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Prediction of shallow foundation settlements by stiffness-strain factors

Prévision des tassements des fondations superficielles par des facteurs rigidité/déformation

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ABSTRACT
Soil non linear behaviour has to be taken into account in the study of soil-structure interaction. Elastic approaches are generally adopted for settlement prediction, but suitable soil stiffness values should be chosen. The soil behaviour dependence on strain and stress levels is evident even in the serviceability state of a foundation. In the present work, a simplified non-linear method and operational stiffness values are adopted for the estimate of shallow foundation settlements. The reliability of the approach has been considered by the study of case records and numerical analyses.

RÉSUMÉ
Dans l'analyse de l'interaction sol-structure on peut tenir compte du comportement non linéaire du terrain. Des approches élastiques sont generalement adoptées pour la prévision du tassement, mais il faut choisir des valeurs de rigidité du sol appropriées. Le rapport de dépendance entre comportement du sol et niveaux de déformation et de contrainte est meme évident pour une fondation en état d'exercice. Dans ce travail, une méthode non linéaire simplifiée et des valeurs courantes de rigidité sont adoptés pour estimer le tassement des fondations superficielles. La fiabilité de cette approche est verifiée par une étude des cas mentionnés et par des analyses numériques.

1 INTRODUCTION

In common practice, the foundation settlements induced by the working loads are generally evaluated by elastic approaches. The semplicity of this methodology has to be accompanied by a great care in the selection of suitable soil stiffness values.

As demonstrated by many researches in the last years, non-linearity has to be taken into account in the analysis of soil-structure interaction. The estimate of the soil stiffness parameters, describing the so called "*field non-linearity*", is one of the most important issues in geotechnical engineering design.

The non-linear behaviour of the soil interacting with a foundation can be analysed, for practical purposes, by simplified non-linear methods or considering factors which allow one to deduce the field soil stiffness degradation curve from the knowledge of the laboratory one, taking into account both the dependence on the strain and stress levels.

In the present paper the latter approach is developed, defining stiffness factors and parameters that can be applyed into an elastic framework. Some case records will be examined in order to show the practical use and significance of the introduced stiffness factors.

As illustrated in the paper, the outlined procedure allows one to capture the non-linearity which is evident in any foundation load-settlement response; moreover it seems to be a useful tool when indications about the settlements expected for given load values are necessary.

Numerical analyses have been carried out in order to check the validity of the proposed approach.

2 ELASTIC APPROACH FOR SETTLEMENT ASSESSMENT

As mentioned above, the settlements of a shallow foundation can be generally calculated by an elastic approach, applying, for instance, Equation 1, considering foundations on sandy soils, q can be regarded as the net average applied pressure; ν' as the Poisson's ratio; E' as the elastic soil modulus while I_S is an influence factor.

$$\frac{s}{B} = \frac{q}{E'}\left(1 - \nu'^2\right) I_S \qquad (1)$$

Soil non-linearity can be taken into account by assuming a pre-defined (but uncertain) modulus value depending on strain and stress levels; this procedure altough practical and useful, is not completely satisfactory from a conceptual standpoint. The simplicity of the approach is only apparent. In fact, the choice of suitable values of stiffness, describing the behaviour of the soil interacting with a foundation, is particularly delicate.

Moreover, if modulus values are obtained directly from laboratory results, it is necessary to consider that both "*field values*" and *field* non linear behaviour could be different from the laboratory ones.

In the sequel the secant values of Young's modulus obtained by laboratory tests will be indicated as $E'_S(l)$, while the values for the "*in field*" response will be distinguished by the letter "*(f)*".

3 STIFFNESS VALUES BY THE USE OF CORRECTING FACTORS

A research (Bovolenta, 2003) has been conducted in order to study the relationship between the "*laboratory*" stiffness values and the "*in field*" ones, referring to the case of shallow foundations.

Two stiffness – strain factors have been obtained, as it will be specified in the following, allowing one to account for field behaviour starting from laboratory evidences. This approach is similar to the one outlined, e.g., by Atkinson (2000).

The *Strain Correcting Factor (SCF)* is defined by Equation 2. It relates the axial strain value ε_a obtained in laboratory for a fixed value of stiffness ratio E'_S/E'_0 with the corresponding normalised settlement s/B.

Analogously, the *Stiffness Modulus Correcting Factor* (*MCF*), defined by Equation 3, relates the stiffness ratio deduced in laboratory for a fixed value of axial strain with the corresponding stiffness ratio in field.

$$SCF = (s/B)/\varepsilon_a \qquad for \quad E_S'(f)/E_0'(f) = E_S'(l)/E_0'(l) \tag{2}$$

$$MCF = [E_S'(f)/E_0'(f)]/[E_S'(l)/E_0'(l)] \qquad for \quad s/B = \varepsilon_a \tag{3}$$

Operational values of the above correcting factors have been provided by a back-analysis of accurate tests, aimed at comparing soil deformability under different loading conditions.

A set of 9 isotropically consolidated triaxial tests and 29 plate loading tests performed on a prototype foundation (diameter equal to 104 mm) in calibration chamber (Ghionna et al., 1994; Bovolenta, 2003) have been analysed. All the tests, run under two levels of relative density (almost equal to 50% and 90%), have been performed on a medium fine silica sand (Ticino Sand) in normally consolidated conditions.

In order to define the stiffness-strain factors, the modulus values E'_0 and E'_S have been deternined directly from test results and by a fitting procedure. As far as the secant modulus E'_S is concerned, the relationship introduced by Yamashita et al. (2000) (for triaxial testing) and an analogous one derived for PLTs (Bovolenta, 2003), have been adopted in order to deduce the stiffness decays and stiffness ratio values.

It is worth specifying that the stiffness values, back-figured from the plate loading tests, represent an overall behaviour, not the response of a volume element.

Figures 1 and 2 show the values of the introduced factors for three different levels of vertical effective consolidation stress, σ'_{vc}. By the observation of Fig. 1, one can state that for a given strain level (properly the strain for a laboratory sample and the normalised settlement in situ) the "*in field*" stiffness values are greater than the laboratory ones, as it was expected (see also Fig. 3). It is worth underlining that the ratio between the normalised moduli, i.e. *SCF*, is not constant.

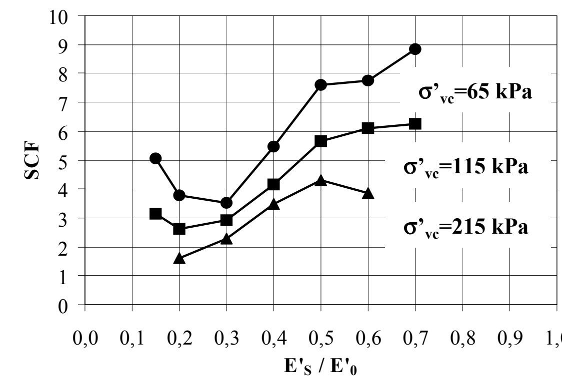

Figure 1. Values of *SCF* for Ticino Sand.

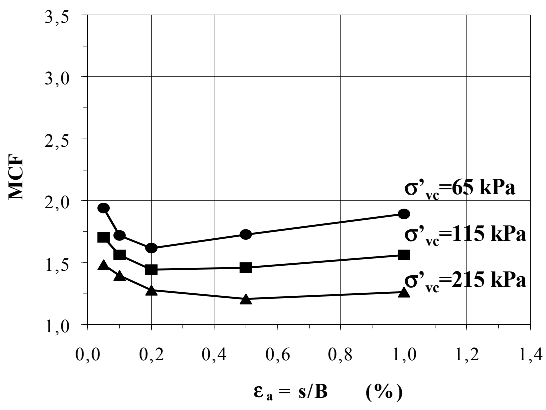

Figure 2. Values of *MCF* for Ticino Sand.

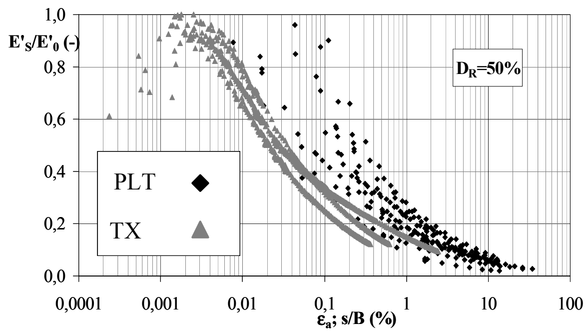

Figure 3. Stiffness decays curves of the triaxial (TX) and plate loading (PLT) tests on Ticino Sand ($D_R \cong$ 50%).

The so defined operational stiffness values allow one to correct the laboratory stiffness decays in order to define the in field values taking into account both the dependence on the strain and stress levels.

4 NUMERICAL ANALYSES

In order to check the validity of the correcting factor values, experimentally determined, finite element analyses have been performed. The Critical State Program CRISP (Britto and Gunn, 1987) has been adopted, having recourse to the Three Surface Kinematic Hardening model (Stallebrass, 1990) for the description of the soil behaviour. Soil parameters have been chosen referring to the results of the triaxial tests.

The FEM analyses have supplied values of *SCF* and *MCF* which agree with those obtained considering the test results. Figures 4 and 5 show, as an example, the correcting factors deduced for Ticino Sand with a relative density equal to 50% and a vertical effective consolidation pressure equal to 115kPa.

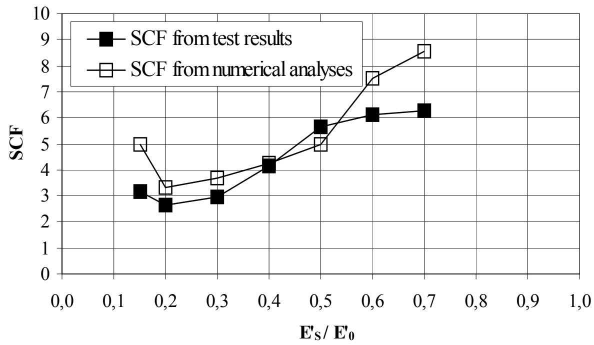

Figure 4. *SCF* values for Ticino Sand by experimental data and numerical analyses.

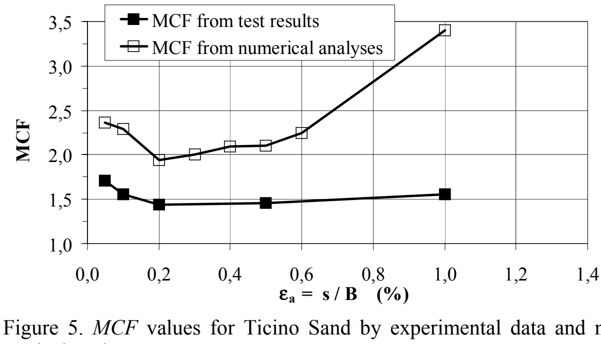

Figure 5. *MCF* values for Ticino Sand by experimental data and numerical analyses.

The numerical modelling has been useful in the analysis of the relationships, previously obtained empirically, between the behaviour of the soil sample and that of single elements of soil placed under the plate, during the load test in calibration chamber. In this respect Figures 4 and 5 indicate a good agreement among experimental and numerical results, as well as the evident non-linearity in "*laboratory*" and "*in field*" behaviours. Possible differences and scattering could be due to the choice of the soil element representative of axial strain in the numerical analyses.

In Fig. 6 the normalised stiffness decays have been represented as functions of the vertical strain in a triaxial test and for some elements of the mesh (adopted for the simulation of the PLTs in CC) under the plate.

For fixed normalised stiffness values, the stiffness degradation curves of a "single element of soil" in the model show greater values of vertical strain than the laboratory ones. This highlights again the difference between laboratory and field performance and, as already pointed out, how a simple rigid translation of the laboratory stiffness degradation curve could be not sufficient, being *SCF* not constant.

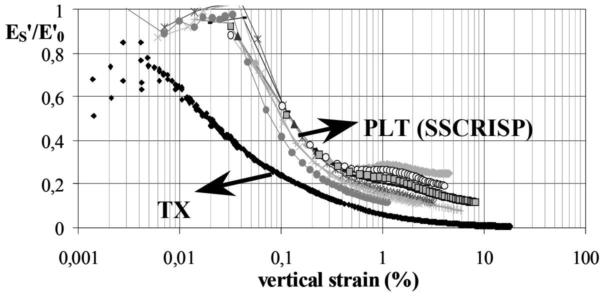

Figure 6. Comparison among the stiffness decay curves.

5 SETTLEMENT PREDICTION EXAMPLES

As described above, by the use of the correcting factors the stiffness decay of the soil interacting with a foundation can be deduced by the knowledge of the one defined by a laboratory test.

Therefore *SCF* and *MCF* can be used for the estimate of the stiffness values to be adopted in Equation 1.

The operational factor values obtained for Ticino Sand can be applied in Equation 1 in order to estimate the settlements of a foundation.

This procedure has been followed for the analyses of some case histories.

Five examples are illustrated in the sequel.

5.1 *Case N. 1*

A square model foundation, 1 m wide, resting on a 3 m deep sand layer (Ticino Sand with D_R = 85%) has been considered. Details are reported in Gabrielaitis et al. (2000).

The foundation model is a rigid square plate which is 1 m embedded and has been loaded vertically up to 300 kN by linear loads of about 50 kN; the settlement values reported in Fig. 7 are the ones measured at the end of each loading step.

Sand properties are analogous to the ones of the sand previously considered for the deduction of the *SCF* and *MCF* values.

The load-settlement curve (Bovolenta, 2003) has been evaluated by Equation 1 assuming $E'_S(f) = MCF \cdot E'_S(l)$.

The values of *MCF* have been obtained computing σ'_{vc} at the depth of 1 m below the foundation; Poisson's ratio ν' has been assumed equal to 0.2; the influence factor I_S has been chosen considering an active zone equal to 2*B* (Berardi and Lancellotta, 1992).

The use of *MCF* has led to an accurate estimate of the foundation performance (Fig. 7).

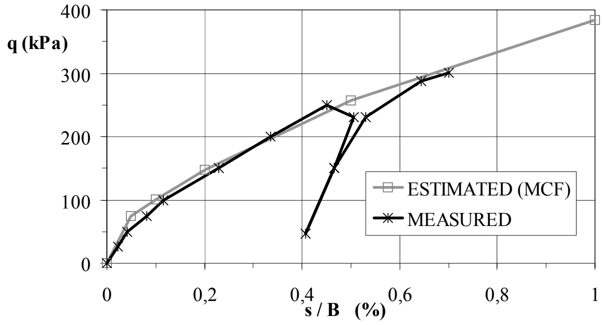

Figure 7. Computed and measured load-settlement curves.

5.2 *Case N. 2*

A square foundation (3 m wide) has been considered. It was tested on occasion of a Prediction Symposium at the Texas A&M University (Briaud and Gibbens, 1994).

A good characterisation (triaxial tests, resonant column tests, in situ tests) of the soil (mainly sand) was supplied.

Analogously to Case N. 1, the stiffness moduli to be adopted in Equation 1 have been evaluated by the application of the Stiffness Modulus Correcting Factor.

The so deduced stiffness values are in accordance with those back-figured from the loading tests.

Poisson's ratio ν' has been assumed equal to 0.2 the influence factor I_S has been chosen considering an active zone equal to 2*B*.

The comparison between the measured load-settlement curve and the one obtained using the stiffness modulus correcting factor is reported in Fig. 8.

As one can observe the non-linear behaviour is caught by the proposed procedure.

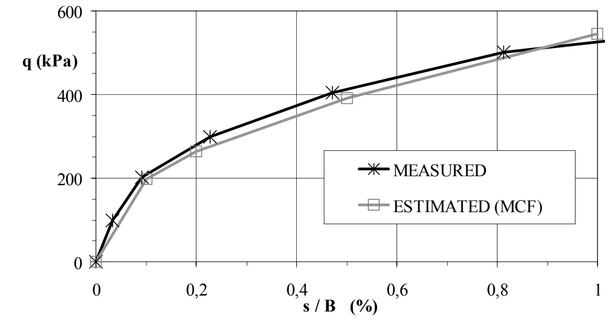

Figure 8. Computed and measured load-settlement curves.

5.3 *Case N. 3*

Two shallow foundations (A: 4x4.6m and B: 4x10.6m) have been considered; the foundation A is 0.5m embedded, while foundation B is 2.0m under the ground level (further details are reported in Ghionna et al., 1991). The footings are part of the foundation structure of the football stadium in Torino (Italy).

The deposit is composed of gravelly sand; the water table is placed at a depth of 17 m.

It is worth observing that, because of the lack of the laboratory test results, the stiffness values of the soil element have been considered as it was Ticino Sand.

The values of *MCF* have been obtained by Fig. 2, computing σ'_{vc} at a depth equal to 4 m below the foundation. The influence factor I_S has been chosen considering an active zone equal to 2*B* and ν' has been assumed equal to 0.3.

Fig. 9 and 10 show the measured and computed load-settlement curves.

In both cases satisfactory results have been obtained; it is worth observing the almost linear behaviour of the actual foundations due to the low strain levels.

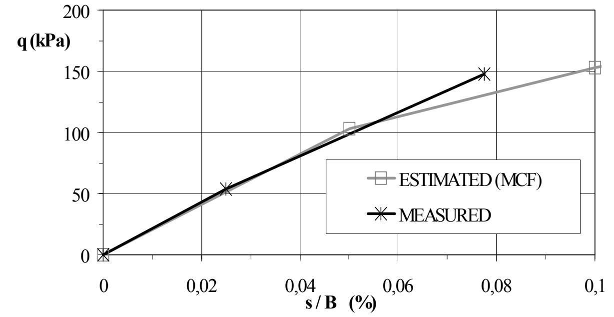

Figure 9. Computed and measured loading curves of foundation A.

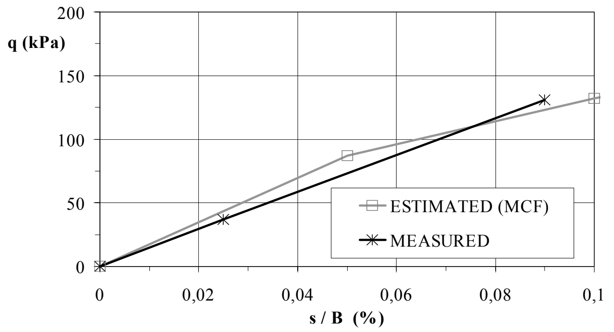

Figure 10. Computed and measured loading curves of foundation B.

5.4 *Case N. 4*

Finally, the performance of a rectangular (13.1x23.9m) foundation located in Fondachello (Catania, Italy) is considered.
It consists of a reinforced concrete mat on which there is a mesh of grade beams with an overall height of 1.2 m. The foundation embedment is 3.0 m.

The deposit is composed of gravelly sands. Further details are reported in Maugeri et al. (1998).

As in the previous example, the laboratory test results were not available. Consequently the laboratory stiffness decay of the soil has been evaluated as it was Ticino Sand.

The vertical effective consolidation stress σ'_{vc} for the definition of *MCF* values have been computed at a depth equal to 13.1 m below the foundation.

For the analysis ν' has been assumed equal to 0.3 and I_S chosen for an active zone equal to 2*B*.

The use of *MCF* allows the assessment of settlement values, which agree to the measured ones, as one can state observing Fig. 11.

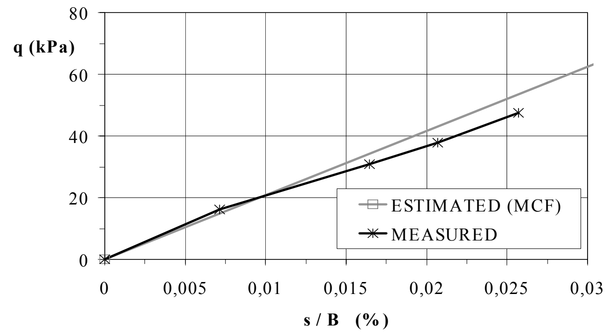

Figure 11. Computed and measured load-settlement curves.

6 SUMMARY AND CONCLUSIONS

In the present paper a simple procedure for the evaluation of shallow foundation settlements has been illustrated.

Settlements are calculated by an elastic approach as usual, but the non linear behaviour of the soil is taken into account.

The stiffness decay of the soil interacting with a foundation is deduced from the stiffness values obtained by laboratory tests, by means of two correcting factors, called *SCF* and *MCF*, which have been determined by the analysis of triaxial and plate loading tests, performed on a normally consolidated silica sand.

Finite element analyses have been performed, confirming the trends and values of the correcting factors obtained back-analysing the experimental data.

Notwithstanding the outlined procedure is based on results obtained by testing a specific soil type (Ticino Sand), the decays of stiffness moduli can be considered representative also for other coarse grained soils. Confirmation of the applicability of the illustrated methodology and the supplied values has been obtained by the analysis of case histories.

Four cases have been presented. The foundation and deposit features, examined in the illustrated examples are rather dissimilar. Despite this difference, in each case the non linear behaviour is caught and the settlements of the actual foundations have been estimated quite accurately.

Finally, as in any analysis where deformability is the main issue, and stiffness decay curves are considered, it is necessary to stress the importance of an accurate evaluation of the initial stiffness values.

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