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AN INVESTIGATION OF CORRELATION FACTORS LINKING FOOTING RESISTANCE ON SAND WITH CONE PENETRATION TEST RESULTS

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Abstract:

Significant research effort has led to improvements in our ability to estimate the ultimate bearing resistance of footings in sand. These techniques often estimate the footing resistance at relatively large displacements, typically 10% of the footing width, $q_{b0.1}$. Cone Penetration Test (CPT) design methods typically link $q_{b0.1}$ and q_c through a constant reduction factor, α . A range of α factors for shallow footings have been proposed, some methods suggest that α is constant and while others that it varies with footing width and depth (or stress level). There is a dearth of field data with which to compare these correlation factors, in particular where foundation width and depth have been varied in the same ground conditions. For this reason finite element analyses have proven to be a useful tool for performing the parametric studies required to assess factors controlling α . This paper describes the results of numerical analyses performed to investigate α factors for soil profiles which were calibrated using the results of the CPT tests performed at a dense sand test-bed site. The numerical model was first used to perform parametric

analyses to consider the effect of footing width, B and footing depth, D on the α factor mobilised in dense Blessington sand. In order to assess the effects of relative density, footing tests in a range of natural sands with variable in-situ densities were modeled. The results of the finite element analyses suggest that a direct correlation between $q_{b0.1}$ and q_c can be established at a given test site which is independent of footing width and depth and is relatively weakly dependent on the sand's relative density if the zone of influence of the foundation considered is large enough.

Keywords: Cone penetration test, Finite element analysis, Sand, Footing, Bearing resistance

1 Introduction

Due to their relatively low cost, shallow foundations are widely used as supports for both onshore and offshore structures. Most textbooks and design codes recommend conventional bearing capacity approaches to calculate the ultimate bearing capacity (q_{ult}) of footing on sand:

$$[1] \quad q_{ult} = 0.5 B \gamma N_{\gamma} s_{\gamma} d_{\gamma} + \gamma D N_q s_q d_q$$

in which B is the footing width; γ is the unit weight of the ground;; D is the embedment depth; N_{γ} , and N_q are bearing capacity factors that depend on the footing shape and the effective friction angle (ϕ) of the soil while factors s_{γ} , s_q , d_{γ} and d_q take account of the footing shape and embedment depth.

Notwithstanding the difficulty of choosing a design value of ϕ (see Randolph et al. 2004), Briaud (2007) argued that whilst Eqn. 1 would produce good estimates of q_{ult} of footings in soil profiles where the soil strength increased linearly with depth. In over-consolidated sand or in deposits where the near surface soil is unsaturated the soils strength is often relatively constant with depth and the assumption that q_{ult} increases with footing width, B or footing depth, D is not valid. By compiling data from a number of full-scale footing tests he demonstrated that when the mobilised bearing pressure q , was normalised by an in-situ measurement of soil strength such as the Standard Penetration Test (SPT) N value or the Cone Penetration Test (CPT) end resistance (q_c) value

averaged over the zone of influence of the footing, a unique normalised load-settlement response was obtained for a given site. This approach is illustrated in Figure 1a which shows the pressure-settlement response measured during load tests performed at Texas A&M University reported by Briaud and Gibbens (1999). The tests were performed on square footings those width varied from 1 m to 3 m, which were founded 0.75 m below the ground surface. The recently deposited medium-dense sand was in a lightly over-consolidated state ($OCR \approx 2$) following the removal of about 1.0 m overburden depth. The mean CPT q_c resistance ranged from 5 to 7.25 MPa in the zone of influence of the footing. When the bearing pressure was normalised by the average q_c value and the settlement s , was normalised by the footing width, B , a direct relationship was suggested for the normalised pressure-settlement response, which was independent of footing width or relative embedment (D/B) was obtained (See Figure 1b). The footing resistance mobilised when the settlement reached 10% of the footing width, $q_{b0.1}$ could be given as:

$$[2] \quad q_{b0.1} = \alpha q_c$$

an α value of 0.25 was suggested by Briaud and Gibbens (1999) to provide a good fit to the measured data.

Whilst the Texas A&M footing tests were performed on medium dense sand at a constant depth of embedment, Eslaamizaad and Robertson (1996) compiled a database of footing tests and found that the back-figured α values varied with soil density, relative embedment and footing shape. Randolph et al. (2004) summarised the results of

laboratory and field tests and numerical analyses performed on shallow footings and buried piles. Although a relatively wide range of α values were reported, with α varying from 0.13–0.21, there was no evidence that α varied with footing width or sand state.

Atkinson (2000) noted that the non-linear pressure-settlement response of footings depended on the rigidity index I_R (defined as the ratio of small strain stiffness to strength. Gavin et al. (2009) compared the normalised stiffness degradation (β is the ratio of secant stiffness to q_c) from shallow footing tests performed at three sites, Texas A&M (Site A), Shenton Park (Site C) and Blessington (Site C). The data shown in Figure 2 confirms Atkinson's suggestion that the degree of non-linearity increases as the rigidity index increases. Interestingly, once the normalised settlement s/B exceeded $\approx 5\%$, the ratio E'_s/q_c converged, suggesting that soil stiffness would not affect α values at normalised settlements of 10%.

The selection of a unique α value which is independent of soil state or footing geometry is in keeping with observations from tests that measured the base resistance of piles installed in sand. De Cock et al. (2003) and Cadogan and Gavin (2006) report on tests on full replacement bored piles and note that an α value of approximately 0.2 provided a good fit to a database of model and full-scale footing tests where the pile diameters ranged from 0.1 m to 1.5 m and the length ranged from 2 m to 18.2 m. Jardine et al. (2005) compiled a database of load tests performed on full-replacement, closed-ended piles in sand with a wide range of in-situ density and suggested that a diameter dependent α value reduced from 0.63 to 0.43 as the pile diameter increased from 200 mm to 500

mm. Randolph (2003) and White and Bolton (2005) argued that once appropriate averaging techniques were adopted to derive design q_c values and the effects of residual loads were accounted for, a constant α factor can be adopted which is independent of pile diameter. Lehane et al. (2005) re-interpreted the database study and found that once these corrections were considered a α value of 0.6 which was independent of pile diameter gave the best-fit.

For partial displacement (open-ended) pipe piles, model and full-scale pile tests reported by Lehane and Gavin (2001) and Foye et al. (2009) show that direct correlations between α (based on the average pressure mobilised over the entire pile base area) and q_c which are independent of pile diameter or sand state can be determined once the effect of sand displacement at the pile base during pile installation are included. An α value of 0.6 is suggested by Lehane et al. (2005) for a pile which formed a complete plug during installation (in effect became closed-ended). Minimum α values in the range 0.15 to 0.2 have been suggested by Lehane and Randolph (2002) and Gavin and Lehane (2003) for fully-coring piles.

Evidence from a limited number of shallow footing tests and relatively extensive pile testing suggest that unique α factors may be suggested for shallow footings which are in the range 0.15 to 0.2. Large-scale footing tests are expensive and time consuming and therefore most field tests consider a relatively limited range of either footing width and/or depths in sand where the relative density is relatively constant. Finite Element (FE)

analyses present an ideal environment to consider sensitivity analyses for foundations. Lee and Salgado (2005) reported a significant study of FE analyses in which they investigated the effect of footing width and relative density (D_r) on the mobilised bearing resistance. They used the FE program ABAQUS with a user defined soil model which incorporated non-linear stiffness to investigate α values for a range of footing widths and relative densities. The CPT q_c values used to normalise the footing resistance were derived from a different program CONPOINT (Salgado and Randolph, 2001). Their data shown in Figure 3 indicate that α increased when the relative density of the soil reduced and the footing width increased. The rate at which α increased with the footing width depended on the relative density of the soil, with an increase of 35% being noted for $D_r = 90\%$ when the footing width increased from 1 m to 3 m, whilst the increase was only 5% for $D_r = 30\%$.

It is obvious from the foregoing that there is considerable uncertainty with regard to the choice of a suitable α value for use in the design of shallow foundation on sand. In an attempt to understand the parameters affecting α values a suite of finite element analyses were performed. As a first step FE analyses were concentrated on modelling the CPT q_c profiles at the University College Dublin (UCD) dense sand test bed site in Blessington, County Wicklow. The FE model thus calibrated was used to predict the response of shallow footing tests of a range of footings with widths varying from 2 m to 5 m, and embedment depths varying from 1 m to 3 m. The results are then compared to model footing tests performed at the site. The effect of relative density was then considered by

repeating the analyses using well characterised sand deposits from the geotechnical literature.

2 Finite Element Analyses

2.1 Determination of FE model parameters for Blessington Sand

The FEM analyses performed in this paper were undertaken using Plaxis version 8 (2002). The sand was modelled using the Hardening Soil (HS) model described by Schanz et al. (1999). The primary aim of these analyses was to investigate circular footing response in dense sand found at the University College Dublin (UCD) test bed site is located in Blessington, County Wicklow approximately 25 km south-west of Dublin. The deposit is in an over-consolidated state due to glacial action, ground water level changes, and recent sand extraction. Extensive CPT testing has been performed at the site in association with model pile and footing tests described by Gavin and O’Kelly (2007), Gavin and Lehane (2007) and Gavin et al. (2009). The water table was approximately 13 m below the ground level (bgl) at which the CPT profiles were measured. The unit weight of the material calculated from sand replacement tests was 20 kN/m³, and the degree of saturation was 71%. In order to provide input parameters for FEM soil models, triaxial compression tests and oedometer tests on representative soil samples were performed (See Tolooiyan, 2011). The triaxial tests revealed that the constant volume friction angle of this well-graded, angular sand was 37°, and that the

dilation angle which varied with the confining pressure, was 5.4° at the reference pressure of 100 kPa. Tolooiyan and Gavin (2011) describe a calibration procedure whereby the triaxial test results were modelled as element tests in Plaxis and the HS soil parameters were varied to match the measured data. The HS soil parameters thus obtained are shown in Table 1.

2.2 FE Modelling of Cavity Expansion and Developing the CPT Profile

Spherical cavity expansion analyses were undertaken to estimate the pressuremeter limit pressure p_{limit} and hence predict a q_c value using the approach suggested by Randolph et al. (1994) in Equation 3a and Yu et al. (1996) in Equation 3b:

$$[3a] \quad q_c = p_{\text{limit}} (1 + \tan\theta \cdot \tan\phi)$$

$$[3b] \quad q_c = p_{\text{limit}} (1 + \sqrt{3} \cdot \tan\phi)$$

Where θ is the cone tip angle or the cone face with the horizontal which is 60 degree and ϕ is the friction angle of the soil.

Although the field measured CPT data for Blessington sand were available (see Figure a), the CPT profile was modelled in the FEM analysis to ensure that the constitutive model parameters for the soil derived from calibration of the laboratory tests matched the in-

situ CPT test results. Once this was achieved it allowed for confidence in the results of the FE analyses of the footing tests presented in the next section.

Axisymmetric analyses were performed using the mesh shown in Figure 4b. The left-hand boundary was the axis of symmetry, the vertical and horizontal boundaries were fixed at the base, and horizontal displacements were restrained at the right-hand boundary. The mesh was 10 m wide and 21 m deep. Significant numerical efficiencies were achieved by placing a 1 m dummy layer at the top of the 20 m deep weightless soil deposit. Cavity expansion analyses were thus performed using a single mesh and modelling an increase in stress level (due to increasing penetrometer depth) by varying the unit weight of the material in the dummy layer. Because the CPT resistance is dependent on both soil strength and stiffness, the HS soil model was used in the cavity expansion analyses to model the actual stiffness value corresponding to applied stress level. The cavity expansion analyses were performed using a procedure described by Xu (2007), Xu and Lehane (2008) and Tolooiyan and Gavin (2011). The CPT q_c values predicted using the FE models are compared in Figure 4a with the q_c profile measured at Blessington. It is clearly evident that the results from the HS model, which was implemented using the soil properties derived from the lab test calibration procedure, provided a reasonably good estimate of the measured CPT q_c profile.

2.3 Footing Tests

The footing tests were modelled using Plaxis with 15 noded axisymmetric elements. A range of footing widths B , from 2 m to 5 m and depths, D of 1 m to 3 m was considered in the analyses. Because the results depend on the mesh size and density, a normalised model geometry, which depended on the footing width (See Figure 5a), was adopted. Each mesh thus contained 650 elements, 5600 nodes and 7900 stress points (See Figure 5a). In order to provide a realistic representation of the over-consolidated Blessington sand deposit, the soil was modelled in 2 m depth intervals with approximate (average) OCR and K_o values assigned to each layer.

The HS parameters from Table 1 were assigned to the soil elements whilst a linear elastic model with $\gamma = 23 \text{ kN/m}^3$ and $E = 2 \times 10^8 \text{ kPa}$ was used for the footing. An interface reduction factor (R_{int}) of 0.9 was applied at the soil-structure interface at the base of the footing, while $R_{\text{int}} \approx 0$ was used along the sides of the footing to prevent the development of shear stress. The concrete footing was loaded axially until the footing settlement reached $0.1 B$. Automatic mesh updating using the updated langrangian procedure was activated to accommodate the relatively large strains experienced. The condition of soil elements following the application of the maximum footing settlement (equal to 10% of the footing width) are show in Figure 5b. This shows soils elements which have reached the plastic state, cap and hardening points. Cap points occur when the stress state is equivalent to the pre-consolidation stress or the maximum stress level that has previously been reached. Hardening points occur when the stress state corresponds to the maximum mobilised friction angle that has previously been reached.

3 Results of FE Analyses

3.1 Effect of Footing Width and Depth on Bearing Pressure in Blessington Sand

The bearing resistance predicted for circular footings of varying width founded at 1 m depth in Blessington sand are shown in Figure 6a. The mobilised bearing resistance increased by 11 % as the footing width increased from 2 m to 5 m. In analyses where the footing width was constant ($B = 3$ m) and the footing depth was increased from 1 m to 3 m (See Figure 6b) the mobilised bearing resistance did not increase significantly as the footing depth increased.

The effect of footing width on the back-figured α values derived for all footing tests performed at $D = 1$ m are shown in Table 2. Whilst the 2 m wide footing mobilised the lowest bearing resistance, the back-figured α value ($\alpha = 0.45$) was highest for this footing at the average q_c value was also low. The α value mobilised for the other footing widths considered were identical ($\alpha = 0.43$).

The effect of footing depth on α values back-figured from tests where $B = 3$ m and the footing depth varied are shown in Table 2. It is clear therefore that although the bearing pressure increased only slightly as the footing depth increased, the average q_c value increased at a faster rate and therefore α reduced with increasing footing depth.

Whilst the trend for α values determined in the FE analyses performed on the Blessington dense sand to be relatively insensitive to variations in footing width disagree with observations from the FE analyses reported by Lee and Salgado (2005), the tendency for α to reduce with increasing embedment depth agrees with analyses reported by the same authors when they considered the effect of relative depth (D/B) on the α values mobilised by bored piles in sand. Lee and Salgado (1999) and others had suggested a dependence of α on the soils relative density D_r . In order to further investigate this effect, a suite of analyses were performed on sand deposits whose soil properties (required for definitions of the hardening soil model) were well defined and which had a range of relative density from 50% to 75%.

3.2 Effect of Relative Density

The sands chosen for this sensitivity analysis were Tanta sand from Egypt which has an in-situ $D_r = 75\%$ (reported by El Sawwaf, 2005 & 2009), Monterey sand from the United States placed at $D_r = 65\%$ (Wu et al., 2004 and Yang et al., 2008), and Hokksund sand from Norway with $D_r = 50\%$ (Tefera et al., 2006). The HS soil parameters considered for these sands are presented in Table 3. The CPT q_c profiles derived for these deposits are shown in Figure 7.

The variation of bearing pressure q_b as the footing width is shown in Table 4. All footings show a clear trend for q_b to increase as the footing width increased. The percentage

increase predicted as the footing width increased from 2 m to 5 m was 71% for Tanta sand, 53% for Monterey sand and 61% for Hokksund sand.

The variation of q_b as the footing depth increased (the footing width was constant at 3 m in these analyses) is shown in Table 4. An increase of bearing resistance with footing depth is evident at all sites. The percentage increase predicted as the footing depth increased from 1 m to 3 m was 19% for Tanta sand, 15% for Monterey sand and 20% for Hokksund sand. A tendency for q_b to increase with footing width and depth in soil deposits where the soil strength (as reflected by the CPT q_c profile) increases with depth is in keeping with the observations of Briaud (2007).

The $q_{b0.1}$ values normalised by the CPT q_c resistance (averaged over one footing width below the foundation to give α values) at all sites are compared in Figure 8. The following trends are observed:

- The largest α values were measured in Tanta sand and the lowest were measured in Blessington sand. There was no unique dependence of D_r on the α value developed, as the Tanta and Blessington sands had the highest D_r values of the four sites considered.
- Considering the effect of footing width B , in Figure 8a, relatively constant α factors were determined at all sites.

- The effect of footing depth on the α factors mobilised at all sites is shown in Figure 8b. A constant footing width of 3 m was used in the analyses. Whilst it was noted that a slight reduction of mobilised α with depth occurred at Blessington, reductions at other sites were more significant. The largest reductions occurred in Tanta sand, where α reduced from 0.78 ($D = 1$ m) to 0.58 (for $D = 3$ m).

3.3 Effect of Zone of Influence

In the normalisations performed to date a zone of influence corresponding to one footing width was used to calculate α values. However, a number of alternative depths for the zone of influence have been suggested in the literature. In order to investigate the possible influence of the zone of influence over which the CPT q_c values were averaged had on the results, a sensitivity analyses was performed using FEM analyses. In these analyses the effect of averaging the q_c values over a three possible zones of influence, z_i were compared. Burland and Burbridge (1985) and Lehane et al. (2008) consider the zone of influence to extend to $B^{0.75}$ beneath the footing and this was considered as a minimum value. Many of the empirical CPT methods considered in section 1 consider $z_i = B$. Since Tanta sand appeared to exhibit the largest range of predicted α values, the vertical displacement profile predicted during a series of analyses where the footing width was increased in this sand deposit were considered (See Figure 9a). It is clear that the depth of influence increased as the footing width increased. When the settlement and depth were normalised by footing width (See Figure 9b) it is apparent that displacements

were concentrated in a zone of influence extending a distance of up to $4.5 B$ below the footing. When the other sands were considered (See Figure 9c) it was shown that although the normalised settlement varied for the sands considered, with the zone of influence being smallest for Blessington sand, the z_i value of $3.5 B$ represented a reasonable depth at which normalised settlements could be considered to be small at all the sites considered.

The $q_{b0.1}$ values at all sites, normalised by the q_c values within the three zone of influence depths considered are shown in Figure 10. Considering the zone of influence to extend to $B^{0.75}$ below the footing, it is clear that α factors are relatively high, site dependent and increase strongly as the footing width increased or footing depth decreased. These effects are still evident when the zone of influence considered is $1.0B$. When the zone of influence is extended to $3.5B$, the α factors were seen to be quite close at all sites falling in a relatively narrow range of 0.32 to 0.39, with α reducing very slightly as the footing width increased or footing depth decreased.

In the FE analyses which considered a relatively large zone of influence ($z_i = 3.5 B$) back-calculated α values in a relatively narrow range from 0.32 to 0.39 were determined. These values did not vary significantly for the values of B , D_r and D considered. These trends are in keeping with database studies of field tests, although the α values are higher (by $\approx 50\%$) than those normally reported for footing tests. The possible effect of creep (which was not considered in the FE models described previously) on the α values mobilised in footing tests is described in the following section.

4 Effects of Creep from Field Tests

Significant creep effects have been reported in field studies of footing behaviour in sand by Briaud and Gibbens (1999) at the Texas A&M site and Lehane et al. (2008) from tests performed at the Shenton Park site, in Perth, Western Australia. In footing tests performed in Texas, (See Figure 11) creep became significant when the footing pressure exceeded 600 kPa. Both Briaud and Gibbens and Lehane et al, proposed creep models, that were essential to model the settlement response at their test sites.

Gavin et al. (2009) present the results of model footing tests performed in Blessington sand. Tests were performed on 100 mm, 250 mm and 400 mm wide square footings, at relative depths D/B varying from 0.4 to 2.0. The measured pressure settlement response is shown in Figure 12a and the normalised footing settlement response in Figure 12b. The data suggests an α value of ≈ 0.2 , which is independent of footing width or depth is appropriate for dense Blessington sand. This value is clearly significantly lower than the α values of 0.32 to 0.39 suggested by the FE analyses.

The model test data in Figure 12 represents a summary plot (which includes only the maximum settlement measured during a given time increment) of measured and predicted response from footing tests in which the applied pressure was applied in increments (each increment corresponded approximately to 10% of the ultimate resistance) for a time period of 10 minutes or until the settlement reduced below 0.2 mm/minute. When the full pressure-settlement curve for the test on the 250 mm square plate is plotted in Figure 13,

it is clear that the effects of creep were significant when the applied pressure exceeded 1,000 kPa. Similar behaviour was measured in all of the tests.

In order to eliminate creep effects in tests performed in dense Blessington sand, a fast load test was performed on a 200 mm plate, where the total loading time was 370 seconds. The pressure-settlement response from this fast test is compared with that measured in the Maintained Load Test (MLT) using the same plate tested within 2m of the fast load test in Figure 14. At a normalised settlement of less than 10%, the footing loaded quickly mobilised a much higher (1000kPa) bearing pressure than the footing loading using the maintained load test procedure. As a consequence the α factor ($\alpha = 0.39$) measured for the fast load test shown in Figure 14 was significantly higher than suggested from the maintained load tests ($\alpha = 0.3$) and is compatible with the FE analyses which ignore creep effects.

The footing tests performed at Texas A&M (See Figure 1) were maintained load tests with each load increment being maintained for a period of 30 minutes. The pressure settlement model developed by Gavin et al., 2009, which includes for creep response using the model developed by Lehane et al. (2008) was used to predict the footing settlement was used to investigate the effect of creep on the measured response of the 1.5 m wide footing in Figure 15. It is clear that the α value was quite sensitive to creep time and varying between 0.26 when the full 30 minute creep time was modelled and increasing to 0.34 when the creep time was set to zero.

5 Conclusions

Finite element analyses of footings tests on sand was performed in order to investigate the effects of footing width, B footing depth, D and relative density, D_r of sand on the empirical correlation factor α linking the pressure mobilised at a footing displacement of 10% of the footing diameter and the CPT q_c resistance mobilised in the zone of influence. Whilst α factors varied with B , D and very significantly with D_r , when the zone of influence, z_i over which q_c values were averaged was calculated using approaches commonly used in practice (i.e. $z_i = B$ or $B^{0.75}$). When a much larger zone of influence was considered ($z_i = 3.5B$) relatively constant α values were determined for all D_r profiles considered. These latter α values did not vary significantly with B or D and were in keeping therefore with trends observed from footing tests performed in the field.

Notwithstanding the relatively constant α factors determined in the FE analyses, these factors were typically 33 to 50% higher than those reported from field tests. The effects of creep settlements which develop at relatively high applied stress level, and therefore affect the α value mobilised in field tests was discussed. The α values derived from numerical analyses in this paper are appropriate to situations where creep settlement do not occur, for example for very rapid loading or alternatively could be adopted where numerical analyses are developed to allow for quantification of creep effects in sand.

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(a) applied pressure versus settlement

(b) normalised pressure versus settlement

Fig. 1. Footing test results from Texas A&M University (after Briaud and Gibbens, 1999)

Figure 2 Effect of soil stiffness of non-linear response of footings (Gavin et al. 2009)

Fig. 3. Trend for $q_{b0.1}/q_c$ to vary with footing width and sand relative density (Lee & Salgado, 2005)

Fig. 4a CPT q_c profile estimated using cavity expansion analysis

Fig. 4b. Plaxis FEM geometry and cavity area

Fig. 5. (a) Model geometry and mesh elements for a footing test, (b) Plastic and hardening points when footing settlement is reached to $0.1B$

(a)

(b)

Fig. 6. Results of FE analyses for Blessington Sand, (a) effect of footing width (b) effect of footing depth

Fig. 7. Estimated CPT q_c profile for Hokksund, Monterey and Tanta sand

Fig. 8. Effect of relative density on predicted $q_{b0.1}/q_{c1.0B}$

(a)

(b)

(c)

Fig. 9. Depth of influence depth below the centre of footings (a) Tanta sand, (b) normalised influence depth in Tanta sand, (c) normalised influence depth in all sands

(a)

(b)

Fig. 10. Normalised $q_{b0.1}$ using varying influence depth; (a) different width, (b) different depth

(a)

(b)

Fig. 11. Pressure-settlement curves showing creep from footing tests in Texas A&M

(a)

Fig. 12. Effect of footing size on bearing resistances at Blessington site: (a) pressure-settlement response; (b) normalised pressure-settlement response (Gavin et al., 2009)

Fig. 13. Pressure-settlement plot for 250 mm footing test at Blessington including creep

Fig. 14. Effect of loading rate on normalised bearing pressure, α

Fig. 15. Comparison between measured and predicted pressure settlement curves
measured on Texas A&M sand (footing width=1.5 m)

Table 1. HS parameters of Blessington sand

Parameter	Parameter Value
Unit Weight γ (kN/m ³)	20
E_{50}^{ref} (P_{ref} =100kPa) (kPa)	44000
E_{ur}^{ref} (P_{ref} =100kPa) (kPa)	155000
E_{oed}^{ref} (P_{ref} =100kPa) (kPa)	25000
Cohesion (kPa)	0.0
Ultimate Friction Angle (°)	42.4
Ultimate Dilatancy Angle* (°)	6.6
Poisson's ratio	0.2
Power m	0.4
R_f	0.8
Tensile Strength (kPa)	0.0
e_{init}	0.373
e_{min}	0.373
e_{max}	0.733
D_r (%)	100
P_{ref} (kPa)	100

* Ultimate dilatancy angle (ψ_m) has been estimated using $\sin \psi_m = \left(\frac{\sin \phi_m - \sin \phi_{cv}}{1 - \sin \phi_m \sin \phi_{cv}} \right)$

Table 2. Effect of varying footing width and depth on α mobilised at Blessington

Width, B (m)	Depth, D (m)	$q_{b0.1}$ (kPa)	$q_{c\ 1.0B}$ (kPa)	$\alpha = q_{b0.1}/q_{c\ 1.0B}$
2	1	5,301	11,898	0.45
3	1	5,463	12,818	0.43
4	1	5,808	13,568	0.43
5	1	6,064	14,203	0.43
3	1	5,463	12,818	0.43
3	2	5,445	14,544	0.37
3	3	5,589	15,769	0.35

Table 3. HS soil parameters for Tanta, Monterey and Hokksund sand

Parameter	Tanta	Monterey	Hokksund
Unit Weight γ (kN/m ³)	18.90	16.05	15.1
E_{50}^{ref} (P_{ref} =100kPa) (kPa)	40000	35000	20000
E_{ur}^{ref} (P_{ref} =100kPa) (kPa)	120000	105000	100000
E_{oed}^{ref} (P_{ref} =100kPa) (kPa)	40000	35000	25000
Cohesion (kPa)	0.0	0.0	0.0
Peak Friction angle (°)	40	36.7	34
Dilatancy angle (°)	10	6.7	2.5
Poisson's ratio	0.2	0.2	0.2
Power m	0.7	0.5	0.5
R_f	0.9	0.85	0.9
Tensile Strength (kPa)	0.0	0.0	0.0
e_{init}	0.377	0.661	0.760
e_{min}	0.305	0.541	0.570
e_{max}	0.593	0.885	0.950
Dr (%)	75	65	50
P_{ref} (kPa)	100	100	100

**Table 4. Effect of footing width and depth on predicted $q_{b0.1}$ (kPa) of Tanta,
Monterey and Hokksund sand**

Width, B (m)	Depth, D (m)	Tanta	Monterey	Hokksund
2	1	2,436	1,605	881
3	1	3,146	1,946	1,095
4	1	3,683	2,223	1,269
5	1	4,160	2,467	1,424
3	1	3,146	1,946	1,095
3	2	3,497	2,139	1,217
3	3	3,798	2,237	1,301