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Soils and Foundations 64 (2024) 101408
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Technical Note

Settlement prediction of shallow foundations for quality controls of sandy hydraulic fills

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Received 20 February 2023; received in revised form 21 November 2023; accepted 5 December 2023

Abstract

This paper describes a procedure for settlement prediction of shallow foundations on carbonate sands, but it is fully applicable and valid for siliceous sands. For practical purposes, the design of shallow foundations resting on medium dense and dense granular soils is typically governed by limiting settlement to tolerable values. Predicting foundation settlement is therefore important, but in standard practice it is necessarily based on indirect (and therefore often conservative) determinations of soil compressibility (or modulus), due to the intrinsic difficulties in obtaining direct measurements.

While numerical analyses incorporating non-linear soil behaviour may be a preferred method for computing expected total and differential settlement of shallow foundations of given geometry and stiffness on sand under static loading, the method described in this paper consists of a simplified and expeditious method based on equivalent linear elasticity. The method uses: i) the elastic soil stiffness profile at small strain, $E_0(z)$ obtained from the shear wave velocity as the primary measurement of deformability and ii) the reduction in modulus as a function of strain magnitude, $E(\varepsilon)$ to account for stiffness non-linearity. The beneficial effect on the soil initial stiffness of the applied footing load is also considered. The method was developed as an on-site tool for checking the compaction of hydraulic fills made of carbonate sand to form artificial islands, but its application can be extended to other natural and anthropogenic coarse-grained materials.

Keywords: Settlement; Shallow foundations; Equivalent linear elasticity; Land reclamation; Shear wave velocity; Hydraulic fill; Performance-based acceptance criteria

1. Introduction

The application of traditional acceptance criteria for land reclamation and ground improvement, based on prescribed profiles of penetration resistance or shear wave velocity intended to guarantee achievement of minimum target values of relative density throughout the compacted fill, have proven difficult to apply in presence of crushable materials such as calcareous soils and have led to prolonged technical discussion and uncertainties as to the acceptability or otherwise of the works performed.

This is related mainly to the following issues:

- Conceptual and practical difficulties in applying acceptance criteria based on relative density, both in general and in particular for crushable calcareous sands (Hamidi et al., 2013), for which the susceptibility to particle breakage makes the determination of maximum density heavily dependent on the test method. Even where this is nominally overcome by reference to a specific test method, direct application of such criteria

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https://doi.org/10.1016/j.sandf.2023.101408
0038-0806/

to land reclamation is impossible in so far as it is impossible to measure directly the in-situ density of the (compacted) fill below the water table.
- Conceptual and practical difficulties in applying acceptance criteria based on static (CPT/CPTU) and dynamic (SPT/DP) penetration tests in calcareous sand, due to its high and variable compressibility and susceptibility to particle crushing, especially at the relatively high stress levels mobilized by the tests compared to those of actual interest, typically much lower. This makes the use of standard correlations between penetration resistance and relative density inapplicable without specific calibration (Giretti et al. 2018).
- Conceptual and practical difficulties with defining acceptance criteria which aim to assure achievement of the required performance via proxies and go-between parameters such as relative density or penetration resistance, rather than specifying the target performance itself.

Notwithstanding these difficulties, traditional approaches to defining acceptance criteria are often retained because of perceived simplicity of application and presumed difficulties with defining standardized procedures for the direct application of performance-based acceptance criteria.

It is desirable therefore to develop performance-based acceptance criteria for land reclamation and ground improvement, adopting simple and standardized procedures which evaluate the compliance of the improved landfill with the stated criteria. This should be done on the basis of relevant physical parameters directly and reliably measurable. Control parameters should not be affected by potential error associated with high and variable compressibility and susceptibility to particle crushing, which are typical of calcareous sands. As an example, definition of performance requirements for static loading could include shallow foundations bearing capacity and maximum allowable settlement, long-term settlement and creep in fill and foundation soils, as well as settlement and creep under foundation loads. In this context a dedicated procedure has been developed, as described in this paper, to evaluate the expected total and differential settlement of shallow foundations of given geometry and stiffness, based on equivalent linear elasticity and measured profiles of shear wave velocity, Vs in the fill.

Berardi and Lancellotta (1991) proposed one of the earliest methods accounting for soil nonlinearity through an iterative scheme where the soil stiffness is derived from the corrected SPT blow count and varied according to the calculated relative strain levels. The method by Lehane and Fahey (2002) takes into account the strain dependency of the soil stiffness by reducing the small-strain Young's modulus with increasing axial strain. Stokoe et al. (2013) proposed an approach implementing on a commercial FEM program the dynamic nonlinear soil behaviour and field seismic testing to estimate the settlement of footings.

According to the approach here proposed, once the influence depth H_s (i.e. the maximum soil depth affected by the loaded foundation) has been determined for the given foundation geometry, the vertical stress increment within H_s is calculated using the Boussinesq simplified solution. Given this stress increment, the initial small strain stiffness is adjusted. Within the depth of influence, the soil is divided into sub-layers and the vertical strain in each sub-layer is calculated using the Boussinesq stress increment and the stress-normalized modulus. The strains are calculated, and the modulus adjusted iteratively, until eventually the strains in each sub-layer are accumulated to give the settlement at the footing base.

This method can be used in routine engineering practice. It captures all of the physical processes that affect settlement and the input parameters are relatively easy to determine. It was developed as an expeditious tool for evaluation of ground treatment of hydraulic fill consisting of carbonate sands, used to form artificial islands. If, for example, the requirements of ground improvement are that a particular sized footing under a certain load should not settle more than 50 mm, quality control could consist of shear wave velocity measurements in the treated ground and associated calculations based on the method presented here, which does not require apriori definition of Vs minimum profile.

2. Procedure for settlement calculation

The procedure here described assumes that it is not necessary to consider the following aspects:

- compression of the fill induced by its own weight;
- creep effects;
- embedment of foundation (depth of the foundation base from finished level).

It is assumed that compression of the saturated fill induced by its own weight will take place in a very short time, hence it does not affect the settlement of foundations built later. Furthermore, it is assumed that, at the applied load considered in the settlement calculation, relevant to serviceability conditions, the foundation has an adequate factor of safety against failure, typically not less than 3.

Input data of the calculation are related to the soil properties and to the foundation characteristics.

The soil properties required are:

- the profile of shear wave velocity Vs measured in situ via cross hole, downhole, seismic cone, seismic dilatometer, MASW tests;
- the soil stiffness strain dependency curve, as determined from high quality laboratory tests such as cyclic or monotonic triaxial with local measurements of strain

or resonant column; if good quality samples cannot be retrieved, literature curves for similar materials can be used;
- the unit weight γ_t;
- the Poisson ratio ν_s;
- the groundwater depth, where applicable, such as in the case of land reclamation.

If applicable, the experimental degradation curve, specific for the studied material, can be corrected by applying the correction factor Cr, as proposed by Ishihara (1996), defined as the ratio between the shear modulus ratio for the field deposit (G_F/G_{0F}) to that for the laboratory sample (G_L/G_{0L}) as a function of the sampling technique.

The foundation characteristics required are:

- the shape (i.e. rectangular or circular);
- the effective width B' and length L' (or D for circular footing with no eccentricity of loading);
- the thickness T_h;
- the material Young's modulus E_r and Poisson ratio ν_r;
- the applied vertical pressure p.

Effective foundation dimensions and applied pressure are evaluated according to Meyerhof (1953).

Once the foundation geometry and dimensions are defined, the maximum soil depth H_s affected by the loaded foundation is determined according to the method developed by Gorbunov - Possadov et al. (1984), which relates H_s to the foundation width (B') through to the following equation:

$$H_s = 2A_w B' \quad (1)$$

where A_w is a non-dimensional coefficient, defined in Table 1 for rectangular footings.

For circular footings, A_w = 1.13. For both footing shapes, the values of A_w have been determined with reference to a soil Poisson ratio ν_s equal to 0.25.

The soil depth is then subdivided into sublayers and, for each sublayer i, the increment in vertical stress Δp_i below the centre of a nominally flexible foundation of given geometry is determined as follows:

$$\Delta p_i = I_{si} p \quad (2)$$

Table 1
Non-dimensional coefficient for rectangular footings.

L'/B'	A_w
1	1.26
2	1.53
3	1.72
4	2.01
5	2.21
6	2.37
7	2.50
8	2.61
10	2.70
∞	2.79

where:

p = average applied vertical stress over the effective foundation dimensions;

I_{si} = vertical stress influence factor, for the given sublayer i, determined according to the simplified Boussinesq solution (Poulos and Davis, 1974), as a function of the foundation geometry (rectangular or circular) and dimensions (sides or diameter):

$$I_{si} = \frac{1}{2\pi}\left[\tan^{-1}\left(\frac{\frac{B'}{2}\frac{L'}{2}}{zR_3}\right) + \frac{\frac{B'}{2}\frac{L'}{2}z}{R_3}\left(\frac{1}{R_1^2}+\frac{1}{R_2^2}\right)\right] \; rectangular \quad (3)$$

$$I_{si} = 1 - \left[\frac{1}{1+\left(\frac{D}{2Z}\right)^2}\right]^{\frac{3}{2}} \; circular \quad (4)$$

where:

$$R_1 = \left(\frac{L'^2}{4} + z^2\right)^{0.5}$$

$$R_2 = \left(\frac{B'^2}{4} + z^2\right)^{0.5}$$

$$R_3 = \left(\frac{B'^2}{4} + \frac{L'^2}{4} + z^2\right)^{0.5}$$

From the in situ measures of shear wave velocity Vs, the small strain shear modulus G_0 and Young's modulus E_0 are determined, with reference to the following elastic relationships:

$$G_0 = \rho Vs^2 \quad (5)$$

$$E_0 = 2G_0(1+\nu_s) \quad (6)$$

where:

ρ = soil mass density (i.e. ρ = γ/g).

The small strain Young's modulus, determined as per Eq.6, is then corrected considering the beneficial effect of increased effective confining pressure induced by the load. In particular, for each sublayer, the corrected small strain Young's modulus E_{0i}^* is determined as follows:

$$E_{0i}^* = E_{0i}\beta \quad (7)$$

where:

$$\beta = \sqrt{\frac{\sigma'_{v0i} + (\Delta p_i/2)}{\sigma'_{v0i}}} \quad (8)$$

σ'_{v0} = initial vertical overburden effective stress.

For each sublayer i, the actual modulus reduction factor (or E_i/E_{0i}^*) as a function of the associated strain ε_i is determined iteratively as follows:

1. determination of the first tentative value of E_i/E_{0i}^* and the corresponding value of the operational modulus $E_{i,1}$;
2. determination of the associated strain $\varepsilon_{i,1}$ by the application of the modulus degradation curve;

3. calculation of the strain $\varepsilon_{i,tent,1} = \Delta p_i/E_{i,1}$, based on the increment of applied pressure Δp_i at relevant depth;
4. calculation of the difference $\Delta\varepsilon_i = \varepsilon_{i,tent,1} - \varepsilon_{i,1}$;
5. update tentative estimate of E_i/E_{0i}* such that for $E_{i,n+1}$, $\varepsilon_{i,n+1} = \varepsilon_{i,tent,n}$;
6. iterations of steps 1 to 5 until the tentative E_i/E_{0i}* value fits the corresponding value on the degradation curve (i.e. $\Delta\varepsilon \approx 0$);

Once the iterative subroutine is completed for all sublayers, the total settlement at the centre of a flexible foundation of given geometry, s_V is calculated as the sum of the contributions of all sublayers from the ground surface to the maximum depth H_s. The iterative procedure is represented in Fig. 1.

According to the method proposed by Mayne & Poulos (1999), which takes into account the soil-footing stiffness ratio, the settlement s_A under the centre of a foundation of given geometry and stiffness is calculated as follows:

$$s_A = \frac{pB' I_F I_G (1 - {v_s}^2)}{E_{eq}} \tag{9}$$

where:

$E_{eq} = p$ B' $(1-v_s^2)/s_v$ = equivalent soil modulus;
B' = effective footing width (or diameter D);
I_F = foundation rigidity correction factor;
I_G = displacement influence factor.

The foundation rigidity correction factor I_F and the displacement influence factor I_G are both calculated as a function of the soil-footing stiffness ratio K_R:

$$I_F = \frac{\pi}{4} + \frac{1}{(4.6 + 10K_R)} \tag{10}$$

while

$I_G = 1$ for $K_R < 0.05$;
$I_G = 0.787$ for $K_R > 8$;
I_G is obtained by linear interpolation for $0.05 < K_R < 8$.

For rectangular foundations K_R is determined according to the formula proposed by Horikoshi and Randolph (1997):

$$K_R = 5.57 \frac{E_r}{E_s} \frac{(1 - v_s^2)}{(1 - v_r^2)} \left(\frac{B'}{L'}\right)^{0.5} \left(\frac{T_h}{L'}\right)^3 \tag{11}$$

while for circular footings, K_R is determined according to the formula proposed by Clancy (1993):

$$K_R = \frac{E_r}{E_s} \frac{(1 - v_s^2)}{(1 - v_r^2)} \left(\frac{2T_h}{D}\right)^3 \tag{12}$$

where:

E_r = Young's modulus of footing;
$E_s = E_{eq}$;
v_s = Poisson ratio of the soil;
v_r = Poisson ratio of the footing;
B', L' = rectangular footing effective dimensions;
D = circular footing diameter;
T_h = foundation thickness.

The differential settlement Δs between the centre and selected points on the edge of the foundation is calculated as follows:

$$\Delta s = \Delta s^* s_A \tag{13}$$

where:

Δs* = normalized differential settlement, determined as a function of the soil-foundation stiffness ratio K_R as proposed by Horikoshi and Randolph (1997);

s_A = the settlement at the footing centre from Eq. (9).

The settlement calculation, which takes into account the soil-footing relative stiffness is summarized, in Fig. 2. As evidenced by Figs. 1 and 2, the outputs of the calculation are:

- profiles of vertical strain and mobilized stiffness modulus;
- total settlement for an equivalent flexible footing;
- differential settlement within a footing of given geometry and stiffness.

3. Validation

The method described above has been validated by comparing calculated settlements with those measured in two plate load tests carried out in the centrifuge using different sands and foundation geometry. One test was carried out by the Authors within the context of a research study on settlement reducing piles, referring here to the test developed for a reference raft without piles; this test was performed on Venice Lagoon Sand (VLS), a medium to fine sand with 15% fine content, constituted by quartz, feldspar and carbonates (calcite and dolomite). The other test belongs to an experimental campaign carried out to investigate in detail some aspects of the mechanical behaviour of a coarse to medium carbonate sand (about 95 % of calciumcarbonate content) of biogenic origin (CS) used for hydraulic fill construction. The two sands' main characteristics are given in Table 2, while their grain size distribution curves are shown in Fig. 3. Minimum and mximum dry density in Table 2 were measured on fresh samples according to ASTM 4254 and ASTM 4253, respectively. As summarized in Table 3, for the test carried out using VLS (Fioravante and Giretti, 2010), the imposed centrifuge acceleration was equal to 65g, the model raft was a square 115 mm wide and 25 mm thick steel plate (7.5 m and 1.6 at the prototype scale); the sand model was dry and had an in-flight relative density of 70 %.

The test on carbonate sand CS (El-Gharbawy et al., 2014) was carried out at 75 g, using a rectangular steel raft

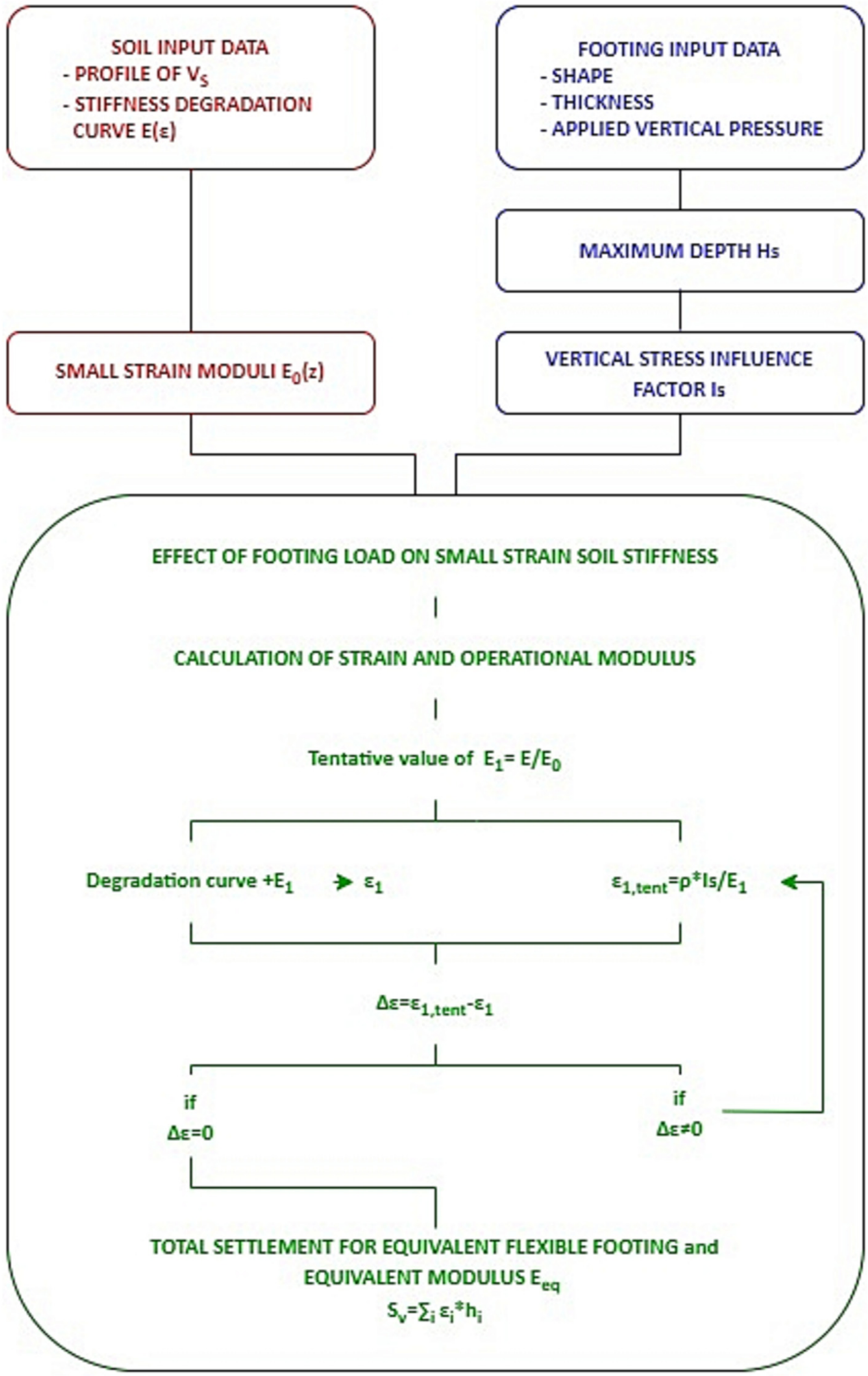

Fig. 1. Iterative procedure for settlement calculation for an equivalent flexible foundation.

240 mm long, 40 mm wide, 40 mm thick (18 m x 3 m x 3 m), the model had a relative density of 65 % and the water table was located 5 m below the ground surface.

In the two models, the loading test was carried out applying a vertical and centred load at the soil surface, with no embedment.

The measured load settlement curves are shown in Fig. 4, at the prototype scale. In both tests, almost linear load transfer relationships can be noted, with development of relatively high settlements, without the attainment of an evident yielding stress. Such a deformation mode is typical of compressible sands, such as VSL and CS. Due to its crushability, the carbonate sand tested has a compressibility significantly higher than a silica sand (critical state line slope $\lambda_{carbonate} \sim 0.3$; λ_{silica} typically 0.025 to 0.06). The venetian silty sand VLS, originated from siliceous-

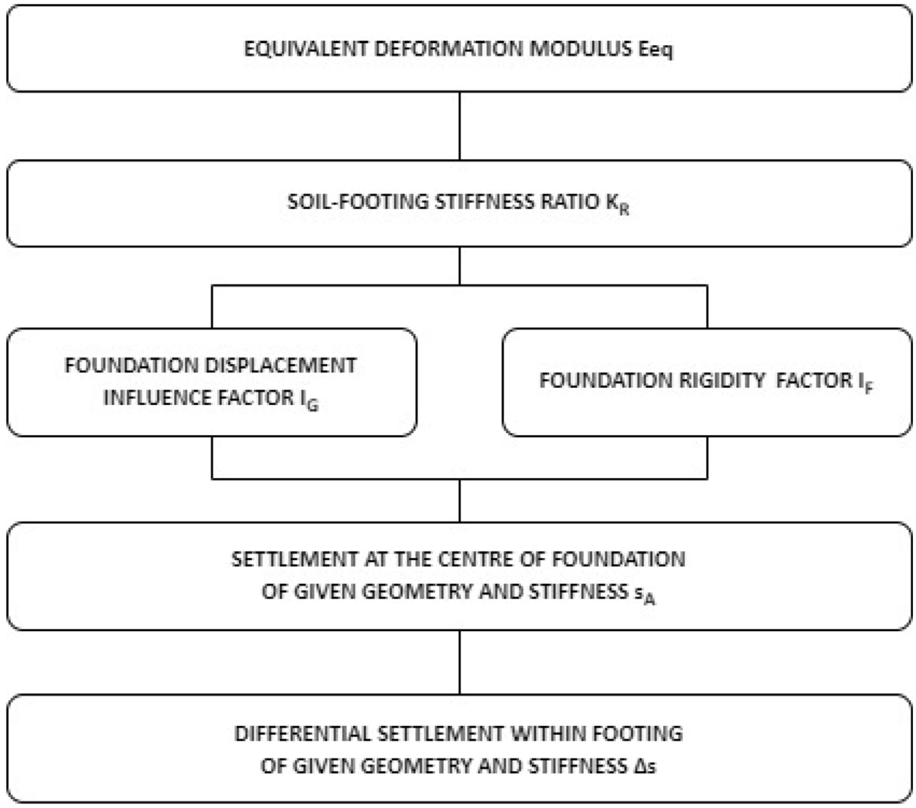

Fig. 2. Settlement calculation for a foundation of given geometry and stiffness.

Table 2
Physical properties of test sands.

Sand	$\gamma_{d,min}$ [kN/m^3]	$\gamma_{d,max}$ [kN/m^3]	G_S [-]	D_{60} [mm]	D_{50} [mm]	D_{10} [mm]	FC [%]	U_C [-]
VLS	13.08	16.5	2.8	0.2	0.18	0.065	14.7	3.33
CS	12.23	16	2.84	0.77	0.59	0.18	1.8	4.1

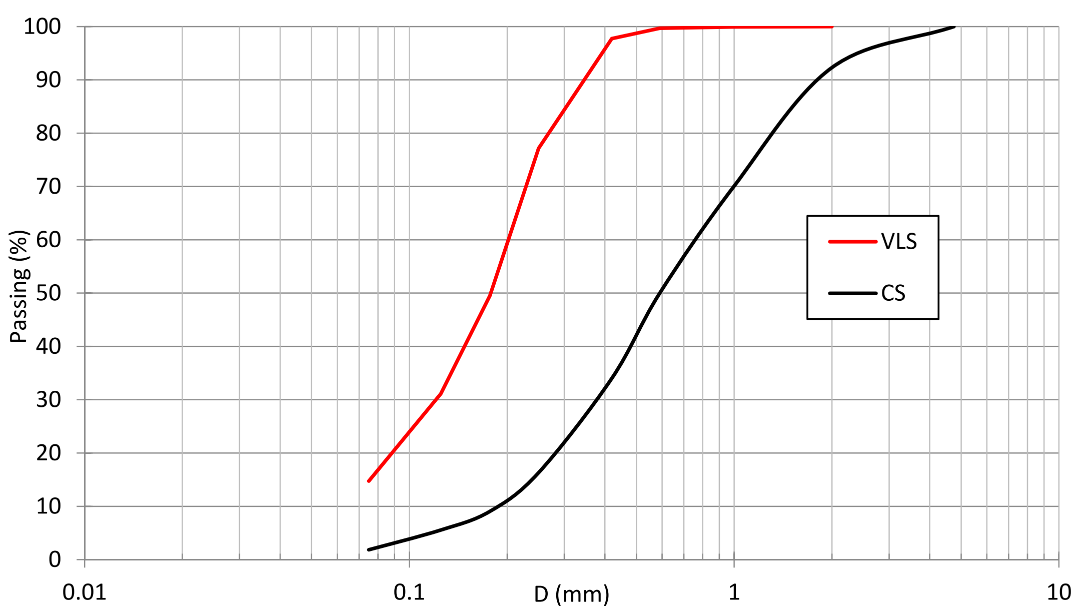

Fig. 3. Grain size distribution of the test sands.

Table 3
Plate load test details (prototype scale).

SOIL	acc [g]	D_R [%]	Length L' [m]	Width B' [m]	Thickness T_h [m]	Water table depth [m]
VLS	65	70	7.5	7.5	1.6	–
CS	75	65	18	3	3	5

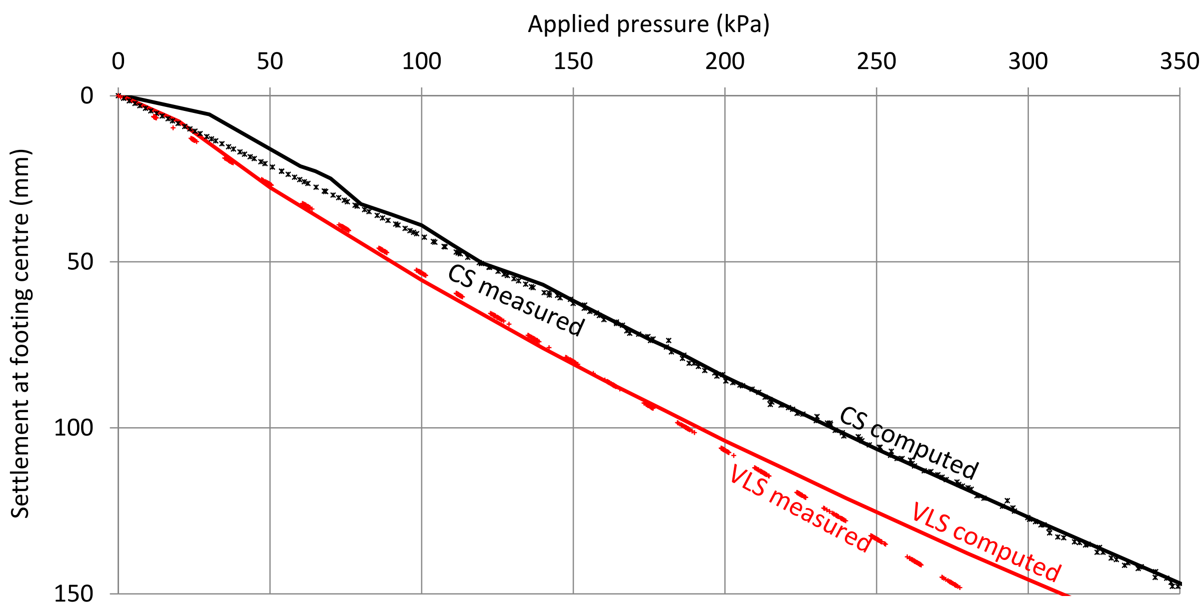

Fig. 4. Centrifuge plate load test: measured load-settlement curves (prototype scale). Superimposed output of the computation.

calcareous sand by crushing and sedimentation, results more compressible than several silica sands ($\lambda_{VLS} \sim 0.09$).

For these two tests, load settlement curves were computed according to the procedure described in the previous section, adopting the soil stiffness strain dependency curves plotted in Fig. 5. The curves were measured on reconstituted samples of the test sands, characterised by the same dry density as the centrifuge models. They compare well with the degradation curve measured on a natural calcareous sand ZS, retrieved from a compacted hydraulic fill,

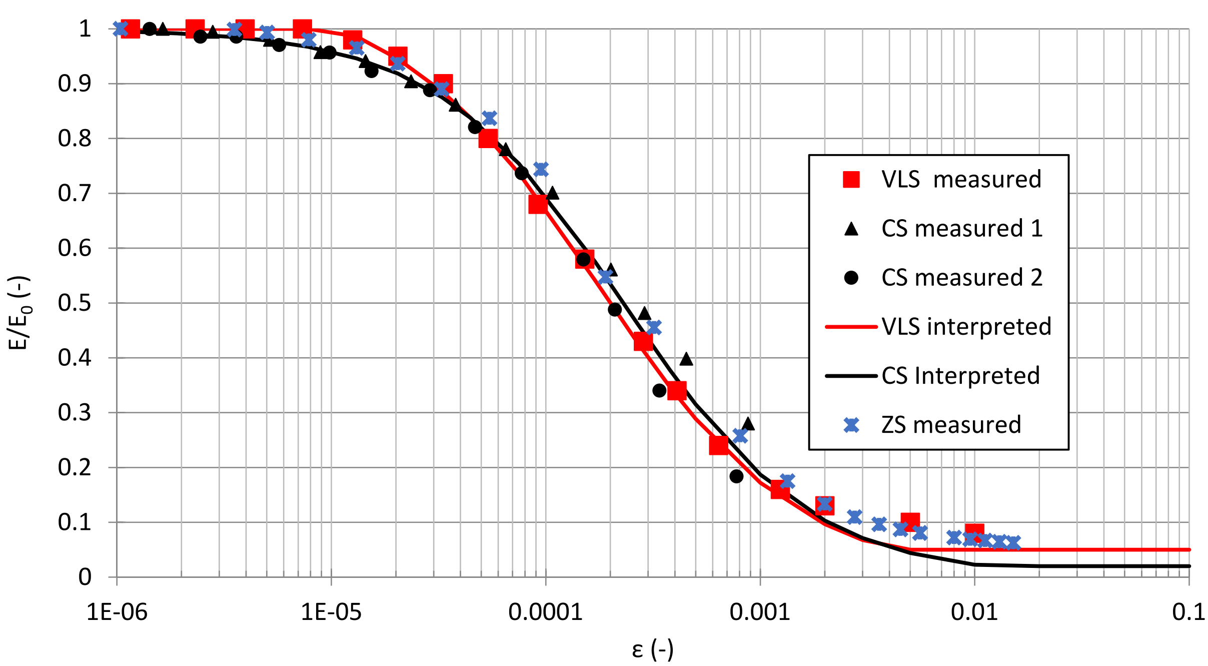

Fig. 5. Measured and interpreted Young's modulus degradation curve.

suggesting that in the absence of specific laboratory tests, reference may be made to appropriate literature degradation curves for comparable materials.

The experimental degradation curves were fitted by with the equation proposed by Lehane and Cosgrove (2000):

$$\frac{E}{E_0} = \frac{1}{1 + \left(\frac{\varepsilon - \varepsilon_{th}}{\varepsilon_r - \varepsilon_{th}}\right)^n} \quad for \quad \varepsilon > \varepsilon_{th} \tag{14}$$

where:

ε_{th} = linear elastic threshold strain;
ε_r = reference strain at $E/E_0 = 0.5$;
n = empirical constant.

The values of ε_r and n define the shape of the stiffness-strain degradation curve and shall be calibrated on the basis of the experimental data available. The stiffness expression of Eq. (14) is comparable to the hyperbolic formulation when n is equal to 1, but the calibration by Lehane and Cosgrove (2000) indicates n values less than unity, which means a stiffness reduction rate lower than predicted by the hyperbolic model. On the basis of the experimental data in Fig. 5, *n* value resulted equal to 1.0 and 0.95 for CS and VLS, respectively.

Another parameter which has to be introduced to interpret numerically an experimental degradation curve is a cut-off (lower bound) value of the ratio E/E_0. It is necessary to consider this lower bound since the degradation curve computed by Eq. (14) tends to zero when approaching very large strains, potentially leading to unrealistic low operational deformation moduli.

This limits the field of applicability of the settlement computation method to situations where the applied pressures are far from those causing failure of the foundation. For the loading tests here presented, in light of the relatively high compressibility of the tested sand, it has been considered appropriate to apply a cut-off value of the ratio E/E_0 in the range of 3–5 %, as shown in Fig. 5.

For the reproduction of the experimental load settlement curves the correction factor Cr was not applied, since the centrifuge models as well as the specimens on which the degradation curves were measured were all made of freshly deposited and normally consolidated sands. For each computation, the in-flight soil unit weight of the centrifuge model was assumed and the ground water table was located as in the centrifuge tests.

The profiles of the shear wave velocity adopted in the computation were derived for the accelerated models as a function of the void ratio and the geostatic stresses, according to the equations below (Fioravante, 2000):

$$Vs = C_S e^{-d} \left(\frac{\sigma\prime_v}{p_a}\right)^{nv} \left(\frac{\sigma\prime_h}{p_a}\right)^{nh} \tag{15}$$

$$Vs = C_S e^{-d} \left(\frac{p\prime}{p_a}\right)^{np} \tag{16}$$

where:

d, nv, nh, np = non–dimensional function exponents;
C_S = material coefficient;
e = void ratio;
σ'_v = vertical effective stress;
σ'_h = horizontal effective stress;
p' = mean effective stress;
p_a = atmospheric pressure.

The values of the correlation coefficients were calibrated for VLS and CS on the basis of bender element tests on reconstituted triaxial samples and are given in Table 4. The computed Vs profiles are shown in Fig. 6. Despite the similar relative density of CS and VLS models, the former is considerably stiffer, in line with literature studies which indicate higher small strain stiffness of carbonate sands with respect to silica sands (Carraro and Bortolotto, 2015). The centrifuge models are typically reconstituted following procedures which guarantee homogeneity of the void ratio, this justifies the adoption of the monotonic Vs curves of Fig. 6. However, the settlement computation method here discussed intrinsically accounts for possible density variation in a compacted fill, since the significant depth is divided into sublayers and each layer is given its own measured value of shear wave velocity, which depends on (and accounts for) the soil type, void ratio, and stress. For the sake of comparison, Fig. 6 reports a post-compaction Vs profile (ZS) measured in a real hydraulic fill, made of dredged calcareous sand. The test was conducted in the context of Quality Control procedures, and, even denoting density variability, it shows similarity to the CS computed profile, strengthening the reliability of the adopted Vs profiles.

The following material characteristics were assumed for the footings in the settlement computation: Young's modulus E_r = 210.000 MPa, Poisson ratio $\nu_r = 0.3$.

Fig. 4 compares the computed load-settlement curves, obtained for a sequence of increasing pressures up to the maximum value applied in the centrifuge, with the experimental results. A good agreement between the measured and computed settlement can be observed since, as evidenced before, the prediction was carried out allowing the modulus to decay towards the strain region where plastic shearing occurs, in order to reproduce the deformations measured. The slightly upward concavity of the VLS computed curve may be interpreted as a modest increase in stiff-

Table 4
Correlation coefficients.

Sand	C_s [m/s]	d [-]	n_v [-]	n_h [-]	n_p [-]
VLS‡	203	−0.34	–	–	0.26
CS‖	274	−0.39	0.16	0.15	–

‡ Saccenti (2005).
‖ Van Impe et al. (2015).

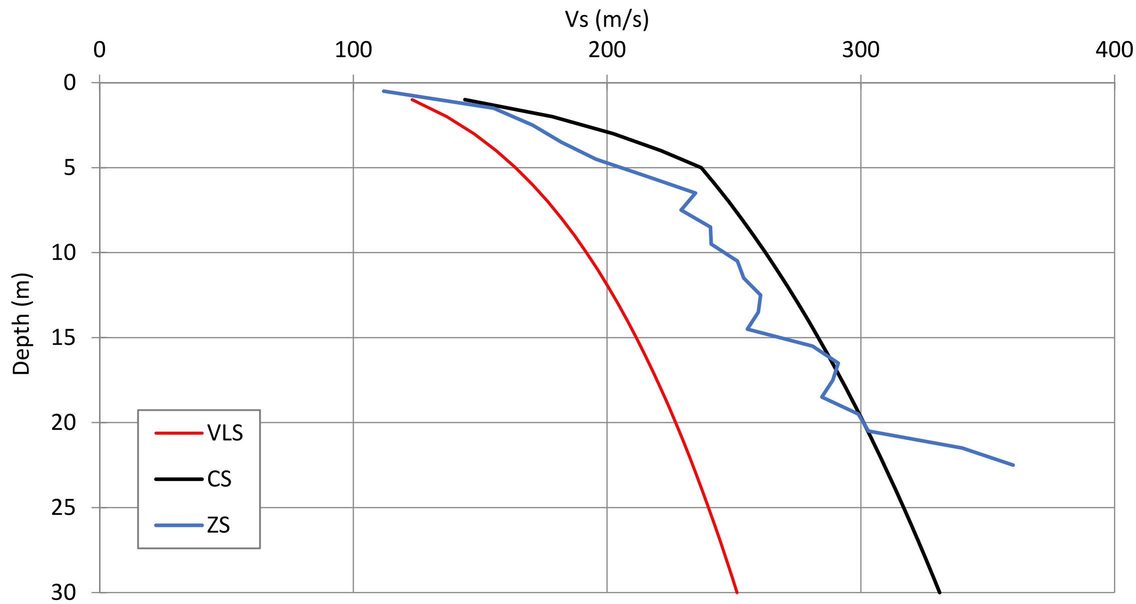

Fig. 6. Shear wave velocity profiles.

ness due to the increment in the effective pressure in consequence of the applied load, accounted for through Eqs. (7) and (8).

Fig. 7 shows some relevant parameters computed for the centre of a flexible foundation (results of the iterative procedure, before accounting for the foundation stiffness) and for an applied pressure p = 200 kPa, for the case of the strip footing in CS. E_0 in the Fig. 7a is the initial Young's modulus derived from the Vs profile in Fig. 6; the corrected/hardened value, enhanced as a function of the applied pressure, is also shown. Fig. 7b shows the reduced stiffness ratio for p = 200 kPa; Fig. 7c and 7d show the associated strains and the settlement profile down to the maximum soil depth affected by the loaded foundation which, for the tested geometry, is H_s = 15 m. The shallowest value of the settlement profile represents the displacement at the centre of a flexible footing of the reference geometry, s_v in Fig. 1. At pressure equal to 200 kPa, the sublayers within 6 m of depth from the foundation base (assumed without embedment) achieve large deformations such to attain the lower bound of the stiffness ratio; the contribution to the total settlement of the deeper sublayers

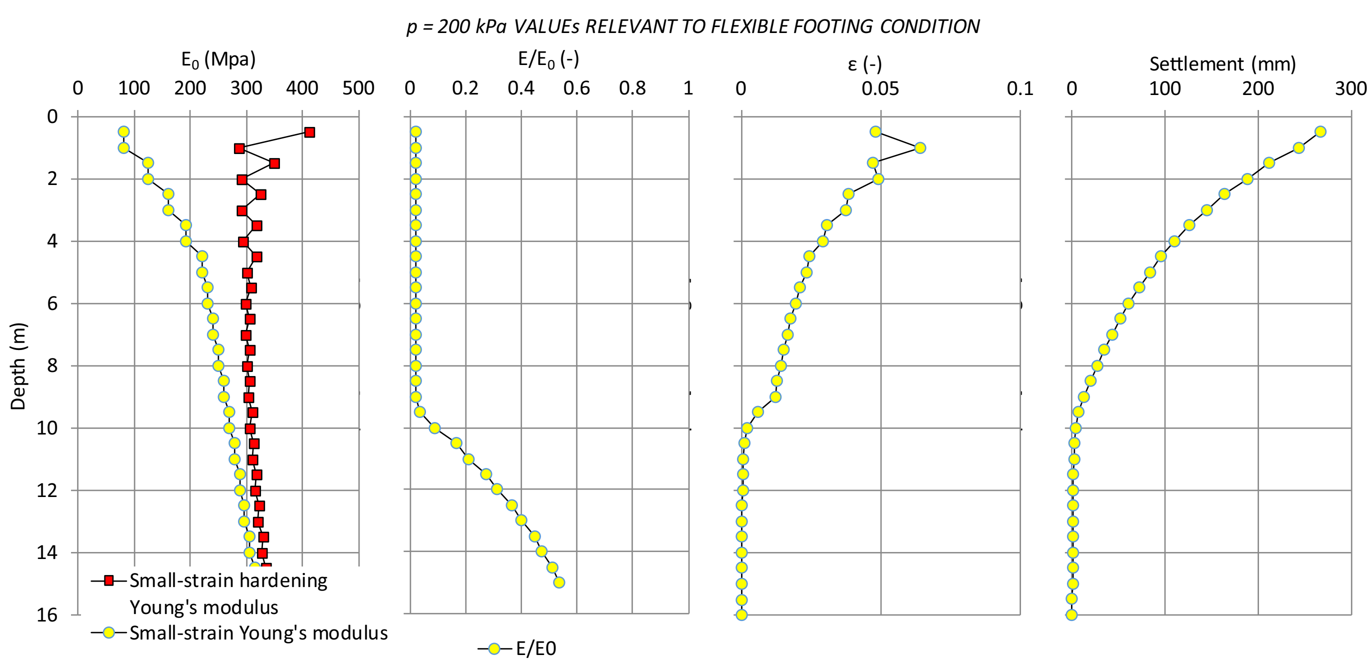

Fig. 7. Output of CS test simulation: a) small strain stiffness and hardened small strain stiffness, b) degraded stiffness ratio, c) strains, d) settlement profile computed for p = 200 kPa at the centre of a flexible foundation.

is almost negligible. Applying lower pressure, this condition tends to move upwards, being thinner the subsoil involved in plastic straining.

The results of the validation described in this section can be considered an ex-post prediction and highlight that the proposed method allows to estimate the settlements of shallow foundations even under limit conditions as those of the centrifuge tests. The validation needs to be repeated for full scale foundations, to be reliably employed for class A predictions.

4. Conclusions

The governing design criterion for shallow foundations on medium dense to dense granular soil is usually related to settlement more than to bearing capacity. Due to difficulties in retrieving undisturbed samples, routine settlement estimation is traditionally carried out using empirical correlations with in-situ penetration resistance.

This paper presents a simplified and expeditious procedure to evaluate the expected total and differential settlement of shallow foundations of given geometry and stiffness, accounting for the non-linear soil behaviour and using shear wave velocity from in situ test as the measurement of deformability. The method is based on equivalent linear elasticity; the Young's modulus is varied as a function of stress, considering the beneficial effect of the applied footing loading; non-linearity is considered through the reduction in modulus as a function of strain magnitude.

The method was developed as a tool for assessment of ground treatment of hydraulic fill consisting of carbonate sands, in the perspective of project specification and acceptance criteria for land reclamation based on the performance of the compacted fill under static operating loads rather than on prescribed profiles of penetration resistance or shear wave velocity intended to guarantee achievement of minimum target values of relative density throughout the fill, which are difficult to apply in cases involving carbonate sands, characterised by high and variable compressibility and susceptibility to particle crushing.

The procedure presented here is validated by satisfactory comparison between the footing settlements predicted by the proposed method with the results of load-settlement tests carried out in the centrifuge. Further validation by comparison with well documented case histories of real foundations is recommended and will be the subject of further studies.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgments

The Authors are indebted to Prof. M. Jamiolkowski for his precious advice during the original development of the procedure presented here.

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