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Computation of bearing capacity in a multi-layered soil unit: A case study from Dharan, Eastern Nepal

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ABSTRACT

In this paper, a process of input data preparation for bearing capacity analysis in a multi-layered soil unit is presented. The method first takes into account of the engineering geological conditions of the construction site followed by geotechnical investigations that includes, among others, drilling boreholes at particulars sites also performing standard penetration test (SPT). Besides, various laboratory tests, including consolidation test, were also performed on soil samples collected from different soil layers. Finally, the soil is generalized into certain layer system and the bearing capacity of the foundation soil is computed using both shear failure as well as settlement criteria.

INTRODUCTION

Computation of bearing capacity in foundation analysis is an important task. Application of a downward load by a civil structure may cause settlement or shear failure of the foundation depending upon the nature of the foundation soil. In general, shear failure may occur in cohesive soil such as clay, and settlement may take place in non-cohesive soil such as sand. Both, the settlement and the shear failure may take place in various ways and each affects the performance of the structure differently.

Burland and Worth (1974) have discussed the difficulties in computing a numerical magnitude for the settlement that would cause problems in civil structures. Such a computation would require a complex indeterminate analysis (Wahls 1981). Because the problem is so complex, one has to rely on empirical correlations between the observed behavior of structures and one or more of soil parameters.

The magnitudes of the shear stresses in the soil beneath a footing depend largely on the net bearing pressure and the size of the footing. If the bearing pressure is large enough, or the footing is small enough, these shear stresses may exceed the shear strength of the soil, resulting in a bearing capacity failure. Vesic (1963) has identified three types of bearing capacity failures: a general shear failure, a local shear failure, and a punching shear failure. The general shear failure occurs in soils that are relatively incompressible and reasonably strong (relative density greater than 67 %), or in saturated, normally consolidated clays that are loaded rapidly enough that the undrained strength prevails. The punching shear failure occurs in very loose sands (relative density less than 30 %), in a thin crust of strong soil underlain by a very weak soil, or in weak clays loaded under slow, drained conditions. The local shear failure occurs in loose to medium dense sands (relative density between 30 and 67 %). Vesic (1973) has suggested that for most practical design problems, it is only necessary to check the general shear case, and then conduct settlement analysis to verify that the footing will not settle excessively. These settlement analyses implicitly protect against local and punching shear failures.

The commonly used analysis techniques for predicting bearing capacity are based on soil profiles that are either entirely cohesive or entirely non-cohesive. This creates difficulties when one has to deal with multi-layering soil profiles that also include thin lenses on it. At many circumstances, foundations are also designed based on empirical values of bearing capacity without carrying out proper soil investigation. In addition, recently people have started constructing multi-storied buildings in many parts of Nepal for commercial activities and public utilities but practice of consulting engineering geologists and foundation engineers to design foundation is still in infant stage in Nepal.

In this paper, a process of input data preparation for bearing capacity analysis in a multi-layered soil unit is presented. The method first takes into account of the engineering geological conditions of the construction site followed by geotechnical investigations that includes, among others, drilling boreholes at particular sites also performing standard penetration test (SPT). Besides, various laboratory tests, including consolidation test, are also performed on soil samples collected from different soil layers. Finally, the soil is generalized into certain layer system and the bearing capacity of the soil is computed using both shear failure as well as settlement criteria.

STUDY AREA

The study site lies in Sunsari District, Eastern Nepal at the Bus Park of Dharan Bazar, also known as Bhanu Chowk. It is on the western side of the Dharan-Dhankuta Road and is surrounded by heavily built-up area of Dharan Bazar. The site features an open ground having about 60 m in length and 26 m in width that slopes gently towards south. The southern margin of the site is at an elevation of 97.30 m amsl, whereas the northern boundary follows 99.50 m contour line resulting an elevation difference of 2.20 m between the northern and southern sides of the site.

REGIONAL GEOLOGY

The study area lies on the Terai Zone made up of alluvium of Pleistocene to Recent age. Lying on the southern most part of Nepal, the Terai zone depicts three distinct geomorphologic units namely, from south to north, Southern Terai, Middle Terai and Bhaber Zone. The grain size of the sediments in the Terai Zone decreases from north to south having coarser sediments like gravel, cobbles and boulders in the Bhabar Zone and finer sediments such as silt and clay in the Southern Terai. In the Middle Terai, the sediments are of intermediate nature consisting of gravel and sand intermixed with silts and clays.

The study area, Dharan, belongs to the Bhaber Zone and is situated at the foothill of the Churia range. The Bhaber Zone at this place is represented by the Dharan Formation (Anon 2001). The Dharan Formation at this place is composed of ill-sorted deposits of sub-angular to sub-rounded boulders, gravels, cobbles and pebbles of quartzite, sandstone and gneiss derived from the Siwalik and Lesser Himalayan mountains and intermixed with sand and silt. The Dharan Formation is upto 25 m thick underlain by Siwalik rocks. The surface humus soil is brownish gray in color and is less than one meter thick.

SITE CHARACTERIZATION

Boring

Six boreholes were drilled by Percussion drilling so as to extract soil samples and, among others, also to reveal the sub-surface soil strata of the site. The procedure followed for boring was as per IS 1892-1979. The boreholes range in depth from 10 to 12 m.

Geological profile

Based on the information reveled by the borehole-logs, geological profiles of the construction site were prepared. As shown in Figures 1 and 2, the surface sediment (fill materials), 0.4 to 1.2 m thick, is underlain by gravel-mixed clayey silt, 2.1 to 4.4 m thick, followed by sandy gravel of 2.5 to 7.3 m thick. Except at northwest corner (borehole 6), the sandy gravel layer contains some lenses of gravel-mixed clayey silt and silty gravel. The former are 1.0 to 1.8 m thick and are observed at different depths in the southern and western part of the site (borehole 1, 2 and 3); whereas the later ones are 1.05 to 1.25 m thick and are encountered in the northeast side of the site (boreholes 4 and 5).

Since the lenses of gravel-mixed clayey silt and silty gravel occurs as isolated patches of comparatively smaller thickness at different depths and locations, the sub-surface soil of the site may be considered two layer system composed of gravel-mixed clayey silt, 3.25 m thick, in the upper part followed by sandy gravel, 4.9 m thick, in the lower part for the purpose of bearing capacity and settlement computation.

Standard penetration test (SPT)

The Standard Penetration Test (SPT) was carried out in each borehole following IS: 2131-1981. An average N-value obtained for each soil layers is depicted in Figures 1 and 2. It is important to note that the soils that contain a large percentage of gravel and those that contain cobbles or boulders create problems in SPT measurement. Often, the in-situ test device is not able to penetrate through such soils or the results are not representative because the particles are about the same size as the test device. The higher values, more than 50, shown in Figures 1 and 2 should be viewed in this line as it corresponds to the presence of cobbles in the soil. Thus, these higher values are not appropriate in using for engineering practices and are therefore excluded in this analysis.

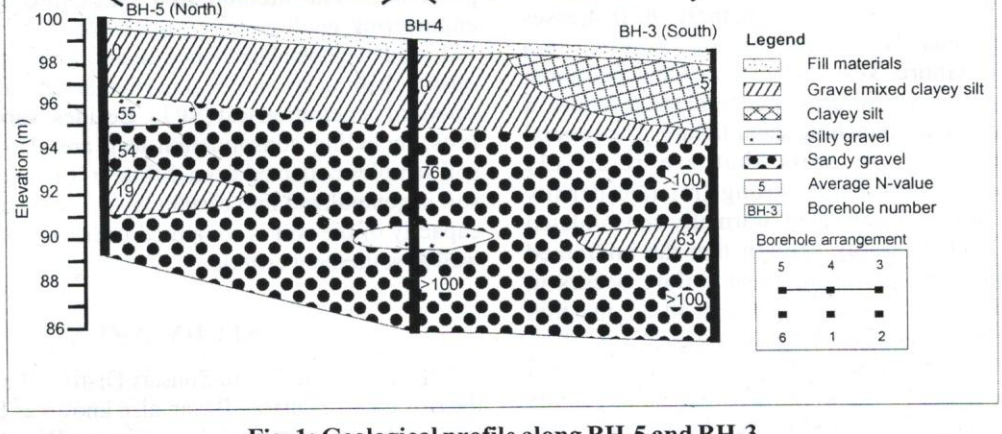

Fig. 1: Geological profile along BH-5 and BH-3

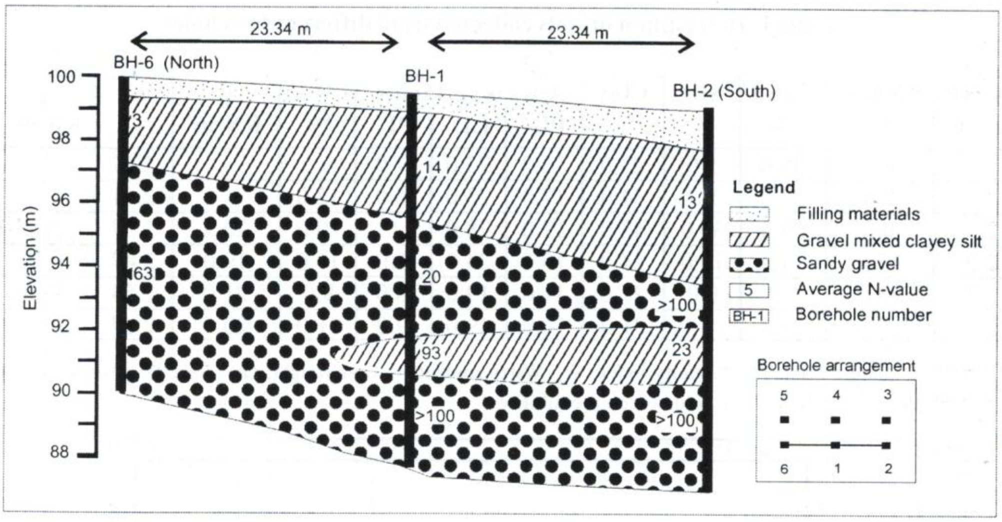

Fig. 2: Geological profile along BH-6 to BH-2

Groundwater table

The groundwater table was not encountered in any of the boreholes drilled. As already mentioned above, the area lies in the Bhaber Zone, which slopes to the south with 100 to 200 m per km. Besides it is composed of coarser sediments compared to the sediments of Middle and Southern Terai, both of which also slope to the south but with lesser gradient (1:200 and 1: 500), respectively (Bhattarai and Rao 1989). These points lead to a fact that the Bhaber Zone forms a recharge area for the groundwater of whole Terai Zone and therefore the groundwater table is not observed at a shallow depth in and around the study site.

LABORATORY TEST

During the drilling, disturbed as well as undisturbed samples were extracted from different depths of the boreholes as per the procedure suggested by IS 1892-1979. Various laboratory tests, namely, grain size analysis (Fig. 3), Atterberg's limit, natural water content, unit weight, specific gravity, unconfined compression test (Fig. 4) and consolidation test (Fig. 5) were conducted to determine related soil properties. Table 1 summarizes the results of the laboratory tests.

COMPUTATION OF BEARING CAPACITY

General

The construction and loading of a footing generates both normal and shear stresses on the ground. This introduces two concerns. First, will the shear stress exceed the shear strength of the soil resulting in a shear failure? Second, will the normal strains induced by the increased normal stresses cause the footing to settle excessively? In this analysis, the bearing capacity has been computed in view of both of these two criteria and minimum of the two values are recommended as the safe bearing capacity of the soil.

Like in most cases, people in Dharan area also have not realized the importance of foundation design. In general, they do not seek geological/geotechnical input from engineering geologists to design foundations for their residential buildings. For the ease of excavation, foundations are generally put at about 1.5 m depth from the existing ground surface. As already mentioned above, the soil upto

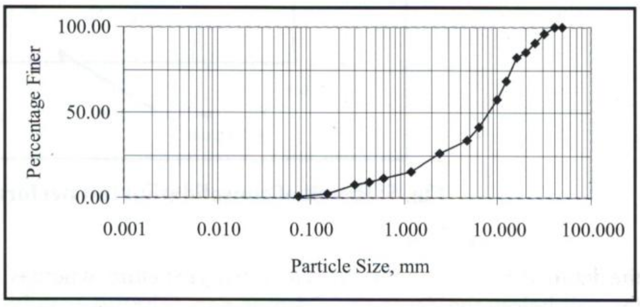

Fig. 3: Grain size distribution curve for a soil sample collected from borehole 6, depth: 5.0 m.

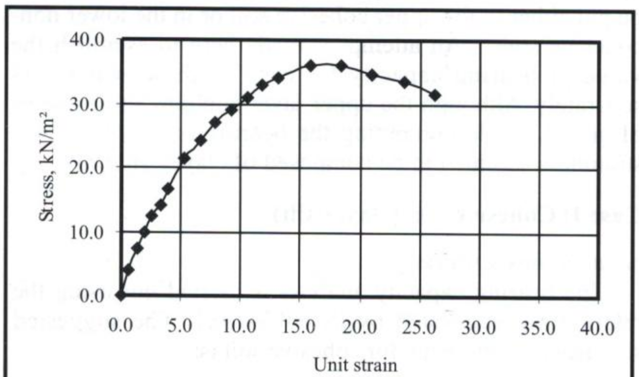

Fig. 4: Stress-strain curve for a soil sample collected from borehole 3, depth: 3.5 m

Table 1: Description of soils collected from different boreholes.

BH N0.	Depth (m)	Gravel %	Sand %	Silt %	Clay %	Atterberg Limits PL	LL	PI	W_c %	γ KN/m^3	G_s	q_u kN/m^2	m_v m^2/kN
1	7.0	40.0	30.0	5.0	25.0	16.0	23.0	7.0	6.0	-	2.37	-	0.0005
2	7.5	22.0	23.0	21.0	34.0	17.0	23.0	6.0	12.0	-	2.61	-	0.0009
3	2.0	0.0	28.0	30.0	42.0	17.0	28.0	11.0	10.0	20.5	2.37	18.0	0.0007
4	2.0	47.0	14.0	15.0	24.0	19.0	24.0	5.0	10.0	19.0	2.47	18.0	0.0005
5	2.0	33.0	31.0	9.0	27.0	11.0	19.0	8.0	9.0	21.0	2.58	35.0	0.0007
6	1.5	12.0	29.0	28.0	31.0	10.0	19.0	9.0	11.0	20.6	2.67	33.0	0.0005

PL: Plastic limit, LL: Liquid limit, PI: Plasticity index, W_c: Water content; γ: Unit weight, Gs: Specific gravity; q_u: Unconfined compressive strength; m_v; Coefficient of volume compressibility.

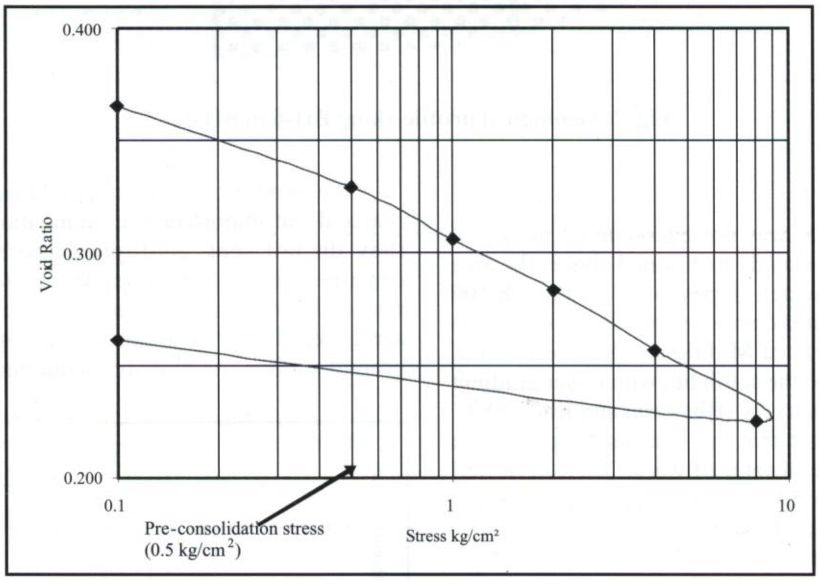

Fig. 5: Result of consolidation test performed in clayey silt (borehole: 4; depth: 2 m).

the depth of 3.25 m is cohesive (clayey silt) in nature, whereas the soil below the depth of 3.25 m is non-cohesive (sandy gravel). Usually, bearing capacity of cohesive soil is smaller than that of the non-cohesive soil. Consequently, depending upon the structural design of buildings, a foundation may be put either in the upper cohesive soil or in the lower non-cohesive soils. An attempt is made here to establish the value of bearing capacity for each of these soil layers separately. Although the upper layer contains some pieces of gravel, while computing the bearing capacity the soil stratum is assumed to be composed of clayey silt only.

Case 1: Cohesive Soil (clayey silt)

Shear failure criteria

The bearing capacity analysis is carried out using the relationship suggested by IS 6403-1981. The suggested simplified relationship for cohesive soil is:

$$q_d = c*N_c*s_c*d_c*i_c \tag{1}$$

Where,

q_d=Net ultimate bearing capacity based on general shear failure in kgf/cm^2; c= Cohesion in kgf/cm^2; N_c=Bearing capacity factor; s_c=Shape factor; d_c=Depth factor; i_c=inclination factor.

The above factors are quantified using the following relations:

s_c=1+0.2*B/L for strip and rectangular footings and 1.3 for square footings

d_c=1+ 0.2*D_f/B; i_c= (1- α/90)^2

Where,

α = Inclination of the load to the vertical in degrees; B = Width of the footing in cm; D_f = Depth of foundation in cm; L = Length of the footing in cm

Settlement criteria

The result of consolidation test (Fig. 5) suggests that the clayey silt occuring at the site is overconsolidated. The relationship suggested in Tomlinson (1969) is used to compute the net bearing pressure based on allowable settlement. The relationship used is:

$$\delta = m_v \times \Delta p \times H \tag{2}$$

Where,

Δp = Increase in pressure at center of clay layer or of the significant depth; δ = Permissible settlement; m_v = Coefficient of volume compressibility; H = Thickness of compressive layer (significant depth) = 1.5B

Case 2: Non-cohesive soil (sandy gravel)

Shear failure criteria

The bearing capacity analysis is carried out based on Standard Penetration Resistance Value using the relationship suggested by IS 6403-1981. The relationship suggested is:

$$q_d = q(N_q - 1)\, s_q d_q i_q + 0.5\, B\gamma N_\gamma\, s_\gamma d_\gamma i_\gamma\, W' \tag{3}$$

Where,

q_d = Net ultimate bearing capacity; q = Effective surcharge at the base level of the foundation; N_q, N_γ = Bearing capacity factor; s_q, s_γ = Shape factor; d_q, d_γ = Depth factor; i_q, i_γ = Inclination factor; W' = Correction factor for location of water table; B = Width of footing; γ = Unit weight of foundation soil.

Settlement criteria

The settlement analysis is computed based on Standard Penetration Test (SPT) values. The following relation proposed by Burland and Burbidge (1985) is used for the purpose.

$$\delta / B_r = 0.14\, C_S C_I I_C\, (B/B_r)^{0.7} (q'/\sigma_r) \tag{4}$$

Where,

δ: Settlement; I_C: Compressibility index; C_I Depth of influence correction factor; C_S Shape factor; q': Net bearing pressure; B_r: Reference width = 1 ft = 0.3 m = 12 in = 300 mm; σ_r: Reference stress = 2000 lb/ft2 = 100 kPa.

Safe bearing capacity

Using the appropriate equations presented above the foundation analysis was carried out for both types of soils. The results are shown below.

For cohesive soil (clayey silt) at 2.0 m bgl:

Settlement criteria: 160 kN/m^2 (with expected settlement of 75 mm)

Shear failure criteria: 80 kN/m^2 (considering safety factor of 3)

For non-cohesive soil (sandy gravel) at 3.25 m bgl:

Settlement criteria: 230 kN/m^2 (with expected settlement of 25 mm)

Shear failure criteria: 1338 kN/m^2 (considering safety factor of 3).

It is evident from the above-mentioned computations that the bearing capacity of the upper soil horizon (clayey silt) is governed by the shear failure criteria and a foundation is to be designed based on the value of 80 kN/m^2. But in case of lower soil horizon (sandy gravel), the bearing capacity is controlled by settlement criteria and a foundation in this soil may be planned based on the value of 230 kN/m^2.

CONCLUSIONS

Lying in the Bhaber Zone of Terai plain, the study site is composed of 25 m thick alluvium underlain by bed rock belonging to Siwaliks. The sub-surface soil of the site may be simplified into two-layer system composed of gravel-mixed clayey silt, 3.25 m thick, in the upper part followed by sandy gravel, 4.9 m thick, in the lower part for the purpose of bearing capacity and settlement computation. Foundation analysis carried out on the basis of settlement and shear failure criteria reveals that the safe bearing capacity for the upper soil horizon (gravel-mixed clayey silt) and the lower soil horizon (sandy gravel) to be 80 kN/m^2 and 230 kN/m^2, respectively. Since the sandy gravel also contains some isolated patches of clayey silt, the value (230 kN/m^2) should be used with caution.

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