Journal of Civil Engineering (IEB), 42 (2) (2013) 171-188

Journal of
Civil Engineering
IEB

Elasto-plastic constitutive model parameters and their application to bearing capacity estimation for Dhaka sub-soil

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Received 12 January 2014

Abstract

Elastoplastic constitutive model parameter identification is an improtant task for proper modeling of any soil. In this paper, subsoil characteristics of Dhaka sub-soil is presented based on field and laboratory test results. This paper summarizes the charctritics of the red clay and fine sand layer of Dhaka sub-soil. It is found that Dhaka sub-soil is mainly consists of top red clay layer underlying by a fine sand layer up to infinite extent. Elasto-plastic constitutive model paramters of Dhaka soil has been determined for extended subloading t_{ij}model. Using these parameters, bearing capacity of Dhaka sub-soil has been estimated for different OCRs and considering bonding effect which can not be obtained uisng classical theories.Considering the effect of shape factor for shallow foundation both 2D and 3D FE analyses have been conducted. It is found that bearing capacity determined by the conventional methods match well with the results of the numerical simulations.

Keywords: Bearing capapcity, elastoplastic constitutive model, Dhaka soil, fine sand, finite element analysis

1. Introduction

Bangladesh including capital city Dhaka is largely an alluvial plain consisting of fine sand and silt deposits with shallow ground water table in most places. Dhaka soil largely consists of fine sand with clay deposit at uppermost layer. Generally, bearing capacity is estimated based on classical theories using limited sub-soil test data mainly SPT N-value. Finite element analysis in geotechnical problem is used widespread now-a-days. Almost all the geotechnical problems solved by finite element method should be related to a non-linear elastoplastic constitutive model since soil is a natural non-linear material.

Bearing capacity is estimated by limit analyses using upper bound and lower bound theory. But the limit state analyses cannot consider the effect of Over Consolidation Ratio (OCR), bonding effect of soil. Therefore, in estimation of bearing capacity such parameters should be considered. A now-a-days FE method is widely used in different fields of Geotechnical Engineering. So, such condition can also be applied for bearing capacity estimation. However, the accuracy of the FE analyses depends on the constitutive models of soils. Available constitutive models such as Camclay model (Roscoe and Burland, 1968), Drucker-Prager Model, Mohr-Coloumb Model cannot properly consider or explain soil behavior of different densities. However, in this paper extended sub-loading t_{ij} model (Nakai and Hinokio, 2004; Nakai et al., 2011) is used which can consider influence of intermediate principal stress on the deformation and strength of soils, dependence of the direction of plastic flow on the stress paths, influence of density and/or confining pressure and bonding effect on the deformation and strength of soils (Shahin et al,. 2004; Nakai et al., 2010; Nakai et al., 2011).

The red tropical clay soils of Dhaka are mainly composed of Illite, Kaolinite, Chlorite and some non-clay minerals mainly quartz and feldspar. Under undrained shearing, a significant difference between the natural and destructured failure surface has been observed due to the presence of bonding in the natural soils. It is established that these clay soils of Dhaka are bonded (Hossain and Toll, 2008).

Few efforts have been taken to characterize Dhaka soil (Ameen, 1985, Uddin, 1990 Siddique, 1986, Islam, 1999). Strength and deformation anisotropy of Dhaka clay was studied (Islam, 1999, Islam and Hoque, 2010 and 2011). Ahmed (2005) determined the liquefaction potential of reclaimed areas of Dhaka. Islam and Ahmed (2009) measured shear wave velocity at selected locations in Dhaka city using Small Scale Micrometer (SSMM) method and made a correlation of shear wave velocity with SPT N-value. Islam and Hossain (2010) evaluated earthquake induced liquefaction vulnerability of reclaimed areas of Dhaka city. Dynamic properties and liquefaction potential of Dhaka sub-soils were also studied by several researchers. Islam et al. (2010) measured the dynamic stiffness of laterally loaded pile foundation of Dhaka soil. However, none of these studies were focused on determination of elastoplastic constitutive model parameters for Dhaka sub-soil.

The present study is limited to sub-soil properties parameters for constitutive modeling Dhaka soil. Index properties and strength properties were determined at four selected locations (i.e., Farmgate, Dhaka University Campus, Gulshan and Uttara). The main objectives of the study are:

(1) Determination of elasto-plastic constitutive model parameters for extended subloading t_{ij} model.

(2) Bearing capacity considering OCR and bonding effect for structured Dhaka sub-soil.

2. Geology of Dhaka Soil

Dhaka is situated at the southern tip of a Pleistocene terrace, the *Madhupur* tract. Two characteristic geological units cover the city and surroundings, viz. *Madhupur* clay of the Pleistocene age and alluvial deposits of recent age. The *Madhupur c*lay is the oldest sediment exposed in and around the city area having characteristic topography and drainage. The major geomorphic units of the city are: the high land or the Dhaka terrace, the low lands or floodplains, depressions and abandoned channels. Low lying swamps and marshes located in and around the city are other major topographic features.

The subsurface sedimentary sequence, up to the explored depth of 300m, shows three distinct entities: first one is the *Madhupur c*lay of the Pleistocene age, characterized by reddish plastic clay with silt and very fine sand particles. Second one is this *Madhupur* clay uncomfortably overlies the *DupiTila* formation of the *Plio-Pleistocene* age, composed of medium to coarse yellowish brown sand and occasional gravel. And the third one is the incised channels and depressions within the city are floored by recent alluvial floodplain deposits and are further subdivided into lowland alluvium and highland alluvium. Geotechnical characteristics of the *Madhupur clay* in Dhaka city and its surroundings vary significantly both aerially and vertically. The moisture content and plastic limit results show that *Madhupur clay* is normally consolidated to over-consolidated clay. The clay is normal to active and has intermediate to high plasticity. The compressibility values suggest that the clay is very low to highly compressible at different locations.

The *DupiTila* sands aquifer is the main source of water in Dhaka city. *Madhupur clay* overlies the Aquifer with a thickness of 8 to 45m.The aquifer varies in thickness from 100 to 200m. Groundwater occurs at a depth of 25 to 30m in the central part of the city. In the periphery the ground water table lies at a depth of 15 to 20m.

Dhaka city and its surroundings are situated in the seismic Zone 2, the medium hazard zone which has a basic seismic coefficient, Z=0.15 (BNBC, 1993). It has been observed that the NNW-SSE trending *Madhupur* fault is associated with two earthquake epicenters of magnitude 6 and 7 (Hoque et al., 1994). The *Madhupur* and another nearby fault Bansi are very close to the metropolis Dhaka and magnitudes with which they are associated are remarkably high in the context of seismicity. It is evident that the metropolis and its surroundings have high probability of seismic hazard (Ansary et al., 2004).

3. Sub-soil Characteristics

Field investigations were carried out at four selected sites in Dhaka city. These locations have been selected based on prime importance of the city. The selected sites are Dhaka University Campus, Farmgate, Mohakhali and Uttara. Locations of the selected areas on Dhaka city map have been shown in Fig. 1. Standard Penetration Test (SPT) was conducted at all selected locations. Laboratory tests were conducted to determine the index properties, strength properties and compressibility properties of the soil.

3.1 Field Tests

Standard Penetration Test (SPT) was conducted in all the selected sites. Wash boring technique was used for SPT. Disturbed samples have been collected and SPT N-value has been recorded at every 1.5m depth interval up to 15 to 30m from Existing Ground Level (EGL).The test procedure is described in ASTM D1586 (ASTM, 1989). Sub-soil profile obtained in the study has been presented in Fig. 2. From the bore logs, it is seen that the upper layer is mainly consist of red clay and the next layer is fine sand layer.

3.2 Laboratory Tests

Disturbed and undisturbed samples were collected during SPT tests. Tests were conducted according to ASTM standards (ASTM, 1989). Index and strength properties were determined to evaluate the sub-soil condition of Dhaka city. Atterberg limits, grain size distribution, unconfined compression test, direct shear tests were conducted at four selected sites. Tri-axial test and 1-D consolidation tests were also conducted for some selected sites.

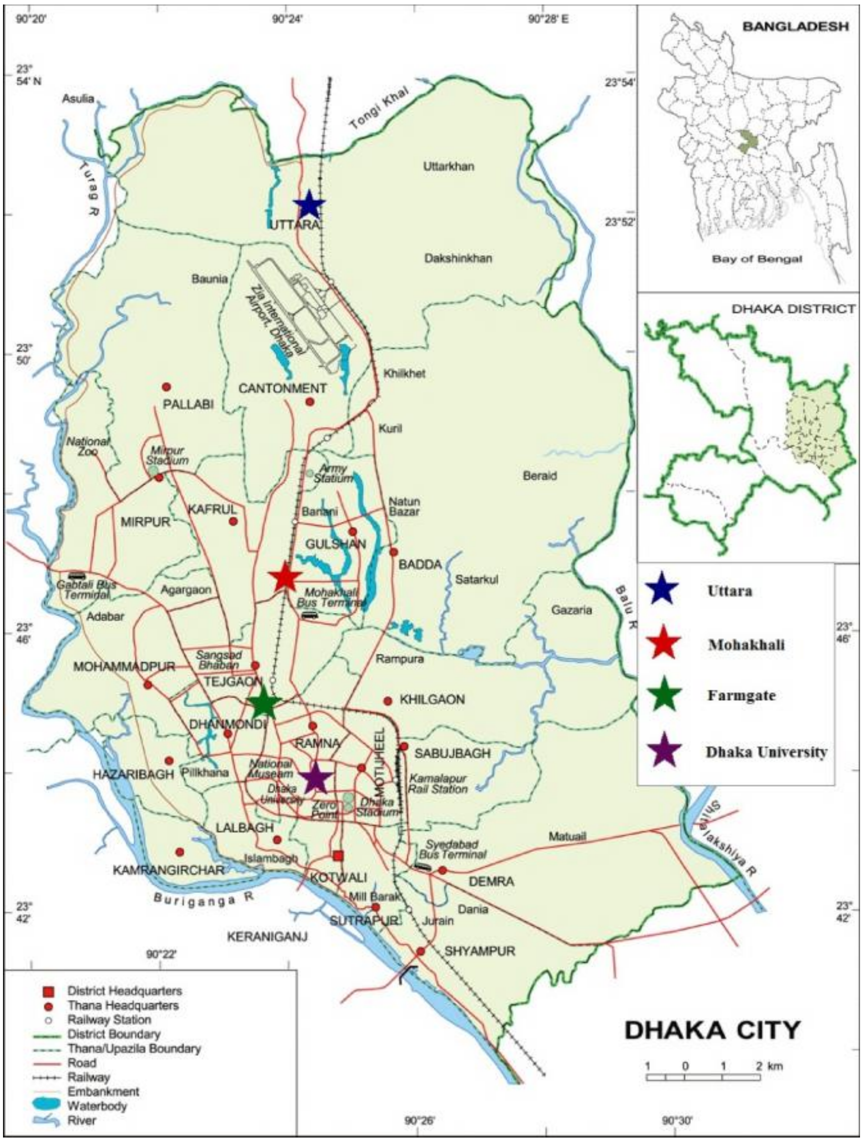

Fig. 1. Dhaka city map showing the location of the study areas

3.2.1 Index Properties

Index properties of the clay layer have been presented in Table 1. Specific gravity of the clay layer varies in the range 2.68 to 2.71. Dry unit weight of the soil varies between 14.11 and 16.20 kN/m^3. Natural moisture content of the soil varies in the range 23.0 and 40.0. Liquid limit and plastic limit of the clay layer vary in the range 48 and 56, 21 and 28, respectively. Sand, silt and clay content of the layer are 3~10%, 49~70% and 27~41%, respectively.

3.2.2 Strength Properties of Dhaka Soil

Strength properties determined from unconfined compression test, triaxial test and direct shear tests are presented in this section. Typical stress-strain relationship obtained from unconfined compression test is presented in Fig. 4a. Table 3 presents the unconfined compressive strength (q_u) of the undisturbed samples collected from different study areas. It is seen that q_u varies significantly in the range between 62 and 139 kPa. This variation might be due to large variation in the natural moisture content (23.0 to 34.5%, Fig. 4b). From consolidated undrained triaxial test, it is found that the cohesion of the clay is about 80 kPa and angle of internal friction is about 6.2º (Fig. 5).

Table 1
Index properties of the clay layer

Soil Parameters	DU Campus	Farmgate	Mohakhali	Uttara
Specific gravity, G_s	2.70~2.71	2.68	2.68~2.69	2.66~2.68
Dry unit weight, γ_d (kN/m^3)	15.79~16.20	14.65~14.96	15.07~15.86	14.11
Natural moisture content (%)	23.0~23.2	24.5~26.7	23.7~26.0	26.0~40.0
Liquid limit, LL (%)	49~50	52~53	48	45~56
Plastic limit, PL (%)	18~22	24~25	21	25~28

Table 2
Grain size distributions of the silty clay and fine sand layer

Layer	Sand (%)	Silt (%)	Clay (%)	D_{50} (mm)
Silty clay	2~12	47~68	26~41	-
Silty fine sand	45~89	11~45	0~14	0.035~0.150

Table 3
Strength properties of clay layer

Soil Parameters	DU Campus	Farmgate	Mohakhali	Uttara
Unconfined compressive strength, q_u (kPa)	110~139	112~119	89~134	62
Failure strain, ε_f (%)	6~8	10~12	6~10	14
Dry unit weight, γ_d (kN/m^3)	15.5~15.9	14.9~14.9	15.0~15.8	14.1
Moisture content (%)	23.0~23.8	24.5~24.8	23.7~26.0	34.5

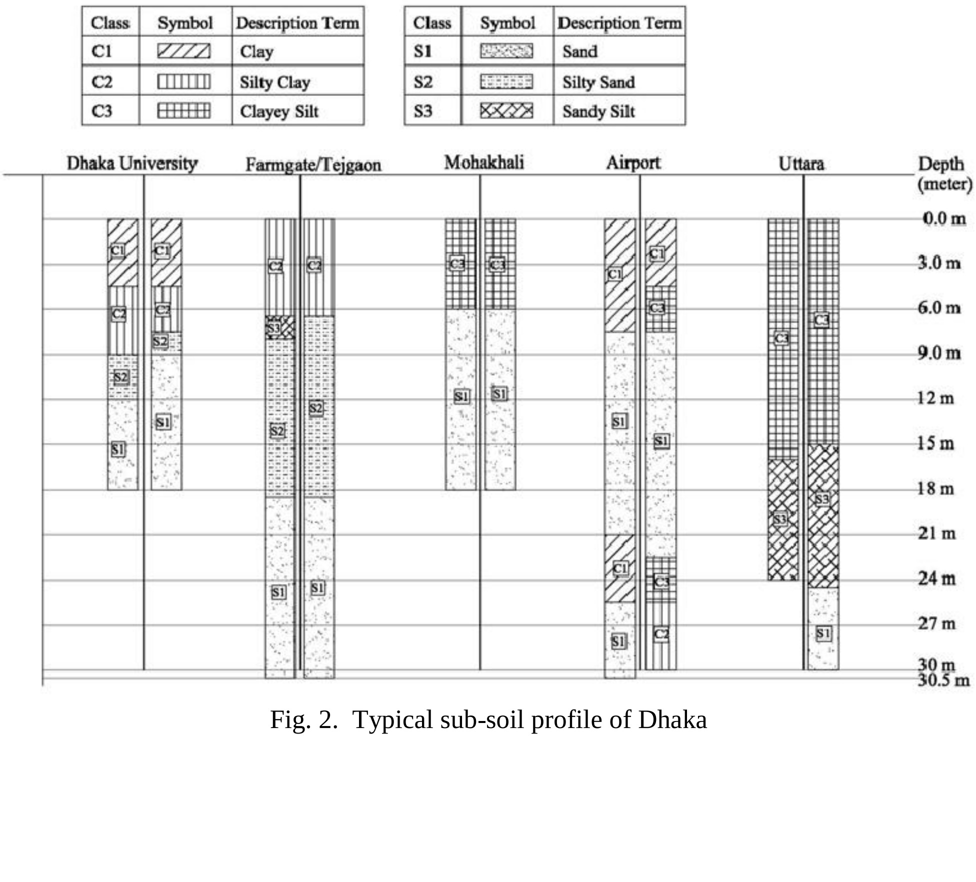

Fig. 2. Typical sub-soil profile of Dhaka

Consolidated undrained direct shear tests were conducted on laboratory reconstituted soil samples. Dry density and water content of the samples were in the range 15.5 to 16.16 kN/m3 and 16.12 to 23.18%. Angle of internal friction of the fine sand layer varies between 31 and 36º.

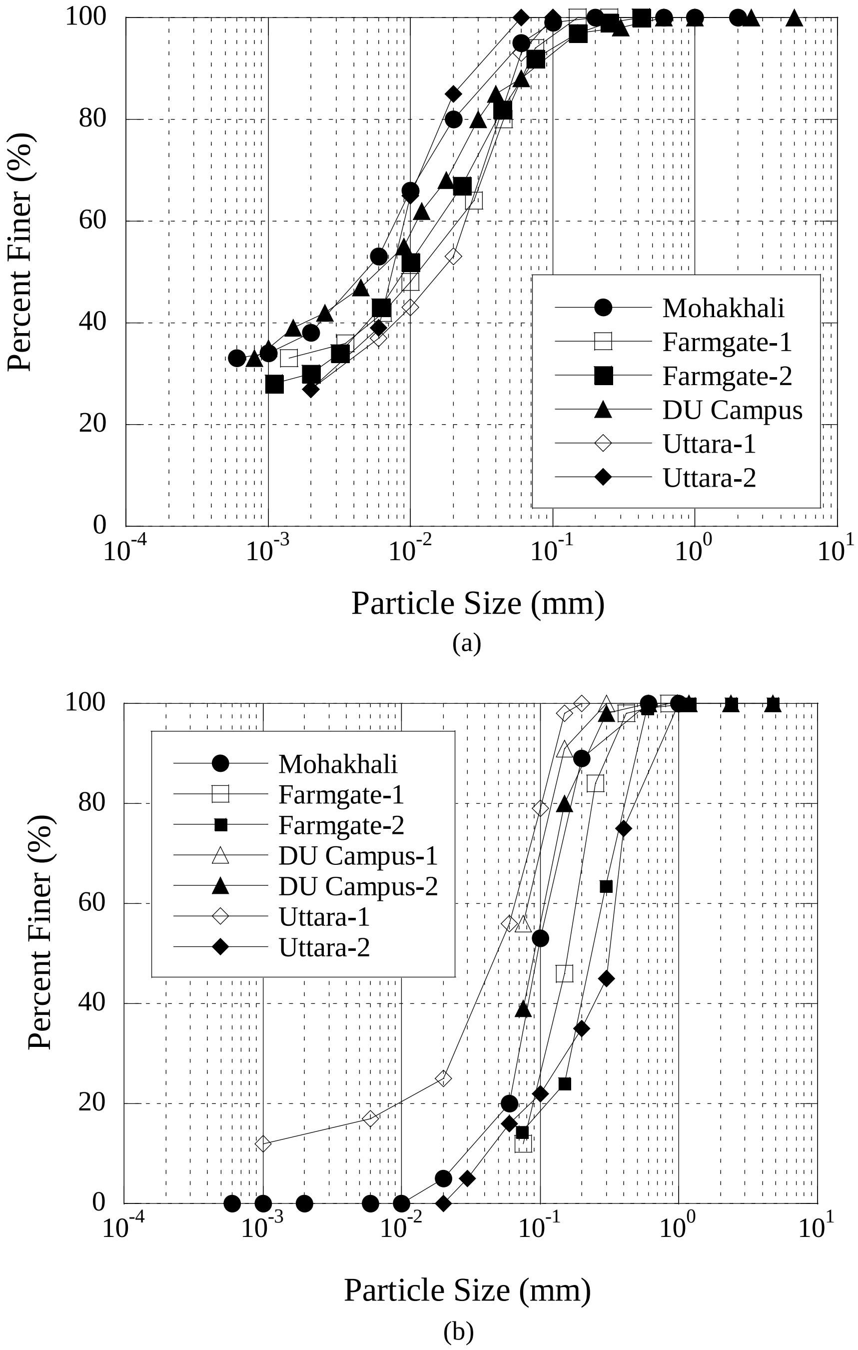

Fig. 3. Grain size distributions of selected samples: (a) sand layer and (b) clay layer

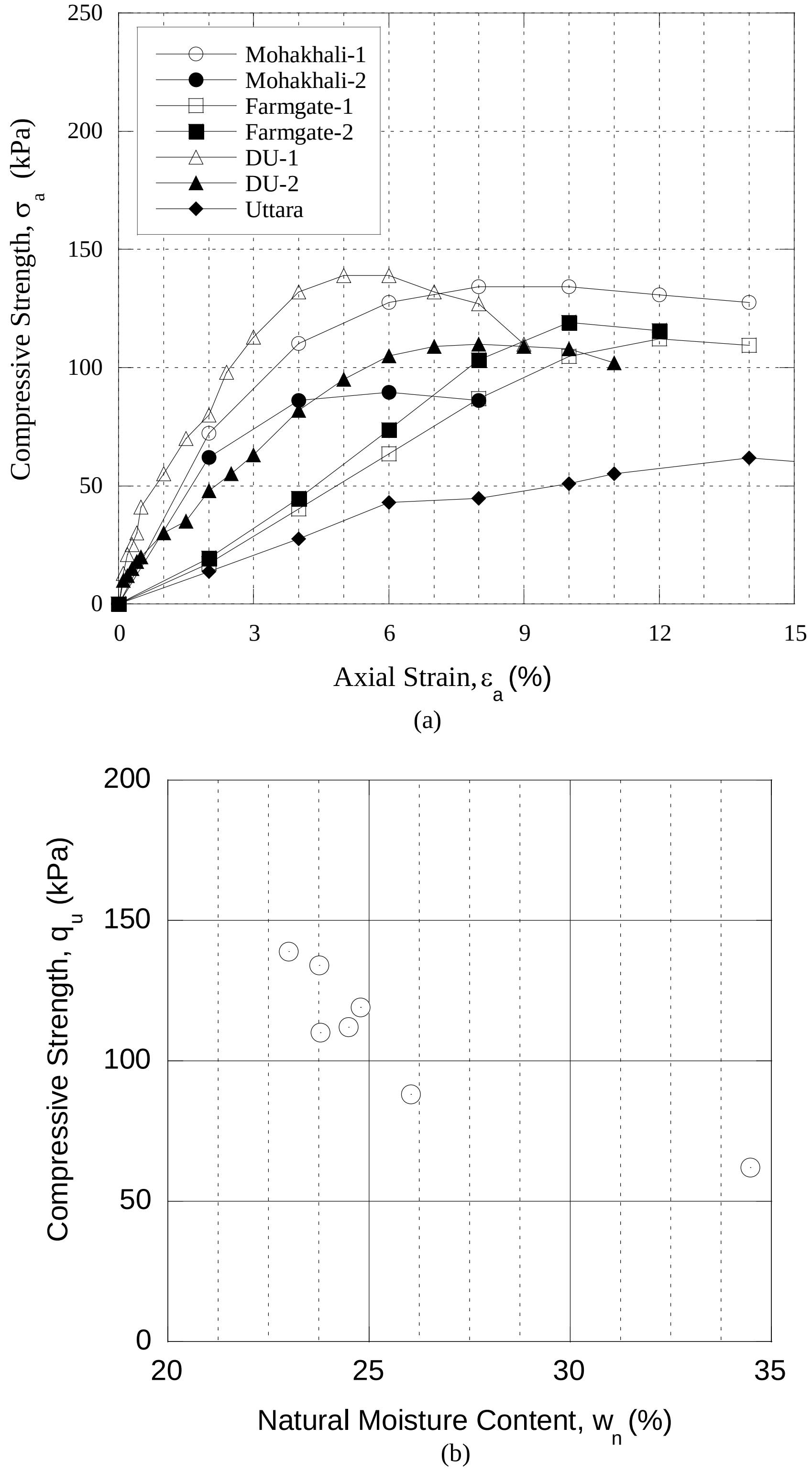

Fig. 4. (a) Stress-strain relationship in unconfined compression test and (b) compressive strength vs. moisture content

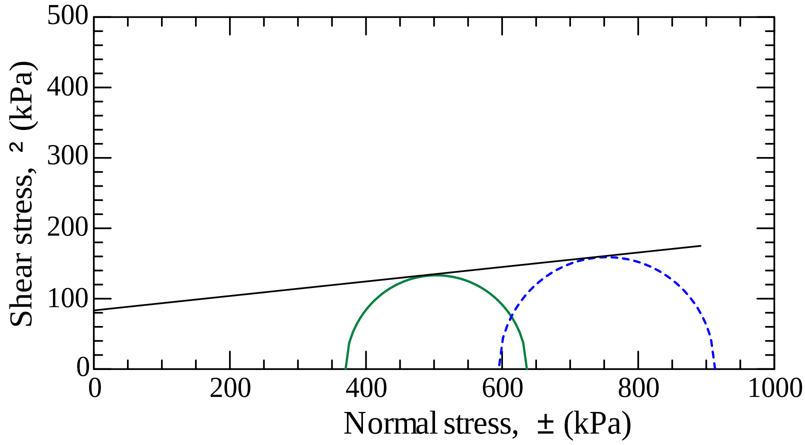

Fig. 5. Failure envelope in triaxial test.

3.2.3 *Compressibility Properties*

1-D consolidation tests were conducted on undisturbed samples collected from DU campus and Farmgate. elog- P curves obtained from 1-D consolidation tests are presented in Figure 7. Initial void ratio of the sample varies between 0.710 and 0.757. Compression index, C_c or λ vary between 0.115 and 0.200. Recompression index or swelling index, C_r or κ vary between 0.0125 and 0.04.

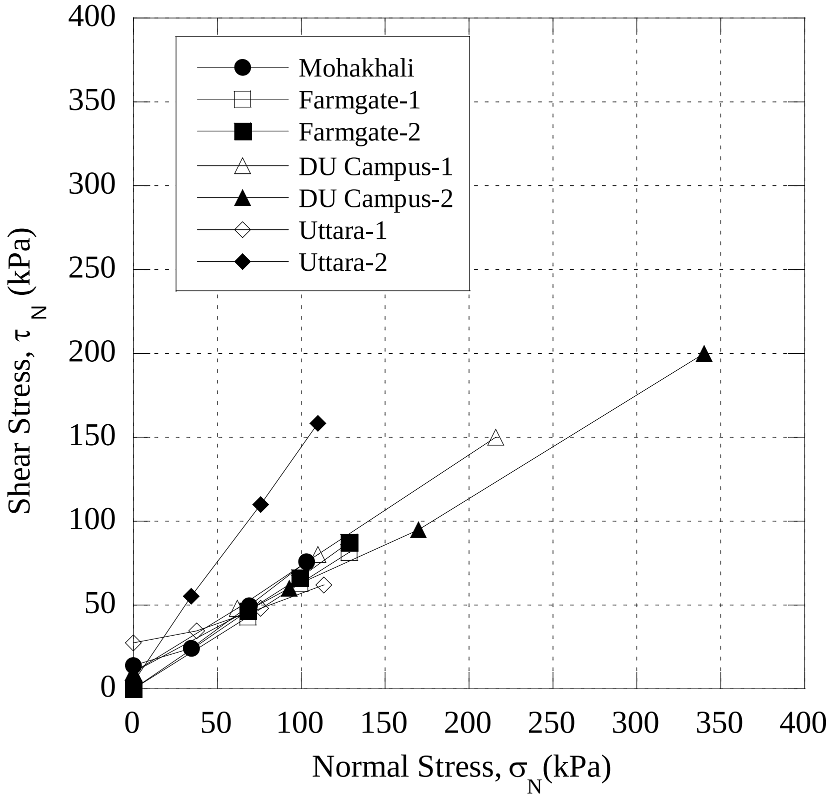

Fig. 6. Failure envelopes in direct shear test

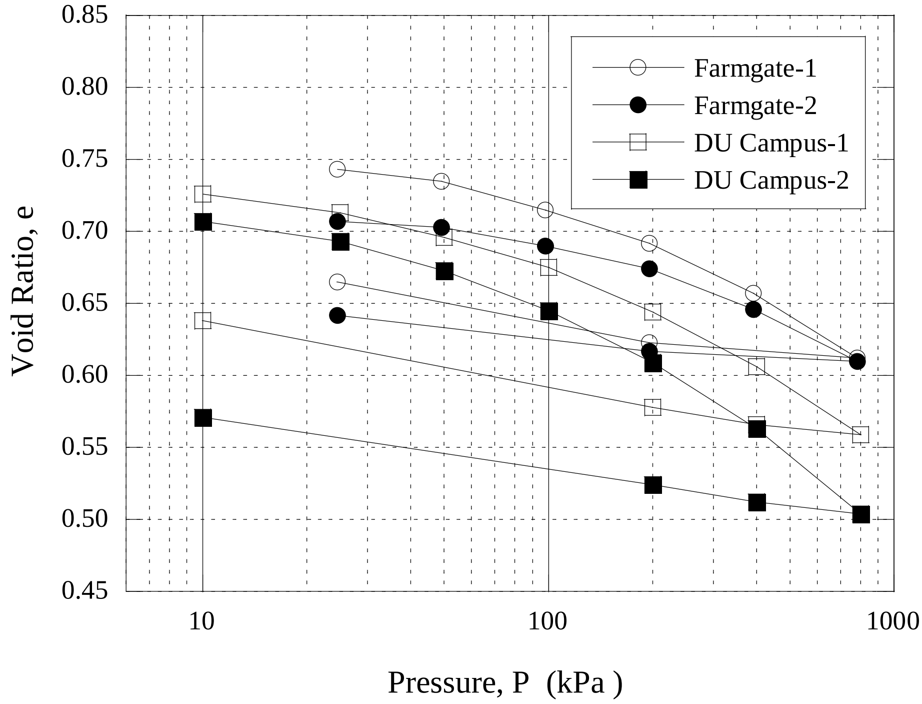

Fig.7. e-logP curves of clay samples obtained from 1-D consolidation test.

4. Parameter Identification for Constitutive Modelling

4.1 Some Important Features of Extended Sub-loading t_{ij} Model

An elastoplastic constitutive model for soils, called the extended subloading t_{ij}-model (Nakai et al., 2011), is used in the finite element analyses. This model, despite the use of a small number of material parameters, can describe properly the following typical features of soil behaviours (Nakai and Hinokio, 2004 & Nakai et al., 2011).

- Influence of intermediate principal stress on the deformation and strength of geomaterials.
- Dependence of the direction of plastic flow on the stress paths.
- Influence of density and/or confining pressure on the deformation and strength of geomaterials.
- The behavior of structured soils such as naturally deposited soils.

A brief description of the above mentioned features of this model can be made as follows:
Influence of intermediate principal stress is considered by defining yield function f with modified stress t_{ij} (i.e., defining the yield function with the stress invariants (t_N and t_S) instead of (p and q). The yield function is written as a function of the mean stress t_N and stress ratio $X \equiv t_S/t_N$ based on t_{ij} by Eq.(1).

$$f = \ln\frac{t_N}{t_{N0}} + \varsigma(X) - \left(\ln\frac{t_{N1e}}{t_{N0}} - \ln\frac{t_{N1e}}{t_{N1}}\right) = 0 \tag{1}$$

Here, t_{N1} determines the size of the yield surface (the value of t_N at X=0), t_{N0} is the value of t_N at reference state and t_{N1e} is the mean stress t_N equivalent to the present plastic volumetric strain which is related to the plastic volumetric strain $£_v^p$ as

$$£_v^p = \frac{ª - ©}{1+e_0} \ln\left(\frac{t_{N1e}}{t_{N1}}\right) \tag{2}$$

The symbols ª and © denote compression index and swelling index, respectively, and e_0 is the void ratio at reference state. In this research, the expression for ς(X) is assumed as,

$$s\left(X\right) = \frac{1}{}\left(\frac{X}{\mathrm{M}^*}\right) \quad (\quad \text{: material parameter}) \tag{3}$$

The value of M* in Eq.(4) is expressed as follows using principal stress ratio X_{CS}≡$(t_S/t_N)_{CS}$ and plastic strain increment ratio Y_{CS}≡$(d£_{SMP}{}^{*p}/d¥_{SMP}{}^{*p})_{CS}$ at critical state:

$$\mathrm{M}^* = \left(X_{CS} + X_{CS}{}^{-1} Y_{CS}\right)^{1/} \tag{4}$$

and these ratios X_{CS} and Y_{CS} are represented by the principal stress ratio at critical state in triaxial compression R_{CS}.
In elasto-plastic theory, total strain increment consists of elastic and plastic strain increments as

$$d£_{ij} = d£_{ij}^e + d£_{ij}^p \tag{5}$$

Here, plastic strain increment is divided into component $d£_{ij}{}^{p(AF)}$, which satisfies associate flow rule in the space of modified stress t_{ij}, and isotropic compression component $d£_{ij}{}^{p(IC)}$as given in Eq.(6).

$$d£_{ij}^p = d£_{ij}^{p(AF)} + d£_{ij}^{p(IC)} \tag{6}$$

The components of strain increment are expressed as,

$$d£_{ij}^{p(AF)} = \Lambda \frac{\partial f}{ô\, t_{ij}} \quad and \quad d£_{ij}^{p(IC)} = K\left\langle dt_N \right\rangle \frac{¢_{ij}}{3} \tag{7}$$

Here, Λ is the proportionality constant, $¢_{ij}$ is Kronecker's delta and <> denotes Macauley bracket. Dividing plastic strain increment into two components as in Eqs.(6) and (7), for the same yield function, this model can take into consideration feature (ii), i.e., the dependence of the direction of plastic flow on the stress paths. Adding the term $G(°)$ in the denominator of the proportionality constant Λ of normal consolidated condition, influence of density is considered. The proportionality constant Λ is expressed as

$$\Lambda = \frac{\dfrac{\partial f}{\partial ±_{ij}} d±_{ij}}{\dfrac{1+e_0}{ª - ©}\left(\dfrac{\partial f}{\partial t_{kk}} + \dfrac{G(°)}{t_N} + \dfrac{Q(µ)}{t_N}\right)} = \frac{df_±}{h^p} \tag{8}$$

$$\text{and } K = \frac{1}{\frac{1+e_0}{ª-©}\left(1+\frac{G(°)}{a_{kk}}\right)} \cdot \frac{1}{t_{N1}} \quad (9)$$

In feature (iv), the stress-strain behavior of structured soil can be described by considering not only the effect of density described above but also the effect of bonding. Two state variables ° related to density and µ representing the bonding effect are used to consider feature (iv). The following relationships for *G(°)* and *Q*(µ) are adopted in the model:

$$G(°) = sign(°)a°^2 \quad and\ Q(µ) = bµ \quad (10)$$

where, *a* and *b* are material parameters.

The parameters of subloading t_{ij} model are fundamentally the same as those of the Cam clay model (Roscoe and Burland, 1968), except for the parameter *a*, which is responsible for the influence of the density and the confining pressure. Parameter controls the shape of the yield surface. The performance of the constitutive model has already been checked in numerical simulations (Shahin et al. 2004, Shahin *et al.*, 2011; Nakai *et al.*, 2010).

4.2 Determination of Soil Parameters

For getting parameters of the constitutive model, 1-D consolidation tests for Farmgate and DU campus soils have been carried out in laboratory. Fig.8 shows the relations between void ratios and mean effective stress in logarithmic scale for Farmgate soil. From these curves, compression index ª, swelling index © and void ratio at 98kPa, N are obtained for both sites. Using these values and fitting the computed curve parameter *a* (density parameter) of subloading t_{ij} model is obtained. Drained triaxial test for Farmgate soils have also been conducted in laboratory to capture the properties of the soils in terms of the stress-strain relations. Figure 9 represents the results of the triaxial compression drained test for Farmgate clay. In numerical analysis, triaxial simulation is carried out considering one element in axisymmetric condition the same way as the laboratory tests. Parameters obtained are presented in Table 6. Compression index, λ is 0.080, swelling index, © is 0.078, ρ reference void ratio on normally consolidation line at p= 98 kPa& q= 0 kPa, *N* is 0.80, critical state stress ratio $R_{cs}= (\sigma_1/\sigma_3)_{cs(comp.)}$ is3.82,Poisson's ratio, $\neg_e$ is 0.20 shape of yield surface (same as original Cam clay at = *1)*is 1.50, influence of density and confining pressure is 600, influence of structure of soil, *b*is 2.5.

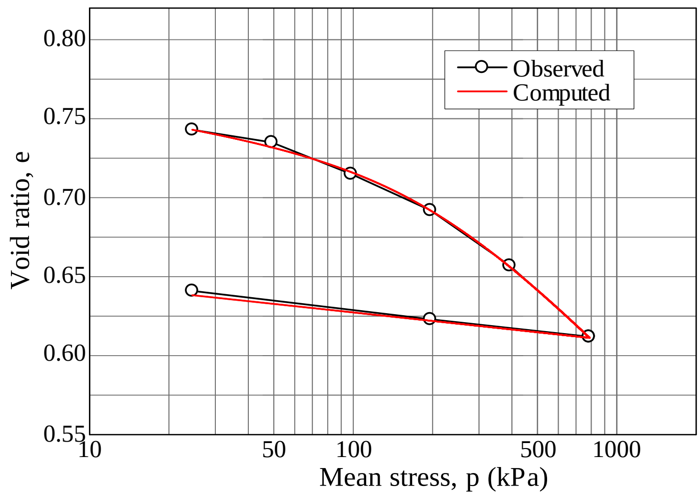

Fig. 8. Comparison of e-logP curves of clay samples collected from Farmgate site.

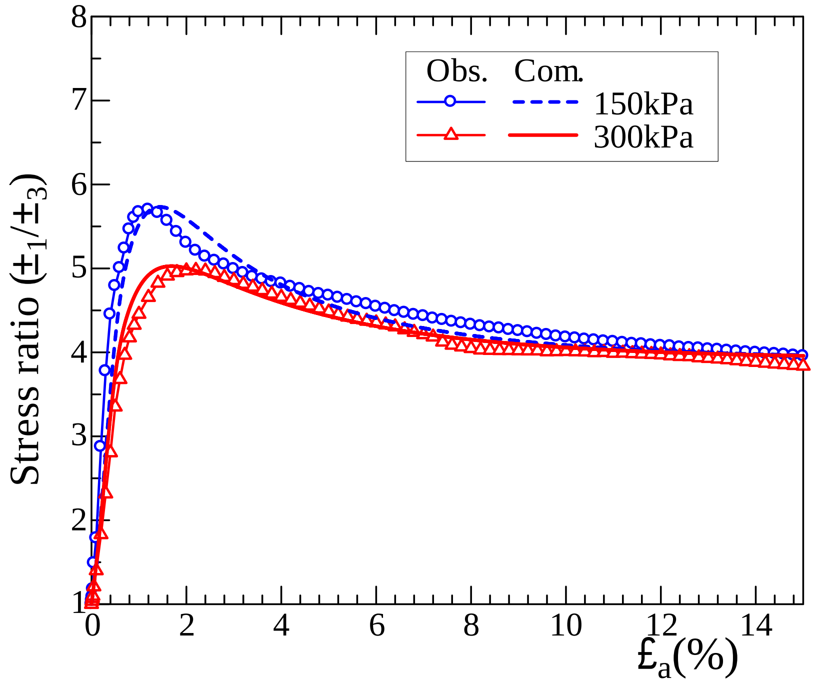

Fig. 9. Comparison of stress ratio and axial strain graphs of Dhaka clay.

Parameters obtained in the study has been compared with those of Fujinimori clay (Grain size: sand 13.5%, silt 69.5% and clay 17%; specific gravity G_s=2.67; Index properties: Liquid limit, LL=41%, Plastic limit, PL=23.0%, and plasticity index, PI=18%) in Table 6. It is seen that most of the elasto-plastic constitutive model parameters are similar to those of Fujinimori clay.

4.3 Numerical Integration

In the numerical integration, sub-stepping step (Shahin *et al.*, 2011) with Modified Euler Method is the finite element analyses. The essence of the method is to find the appropriate size of the time increments in such a way that the local truncation error during each sub-step is kept within an acceptable tolerance. For that purpose at least two evaluations of the final solution have to be computed. The first uses a simple first order Forward-Euler approximation:

$$\sigma_{ij,k+1} = \sigma_{ij,k} + \Delta\sigma_{ij,1} \tag{11}$$

in which

$$\Delta\sigma_{ij,1} = D^{ep}_{ij}(\sigma_{ij,k}, z_k) : \Delta\varepsilon_{ij,k} \tag{12}$$

The second approximation uses a second order Modified-Euler scheme:

$$\hat{\sigma}_{ij,k+1} = \sigma_{ij,k} + \frac{1}{2}\left(\Delta\sigma_{ij,1} + \Delta\sigma_{ij,2}\right) \tag{13}$$

in which

$$\Delta\sigma_{ij,2} = D^{ep}_{ij}\left(\sigma_{ij,k+1}, z_{k+1}\right) : \Delta\varepsilon_{ij,k} \tag{14}$$

Thus, an error estimate can be obtained from the difference between the first and second orders approximations:

$$\tilde{E}_k \approx \frac{1}{2}\left(-\Delta\tilde{\sigma}_1 + \Delta\tilde{\sigma}_2\right) \tag{15}$$

and a relative error measure is defined as:

$$Err_k = \frac{\left\|\tilde{E}_k\right\|}{\left\|\tilde{\sigma}_{k+1}\right\|} \tag{16}$$

This relative error measure is compared with a pre-defined error tolerance (*TOL*) and used to decide whether the present increment size is acceptable or not. Based on the previous study (Shahin *et al.*, 2011) the optimum value of *TOL* for Modified Euler Method was found 10^{-4} which was also used in the subsequent 2D and 3D bearing capacity analyses.

Table 4
Strength properties of fine sand layer

Soil Parameters	DU Campus	Farmgate	Mohakhali	Uttara
Angle of Internal Friction, ϕ(deg.)	32~35	31~36	33	35
Cohesion, c (kPa)	0	0	0	4.0
Dry unit weight, γ_d (kN/m^3)	15.79~16.16	15.5	15.8	15.6
Moisture content (%)	16.12~23.18	-	-	-

Table 5
Physical and compressibility properties of clay layer

Soil Parameters	DU Campus	Farmgate
Void ratio, e_0	0.735~0.749	0.710~0.757
Compression index, C_c or λ	0.160~0.200	0.115~0.135
Recompression index or swelling index, C_r or κ	0.0400	0.0125~0.0189
e_N	0.71~0.695	0.713~0.742

5. Results and Discussion on Bearing Capacity of Dhaka Sub-soil

5.1 Two-dimensional Analyses

Bearing capacity for Dhaka Clay is simulated with finite element analyses. The simulations are carried out considering plane strain undrained conditions using FEMtij-2D. Figure 10 shows a typical mesh for two dimensional analyses, where the bottom boundary is fixed, and the lateral boundaries are free in the vertical direction. Isoperimetric 4-noded quadrilateral elements are used to model the ground in the analysis. The same parameters for Dhaka Clay shown in Table 6 are used in the analyses. In every analysis, the 2D model ground is made by self-consolidating Dhaka Clay (unit weight=16.66 kN/m^3) at a very low confining stress $p_0=9.8\times10^{-6}$kPa). For the ground condition, a strip foundation of 4m is used which is assumed to be an elastic material with large stiffness, and the frictional behavior between the foundation and the ground is simulated by an elastoplastic joint element (Nakai, 1985). The friction angle between the foundation and the soils is assumed to be 15°. After making initial ground displacement is applied at central node on the strip foundation.

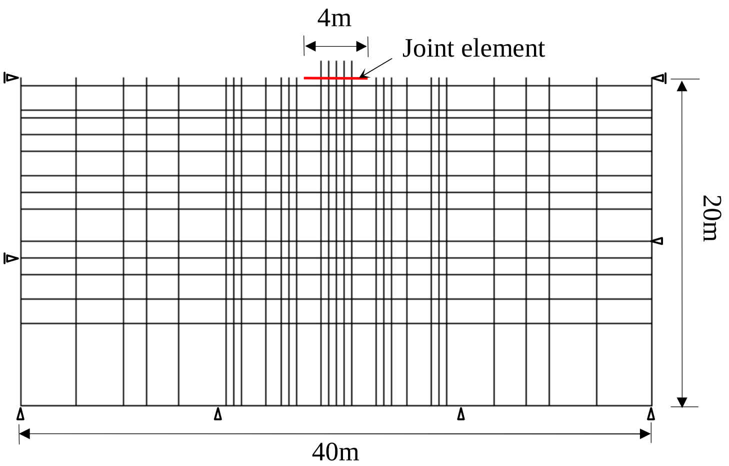

Fig. 10. Finite element mesh for bearing capacity analysis

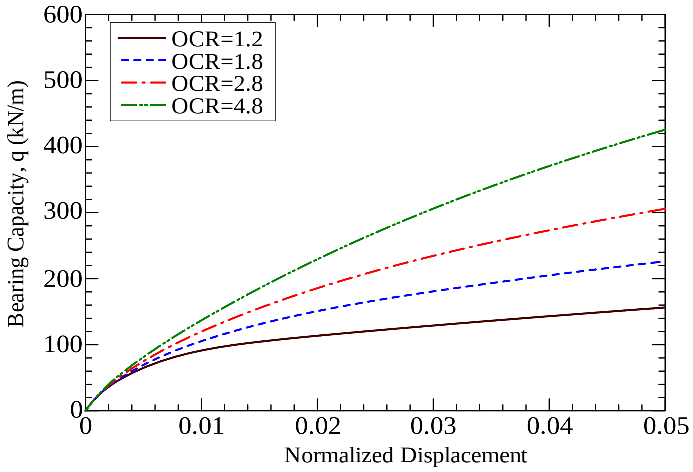

Fig. 11. Bearing capacity of Dhaka Clay for different OCR: strip foundation

Table 6
Parameters obtained from extended subloading t_{ij} model for Dhaka clay

Parameter	Notation	Value		Remarks
		Dhaka Clay	Fujinomori Clay	
Compression index	λ	0.080	0.1039	Same parameters as Cam-clay model
Swelling index	©	0.078	0.0099	
Reference void ratio on normally consolidation line at p= 98 kPa& q= 0 kPa	*N*	0.80	0.922	
Critical state stress ratio $(\sigma_1/\sigma_3)_{cs(comp.)}$	$R_{cs}=R_{CS}=(\pm_1/\pm_3)_{CS(comp.)}$	3.82	3.20	
Poisson's ratio	¬$_e$=	0.20	0.20	
Shape of yield surface (same as original Cam clay at = 1)		1.50	1.50	
Influence of density and confining pressure	*a*	600	500	
Influence of structure of soil	*b*	2.5	-	

Figure 11 shows the load-displacement curves of Dhaka Clay subjected to vertical concentric loading for different Over Consolidation Ratio (OCR).The vertical axis indicates the load normalized with the width of the strip foundation, and the horizontal axis is the displacement normalized with the width of the foundation, *B*. It is seen that the load carrying capacity of stiffer ground is larger than that of the soft ground as can be expected.

5.2 Bearing Capacity Obtained from Equation

The most commonly used bearing capacity equation is that equation developed by Terzaghi (1943).For a uniform vertical loading of a strip footing, Terzaghi (1943) assumed a general shear failurein order to develop the following bearing capacity Equation 17,

$$q_{ult}= cN_c+0.5\gamma_t BN_\gamma+\gamma_t D_f N_q \qquad (17)$$

where, q_{ult} = ultimate bearing capacity for a strip footing
B = width of the strip footing
L = length of the strip footing
γ_t = total unit weight of the soil
D_f = vertical distance from ground surface to bottom of strip footing
c = cohesion of the soil underlying the strip footing
N_c = dimensionless bearing capacity factors
N_q = dimensionless bearing capacity factors
N_γ = dimensionless bearing capacity factors

According to equation (17) and for clay (c_u= 40 kPa, ϕ=0, N_c=5.5), bearing capacity of a strip foundation is 220kPa which corresponds to the bearing capacity obtained from the numerical analyses having OCR=1.8 (Fig. 11). The results of the numerical simulation well match the results of the conventional analysis. In addition, as the numerical analysis can estimate the bearing capacity for different ground densities (OCRs), the numerical tools can be used in calculating the bearing capacity of the ground.

5.3 Three-dimensional Analysis

Three-dimensional finite element analysis is carried out with FEMtij-3D program. Figure 12 shows a mesh used in the finite element analysis. As concentric vertical load was applied in this research, one-fourth of the ground is taken for the analyses considering symmetric conditions in both x and y directions. Smooth boundary conditions are applied to all vertical faces, while the nodes in the boundary are kept fixed in the all directions.

The width of the foundation is 5m in both x and y directions in full scale, in one-fourth scale it is 2.5m. Load is applied at the top corner node as depicted in the figure. 8-noded brick element is used as soil element. Elasto-plastic joint element (Nakai, 1985) is used to model interface element between foundation and the ground. The same parameters of Dhaka Clay (Table 6), are used for the ground materials. Here, only the ground having OCR of 1.2 is analyzed and compared with the results of the plane strain condition. The analysis is done in undrained condition.

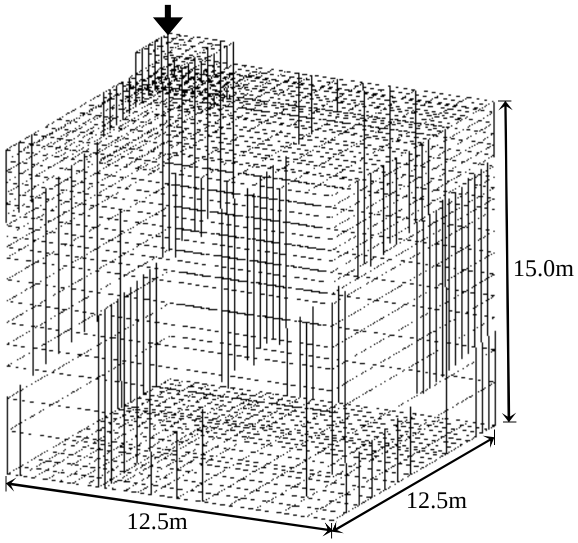

Fig. 12. Typical finite element mesh for prototype analysis

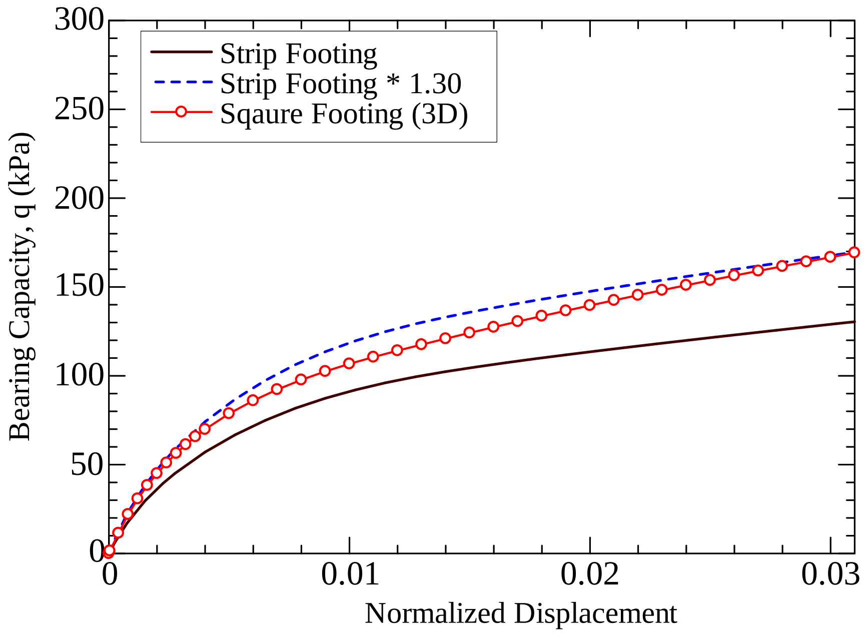

Fig. 13. Bearing capacity of Dhaka Clay – influence of the shape of the footing (OCR=1.2)

Figure 13 shows the load-displacement curves of the 3D analysis. For comparison load-displacement curve for the strip footing of 2D analysis having the same ground condition (OCR=1.2) is plotted in this figure. It is found that if the result of the strip footing is multiplied by 1.3, the curve is almost coincides with the curve of the square footing (3D analysis). In the conventional method of estimating the bearing capacity, the shape factor is assumed as 1.3. Therefore, it can be said that the numerical analyses can well capture the effect of shape factor in predicting the bearing capacity of the footing.

6. Conclusions

Elasto-plastic constitutive model parameters and their application to bearing capacity estimation for Dhaka sub-soil is investigated in this study. The main findings of the study are:

- Dhaka soil largely consists of fine sand with clay deposit at upper most layer. Specific gravity of the clay layer varies in the range 2.68 to 2.71. Dry unit weight of the soil varies between 14.11 and 16.20 kN/m^3. Natural moisture content of the soil varies in the range 23.0 and 40. Liquid limit and plastic limit of the clay layer vary in the range 48 and 56, 21 and 28, respectively. Sand, silt and clay content of the layer are 3~10%, 49~70% and 27~41%. Mean grain size, D_{50}varies in the range between 0.035 and 0.15 mm. Undrained shear strength varies between 30 and 70 kPa.
- Compression index, λ is 0.080, swelling index, κ is 0.078, ρ reference void ratio on normally consolidation line at p= 98 kPa & q= 0 kPa, N is 0.80, critical state stress ratio $R_{cs}= (\sigma_1/\sigma_3)_{cs(comp.)}$ is 3.82, Poisson's ratio, ν_e is 0.20, shape of yield surface (same as original Cam clay at β= 1)is 1.50, influence of density and confining pressure is 600, influence of structure of soil, *b* is 2.5. These parameters are almost similar to those of Fujinomori clay. These parameters will be useful for constitutive modeling of excavation problem, such as open-cut excavation, tunneling, retaining wall problems, bearing capacity estimation.
- Both 2D and 3D analyses were conducted considering OCRs and bonding effect to determine the bearing capacity of shall foundation considering shape factors. It is found that bearing capacity obtained from classical theories almost the same as result of OCR=1.2. As the numerical analysis can estimate the bearing capacity for different ground densities (OCRs), it can be used in calculating the bearing capacity of different grounds. The numerical analyses can well capture the effect of shape factor in predicting the bearing capacity of the footing.

Acknowledgements
The authors greatly acknowledge the support provided by Bangladesh University of Engineering and Technology (BUET), Bangladesh and Department Civil Engineering, Nagoya Institute of Technology, Japan for this research. The authors are also grateful to Dhaka Soil and Moderns Khawaja Soil Engineers for providing necessary data.

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