is obtained between the values of the modulus parameter m, calculated according to equation (9), and the range of values proposed by Janbu (1963) and the Canadian Foundation Engineering Manual (1985), cf. Table 2. The advantage of using the above proposed approach is that the modulus value m can be determined, based on CPT data. The method takes also into account in a rational manner the effect of two important parameters for settlement analysis, the mean effective stress and the soil density.

STRESS HISTORY AND PRECONSOLIDATION EFFECT

A recently deposited hydraulic fill can be assumed to be normally consolidated. However, soil compaction causes a substantial overconsolidation effect. Leonards and Frost (1988) pointed out the importance of stress history for the realistic assessment of settlements. Fig. 2 shows the stress path for a soil element before, during and after compaction.

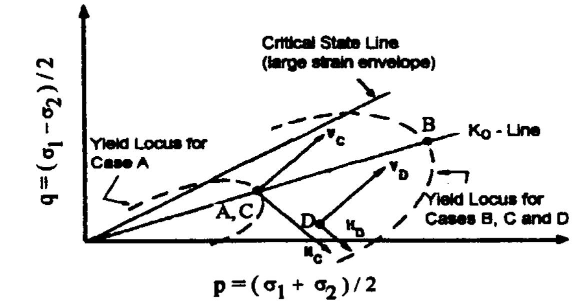

Fig. 2. Stress Path for a soil element before (A), during (B) and after compaction (C, D), after Leonards and Frost (1988)

The initial state of effective stresses is indicated by point A in Fig. 2. During compaction, the stress conditions are changed approximately along the K_0-line to point B. After compaction, if lateral strain allows to maintain the K_0-stress path, the soil element reaches location C. If however, the stress state at B is changed at zero lateral strain, point D is reached. Although the overconsolidation ratio with respect to vertical stresses, OCR_v is the same for conditions C and D, the overconsolidation ratio with respect to horizontal stresses is different, $OCR_h = \sigma'_{H(B)}/\sigma'_{H(D)}$. This aspect is of practical importance, as soil deformations during subsequent loading will be strongly influenced by the value of OCR_h. Leonards and Frost (1988) suggested (based on limited data from reconstituted samples of silica sand) a value of OCR_h = 1,25. Leonards and Frost (1988) point out that for loading in the horizontal H_C and vertical V_C directions from point C, and in the vertical direction V_D from point D, the sand remains in the overconsolidated ("elastic") range for substantial stress increments, while loading in the horizontal direction H_C for case D reaches the yield locus at a relatively smaller stress increment, Fig. 2. For most conventional loading conditions (vertical stress increase), a stress path V_D from point D will be followed. Calibration chamber tests have shown that different types of CPT measurements (cone penetration resistance q_c and sleeve friction f_s) are strongly affected by the horizontal effective stress, σ'_{ho}.

The sleeve friction is normally used to determine the friction ration, FR, which is a sensitive indicator of soil type. In the geotechnical literature, the absolute value of the sleeve friction is rarely used for geotechnical design purposes. It is difficult to determine the absolute value of the horizontal effective stress from f_s. However, f_s is sensitive to changes in horizontal effective stress, e.g. which are caused by compaction. In the following paragraphs, a simple procedure is proposed, which makes it possible to assess quantitatively the increase of lateral and vertical effective stress resulting from compaction. From the ratio of the sleeve friction value after compaction f_{sa} and before compaction, f_{sb}, an improvement factor n can be determined, which reflects the increase in lateral stress,

$$n = f_{sa} / f_{sb} \cong \sigma'_{hD} / \sigma'_{hA} \tag{10}$$

where σ'_{hA} and σ'_{hD} correspond to the stress conditions in Fig. 2. The preconsolidation effect in the horizontal direction OCR_h can only be estimated, as little information is available in the literature for granular soils. Leonards and Frost (1988) suggest for silica sand $OCR_h \cong 1,25$. Thus the horizontal preconsolidation stress σ'_{hB} (at point B) can be estimated according to

$$\sigma'_{hB} = OCR_h \cdot \sigma'_{hD} \cong OCR_h \cdot n \cdot \sigma'_{hA} \tag{11}$$

The overconsolidation ration in the vertical direction OCR_v can now be determined from the following approximate relationship

$$OCR_v = \sigma'_{vB} / \sigma'_{vA} \cong n \cdot OCR_h \cdot (K_{0A}/K_{0B}) \tag{12}$$

where K_{0A} and K_{0B} are the coefficient of earth pressure at rest at point A and B, respectively, in Fig. 2.

CASE HISTORY

The method of settlement prediction as outlined above, is exemplified by a case history, where an about 9 m thick hydraulic fill was densified by vibratory compaction, using the resonance compaction system (MRC System). A variable frequency vibrator is used in combination with a flexible compaction probe. The vibrator frequency is varied during the different phases of the compaction process to achieve optimal soil densification. A detailed description of the MRC System has been given by Massarsch and Heppel (1991). Figure 3 shows the fully automated and computer-controlled MRC machine and a detail of the low-impedance compaction probe.

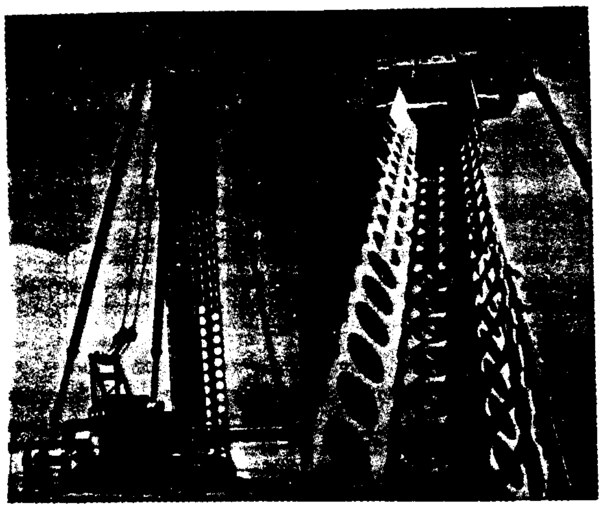

Fig. 3 MRC compaction machine and detail of low-impedance compaction probe (Flexi Probe)

The MRC system was used to compact a deposit of hydraulic fill for a land reclamation project. The ground water table at the site was located 4 m below the ground surface. The geotechnical properties of the soil deposit were investigated by extensive field and laboratory tests. In addition to a limited number of SPT tests, extensive CPT tests were carried out before, during and after compaction. Figure 4 shows results of a cone penetration test in the trial area before compaction, with sleeve friction and friction ratio measurements.

Above the ground water table the hydraulic fill is loose to compact (q_c: 10-16, f_s: 35-45 kPa). Below the ground water table, the soil becomes loose to very loose down to a depth of 9 m (q_c: 3-8, f_s: 2-10 kPa). Some layers of silty sand and clay occurred in the hydraulic fill and below, which is exemplified by an increase of the friction ratio, FR.

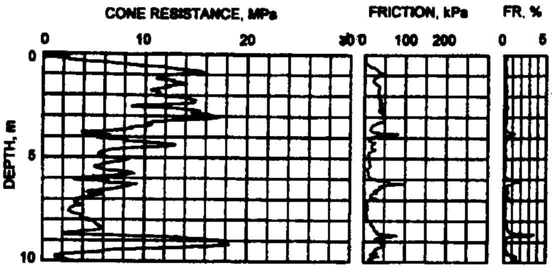

Fig. 4. Cone penetration test before compaction of the hydraulic fill

Compaction trials were carried out at different grid spacings, compaction sequences and varying the vibration duration. The compaction effect was checked by extensive CPT investigations. A cone penetration test performed one day after test compaction in the trial area, Fig. 5. The maximum probe penetration depth was 6,5 m. It can be concluded that the compaction effect extends to the depth of probe penetration. The average CPT resistance was increased to values ranging between 20 and 30 MPa in the upper sand layer and to 8 - 15 MPa in the lower, silty sand layer. The sleeve friction increased in the upper sand layer to 50 - 100 kPa, and to 150 - 200 kPa in the silty sand. The increase of sleeve friction was larger in the silty sand than in the fine sand.

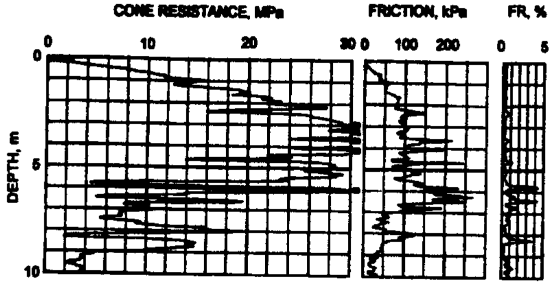

Fig. 5. Cone penetration test one day after compaction of the hydraulic fill, depth of probe insertion 6,5 m

Settlements were calculated for a tank of 10 m diameter and a surface load intensity of 250 kPa according to the tangent modulus method as outlined above. The cone penetration resistance was corrected for the mean effective stress, using the C_M-value, equation (7). The coefficient of lateral earth pressure was calculated, using a friction angle of 30°. The a-values were chosen according to Table 3 for compact sand (a:18), loose sand (a: 16) and silty clayey silty sand (a: 10). The entire hydraulic fill was assumed to be normally consolidated prior to compaction, in spite of the apparent overconsolidation effect in the upper sand layer. The settlements were then calculated for two assumptions after compaction, with and without accounting for the increase in horizontal stresses. For the case of a "normally consolidated" compacted fill, the settlements were determined, using a-values for dense sand and dense silt in the respective layers. The preconsolidation effect was accounted for according to the concept outlined above. A comparison of friction sleeve measurements before and after compaction yields n-values of about 2-3 in the upper sand layer, and much higher values in the lower silty sand, ranging between 3-25. According to equation (12), a conservative OCR_V-value of 5 was obtained. The vertical stress distribution below the tank was calculated according to the 2 (vertical) : 1 (horizontal) load distribution. The settlements in the overconsolidated stress range were neglected. The results of the settlement analysis are summarised in Table 4.

Table 4. Results of Settlement Analysis Before and After Compaction, Surface Load of 250 kPa, Width of Loaded area 10m

Pre-compaction	Post-compaction "normally consolidated"	Post-compaction "overconsolidated"
(OCR = 1)	(OCR = 1)	(OCR = 5)
73 mm	49 mm	14 mm

It can be concluded that if the effect of preconsolidation, resulting from soil compaction, is taken into account in the settlement analysis, a substantial reduction of estimated settlements is obtained. This conclusion supports the findings by Leonards and Frost (1988)

SUMMARY AND CONCLUSION

A unified method for the determination of settlements in granular soil is presented. The tangent modulus can be estimated from the cone penetration resistance, provided that the effect of the mean effective stress (influence of lateral effective stress) is taken into account. A simple relationship can be used to normalise the CPT resistance with respect to a reference stress of 100 kPa, similar to the SPT.

An empirical correlation is proposed which permits the estimate of the modulus value m, based on the normalised CPT resistance. Good agreement has been obtained with the m-values proposed by Janbu (1963) and in the Canadian Foundation Engineering Manual (1985).

Based on friction sleeve measurements, it possible to assess the increase of lateral effective stress after soil densification. The present paper supports the findings reported by Leonards and Frost (1988).

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- Kd highly reproducible. But f_s ?...
- Kd directly related to σ_h (horizontal). While $f_s = \sigma_h \, tg\delta$. (Sleeve measures vertical force. Then evaluates σ_h via $tg\delta$)
- Cilyndrical probes : *arching* reduces sensitivity to σ_h. Flat shape much less.