Investigation Of Compressibility Characteristics Of Addis Ababa (Gelan) Subcity Soils For Shallow Foundation Design Input
(Illustration and Comparison using different Building Codes)

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Abstract
Any engineered structures constructed on the soil need to respond safely for static and dynamic loading to insure the bearing capacity and compressibility characteristics of the soil. Unbalanced load of soil leads excessive structural distortion and mass collapse against shear failure and excessive settlement which result, structural damage to building frames, cracks, sticking of doors and windows. The unbalanced super structural load is factored to account several factors among these are soil types on which foundation laid on, foundation shape, and depth and design approach using different codes. This study tried to determine the compressibility characteristics of southern Addis Ababa sub city (Gelan) soils with illustration of different codes and the evaluation of Ethiopian building code compared to other codes.

From disturbed samples dry sieve analysis, moisture content, specific gravity, bulk density, Atterberg limits, unconfined compressive strength; direct shear test and consolidation tests and free swell test were done using undisturbed samples. From the test results, drained cohesive strength was 10.5-21kN/m², angle of internal friction ranges from 15-23º, undrained cohesive strength was 35.7-68Mpa, swelling index/recompression index and coefficient of compressibility range was 0.037-0.071and 0.214-0.286 respectively. It is known that the expansive potential of the soil depends on the swelling pressure fully and it is observed in all the study area that the soils are under medium expansiveness and are grouped under Kaolinite clay type with the values swelling index of 40-78% and swelling pressure 60-89kpa with initial moisture content of 29-39.1%. Using these test results,

Settlement values determined by EBCS-7 are conservative and demands some revision to make it more economical. The study suggests that the method of estimating should need further investigation and relevant revision has been made in EBCS-7.

Keywords: Compressibility, Shallow Foundation, Illustration, EBCS-7, building Codes

1. Introduction
1.1 BACKGROUND OF THE STUDY
1.1.1 General Description of the study area and the problem
Gelan is a sub city found in Addis Ababa with a population of 26753 as shown Figure 1.1. It is located with latitude of 8°52’04”E and a longitude of 37°47’S in Ethiopia, 2200m above mean sea (1). It is a highly flooded area and steeply slopes with a daily temperature of 10-15°C (1). Since it is a village where extensive construction activities are underway, the execution of safe and economical Geotechnical and structural design for structures is essential. In this regard, a proper investigation of the soil and a reliable settlement (compressibility) analysis, in static loading cases have a fundamental importance as this thesis tries to demonstrate.
Soil mechanics involve a combination of applying the principles of mechanics and hydraulicsto engineering problems dealing with soil as an engineering material. In many regions; including arid and semi-arid regions, shallow foundations are usually used above the ground water table where the soil is typically in a state of unsaturated condition. Compressibility characteristics of the foundation soil are the key parameters required in the design of shallow foundations. The response and safety of any engineered structures constructed on the soil due to static and dynamic loading depend on the bearing capacity and compressibility characteristics of the soil. For stable and safe structures founded in the ground, the soil must be capable of carrying the loads from any engineered structure placed upon it without shear failure and excessive settlement beyond the recommended limit. A soil shear failure can result in excessive building structural distortion and even collapse and excessive settlement can result in structural damage to frame elements, nuisances such as sticking doors and windows cracks on finishing layers and excessive wear or equipment failure from misalignment resulting from foundation settlement shear failure and with the resulting settlements being tolerable for that structure.

Shallow foundation design practices particular to the analytical approach/computations of bearing capacity and settlement of soils in Ethiopia reveal the use of previous Geotechnical data and follow mixed approaches from the empirical formulations developed from American and European experience found in different codes of standards without a specific understanding of soils shear resisting capacity and compressibility nature of the specific site. Unless we care, estimation of the bearing capacity and settlement of shallow foundations using the conventional approaches practiced in Ethiopia might lead to uneconomical and unsafe designs which may eventually lead failure. The misconception practiced in using the same approach for different magnitude of design leads to different types and erratic nature of the soils currently showing various failure symptoms. This is typically on shallow foundations used in substructure of buildings in Addis Ababa and the surrounding areas. Some of the damaged buildings are shown in figure 1.2 structures caused by the any combination of factors such as design and construction, but are related to the soil around the study area.

Figure 1.1. Geographical location of the study area

Figure 1.2. Shear of tie beam in Gelan city car parking basement building around the asphalt area
Several literatures indicated that the ultimate bearing capacity and compressibility value for the same engineering property of the soil is not similar. This is basically due to the difference in their conceptual understanding of the parameters and methods they considered.

Any structure built on the ground causes an increase of pressures on the underlying soil layers and contributes for settlement in structures. So, settlement determination plays a considerable role in assessing the safety and architectural aesthetics of any structure [3]. If a structure settles uniformly, no damage will be done as such, but the function of the structure may be affected. A more serious case is non-uniform or differential settlement of the structures, in which part of a foundation or two adjoining footings settle differently. If the effect of the differential settlement is not taken into the design of the structure, the structure may crack very badly and the safety of the structure becomes questionable [3]. The magnitude of the differential settlement that can be accommodated by a structure depends upon the soil type, the type of construction, the type foundation and column spacing.

The soil layers are unable to spread laterally as the surrounding soil strata confines them. Hence the soil reacts by the adjustment to the new pressure by deforming vertically. The compression of the soil mass leads to the decrease in the volume of the mass, which result in the settlement of the structure, built on the mass. The vertical compression of the soil mass under increased pressures is thus made up of the following components [4]: deformation/particle rearrangement of the soil grains, the compression of air within the voids and an escape of water and air from the Voids.

Deformation of the soil grains is the primary feature of elastic settlement also named as distortion settlement or contact settlement and usually taken to occur immediately on the application of the foundation load. Such immediate settlement in the case of partially saturated soils is primarily due to the expulsion of gases and to the elastic compression and rearrangement of particles. In the case of saturated soils immediate settlement is considered to be the result of vertical soil compression, before any change in volume occurs. The other two components of settlement result of the gradual expulsion of water from the voids and the concurrent compression of the soil skeleton are characterized by consolidation settlement. Primary consolidation settlement of soil is the gradual reduction in volume of voids upon loading. In saturated soils (Clays), the reduction of void volume occurs due to dissipation of excess pore-water pressure. The theory for the time rate of one-dimensional consolidation was first
proposed by Terzaghi and Peck for Granular soils \[2\]. The underlying assumptions in the derivation of
the mathematical equation are the following:
a) The soil is homogeneous and isotropic
b) The soil is fully saturated the soil particles and the water in the voids are incompressible. The
consolidation occurs due to expulsion of water from the voids
c) Darcy’s law is valid throughout the consolidation process
d) Soil is laterally confined and the consolidation takes place only in the axial direction.
e) Drainage of water also occurs only in the vertical direction
The assumptions made by Terzaghi are not fully satisfied in actual field conditions. The results
obtained from the use of the theory to practical problem are approximate. However, considering
complexity of the problem, the theory gives reasonably accurate estimate of the time rate of settlement,
the total consolidation settlement and elastic settlement of a structure built on the soil\[5\].

1.1.2 Settlement Analysis by Skempton and Macdonald Approach
From statically analysis Skempton and MacDonald concluded that as long as the angular
distortion \((\delta/l)\) of a building is less than 1/300, there should be no settlement damage \[3\]. This finding
was also substantiated by having established the permissible limits of differential settlement.

Table 1.1. Maximum permissible settlement based on Skempton and MacDonald \[3\]

<table>
<thead>
<tr>
<th>Type of Soil and Footing</th>
<th>Empirical Formula</th>
<th>Total Settlement (mm)</th>
<th>Angular Distortion</th>
</tr>
</thead>
<tbody>
<tr>
<td>For Sand soil</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Isolated Footing</td>
<td>(\frac{d}{l} = \frac{s_{\text{max}}}{600})</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Raft Foundation</td>
<td>(\frac{d}{l} = \frac{s_{\text{max}}}{750})</td>
<td>60 to keep (\leq 1300)</td>
<td></td>
</tr>
<tr>
<td>For Clay soil</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Isolated Footing</td>
<td>(\frac{d}{l} = \frac{s_{\text{max}}}{1000})</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>Raft Foundation</td>
<td>(\frac{\delta}{l} = \frac{s_{\text{max}}}{1250})</td>
<td>90</td>
<td></td>
</tr>
</tbody>
</table>

1.1.3 Presumptive values of Total Settlement
Taking into consideration the recommendation of Skempton and MacDonald, bowels
recommended the total permissible settlement as follows

Table 1.2 Bowels Maximum Permissible Settlement \[3\]

<table>
<thead>
<tr>
<th>Type of Soil and Footing</th>
<th>Total Settlement (mm)</th>
<th>Angular Distortion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Isolated Footing</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Raft Foundation</td>
<td>50-75</td>
<td>to keep (1/1300)</td>
</tr>
<tr>
<td>Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Isolated Footing</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>Raft Foundation</td>
<td>75-125</td>
<td></td>
</tr>
</tbody>
</table>

The Ethiopian Building code of standard for foundation recommends a permissible total
settlement of 50mm and 75mm on sandy and clayey soils, respectively and this is for an angular
distortion not exceeding 1/300.

Table 1.3 Maximum Permissible Settlement according to Indian Standard Based on Structural
Materials\[6\].

<table>
<thead>
<tr>
<th>Type of Footing and Structural Material</th>
<th>Total Settlement (mm)</th>
<th>Angular Distortion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Structure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Isolated Footing</td>
<td>50</td>
<td>to keep (\leq 1300)</td>
</tr>
<tr>
<td>Raft Foundation</td>
<td>75</td>
<td></td>
</tr>
</tbody>
</table>
1.1.4 Immediate Settlement determination

This part occurs immediately after the construction. Based on the soil type considered, this is computed using different formulae.

Immediate Settlement of Cohesion less Soils

Because of the high permeability, the elastic as well as the primary compression effects occur more or less together in the case of cohesion less soils [7]. The method available for predicting this settlement is far from perfect; either the standard penetration test or the use of charts is resorted. The result which is the number of blows required for causing a standard depth of penetration under specified standard conditions can be used to evaluate immediate settlement in cohesion less soil (De Beer and Martens, 1957). Relationship between pressure and settlement of a 305mm square plate, for different values of N, in cohesion less soils is shown Figure (After Thornburn, 1963). Another method has been developed for use with the Dutch cone penetrometer but can also be adapted for the standard penetration test.

\[ s_i = \frac{1.5 \rho_o H \log_e \left( \frac{p_o + \Delta p}{p_o} \right)}{c_i} \]  

(1.1)

Where:
- \( H \) = thickness,
- \( \rho_o \) = over burden pressure
- \( c_i \) = recompressibility index
- \( s_i \) = immediate settlement

![Graph showing relationship between pressure and settlement](image)

Figure 1.5 Relationship between pressure and settlement of a 305mm square plate, for different values of N, in cohesion less soils (After Thornburn, 1963).

Immediate Settlement in Cohesive Soils (Terzaghi)

Using theory of elasticity of the soil in saturated or partially saturated clay, the immediate settlement occurred is the vertical deformation due to the change in shape but virtually no volume change of the soil [7]. The immediate settlement of flexible foundation according to Terzaghi (1943) at the center of the footing is given by.

\[ s_i = \frac{2qBI_v}{E_s} \left(1 - v^2\right) \]  

(1.2)

Where:
Si = immediate settlement  
q = surcharge load  
Is = shape factor for settlement  
If = Depth factor  
v = poisonous ratio

The poison's ratio is defined as an elastic settlement variable which is determined by the ratio of the axial compressive strain to lateral compressive strain using Triaxial test, but for simplicity, it has been taken with the correlated value with a plasticity index of the soil. For undrained clay soils the piousness ratio is 0.5 and for partially saturated cohesive soils Poison's ratio is represented by the following equation.

\[ n = 0.25 + 0.00225 \text{(PI)} \]  

(1.3)

In computing If factor:

\[ I_s = F_1 + \frac{1-2v}{1-v} F_2 F_2 = \frac{n}{2\pi} \tan^{-1} A_2 \quad F_1 = \frac{1}{\pi} (A_o + A_i) \]

\[ A_o = m \ln \left( \frac{1+\sqrt{m^2+1}\sqrt{m^2+n^2}}{m(1+\sqrt{m^2+n^2}+1)} \right) \]

\[ A_i = \ln \left( \frac{m\sqrt{m^2+1}+\sqrt{1+n^2}}{m+\sqrt{m^2+n^2+1}} \right) \]

\[ A_2 = \frac{m}{m+\sqrt{m^2+n^2+1}} \quad m = \frac{L}{B} \quad n = \frac{2H}{B} \quad I_f = f \left( \frac{D_i}{B}, \nu, \frac{L}{B} \right) \]
Generalized Average Elastic Settlement Method

Janbu (1956) proposed a generalized equation for average elastic settlement for uniformity loaded flexible foundation, but this is done for soil with piousness ratio of 0.5 or in the state of saturation of the soil.

\[ s_{\text{(flexible)}} = m_o m_i \frac{qB}{E_s} \]  

(1.4)

Where:
- \( m_o \) = constant
- \( m_i \) = constant
- \( q \) = surcharge load
- \( E_s \) = modulus of elasticity

Figure 1.2. Correction Factor for depth of embedment of foundation [7]
Mayne and Poulos (1999) presented an improved formula for calculating the elastic settlement of foundations. The formula takes into account the rigidity of the foundation, the depth of the embodiment of the foundation, the increase in modulus of elasticity of the soil with depth, and the location of the rigid layer at a limited depth. Even though it is improved method for settlement calculation, there are limitations to involve in this study, the first one is the scope of study to determine sub grade modulus using plate load test, the second is the equation use soil sub grade modulus as a variable for calculation but the target of this study is to get a reasonable value of settlement to reach the appropriate sub grade modulus value.

\[
s_i = \frac{qB_e I_c I_f I_E}{E_o} (1 - \nu^2)
\]  

Where Be is an equivalent width or diameter of the foundation

\[
B_e = \sqrt{\frac{4BL}{\pi}}
\]

1.1.5 Calculation of Consolidation Settlement

Consolidation settlement is calculated using various analytical methods and using Geotechnical software.

Using \( e - \log p \) Plot

Settlement due to consolidation in this case is computed with two methods of the theory of elasticity. The formulae below is given according to the type of consolidation: Saturated clay soil layer
of thickness H, cross-sectional area A, existing overburden pressure Po, increase in pressure $\Delta p$, and resulting ultimate primary consolidation settlement $s_c$.

METHOD I

For Normally Consolidated Soils ($p_c = p_o + \Delta p$)

$$s_c = \frac{CcH}{1+e_o} \log \left( \frac{p_o + \Delta p}{p_o} \right)$$  \hspace{1cm} (1.6)

For Over Consolidated Soils ($p_c > p_o + \Delta p$)

$$s_c = \frac{C_cH}{1+e_o} \log \left( \frac{p_o + \Delta p}{p_o} \right)$$  \hspace{1cm} (1.7)

For Over Consolidated and ($p_c < p_o + \Delta p$)

$$s_c = \frac{C_cH}{1+e_o} \log \left( \frac{p_o}{p_c} \right) + \frac{C_cH}{1+e_o} \log \left( \frac{p_o + \Delta p}{p_c} \right)$$  \hspace{1cm} (1.8)

For evaluation of the increased stress use a general chart of stress distribution beneath rectangular and strip footing shown below[2].

$$\Delta p = q_o I_s$$  \hspace{1cm} (1.9)

The average value of applied stress variation in depth at the center of the footing

$$\Delta p = q_o \left( 1 - \frac{1}{\left( \frac{B}{2z} \right)^2 + 1} \right)$$  \hspace{1cm} (1.10)

Figure 1.7. Stress distribution on the foundation with depth

METHOD II
From the e-log⁰ method of consolidation, analysis Dimitri’s Pitilakis modified the equation only using compression coefficient shown below, but the stress history is the same as above.

For Over Consolidated Soils and \( p_c < p_o + \Delta p \)

\[
S_c = \frac{0.1C \cdot H}{1 + e_o} \log \left( \frac{p_c}{p_o} \right) + \frac{C \cdot H}{1 + e_o} \log \left( \frac{p_o + \Delta p}{p_c} \right)
\]  

(1.11)

For Over Consolidated Soils and \( p_c > p_o + \Delta p \)

\[
S_c = \frac{0.1C \cdot H}{1 + e_o} \log \left( \frac{p_o + \Delta p}{p_o} \right)
\]  

\[
S_c = \frac{0.1C \cdot H}{1 + e_o} \log \left( \frac{p_c}{p_o} \right) + \frac{C \cdot H}{1 + e_o} \log \left( \frac{p_o + \Delta p}{p_c} \right)
\]  

(1.12)

Using \( m_c \) Method

This method of consolidation settlement was analyzed by Skempton and Bjerrum [8]. In this method the value of \( m_c \) even for a particular soil is not constant it mainly depend on the stress range over which it is calculated.

\[
s_c = \frac{\Delta p H_o}{1 + e_o} \left( \frac{e_o - e_i}{p_i - p_o} \right)
\]  

(1.13)

To address the goals of this study, several soil testing and design approach analysis is performed in stages as follows. In the first part several literatures related to bearing capacity and compressibility are reviewed to obtain sufficient theoretical information about the methods. The second part of the study involves the collection and organization of field data, soil sampling and conducting laboratory experimental soil tests. In these stages disturbed and un-undisturbed soil samples are collected, carrying out Atterberg limits, dry sieving, moisture content, specific gravity, bulk density, direct shear test, consolidation test and unconfined compression strength laboratory tests are conducted. This helps to establish a one dimensional Geotechnical design profile for the site. In the third phase, using the Geotechnical design profile, bearing capacity and compressibility of the soils for different geometry and loading category are determined for shallow foundations at different depth using different empirical formulations included in different codes. The fourth phase of the study includes analysis and discussion of the results obtained in different sections from different codes and empirical formulations. Moreover, this study investigates the effect of depth and geometry when using the different procedures. Finally the conclusion and recommendation of the results encompassed.

This paper focus to study the settlement, determination approaches of shallow foundations using the different empirical formulations included in the above codes as this helps to compare the results obtained from EBCS-7 with others. Therefore, it is recommended to undertake research to facilitate the integration into the EBCS codes of the latest developments in scientific and technological knowledge.

2. Methods And Materials

Proper laboratory testing of soils to determine their physical properties is an integral part in the design and construction of structural foundation, the placement and improvement of soil properties, and the specification and quality control of soil compaction works. It needs to be kept in mind that natural soil deposits often exhibit a high degree of non-homogeneity. The fundamental theoretical and empirical equations that are developed in soil mechanics can be properly used in practice if and only if, the physical parameters used in those equations are properly evaluated in the laboratory. So, learning to perform laboratory tests of soils plays an important role in the Geotechnical engineering profession to conduct the necessary researches.

In this study some base laboratory tests were carried out to get the necessary data of helpful to this study. These are Dry sieve analysis, liquid limit, plastic limit, moisture content, specific gravity,
bulk density, direct shear test, unconfined compressive strength and one dimensional consolidation test.

2.1. Test Results

All the required tests and the corresponding data’s recorded are collected from the laboratory tests shown above. For each disturbed sample, 3 tests per pit were conducted according to ASTM standard and for undisturbed samples from sites were taken and extruded with samples of each test type. The test results were recorded for all tests. The average results of each Geotechnical parameters for each area are used for design as shown in Table 2.1. A direct shear test is carried out at four undisturbed samples collected from Gelan sub city where a better variation is expected. Average bulk density was determined. The ground water table is far below the base of the footing i.e. more than its width and has no effect for the design. The horizontal displacement reading is recorded with an appropriate division interval with application of four normal loads and among the force recorded with the corresponding horizontal dial gauge reading, the maximum value was taken as the shear stress for each normal loading.

Table 2.1. Test results of samples

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth (m)</th>
<th>Moisture content (%)</th>
<th>Liquid limit (%)</th>
<th>Plastic limit (%)</th>
<th>Plasticity index PI</th>
<th>Specific gravity</th>
<th>Bulk density Kg/m³</th>
<th>Percent finer than 0.075mm (%)</th>
<th>Free swell index (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH.F</td>
<td>2</td>
<td>29</td>
<td>54</td>
<td>36.2</td>
<td>17.8</td>
<td>2.68</td>
<td>1.785</td>
<td>61.6</td>
<td>40.1</td>
</tr>
<tr>
<td>BH.R</td>
<td>2</td>
<td>36.6</td>
<td>65.5</td>
<td>24.0</td>
<td>27.9</td>
<td>2.64</td>
<td>1.444</td>
<td>56.8</td>
<td>53.5</td>
</tr>
<tr>
<td>BH.M</td>
<td>2</td>
<td>39.1</td>
<td>80</td>
<td>54.3</td>
<td>25.7</td>
<td>2.66</td>
<td>1.504</td>
<td>63.2</td>
<td>78.4</td>
</tr>
<tr>
<td>BH.A</td>
<td>2</td>
<td>34.0</td>
<td>67.3</td>
<td>38.0</td>
<td>29.3</td>
<td>2.65</td>
<td>1.411</td>
<td>58.4</td>
<td>73.8</td>
</tr>
</tbody>
</table>

![Figure 2.1](image)

Figure 2.1. Typical value of Normal Stress versus Shear Stress of Gelan Area soils

Finally the direct shear strength parameters were estimated from the graph indicated in Figure 2. The normal stress versus shear stress of the appropriate straight line slope relating the points is used.
Drained bearing capacity of these collected samples are later used to determine bearing capacity and settlement using methods found in EBCS-7 and other codes discussed in literature review as well as in SOFA software.

The second test result is an unconfined compressive strength test, on which undrained shear strength was determined. This parameter was determined from the graph of the horizontal dial gauge reading versus the vertical dial gauge reading with the placement of the specimen in a compression device of which the maximum inflection point of the vertical dial gauge with a conversion factor 44.7 was taken as the unconfined compressive strength then the undrained shear strength of the samples of the city was determined by taking the half of unconfined compressive strength. Figure 2.2 represents UCS test results.

![Figure 2.2. Typical value of Axial Strain in percent versus compressive strength of sub city soil](image)

Table 2.2. Summary of direct shear and UCS test results

<table>
<thead>
<tr>
<th>Location</th>
<th>$c'$ (kpa)</th>
<th>$\phi'$ (°)</th>
<th>$C_u$ (Mpa)</th>
<th>$\gamma$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH.F</td>
<td>13.64</td>
<td>19</td>
<td>50.04</td>
<td>17.85</td>
</tr>
<tr>
<td>BH.R</td>
<td>10.5</td>
<td>15</td>
<td>51.68</td>
<td>14.44</td>
</tr>
<tr>
<td>BH.M</td>
<td>18.35</td>
<td>21</td>
<td>46.016</td>
<td>15.04</td>
</tr>
<tr>
<td>BH.A</td>
<td>21.4</td>
<td>23</td>
<td>35.72</td>
<td>14.11</td>
</tr>
</tbody>
</table>

A consolidation test is carried out on four undisturbed samples of Gelan sub city. The test results obtained from consolidation test on undisturbed sample of soil of different density and initial moisture content are represented by dial reading vs. square root of time plots graph, this method is selected according to the type of data relation from Arora[6]. From the graph plotted for each load cases the value of $t_{90}$ and the corresponding Dial gauge readings were determined, then the coefficient of consolidations was found but it is not the concern of the study. The graph of voidratio-logpressure were also represented by the final void ratio at 1440min and log of pressure imposed repeatedly loading-unloading-reloading process were carried out. These results were used to determine the range of value of consolidation parameters $C_c$, $C_s$ and $P_c$. The typical representative Figure below illustrates the theory above. For the purpose of estimating the elastic settlement, the modulus of elasticity and piousness ratio were found with simple mathematical relation. Modulus elasticity of the soil was found with a relation of undrained shear strength estimated above. $Es = (500\text{-}1500)$ times of undrained shear strength whereas piousness ratio is taken simply from reliable empirical formulae[9]. Influence factor of the soil is determined from the geometrical relation of selected footing and stratum thickness. Piousness ratio was determined from general soil characteristics using an empirical formula which correlates with a Plasticity Indexes recommended for partially saturated soils.
2.2. SETTLEMENT ANALYSIS DISCUSSION OF RESULTS

Settlement is computed based on different analytical formulations as described in different codes of standards differently. The soil test results obtained are presented in the Geotechnical soil profile and different scenarios of loading and foundation geometry are discussed. Settlement scenario for square, rectangular and strip footings at 2m and 3m of foundation depth are presented and discussed along with the respective analytical approaches used. The load ranges from this study are divided into four, which is collected from construction sites of structural engineers of which one typical value for each case was taken as 700kN, 950kN, 1450kN and 1950kN.

2.3. SETTLEMENT ANALYSIS OF SQUARE FOOTING DEPTH 2M

In the analysis results it is compulsory to show graphically. As shown from the Figure 2.4, the settlement variation in each code with the value obtained with EBCS-7 and the minimum value is not as such significant. The maximum settlement difference between EBCS-7 and the maximum value for each loading case are respectively for DIN, EAK, EURO, Meyerhof, Hanson, and EBCS 7 is 21.89mm, 24.9mm, 29.24mm and 37.27mm. This indicates that the variation of settlement increases up as the loading increases and the difference of the maximum and EBCS-7 computed values have resulted from loading 700kN-2450kN varied from 1.69-2.05mm.

Table 2.3. Summary of consolidation test results and correlated Modulus of Elasticity

<table>
<thead>
<tr>
<th>Location</th>
<th>Swelling Index-(Cs)</th>
<th>Compression Index-(Cc)</th>
<th>Preconsolidation Pressure-(Pc), kPa</th>
<th>Swelling pressure (kPa)</th>
<th>Overburden Pressure-(Po) kPa</th>
<th>Modulus of Elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH.F</td>
<td>0.047</td>
<td>0.115</td>
<td>178</td>
<td>60</td>
<td>35.7</td>
<td>50.04</td>
</tr>
<tr>
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<td>89</td>
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</table>
Settlement Analysis Of Square Footing Depth 3m

It is a common principle that depth and settlement have an indirect relationship. As shown in Figure 2.5, the settlement in the EBBC-7 method of analytical approaches to the minimum value. The maximum difference of settlement occurred between the maximum values and EBBC-7 method of analysis for each loading case reached up to 4.94mm, 3.53mm, 3.32mm and 4.36mm respectively. Unlike bearing capacity the settlement difference is not regularly decreased as the loading increased. The settlement didn’t exceed the maximum limit for all loading cases encountered. The maximum settlement has been usual in the Hansen’s method of analysis.
Settlement Analysis For Rectangular Footing Depth 2m

Here in this case the unique characteristic is observed for rectangular footing compared to square footing that in the rectangular footing the maximum value fitted with the EBCS-7 method of analysis. The other unique characteristics investigated for rectangular footing are for the maximum loading case typically for 2450kN the settlement exceeds its maximum limit state for on typical borehole whereas for the first loading cases the settlements are diminished in a large extent. The maximum, minimum difference calculated for the four leading cases are 1.94mm, 3.1mm, 6.78mm and 11.55mm respectively. It is numerically observed that the difference is increased for loading increment.
Figure 2. 1. Typical settlement values of rectangular footing depth 2m

**Settlement Analysis For Rectangular Footing Depth 3m**

A settlement with the increment of depth has a common attitude of decrement in each corresponding loading case. The settlement values of rectangular footing in the EBCS-7 method of analysis have corresponded with the maximum value in the four loading cases. The maximum difference of settlement found between maximum (EBCS-7) and minimum values are reached up to 0.63mm, 1.06mm, 2mm and 3.3mm respectively of each loading case. Here the difference goes increased with the increment of loading.
Conclusion

The settlement of the foundation considered for this study for square footing depth 2m and 3m in the EBCS-7 method of analysis for loading cases from 700-2450kN ranged 11.83mm-35.2mm and 8.28-24.52mm respectively. Whereas for rectangular footing the settlement is 3-52mm and 4.5-26mm respectively, of depth 2m and 3m. In square footing the settlement in the EBCS-7 method of analysis result the minimum value whereas in the rectangular footing the values are the maximum result.

This study clearly pointed out that in almost all cases the EBCS-7 method result a maximum settlement.

The study suggests that the EBCS-7 method of estimating settlement should be revised in order not to design conservative value.
References