The Effect of Plastic Strain Variation on Nonlinear Behaviour of Soil

Sriparna Roy and Debjit Bhowmik

Abstract It is well known that the soil exhibits nonlinear behavior when stressed under different condition. Therefore, it is a topic of interest for many researchers to study the failure mechanism of the soil considering its plastic nature. This led to the development of different constitutive modeling techniques. Analysis of geotechnical problems using finite element method (FEM) combined with constitutive laws ensures precise prediction of the behavior of complex problems. Shear strength parameters of soil also depend on amount of strain applied to the soil mass. This paper presents the effects of variation of plastic strain on shear strength parameters of soil. The nonlinear behavior of soil with two types of laboratory tests attributing to variation in plastic strain is simulated numerically using a FORTRAN subroutine linked with ABAQUS. The results are compared which shows how important it is to consider the variation of plastic strain when the numerical analysis is done.

Keyword Plastic strain · Nonlinear behaviour · Finite element method · Constitutive laws · Numerical simulation

1 Introduction

Soil stress–strain behavior is extremely complex as it depends on a number of factors like grain size distribution of soil, density, confining pressure, plasticity index, the degree of saturation, internal friction angle, cohesion, and rate of loading. However, simplified relationships are assumed to develop theories and calculate the transfer of stress in the soil. It is assumed that the soil is homogeneous and isotropic in nature and elastic theory is implemented, whereas soil exhibits nonlinear variation due to its plastic nature. This nonlinear behavior can be measured by assuming that the total strain is the summation of elastic strain and plastic strain. Elastic strain is measured using Hooke's law, and for plastic strain, plasticity theory is taken into account.

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Thus, the study of soil properties considering the plasticity theory became a prime topic of research. It was found that the angle of dilatancy of soil varies with respect to the internal frictional angle, which further depends on plastic strain. Some essential equations were proposed to suggest a relation in between internal friction angle and plastic strain (Vermeer and De Borst 1984). Their experiments showed that plastic strain increment and hardening arise due to friction in between the soil particles. In this relation, the mobilized friction angle depends on plastic strain and increases to peak when the plastic strain reaches its peak. The authors have also given an account of the dependence of dilatancy angle and cohesion on plastic strain by citing. Different types of constitutive modeling theories like Mohr–Coulomb plasticity theory, hyper-elastic model, etc., were used to depict stress–strain curves. Mohr–Coulomb model is one of the most important methods to model soil behavior (Ti et al. 2009). Mathematical equation was used which controlled the shape of the mobilized dilation angle with respect to plastic strain (Soreide et al. 2002). Simple direct shear tests were carried out to find the variation of plastic strain, internal friction angle and dilatancy angle with varying confining pressures. After numerical and experimental analysis, it was stated that plastic strain hardening start after yield point is reached, and this is attributed to models using an increase in internal friction and dilation angles. After peak soil starts to soften, softening is applied by reducing the internal friction angle and dilation angle of the sand. At residual or critical state, the strength parameters are constant (Moradi and Abbasnejad 2014). Fundamental work and research have been done by scientists like Vermeer, De Borston anisotropic properties of soil, and these basic findings are validated and used by scholars like Moradi and Abbasnejad (2014), and Melbouci et al. (2007). Predicting soil behavior with constitutive equations that are based on experimental findings is a significant aspect of soil mechanics. These equations are embodied in the finite element method or discrete element method for analysis of complex geotechnical problems. Since the previous studies reveal that strength parameters will vary as there is plastic strain variation, so this study focuses on the nonlinear behavior in strength parameters of soil with plastic strain variation and its visualization analytically and graphically using ABAQUS and FORTRAN, so that the transfer of stress in the soil stratum can be easily determined, and the affected zones can be provided with more strength.

2 Materials and Methods

2.1 General

The study is divided into two segments. First is a numerical analysis of an unconfined compression test specimen. The material properties are chosen from experimental results of Bhowmik et al. (2013). This step is done to see how the model works with the variation of strength parameters, i.e., internal friction, dilation angle, and cohesion of soil with respect to varying plastic strain as against that of unchanging strength parameters with respect to no plastic strain variation. The numerical and model details are discussed in Sect. 2.2.

In the next segment, the study deals with an experimental program followed by numerical analysis of two sets of plate load tests with two square plates of sizes 10 and 20 cm. This is done to get an idea about the applicability of the numerical analysis of the effect of variation of plastic strain on the nonlinear behaviour of soil for practical cases. Plate load test is helpful in determining the bearing capacity, load and settlement properties of the soil. All the properties of local soil are determined using basic laboratory tests like liquid and plastic limit test, standard Proctor test, triaxial test and unconfined compression test; refer to Table 2. The load-settlement graphs were obtained from the plate load test carried out in a steel tank in the laboratory. The failure load and the corresponding settlement were determined. Then, the same tests were simulated at ABAQUS first with the variation of plastic strain and strength parameters using FORTRAN as done previously in case of UCS test and next without any variation. The results of the numerical tests are then compared with the experimental findings in Sect. 3.

2.2 CAE Model Details of UCS Test

A three-dimensional unconfined compression test model of 76 mm length and 38 mm diameter is created in ABAQUS 6.10 as ABAQUS CAE model with material properties and loading conditions; refer to Fig. 1. The material properties of experimental results are used in order to model nonlinear soil behavior according to (Bhowmik et al. 2013). For this purpose, the properties of soil were chosen from the experimental data of Table 1 above. Meshing is done with hexahedral C3D8R elements with loading on proving ring plate and reaction on the load plate of the 3D model. Data of the third layer are considered to model the unconfined compression test of the soil using ABAQUS. FORTRAN programming is done to see the variation of soil strength parameters with increasing plastic strain. This step is taken up specially to know if it is possible to model the nonlinearity of the soil using a user-defined subroutine program. The program was based on the quasi-empirical formulation given by (Vermeer and De Borst 1984) for frictional hardening and cohesion softening.

Fig. 1 3D UCS model

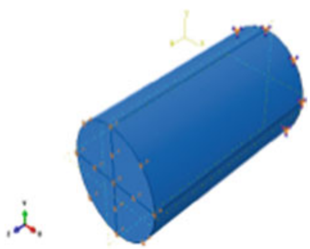

Table 1 Soil properties determined from the laboratory test for UCS test Courtesy Bhowmik et al. (2013)

Characteristics	Description
Layer number	3
LL (%)	43
PL (%)	23
Moisture content (%)	11.5
Unit weight (KN/m^3)	18.520
C(KN/m^2)	113
Φ (°)	16
Description	Brownish inorganic silty clay (CL)
Depth (m)	Below 1.5
Elastic modulus (KN/m^2)	3.4×10^4

The load is given in the form of velocity on the top surface or "proving ring plate". The rear surface of the model, "load plate," was restrained by using encastre boundary condition. The load rate is given keeping in mind that the wave speed to deformation speed should not increase 1% in any of the element. This makes the solution quasi-static. It is important to carry out quasi-static analysis because it will give a realistic solution when the model is three dimensional and goes a good amount of deformation (refer ABAQUS User's Manual). Then a comparison is made for the stress–strain graphs for experimental tests and analytical tests with and without plastic strain variation on strength parameters of soil. The results (Sect. 3) show that it is very important to consider the nonlinear or plastic behavior of soil during numerical analysis.

2.3 *Details of Laboratory Plate Load Test*

For an implementation of the theory of plastic stress–strain variation with respect to the change in friction angle, dilation angle, and cohesion, two plate load tests were carried out. The local c-phi soil which was available in the NIT, Silchar campus, for construction purpose was taken as a case study; refer to Table 2. A standard Proctor test was carried out to find the optimum moisture content and maximum dry density of the soil. A target density of 95% of the dry density is chosen, and the water content value was (20%) considered to be on the wet side of the optimum moisture content (15.9%). This was done as during construction water present in the soil dries up due to the heat during the daytime. The test was carried out in a steel tank of size 1 m × 1 m × 1 m and was conducted in accordance with IS code 1888–1982. The test procedure involves an application of the load and determination of load-settlement behavior of the soil. Plate load test is a field test. The test requires a loading of the rigid plate at the foundation level and determining the settlements corresponding to each load increment; refer to Fig. 2. The settlement is measured with two dial gauges

Table 2 Properties of soil incorporated in the PLT model

Properties of soil	Description
Elastic (kN/m^2)	1.2×10^4
Poisson's ratio	0.2
Φ (°)	20° (changes with strain field variable)
Dilation angle (°)	0.1 (changes with strain field variable)
Cohesion yield stress (kN/m^2)	50 (changes with strain field variable)
User-defined field variable	Used for incorporating changing strain as a field variable
Dependent variables	1 (No. of Field variables to be used)
Density (kg/m^3)	1894

Fig. 2 Laboratory plate load test set up

with a sensitivity of 0.01 mm. A hydraulic jack of capacity 10 ton is required to transfer the load to the plate. Proving ring of capacity 25 KN is used to measure the load applied to the plate. Two tests are done each with the same arrangement but different plate sizes of 10 and 20 cm. The test plate would be placed centered properly over the soil bed with the help of a plumb bob held vertically so that the hydraulic jack, proving ring, and the plate all are in alignment for the load to be applied centrally and spirit level is used for horizontal alignment. The average of the dial gauge readings is taken for measuring the settlement of the soil. The load versus settlement curve is obtained from the plate load tests for plates sizes 10 and 20 cm. All the soil properties are then used as input parameters in the numerical analysis of the problem.

Table 3 Properties of steel plate incorporated in model

Properties of steel plate	Description with units
Elasticity	2×10^8 KN/m^3
Poisson's ratio	0.15
Density	7850 kg/m^3

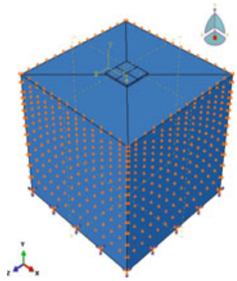

Fig. 3 3D model of laboratory plate load test

2.4 CAE Model Details for Plate Load Tests

Three-dimensional models of the two plate load tests are created. The models are divided into two parts. First part is "c-phi soil" which includes the soil properties derived from the experiment and laboratory tests. The dimension of the first part "c-phi soil" is taken as that of the sandy–silty–clay soil present inside the tank, i.e., a plan area of 1 m × 1 m and height of soil is 0.85 m. The second part of the CAE model is "steel plate" and its dimension is 10 and 20 cm for two different tests, for properties of steel plate refer to Table 3. Hex-type C3D8R elements are used for both the parts and meshing is done. Partition is made at the top of the "c-phi soil" according to the size of the plate to be assembled for respective tests and similar meshing is done for the "steel plate" so that the bottom of the plate and the top partitioned surface of the soil can have surface-to-surface contact effectively; refer to Fig. 3 for 20 cm sized square plate.

Loading in the model is applied as concentrated load acting (it is different for two tests) at the center of the plate. The four sides are restrained using boundary conditions. The base of the "c-phi" soil is fixed. The exterior side walls of the soil mass are restrained in *X*-direction and *Z*-direction. The loading is taken as amplitude where it varies as a smooth step for the entire time period. The contact in between the surfaces of steel plate and soil top is taken as surface-to-surface with soil top as the slave surface and steel plate as the master surface. This is the arrangement of slave and master surface is done because the master surface is the stiffer surface. The soil top is partitioned with a square surface on the top based on the size of the plate used. This sole portion is meshed as per the steel plate so that there is no distortion or isoparametric angle formation. Mass scaling is done for effective quasi-static analysis decrease in computational time and prevention of distortion due to explicit analysis and to provide a much more effective contact in between the steel plate bottom and soil top (refer ABAQUS User's Manual).

3 Result and Discussion

In Fig. 4, below is a comparison between two stress–strain curves of the UCS test. Curve I depicts experimental curve of unconfined compression test from (Bhowmik et al. 2013). Plot digitizer has been used to determine the values on the experimental curve. Curve II is simulated in ABAQUS linked with a FORTRAN user subroutine generated curve. Curve III is also ABAQUS generated curve but without any user subroutine file. It can be seen from the graph below that the stiffness of the curve II is less than that of curve III. This happens due to variation of internal friction and cohesion with varying plastic strain. Also curve II shows close proximity to the curve I in terms of peak values attained than curve III.

The load-settlement curves of laboratory plate load tests of sizes 10 cm and 20 cm plates are compared with numerically obtained curves of the same test simulated in ABAQUS; refer to Table 4. Figures 5 and 6 show that the numerical curve with nonlinear variation of strength parameter with respect to plastic strain shows more proximity to the actual laboratory curve as compared to the one without any variation. Figures 7 and 8 show the deformed shapes of the 3D models of UCS and PLT test showing the maximum strained zones in red and orange color.

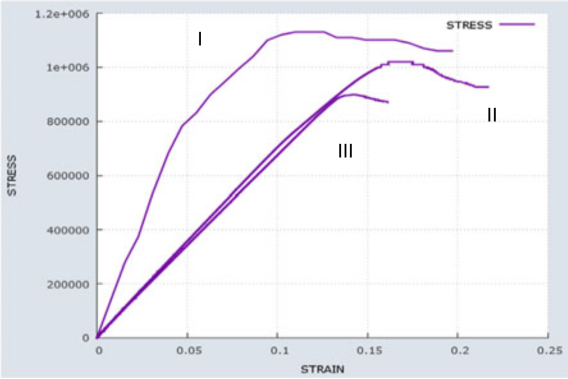

Fig. 4 Stress–strain curve for I Experimental data unconfined compression test; II ABAQUS analysis with variation of plastic strain; III ABAQUS analysis without variation of plastic strain

Table 4 Comparison of numerical and experimental plate load test results

Plate size (cm)	Failure load in experimental curve (N)	Failure load in analysis with plastic strain variation (N)	Failure load in analysis without plastic strain (N)
10 × 10	9553	7600	6800
20 × 20	22,000	19,900	18,000

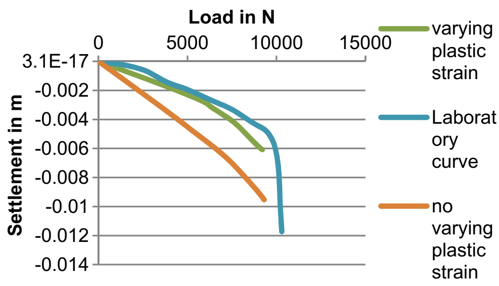

Fig. 5 Experimental and numerical analysis curves for square plate of size 10 cm

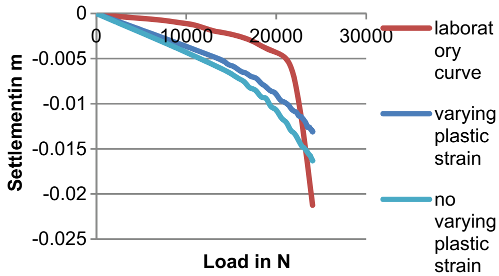

Fig. 6 Experimental and numerical analysis curves for square plate of size 20 cm

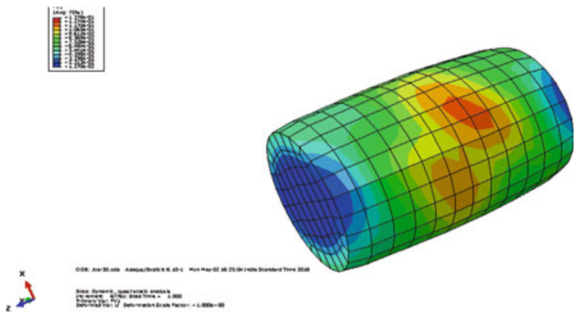

Fig. 7 3D model of UCS test sample showing plastic strain variation

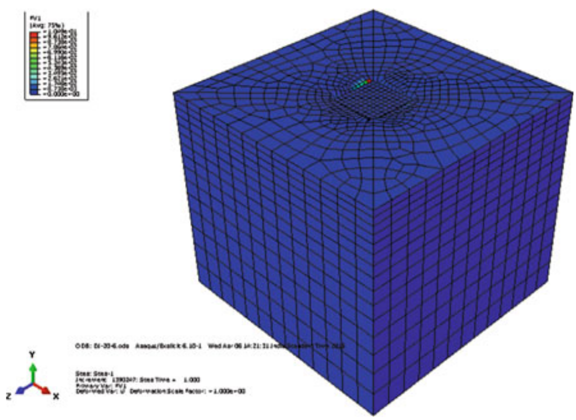

Fig. 8 3D model of plate load test sample showing plastic strain variation

4 Conclusion

To realize the objective of modeling the nonlinear behavior of the soil, experimental data were chosen from unconfined test data of Bhowmik et al. (2013). These values were used in FEM-based software to do the material modeling of UCS test. The hardening and softening behaviors were obtained by writing a user-defined subroutine. It was seen that stress–strain curve generated with FORTRAN subroutine linked ABAQUS program more accurately depicted the actual experimental curve than that which was analyzed in traditional way without any change in soil parameters with respect to varying strain. The visualization module (Fig. 7) shows generation of plastic strain in the sample. Hence, it is very important to consider the variation of plastic strain in numerical modeling to depict the nonlinear behavior of soil.

The objective of practical application of the theory of plasticity of soil is put to test numerically for which two plate load tests were done in a steel tank as a laboratory experiment with two different plate sizes which were simulated in ABAQUS first by using FORTRAN subroutine with varying strain and soil parameters and next without any change. Figures 5 and 6 show that the varying plastic strain curve shows good agreement with the experimental curve than the curve with no plastic strain variation, indicating that variation of plastic strain in numerical modeling can be incorporated to precisely model load-settlement tests and other complex geotechnical problems. The visualization module (Fig. 8) shows plastic strain generation in the plate load test model depicting the zones that can be provided with more strength.

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