Study of Effect of Pre-consolidation Pressure by Simulating Plate Load Test using Hypo-elastic Model for Soil

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Abstract

Full scale field tests on soil with practical foundation size and load intensity can provide reliable information on settlement behaviour which is cost prohibitive, time consuming and sometimes nearly impossible. Next to prototype testing, in the present scenario, plate load test is the only way of determining settlement and allowable bearing capacity not withstanding its limitations. One such limitation of the plate load test is not accounting for the interaction between the adjacent footings in settlement analysis. In such cases numerical simulation by using suitable constitutive model may yield reliable results. However Plate load test results are used to evaluate the performance of the model and enhance the same so that it can be implemented in numerical analysis of any structure.

In this work numerical simulation of plate load test is done by developing a program in FORTRAN 77 using Hypoelastic model for soil. It is observed that the load displacement curve obtained from numerical analysis deviates considerably from plate load test results when the pre-consolidation pressure obtained through isotropic consolidation test is used. As the pre-consolidation pressure is an important parameter, numerical experiments were conducted for different fractions of pre-consolidation pressure which was obtained from laboratory test. It is interesting to note that for any given soil, a correction factor has to be adopted for the pre-consolidation pressure so that the results of the model are reasonably comparable with that of the plate load test results.

Key Words: Hypoelastic, Constitutive, Plate Load Test, Preconsolidation Pressure, Settlement

1. INTRODUCTION

Literature reveals that there are varieties of constitutive models to depict the behaviour of soils. Hypoelastic model is one amongst them. This model is governed by a variety of parameters such as shear modulus, rigidity modulus, coupling modulus, pre-consolidation pressure etc. In the present paper, Plate load test results are used to evaluate the performance of the model and enhance the same so that it can be implemented in numerical analysis of any structure.

Empirical design methods focus on the dependency of settlement calculations from footing size using plate test load results is generally accepted in engineering practice. The state of stress and strain in such a case is complicated and cannot always be discussed by reduction to the equivalent one-dimensional case.

This necessitates the usage of some appropriate numerical techniques to predict the settlement behaviour of structures. Finite element method is reliable and powerful method. The reliability of such settlement forecasting for foundations is very much dependent on two factors, the model for foundation and the model for the soil. Generally the constitutive relation for foundation is linear elastic. However, the constitutive relation for the soil is quite complex. Literature reveals that there are varieties of models to depict the constitutive relations for soil. The applicability of a model is tested for various stress paths by conducting stress controlled drained and un-drained triaxial tests. The present paper adopted the hypoelastic model parameters and compared with plate load test results from the experiments conducted [1]. The suitability of the adopted model for soil can be verified by comparing plate load test results.

The derivation of Hypoelastic model is credited to Truesdell [2]. He proposed constitutive theory in which the stress rate depends on state of stress and the rate of deformation. He examined response of certain hypo elastic materials in hydrostatic compression, simple shear and simple extension. Tokuka [3] demonstrated the correspondence between hypoelasticity and yield condition from plastic theory. The first application of Hypoelasticity to soil mechanics is done by Coon and Evans [4] to describe elastic recoverable deformation of sand and later for concrete in 1972. Romano [5] was the first to use Hypoelasticity to model full range of soil shearing behaviour, including transition from linear elastic behaviour at small shearing load to a gradual yield condition and plastic straining, dilation and strain softening to a critical state. Vagneron [6] modelled the shearing behaviour of loose sand tested in triaxial compression and plain stress at relatively low confining pressure. The predictions of the deviator stresses were satisfactory, but predictions for the volumetric strain were poor and failed to capture dilation.

Davis and Mullenger [7] used material parameters to model behaviour at critical state. They also showed correspondence of hypo-elastic equation with Von mises yield condition. They also state that the hypo-elastic material respond smoothly throughout transition from classical elastic behaviour to plastic flow. Desai and Sirivardane [8] have presented the results of evaluating the parameters for a first order hypoelastic model when the parameters are determined from the tests that is to be predicted agreeing very well, whereas the predictions were extremely poor when the parameters determined from individual tests are used to predict the behaviour for wide range of stress states and stress paths. To consider shear –dilation behaviour of soils, Yin and Yuan [9] suggested a three moduli model. But this model cannot consider mean –stress induced shear strain. Darve et al. [10] suggested models which more than three moduli. However, the determination of those moduli requires special tests and is often difficult.

Collins and Bachus [11] presented first second order hypoelastic model to predict the behaviour of Reid Bedford and Hostun sand for tests conducted along complex loading paths at relatively high confining pressure.

Kolymbus [12] presented a hypoelastic constitutive equation. The model requires 4 material constants which can be determined by a single conventional triaxial compression test without unloading. The model was used to predict the behaviour of sand tested along complex stress paths. The predicted results are shown to agree well with the experimental results. Yin et al [13] developed an improved version of model [9] to consider both shear stress induced volume strain and mean stress induced shear strain using moduli K, G and J for modelling triaxial test. The stress strain relation developed in incremental and three dimensional form. The model considers nonlinearity, dilatancy and the coupled behaviour. This three moduli model was only for triaxial stress state. Krishnamoorthy and Rao [1] verified the applicability of the model for triaxial stress conditions for various stress paths and drainage conditions. Further Krishnamoorthy and Rao [14] used hypo-elastic model to analyze a single and group of piles resting in soil mass and subjected to lateral load are analyzed using non-linear finite element method. In the year 2000, Yin [15] presented extension of Hypo elastic model determining the three moduli K, G and J using the data from an isotropic consolidation test and three triaxial shear tests. The model was verified with measured test data from three drained triaxial shear tests with constant mean effective stress. Two new generalised models were derived based on incremental isotropic hypo elasticity theory giving the tensor form and matrix form of two new generalised stress-strain relationship. This model is implemented in the present paper making minor changes in the evaluation of the modulus G so that model predicts all the stress paths correctly.

The objective of present paper is to simulate field behaviour more closely by capturing the effects of soil nonlinearity and other intricacies like distribution of stresses, displacements and the modes of failure in a plate load test. With this enhancement the model can be implemented in numerical analysis of the prototype capturing various interactions in the structure.

It is observed that the load displacement curve obtained from the model deviates considerably from plate load test results when the pre-consolidation pressure obtained through isotropic consolidation test is used. As the pre-consolidation pressure is an important parameter, numerical experiments were conducted for different fractions of pre-consolidation pressure which was obtained from laboratory test.

2. FOMULATION OF THE MODEL

Model proposed [15] is extended, enhanced and used in this work. Of the two generalised models proposed one model which links incremental strains to incremental stress is chosen and adopted.

It is this constitutive model considered for implementation. The data used is obtained from isotropic consolidation and conventional undrained triaxial compression (CTC) tests on saturated soil.

The model consists of three stress dependent modulus functions, which are as follows,

- Bulk modulus K
- Shear modulus G
- The coupling modulus J that relates effective mean stress p' to shear strain ε_s as well as shear stress q to volumetric strain ε_v .

$$\left.\begin{aligned} d\varepsilon_v &= \frac{1}{K}dp' + \frac{1}{J}dq \qquad ---(a) \\ d\varepsilon_s &= \frac{1}{J}dp' + \frac{1}{3G}dq \qquad ---(b) \end{aligned}\right\} -----(1)$$

Bulk modulus K can be determined from isotropic consolidation test. The coupling modulus J and shear modulus G can be determined either from conventional undrained triaxial compression test (CTC) or conventional drained triaxial compression test.

The model is in incremental form. The change in volumetric strain $d\varepsilon_v$ and shear strain $d\varepsilon_s$ corresponding to the change in effective mean stress dp' as well as shear stress dq' as proposed by Yin et al. are expressed by the relationships

Where, in triaxial stress condition,

ε_v is the volumetric strain , ,

ε_s is the generalised shear strain

p' is effective mean stress,

q is the deviator stress,

The formulation in equation (1) assumes that $dp - d\varepsilon_s$ the coupling and $d\varepsilon_s - dp$ the coupling are controlled by the same J-modulus. Positive dilation, that is, expansion during shearing is associated with J<0. The compression during shearing is associated to J > 0. If there is no dilation or no induced anisotropy, the coupling modulus J= ∞

The above equation can be inversed as,

$$\left.\begin{aligned} dp' &= \bar{K}d\varepsilon_v - \bar{J}d\varepsilon_s \\ dq &= -\bar{J}d\varepsilon_v + 3\bar{G}d\varepsilon_s \end{aligned}\right\} -----------(2)$$

Where are related to $\bar{K}$, $\bar{G}$, $\bar{J}$ as follows.

$$\left.\begin{aligned} \bar{K} &= K\frac{J^2}{J^2 3Kg} \\ \bar{G} &= G\frac{J^2}{J^2 - 3KG} \\ \bar{J} &= \frac{3KGJ}{J^2 - 3KG} \end{aligned}\right\} ---------(3)$$

Formulations in Eq. (1) and Eq. (2) are valid for triaxial state of stress and need to be generalized into 3-D stress state before. Therefore equations (1) and (2) are of theoretical and practical significance.

The two special forms of Hypoelastic relation [4] are as follows:

$$\left.\begin{array}{ll} d\varepsilon_{ij} = C_{ijkl}(\sigma_{mn})d\sigma_{kl} = C \quad d\sigma_{kl} & \text{------(a)} \\ d\sigma_{ij} = D_{ijkl}(\sigma_{mn})d\varepsilon_{kl} = D \quad d\varepsilon_{kl} & \text{------(b)} \end{array}\right\} \text{------4)}$$

Which after simplification result in

$$\begin{Bmatrix} d\varepsilon_{11} \\ d\varepsilon_{22} \\ d\varepsilon_{33} \\ d\gamma_{12} \\ d\gamma_{12} \\ d\gamma_{12} \end{Bmatrix} = \begin{bmatrix} \alpha_1 + 2\alpha_3 & \alpha_2 + \alpha_3 + \alpha_4 & \alpha_2 + \alpha_3 + \alpha_5 & \frac{\sigma_{12}}{qJ} & \frac{\sigma_{23}}{qJ} & \frac{\sigma_{31}}{qJ} \\ \alpha_2 + \alpha_3 + \alpha_4 & \alpha_1 + 2\alpha_4 & \alpha_2 + \alpha_4 + \alpha_5 & \frac{\sigma_{12}}{qJ} & \frac{\sigma_{23}}{qJ} & \frac{\sigma_{31}}{qJ} \\ \alpha_2 + \alpha_3 + \alpha_5 & \alpha_2 + \alpha_4 + \alpha_5 & \alpha_1 + 2\alpha_5 & \frac{\sigma_{12}}{qJ} & \frac{\sigma_{23}}{qJ} & \frac{\sigma_{31}}{qJ} \\ \frac{\sigma_{12}}{qJ} & \frac{\sigma_{12}}{qJ} & \frac{\sigma_{12}}{qJ} & 1/G & 0 & 0 \\ \frac{\sigma_{23}}{qJ} & \frac{\sigma_{23}}{qJ} & \frac{\sigma_{23}}{qJ} & 0 & 1/G & 0 \\ \frac{\sigma_{31}}{qJ} & \frac{\sigma_{31}}{qJ} & \frac{\sigma_{31}}{qJ} & 0 & 0 & 1/G \end{bmatrix}_0 \begin{Bmatrix} d\sigma_{11} \\ d\sigma_{22} \\ d\sigma_{22} \\ d\sigma_{12} \\ d\sigma_{23} \\ d\sigma_{13} \end{Bmatrix} \text{--(5)}$$

Where

$$d\gamma_{12} = 2 \quad \varepsilon_{12}, d\gamma_{23} = 2 \quad \varepsilon_{23}, d\gamma_{31} = 2 \quad \varepsilon_{31}$$

$$\alpha_1 = \frac{1}{9K} + \frac{1}{3G}, \alpha_2 = \frac{1}{9K} - \frac{1}{6G}, \alpha_3 = \frac{2\sigma_{11} - \sigma_{22} - \sigma_{33}}{6qJ},$$

$$\alpha_4 = \frac{2\sigma_{22} - \sigma_{11} - \sigma_{33}}{6qJ}, \alpha_5 = \frac{2\sigma_{33} - \sigma_{11} - \sigma_{33}}{6qJ}$$

$$\begin{Bmatrix} d\sigma_{11} \\ d\sigma_{22} \\ d\sigma_{22} \\ d\sigma_{12} \\ d\sigma_{23} \\ d\sigma_{13} \end{Bmatrix} = \begin{bmatrix} (\beta_1 + 2\beta_3) & \begin{pmatrix} \beta_2 + \beta_3 \\ + \beta_4 \end{pmatrix} & \begin{pmatrix} \beta_2 + \beta_3 \\ + \beta_5 \end{pmatrix} & \frac{-\bar{J}\sigma_{12}}{q} & \frac{-\bar{J}\sigma_{23}}{q} & \frac{-\bar{J}\sigma_{31}}{q} \\ \begin{pmatrix} \beta_2 + \beta_3 \\ + \beta_4 \end{pmatrix} & \beta_1 + 2\beta_4 & \begin{pmatrix} \beta_2 + \beta_4 \\ + \beta_5 \end{pmatrix} & \frac{-\bar{J}\sigma_{12}}{q} & \frac{-\bar{J}\sigma_{23}}{q} & \frac{-\bar{J}\sigma_{31}}{q} \\ \begin{pmatrix} \beta_2 + \beta_3 \\ + \beta_5 \end{pmatrix} & \begin{pmatrix} \beta_2 + \beta_4 \\ + \beta_5 \end{pmatrix} & \beta_1 + 2\beta_5 & \frac{-\bar{J}\sigma_{12}}{q} & \frac{-\bar{J}\sigma_{23}}{q} & \frac{-\bar{J}\sigma_{31}}{q} \\ \frac{-\bar{J}\sigma_{12}}{q} & \frac{-\bar{J}\sigma_{12}}{q} & \frac{-\bar{J}\sigma_{12}}{q} & \bar{G} & 0 & 0 \\ \frac{-\bar{J}\sigma_{23}}{q} & \frac{-\bar{J}\sigma_{23}}{q} & \frac{-\bar{J}\sigma_{23}}{q} & 0 & \bar{G} & 0 \\ \frac{-\bar{J}\sigma_{31}}{q} & \frac{-\bar{J}\sigma_{31}}{q} & \frac{-\bar{J}\sigma_{31}}{q} & 0 & 0 & \bar{G} \end{bmatrix} \begin{Bmatrix} d\varepsilon_{11} \\ d\varepsilon_{22} \\ d\varepsilon_{33} \\ d\gamma_{12} \\ d\gamma_{12} \\ d\gamma_{12} \end{Bmatrix} \text{---(6)}$$

Where

$$\beta_1 = \bar{K} + \frac{4}{3}\bar{G}, \beta_2 = \bar{K} - \frac{2}{3}\bar{G}, \beta_3 = \frac{\bar{J}}{3q}(\sigma_{22} + \sigma_{33} - 2\sigma_{11}),$$

$$\beta_3 = \frac{\bar{J}}{3q}(\sigma_{211} + \sigma_{33} - 2\sigma_{22}), \beta_5 = \frac{\bar{J}}{3q}(\sigma_{22} + \sigma_{11} - 2\sigma_{33})$$

The properties of the soil are shown in Table 1. The model parameters were determined [14] by conducting isotropic consolidation and undrained triaxial compression tests on undisturbed soil samples collected from the respective sites before conducting plate load test.

Table 1. Soil properties and model parameters

Soil properties		Model parameters		
Liquid limit	54	K modulus	λ / V_i	0.020
Plastic limit	40		κ / V_i	0.003
Plasticity Index	14		P'cons	21000 Pa
Shrinkage limit	20	J modulus	A	100
Water content	28		N	100
Specific gravity	2.65	G modulus	E	0.001
Wet density	18.18		F	0.56

3. DEFINITION OF THE PROBLEM

Subsequently the model was applied to the problem of a plate loading test on the soils whose parameters are mentioned above. The load settlement predicted from the Numerical analysis of the plate load tests is compared with the load settlement behaviour obtained from the plate load test.

3.1 Details of Plate Load Test

In order to verify the numerical analysis a few load tests are conducted in the field at different places with with different initial stress conditions and different soil properties. A square pit having a dimension of 1.5 m X 1.5 m is excavated up to required depth. A square rigid plate of size 0.3 m X 0.3 m is placed at the centre of pit is loaded with the help of Hydraulic jack in increments of 2.0 kN to 5.0 kN. The settlement of plate for each increment of load is measured with the help of two dial gauges of least count of 0.01 mm mounted on the opposite edges of plate. Each increment of load is kept constant until the rate of settlement becomes than 0.02 mm/hour. The procedure is repeated until total deformation reached a maximum value of 25.0 mm.

3.2 Details of Numerical model

The finite element model for plate load test adopted was a three dimensional model consisting of 4800 brick elements to represent soil mass of size 1.5 m X1.5 m X 1.2 m with eight nodes. the plate of size 0.3 m X 0.3 m X 0.025m was modelled with 16 plate elements with four nodes having five degrees of freedom per node. The soil properties and soil parameters are given in Table 1.

As in the case of plate load test the initial effective mean stress P0 is taken as 0.056 and q shear stresses zero at the ground level. Below the ground surface the value of effective mean stress and shear stress due to over burden pressure is computed and added to P0 and q0 to obtain initial stress conditions for all the soil elements.

4. RESULTS AND DISCUSSIONS

Numerical analysis of plate load test was done for different values of pre-consolidation pressures. It was observed that settlement increased with decrease in pre-consolidation pressure (Fig. 1).

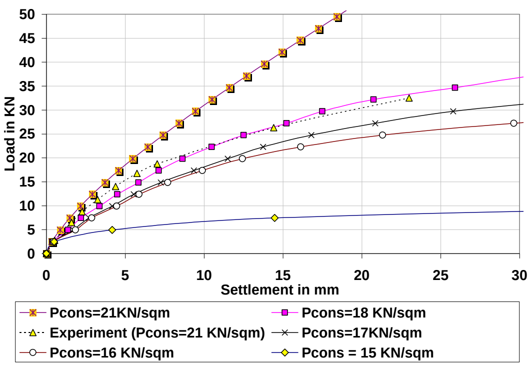

Fig. 1 Load settlement curves for different values of pre-consolidation pressures

Fig. 2 shows the relationship between load and settlement obtained from the plate load test as well as predicted from numerical analysis for a pre-consolidation pressure of 18 kN/sqm (about 85% of actual pre-consolidation pressure). It can be observed that the settlements obtained from the plate load test and predicted by numerical analysis are nearly the same. Fig. 3a and Fig. 3b show the horizontal and vertical displacements below centre line of plate. X/B is the ratio of distance to width of plate.

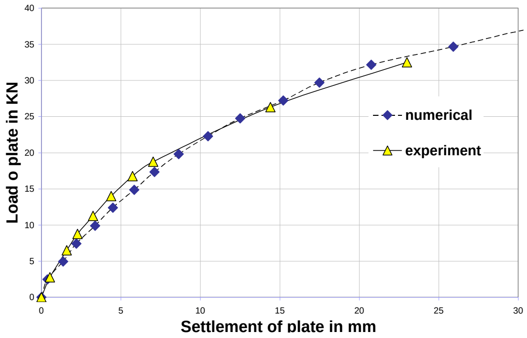

Fig. 2 Load settlement curves for 85% of pre-consolidation pressure

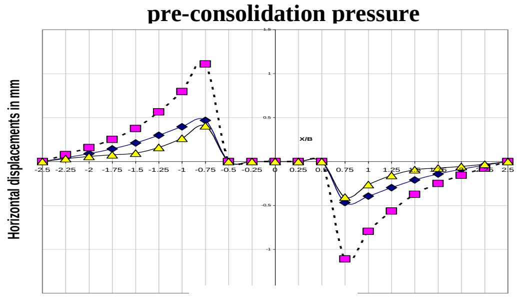

Fig. 3b

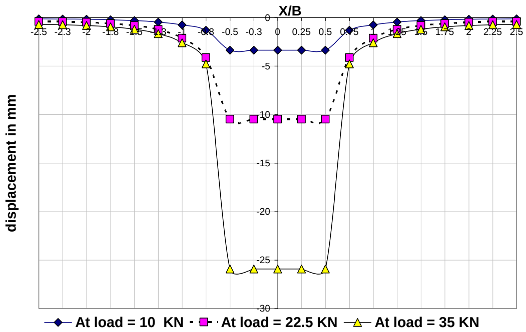

Fig. 3a

Fig. 3 Horizontal (3a) and vertical (3b) displacements below plate displacements

The pressure distribution below the plate for the load 9.5 kN, 22.275 kN and 34.65 kN is shown in Fig. 4. It can be observed that with the increase in load in equal increments the pressure distribution became more uneven. This explains increase in increments of displacements with incrementing of load.

5. CONCLUSIONS

Hypoelastic model yields fairly realistic results when implemented in numerical analysis with modification in pre-consolidation pressure. The necessity of modification in pre-consolidation pressure may be due derivation of model parameters from triaxial tests.

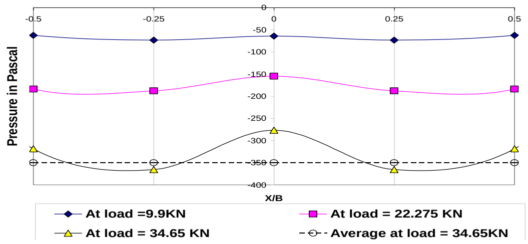

Fig. 4 pressure below plate at the centre line of plate

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