Modelling of shallow foundations on homogeneous and layered soils

Modèle physique du chargement des fondations superficielles sur des sols homogènes et stratifiés

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ABSTRACT: Major Swiss development zones lie mostly upon infilled sedimentary deposits where layering becomes an important feature. Swiss foundation design methods are revisited to focus on these typical soil conditions. Other tools are also introduced, compared and applied to calculation examples to clarify the range of results and interpretation for similar soil properties; e.g. numerical and physical modelling. The physical modelling has been carried out in the geotechnical centrifuge facilities in Bochum and Zürich.

RÉSUMÉ: Les régions majeures de développement en Suisse sont situées dans des zones de remplissement donc les sols ont souvent une structure stratifiée. Les méthodes de construction Suisses sont revues par aux configurations de sol spécifique. Des autres outils de calcul pour des fondations, en comparaison avec ceux appliqués en construction Suisse, sont utilisés pour montrer la vaste zone d'interprétation des résultats; en spécial les méthodes numériques. Une série de tests a été faite en utilisant les centrifuges de l'école polytechnique de Zürich et de la Ruhr-Universität a Bochum pour avoir une base de comparaison pour les résultats analytiques et numériques.

1 INTRODUCTION

Calculating the failure load at ultimate limit state and the settlement of a circular shallow foundation appears to be a relatively straightforward task if axisymmetric geometry and homogeneous soil conditions are assumed. The latter simplification is often made, even though it does not always represent reality. In consequence, high global factors of safety are applied to the ultimate limit state load in order to limit deformations to within tolerance.

For many standard cases, (e.g. dense granular soils) these factors are much higher than necessary, but for some soil configurations, for which the same calculation procedures are used, safety is not always assured. When geometry conditions are more complex or the soil is not homogenous, calculations may generate either hazardous or economically inefficient results.

Suggestions can be made on how to improve the design methods in Switzerland, in order to improve economy while still maintaining a safe design. The recommendations are based on numerical analysis using finite element and finite difference methods as well as physical modelling in the centrifuges installed at the Ruhr-Universität Bochum (Jessberger & Güttler 1988) and at ETH Zürich (Springman et al. 2001).

2 METHODS

There are many different approaches adopted to examine soil-structure interaction. The design methods mostly used in practice are still based on the classical plasticity calculations, which are simplified for easy use. Numerical modelling by computer analysis is used more or less as a control tool to understand the mechanisms arising as a result of the ongoing processes and to estimate the stress distributions and to quantify the deformations.

The constitutive model should set a balance between representing real soil behaviour and the simplicity necessitated in selecting appropriate parameters. Idealisations of the geometry, loading system and boundary conditions will also be made, and the combined effect should be investigated by means of sensitivity analyses to establish the level of uncertainty.

It can be shown, at least in coarse-grained soils, that density has a major impact on all the main parameters used for classical calculations and numerical modelling. It is also one of the easiest to control when building physical models e.g. in a centrifuge. Input values (e.g. stiffness and strength) for many numerical calculations are often derived as functions of density, for which linear assumptions are adequate within the relevant range.

To ensure a comparable database for the numerical and physical analysis, prototypes of the geometry and the soil properties have been assumed. The parameters for the sand relate to the material used in the centrifuge tests at ETHZ. A lacustrine clay from the site of Birmensdorf, deposited in the forefield of a retreating glacier, will be used in the upcoming centrifuge tests on layered soils. Although in situ tests show the soil to be normally consolidated with an undrained shear strength increasing with depth (Panduri 2000), a constant value will be selected for the preliminary analyses to reflect lightly overconsolidated conditions and the constraints of modelmaking in the centrifuge. The values of γ (unit weight), c' and ϕ' (drained shear parameters), s_u (undrained shear strength), and E_0 (one dimensional stiffness) are given in Table 1:

Table 1. Soil properties for numerical analysis.

	general	γ	ϕ'	c'	s_u	E_0
	-	kN/m³	°	kN/m²	kN/m²	kN/m²
Soil 1	Sand	14.7	33	-	-	15000
Soil 2	Clay	15	25	-	20	5000**

* ϕ' is also selected as a variation of γ, ** 1,000,000 kN/m² for undrained calculation

Table 2. Foundation geometry (prototype).

Shape	Type	Diameter
circular	rigid	1.4 m

The diameter of 1.4 m (Table 2) corresponds to the prototype diameter of the foundations tested in Zurich. Parameters for the back analysis of the centrifuge tests at the Ruhr-Universität Bochum - for geometry and soil - are shown as needed later.

The load distribution below a foundation is different for the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS) and also depends on the rigidity of the footing. Plasticity calculations at the ULS after Prandtl (1920) assume nonetheless that this is uniform. The calculation at the SLS generally surmises that a constant pressure is applied to the soil surface, but this is only valid for flexible foundations. In this case, the settlement profile is determined from e.g. a Boussinesq (1885) stress distribution and a simple one-dimensional calculation is carried out at the center and edge of the footing. A uniform settlement is calculated for stiff foundations assuming an even distribution of applied load, together with a one-dimensional response and a characteristic stress distribution beneath a critical

2.1 *Analytical calculation methods - Ultimate Limit State ULS for homogeneous soils*

The design procedure in Switzerland for the geotechnical aspects of many structures is a two-phase process, considering the ULS and the SLS in an uncoupled fashion.

ULS calculation follows the bearing capacity formulas developed originally from Prandtl (1920) for plastic indentation of metals, and is based on two failure mechanisms for homogenous soil separating undrained (constant volume) from drained conditions (e.g. Terzaghi & Peck 1948).

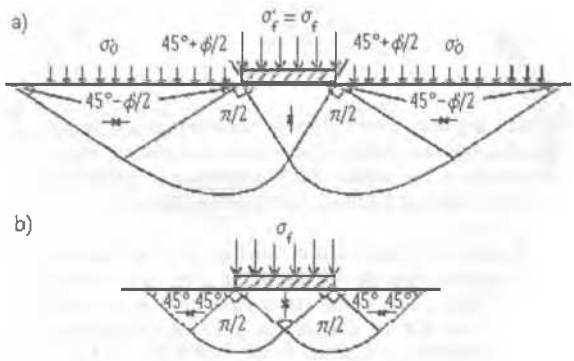

Figure 1. Failure mechanisms: a) drained b) undrained conditions.

In the past, the bearing capacity (σ_f or σ'_f) was divided by a global factor of safety of 2 and compared with an expected working load. However, the new European code philosophy includes partial factors of safety to account for uncertainty in the geometry, magnitude of loads and soil resistance. In this way, a reduced "design" bearing capacity is compared with a design working load.

The solution from Terzaghi & Peck (1948) is quoted for vertical loading of a flexible footing on homogeneous soil in plane strain conditions:

$$\sigma_f = c \cdot N_c + (\gamma \cdot t + q) \cdot N_q + \frac{1}{2} b \cdot \gamma \cdot N_\gamma \qquad (1)$$

c can be either c' or s_u depending upon whether drained or undrained conditions are to be considered, t is the depth of embedment and b is the breadth of the footing, q and/or $\sigma_o = \gamma\, t$ are surface loads and N_c, N_q and N_γ are bearing capacity factors.

For a shallow foundation on the surface of granular material (c' = 0) with no surface load, Equation 1 changes to:

$$\sigma_f = \frac{1}{2} b \cdot \gamma \cdot N_\gamma \cdot s_\gamma \qquad (2)$$

where s_γ = shape correction factor accounting for geometry. For undrained conditions, with shape correction factor s_c:

$$\sigma_f = s_u \cdot (2 + \pi) \cdot (1 + s_c) \qquad (3)$$

The following assumption for N_γ has been taken from Hansen (1968):

$$N_\gamma \cong 1.8 \cdot (N_q - 1) \cdot \tan \phi' \qquad (4)$$

with

$$N_q = e^{\pi \cdot \tan \phi'} \cdot \tan^2 (45° + \frac{1}{2} \phi') \qquad (5)$$

The variation in the calculation of bearing capacity has been reviewed Europe-wide by Sieffert & Bay-Gress (1998), as a function of different calculation methods quoted in design codes of 11 European countries. The results, simply for homogeneous soils, are widely dispersed (by a factor of 2), probably because calculation methods and safety factors have evolved empirically to suit local conditions. This demonstrates a less than convincing case for a unique Paneuropean set of factors and rules.

For design of structures founded on layered (stiff-coarser grained/soft-finer grained) soils, four methods may be used (based on Lang et al. 1996, and common Swiss design practice): 1. model the soil as homogeneous and soft-finer grained (which gives a minimum bearing capacity), 2. model the soil as homogeneous and stiff-coarser grained (which gives a maximum bearing capacity), 3. model the soil as a homogeneously mixed material with average properties out of the two components or 4. search for the critical failure plane with the weakest resistance for given load and geometrical boundary conditions.

The extremes of the ultimate bearing capacity for the first three possibilities are shown in Figure 2 as a function of soil density. The clay is treated as normally consolidated in the drained analysis, so that there is no 'apparent' cohesion c' and nor is there an effect from density. Based on laboratory tests, a variation in the sand density is reflected by changing the shear parameters. To average the properties of the soil layers makes no sense physically in this case, nonetheless it is astonishing how often this 'fudge' is adopted.

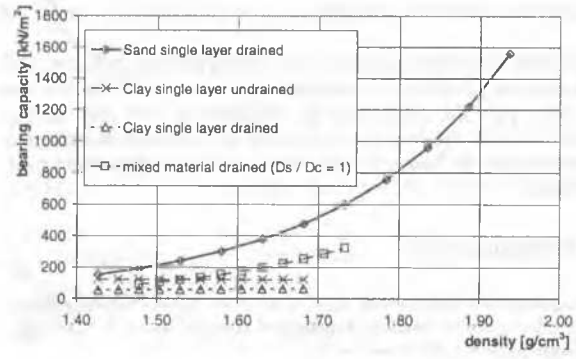

Figure 2. Bearing capacity calculated with the bearing capacity formula of Terzaghi for different soil configurations depending on density.

2.2 *ULS calculation methods for layered soil conditions*

An extrusion mechanism can be proposed for layered soil strata as a more realistic mechanical idealisation of the problem e.g. Bolton (1995) after Calladine (1985). It is assumed that a fine grained soft material (Tresca material, $\tau_{max} = s_u$) lies between comparably stiffer and coarser grained layers.

Uniform states of stress are represented within kites such that the Mohr Circles for each kite are displaced along the σ axis by s_u due to the rotation of principle stress directions through 30° (Figure 3a). The rotation of the principal stress at the edge of the footing from vertical to horizontal is covered in elements a-d with $\Delta\sigma_{f(a\text{-}d)} = 5\ s_u$ (Figure 3b). The pole for element e is P_e, such that the vertical stress is $\sigma_{f(e)} = 5.5\ s_u$ acting on a length $l_e = 1.54$ D (also $\tau = -0.87\ s_u$). This defines the maximum radius

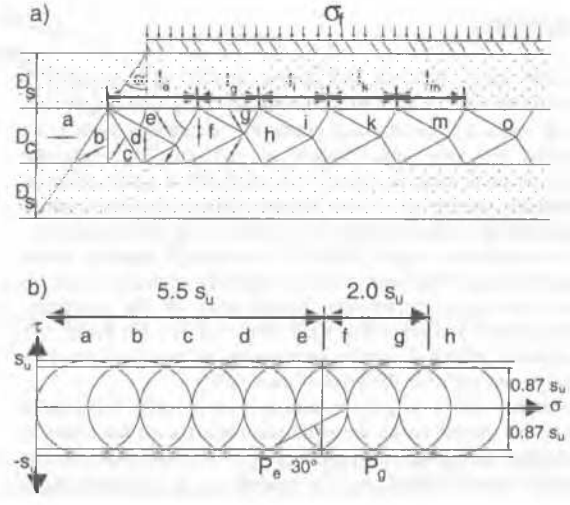

Figure 3. a) extrusion mechanism for layered soils ($D_{sand} = D_{clay}$) for a circular footing of infinite diameter b) related Mohr circles for the stress states for the kites.

for such an extrusion mechanism to develop (fan) and for wider footings, the kites (each containing two right angles) develop with horizontal length and the associated vertical stress, as given in Table 3, e.g. with pole P_g and $\sigma_{f(e-g)} = 2\ s_u$. The value for a system containing the fan and two kites (elements a-i) in axisymmetric plane can be derived e.g. by Equation 6.

Table 3. Kite length and allowable normal load.

kite	e	g	i	k	m	o
length	1.54 D	1.03 D	1.20 D	1.14 D	1.16 D	1.15 D
normal load	5.5 s_u	7.5 s_u	9.5 s_u	11.5 s_u	13.5 s_u	15.5 s_u

$$\sigma_f = A_{(i)} \cdot \sigma_{v(i)} + A_{(g)} \cdot \sigma_{v(g)} + A_{(e)} \cdot \sigma_{v(e)} \qquad (6)$$

with $A_{(i)}$ area of a circle with a radius of the length of kite i and and $A_{(g)}$ and $A_{(e)}$ area of the annuli of width l_g respectively l_e.

Figure 4 shows the influence of the ratio r/D in relation to normalised stress at failure. The load distribution angle is varied between the friction angle ϕ' of the sand and 0°. Most likely, a load distribution angle between $\phi'/4$ and $\phi'/2$ is realistic either following codes (DIN 4017-100 1996) or as measured in model tests (Hartmann 1997). The evaluation of the net vertical load on the top sand layer needs to allow for the fact that the weight of the sand layer of depth D_{sand} (on both sides of the mechanism) can be ignored assuming no penetration prior to failure.

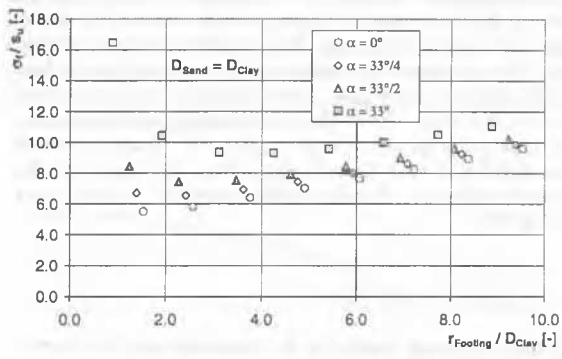

Figure 4. Bearing capacity for layered soil (extrusion mechanism).

2.3 *Serviceability Limit State SLS*

The SLS calculation is based on elastic load-settlement behaviour determined from stress distribution under a structure found via Boussinesq's (1885) solutions. Settlement is calculated from a working load without applying any factors of safety and is compared with allowable SLS deformation.

$$s = \sum_{z=0}^{z=T} \Delta s = \sum_{z=0}^{z=T} \frac{\Delta\sigma_z \cdot \Delta z}{E_{0(z)}} \qquad (7)$$

with s = settlement, T = total depth, z = layer depth, $E_{0(z)}$ = compression modulus in layer, and σ_z = vertical stress in layer. σ_z is derived from the applied load σ multiplied by a vertical influence factor J (Lang et al. 1996).

A calculation of the deformations under undrained loading behaviour gives an even stiffer response to the footing on the layered soil stratum than a pure sand model because there is no volumetric strain in the clay in this condition. Comparable results are shown in Sections 3 & 4 from the numerical and physical modelling.

Traditional calculation methods for determining the ULS have in common that the deformation path during the loading up to failure is unknown since ideal plastic soil behaviour is assumed. Consequently, a working load is defined based on the ULS load and a safety factor, so that the assumption of elastic behaviour is more or less representative for the calculation of deformations, which must lie within the SLS requirements.

3 PHYSICAL MODELLING – CENTRIFUGE TESTS

Centrifuge modelling offers the opportunity to investigate soil-structure interaction under well-defined and controlled conditions with comparable quality to full-scale tests or measurements on site and to obtain data sets for a parametric study of relevant variables. Such data from physical modelling may be validated by existing analytical methods and is a key factor in being able to validate new numerical approaches or to calibrate the selection of appropriate parameters.

3.1 *Physical modelling at the Ruhr-Universität Bochum*

A first series of tests have been conducted at the Geotechnical Centrifuge of the Ruhr-Universität Bochum in Germany (Bucher 1999, Nater 1999). Although the same geometric prototype was modelled, the density and layering was varied with single and two layer sand models and some footings were also exposed to cyclic loads. As the tests have been carried out on sand - the stiff coarser grained material for the layered model - they represent a variation of the upper boundary solution mentioned in chapter 2.1.

All footings for the test series in Bochum were made out of 2 cm thick aluminium plates with diameters of 0.028 m, 0.056 m and 0.075 m, so that they behaved rigidly. They were connected directly to a hydraulic jack assuring central and vertical loading. The prototype for all tests has a diameter of 1.68 m, which is achieved by combining different footing diameters and g levels, following the scaling laws as given by 0.028 m times 60 g or 0.056 m times 30 g or 0.075 m times 22 g (e.g. Schofield 1980). Up to 6 tests could be carried out in the same model container (1 meter diameter) without causing significant interaction between the mechanisms.

All tests presented in this chapter have been run under load-controlled conditions so that load-settlement behaviour is not expected to be similar to the deformation-controlled tests conducted in the ETH Zürich centrifuge described in Section 3.2. All following load-settlement graphs are shown in g invariant measures to maintain comparability.

Modelling of models is well proven in this test series (Figure 5). The load-settlement curves followed a stepped form because local rupture zones appeared at regular intervals due a combina-

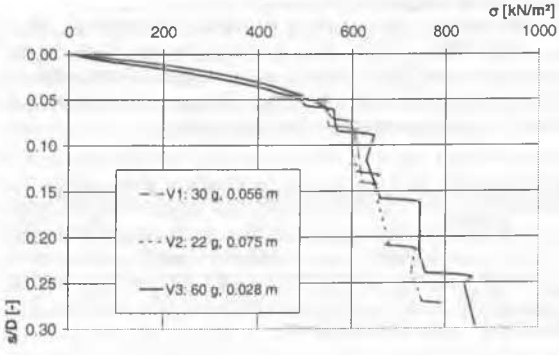

Figure 5. Load-settlement curves (modelling of models) for different centrifuge tests on the same sample (RUB); I_D = 88 %.

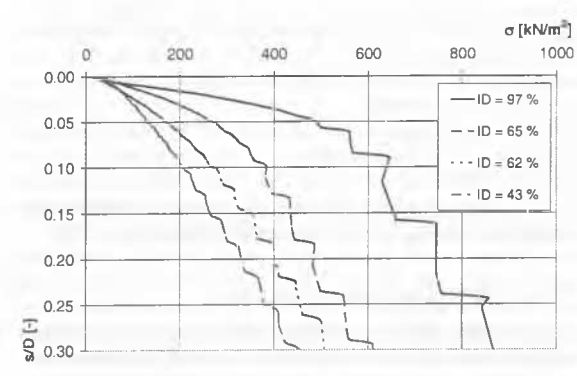

Figure 6. Load-settlement curves for different relative densities (RUB).

tion of dilatancy, crushing and particle rearrangement, which led to progressive failure. This is quite different from the bearing capacity as derived from plasticity calculations. In reality, as the footing becomes more embedded, an asymptotic value of bearing capacity will be approached, reflecting the influence of the $\gamma\, t\, N_q$ term in Equation 1. The influence of the density as the major influencing factor for the ULS calculation is clearly visible in Figure 6.

3.2 *Physical modelling at the ETH Zürich*

The Institute of Geotechnical Engineering at the ETH Zürich has set up a drum centrifuge configured as follows: diameter 2.2 m, max. model weight 2000 kg, max. model width 0.7 m, max. model depth 0.3 m, max g level 440 g (Springman et al. 2001). These tests are the first carried out with the newly commissioned centrally mounted actuators (Ducksch 2001, Graemiger 2001)

An extremely loose homogeneous sample has been built in the centrifuge by spraying sand from a centrally mounted spinning disk to settle through water to deposit on the drum wall. Since relative density is based on the determination of a minimum and a maximum value, which are derived from specified laboratory procedures, it is not surprising that relative densities higher than 100% and lower than 0% are possible. In this case, I_D = -22%. In pluviating the sand through water, sediments laid down in the drum under such buoyant conditions (i.e. lower effective stresses) will be extremely loose, and definitely lower than those established by means of the minimum density determination given by most laboratory testing standards.

A series of 10 displacement controlled footing tests have been carried out on this loose sand sample by loading a footing at different places on the surface with an actuator device. Several loading speeds and 2 footing shapes (slightly curved footing to fit the curved soil surface and a flat footing surface) have been compared without noticing significant differences in the results. Diameters and thickness of the footings are similar to those used for the centrifuge tests at the RUB: d = 0.028 m and d = 0.056 m. The applied g-levels of 25 g, 50 g and 75 g increase the model diameter of 0.028 m to prototype diameters of 0.7 m, 1.4 m and 2.1 m respectively and the model diameter of 0.056 at 25 g to a prototype of 1.4 m. To check the homogeneity of the model, thin-walled tube samples have been taken after the tests to determine density together with 22 CPT soundings. From this data, input parameters have been proposed for numerical modelling.

Figure 7 shows results from this test series. The modelling of models, the middle curves (V2, V3) of Figure 6, representing the same prototype diameter of 1.4 m, does not agree here. It is likely that, for this exceptionally low density, a major stiffening effect takes place in the soil sample with increasing g–level and this is manifest in the stiffer load-deformation response for the same prototype at the higher g-level. The density is no longer similar when comparing the 25 g test with the 50 g test.

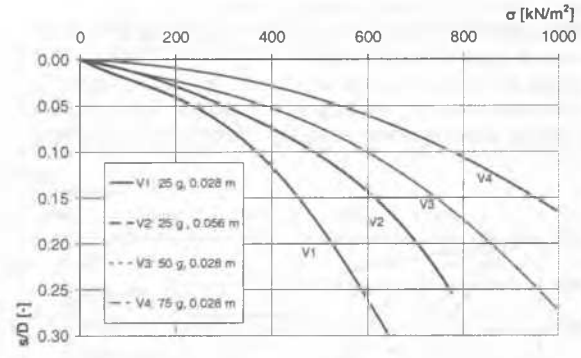

Figure 7. Load-settlement curves including modelling of models (V2, V3) for different centrifuge tests on the sample in Zürich; I_D = - 22 %.

3.3 *Results of tests under layered conditions*

As no tests on layered soils have been conducted yet in Zurich, the results of centrifuge tests on two layer systems with sand and clay are mentioned (Hartmann 1997). These tests have been carried out to check the punching failure calculation method (DIN 4017-100 1996). The load-settlement curves and the bearing capacity of an axisymmetric footing on a stiff sand layer, with varying height between zero and three times the diameter of a model footing, overlying an overconsolidated kaolin sample have been obtained (Figure 8).

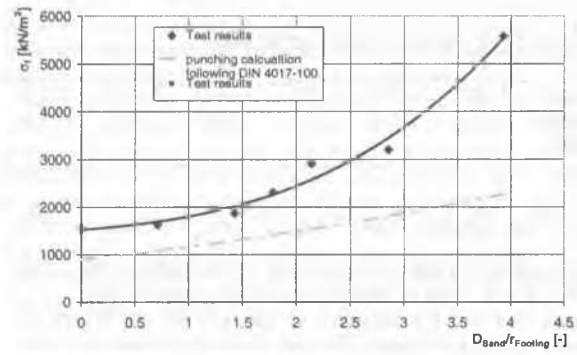

Figure 8. Bearing capacity in dependency of the thickness of the sand layer compared with the result of calculations following DIN 4017-100.

It shows unequivocally that the influences of the different layers have to be taken into account in an ultimate limit state calculation, and that no single mechanism should be assumed. The difference between the measured and calculated bearing capacities is due to bearing capacity factors, which causes the design solution to lie on the safe side. The two curves follow different trends. The calculation for punching failure requires that the load is distributed through the sand layer at an angle of 7° to the vertical (DIN 4017-100 1996). The results show that this assumption is appropriate up to a ratio of D_{sand}/r = 1.5. Load distribution angles determined after the test series from the inprint on the clay layer were up to 23° with increasing ratio D_{sand}/r for a sand with ϕ' of 38°.

4 NUMERICAL MODELLING

Numerical continuum analysis is an appropriate tool for investigating the coupled load-settlement behaviour. Increments of deformation and stress as well as the current state may be examined for each calculation step.

A series of laboratory tests are available, to obtain a set of input parameters for numerical modelling. Oedometer tests for uniaxial compression index E_0, triaxial and simple shear tests under drained conditions for friction angle ϕ', with dilatancy ψ determined from simple shear, were carried out with initial density and controlling stress variations.

The finite element (FE) program PLAXIS (1998) has been selected for this preliminary analysis. It is relatively easy to use but input as well as output tools are limited on some inbuilt features. The Mohr Coulomb failure criterion has been used with parameters representing the physical model in the different centrifuge test series.

FLAC (2000) software is a flexible tool for numerical finite difference (FD) analysis. Nearly every parameter can be controlled and varied as needed using an inbuilt programming language called FISH. This has the disadvantage that even straightforward modelling becomes more complex. Finite difference tools can improve the modelling of large strain problems. The Mohr Coulomb criterion has also been used.

Table 4. Parameters for homogeneous sand for FLAC (RUB at 60 g.).

	general	γ	ν'	E_o**	c'**	ϕ'	Ψ
	-	kN/m³	-	kN/m²	kN/m²	°	°
Soil	Sand	16.9	0.33	15,000*	0.001	38	0

*average value, **needed for calculation

Table 5. Parameters for homogeneous sand and layered model, for both FLAC and PLAXIS (ETH Zürich at 25 g).

	general	γ	ν'	E_o^{**}	c' or s_u^{**}	ϕ'	Ψ
	-	kN/m³	-	kN/m²	kN/m²	°	°
Soil 1	Sand	14.7	0.33	15,000*	0.001	33	0
Soil 2	Clay	15.0	0.30	5000 or 1000000	20	0	0

*average value, **needed for calculation

Both calculations have been run with constant uniaxial stiffness derived from an average value of E_o from the oedometer tests. Other footing tests have been modelled with variable stiffness with depth, which showed slightly better agreement with the test results. Hyperbolic correlations of a stress-deformation relationship could be implemented (e.g. Laue 1996) or more sophisticated constitutive models with stiffness varying with strain could be applied.

Results of both numerical methods are shown in Section 5, in the form of the average stress distributions under the footing together with analytical solutions (Section 2) and centrifuge tests (Section 3), all of which examined the same prototype.

5 COMPARISON OF THE DIFFERENT METHODS

Figure 9 shows a comparison of centrifuge and FD results from loading the 1.68 m diameter footing on a homogeneous sand deposit (RUB). The load-displacement curve fits well over the entire loading range. However, it has not been possible to reproduce numerically the stepwise form indicative of local rupture failures, because constitutive relation adopted is unable to model this aspect of soil behaviour.

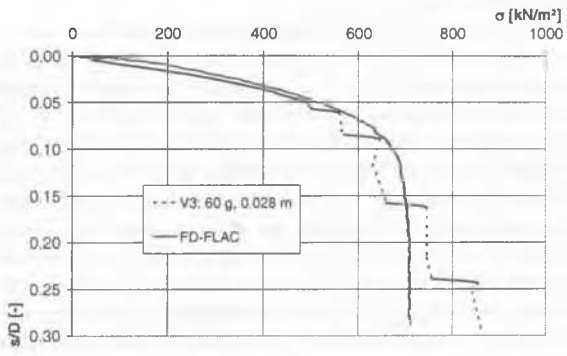

Figure 9. Load-settlement curves: FLAC together with RUB centrifuge model, 60 g test, 0.028 m footing diameter, dense sample; I_D = 97 %.

Figure 10 shows results of loading a 1.4 m diameter footing on homogeneous sand, for different modelling methods. A centrifuge test represents the physical model, and the bearing capacity is defined following the proposal from Bay-Gress et al. (1999), as the intersection point of the tangent on the load-settlement curve at large settlements with the load axis.

The results from two different numerical methods are also added and agree well with the centrifuge data up to 200 kN/m². The classical ideal elastic-plastic analytical approach is shown to be ineffective for this loose soil. Even though the influence of settlement on the ULS calculation in terms of depth of embedment has been neglected, the effect would be marginal.

In a strain controlled footing test with a footing fixed for no rotation and an ideal normal load, no unique ULS will be found in physical models under real stress conditions. The load-settlement curve will always show a small increase of load with increasing settlement (e.g. Laue 1996, Bay-Gress 2000, Ducksch 2001). To compare the ultimate limit state with the analytical solutions, a clear definition of the rupture load has to be found and applied.

Using the Bay-Gress et al. (1999) criterion, the bearing capacity is about 400 kN/m² whereas, that derived from the analytical solution is about 200 kN/m², which is roughly half (Figure 10). In traditional working load design, this value has to be divided by a factor of 2 (Lang et al. 1996) so that the allowable working stress would be reached at about 100 kN/m², whereas a more economic design would base the judgement on tolerable settlement.

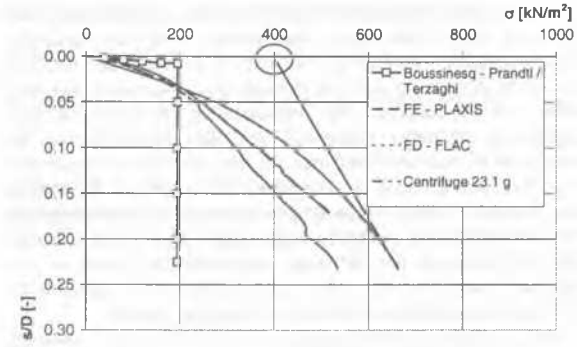

Figure 10. Load-settlement curves: Analytical solutions with Boussinesq SLS and Terzaghi ULS, numerical analysis with PLAXIS and FLAC, ETH centrifuge model at 25 g, 0.056 m diameter footing, loose sample (I_D = -22%) together with construction for bearing capacity determination.

For multilayered problems, different analytical approaches for homogeneous clay or sand bound the results from the FE analysis. For undrained behaviour the extrusion mechanism generates for r/D = 1.4 an ULS in between the two boundaries of the ULS for sand (upper boundary) and for clay (lower boundary). In comparison with the numerical calculation for r/D = 1 the value is still lower. It may be seen from inspection of Figure 4, that there will be a minimum value of σ_f between 1 < r/D < 2. Therefore it is thought that the extrusion calculation for r/D = 1.4 and for α = 33/4° is likely to be a minimum. Another cause may be due to the chosen angle of distribution $\phi'/4$ resulting from Hartmanns test's. An angle of distribution of $\phi'/2$ (compare Figure 4) would shift the predictor to the right closer to the result from the FE calculation (Figure 11).

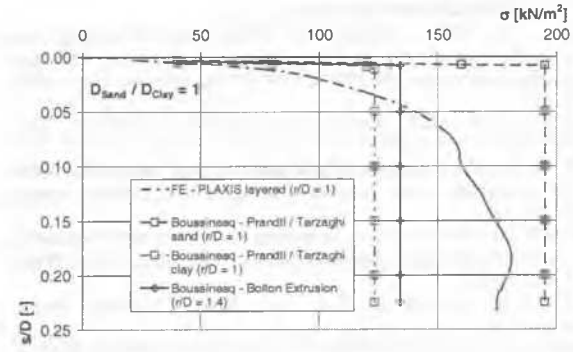

Figure 11. Load-settlement curves: PLAXIS together with analytical solutions and Bolton's extrusion (soil parameters given in Table 5).

6 CONCLUSIONS

Ever more powerful calculation facilities will be available in the future. Numerical analysis is already becoming a cheap and adequate tool for safe and economic design of many geotechnical structures. These aids to design have to be implemented and assessed critically by comparing the predictions with data obtained from closed form solutions, field monitoring, laboratory tests and/or physical modelling under appropriate stress conditions, e.g. in a centrifuge. Data from these physical models or the field are then used to validate new numerical models or to calibrate parameters for use in proven numerical methods.

At small loads, an elastic approach for calculating settlement underneath a footing using Boussinesq's theory seems reasonable for first assumptions but not for sufficient determination of the real load-settlement curve over a wider range of loads. Better simple design methods are available, but they also suffer from significant simplification, especially in the case of layered soils.

Other methods such as numerical modelling using FE and FD

calculations are more suitable for predicting a continuous load-settlement path. Even using basic elastic-plastic soil models in these programs, with coarse assumptions on parameter distribution, better agreement with the physical model data and thus with reality can be achieved. By implementing more sophisticated constitutive relations, numerical modelling can improve the description of real soil behaviour but there may be more parameters, which are difficult to determine. The quality of the modelling is directly related to the quality of the input parameters, the numerical technique (whereby large strains often poses difficulties), and also of the thorough interpretational work of the engineer. Each new numerical model should be validated by physical data according to the main influencing factors.

For practical use the development should focus on a method based on the stress-strain curve in which a tolerable deformation (converted to strain) may be used to read off a maximum allowable working load.

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