Earth Sciences Malaysia (ESMY)

DOI: http://doi.org/10.26480/esmy.02.2022.136.145

ZIBELINE INTERNATIONAL™ PUBLISHING

ISSN: 2521-5035 (Print)
ISSN: 2521-5043 (Online)
CODEN: ESMACU

CrossMark

RESEARCH ARTICLE

GEOTECHNICAL PARAMETER ASSESSMENT AND BEARING CAPACITY ANALYSIS FOR THE FOUNDATION DESIGN

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ARTICLE DETAILS

Article History:

Received 01 July 2022
Accepted 03 August 2022
Available online 16 August 2022

ABSTRACT

The main objective of this research is to determine the soil appropriateness for the construction of buildings, and it encompasses site investigation, a preliminary process for collecting geological, geotechnical, and other engineering information for safe and economical building design. Site investigation provides insight into unforeseen engineering problems; therefore, instability issues can be forestalled if done thoroughly. Residual soils from the research area comprise many clays, some of which can expand upon moisture increase. Therefore, a site investigation must be carried out to assess the site's suitability for the proposed construction. The research includes nine boreholes and laboratory testing demonstrating the soil profile and bearing capacity within the settlement limit. The site's soil is yellowish-brown, weathered, thickly bedded, loosely cemented, friable sandstone consisting of poorly graded sand (SP) and silt/sand (SP-SM) with clayey layers (ML-CL). Uniaxial compressive strength was recorded at 217 to 1238 kPa under natural and saturated conditions. Furthermore, the computed bearing capacity varies from 2.8 to 6.1 tsf using the Terzaghi approach, 7.1 to 8.0 tsf using Bowel's method, and 4.7 to 5.4 tsf using the Meyerhof method. The coefficient of subgrade reaction for an isolated and raft foundation based on Bowels bearing capacity varies between 24.8 to 26.1 MN/m^3 to 13.6 to 15.4 MN/m^3, respectively. Based on the investigation and lab testing, a raft foundation would be appropriate for the structure. The proposed construction location didn't find any significant geological defects; thus, it's suitable for the construction of buildings. However, the paper's recommendations must be implemented.

KEYWORDS

Geotechnical investigation, Subsurface soil profile, SPT, UCS, Bearing capacity.

1. INTRODUCTION

Geotechnical engineering perspectives integrate uncertainty and variability into estimating engineering parameters characterizing the stiffness and strength features of in situ soil and the safety criteria needed to evaluate the safe operation of infrastructures (Phoon et al., 2019). Geotechnical investigations are employed to learn more about the physical characteristics of soil utilized in foundations for planned projects and to repair damage to earthworks and structures brought on by subsurface conditions (Xiao et al., 2022). Geotechnical engineers play an essential role in the construction sector, as geotechnical engineers' suggested site establishes the groundwater, geological, and engineering conditions (Stirling and Nadimi, 2022). Site investigations are often performed before construction, although in other situations, they are performed to check the safety of an existing building or to determine where collapse has happened (Smith and Smith, 1998). High-rise structures have grown increasingly common in many major cities due to population growth and land shortages (Smith et al., 1991). This phenomenon has compelled designers to create an appropriate foundation system to meet the safety and economic requirements for every ground condition and human life (Salehi and Burgueño, 2018).

The Fazaia housing scheme study area is located in the western part of Karachi near Taiser town to construct residential buildings. The heterogeneous nature of the soils in this area is the primary cause of the region's issues since it causes the soils to react differently depending on the particular environmental conditions (Zhang et al., 2022). If the soil in the study area were homogeneous and isotropic, differential settling would not be a problem for buildings constructed on it. On the other hand, homogeneous and isotropic soil is purely imaginative. Soil and rock are often heterogeneous and anisotropic materials until reduced to a fundamental unit where homogeneity and isotropy may be assured. Engineering structures built on sand and clay may suffer the consequences of differential settlement. Therefore, consolidation settlement is the problem that needs to be sorted out on priority (Ding et al., 2022).

Moreover, shear strength tests must be carried out at loads far greater than the total load of the proposed structure (Akbari et al., 2021). It is also crucial that the worst-case scenario for particular soil is determined. When undertaking a shear-box test or a triaxial test, the total load at failure is recorded and used to check whether the entire load of the proposed structure does not exceed that leads to failure. If the entire load at failure recorded during the shear strength tests is less than the total load of the

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Cite the Article: Khaleel Hussain, Dou Bin, Ali Asghar, Javid Hussain, Sayed Muhammad Iqbal, Syed Yasir Ali Shah, Sartaj Hussain (2022). Geotechnical Parameter Assessment and Bearing Capacity Analysis for the Foundation Design. *Earth Science Malaysia, 6(2): 136-145.*

proposed structure, then the structure will not stand. Compaction may be a good option if the soil has a considerably low shear strength. Before compaction, laboratory studies must be performed to estimate the soil's maximum dry density.

A foundation's bearing capacity specifies the overall load that the foundation can withstand without failing within acceptable settlement limitations. The load-bearing capacity of a foundation is determined by its geotechnical and geometrical properties. Geotechnical properties include soil shear strength and deformation factors. Everything must be done following the laboratory test findings before construction can commence. Index, as well as mechanical properties of the soil in the study area, are determined to foresee unfavorable ground conditions that may potentially threaten the proposed construction. Therefore, it is paramount that site investigation is performed before construction. Groundwater level fluctuates up and down, affecting pore water pressure in both consolidated and unconsolidated material, subsequently affecting the effective stress and, ultimately, the shear strength of the soil (Wang and Manga, 2021).

This study highlights the necessity of undertaking a comprehensive geotechnical investigation and using modern in-situ and laboratory testing to analyze geotechnical characteristics, specifically for designing tall building foundations. The several steps of geotechnical evaluation and the approaches available for calculating the geotechnical parameters are covered in detail.

2. Study Area

The Fazaia Housing Scheme is adjacent to the superhighway and the Northern Bypass. Fazaia housing scheme is located in the western part of Karachi near Taiser town. The Fazaia Housing Scheme development is just 20 kilometers from the Jinnah International airport, 35 kilometers from the Civic center, and about 40 kilometers away from the Dream World resort.

3. Geological Setting of the Investigation Area

Karachi and its surrounding land are exposed to middle and upper tertiary rocks and lower tertiary rocks, shale, clay, sandstone, and limestone, among other sedimentary rocks. The three primary formations in this study area are composed of the Nari formation (Oligocene age), the Gaj Formation (Miocene to Pliocene age), and the Manchar Formation (Plio-Pleistocene age), and the recent alluvium Quaternary deposits as shown is (Figure 2) (Kazmi, 1984).

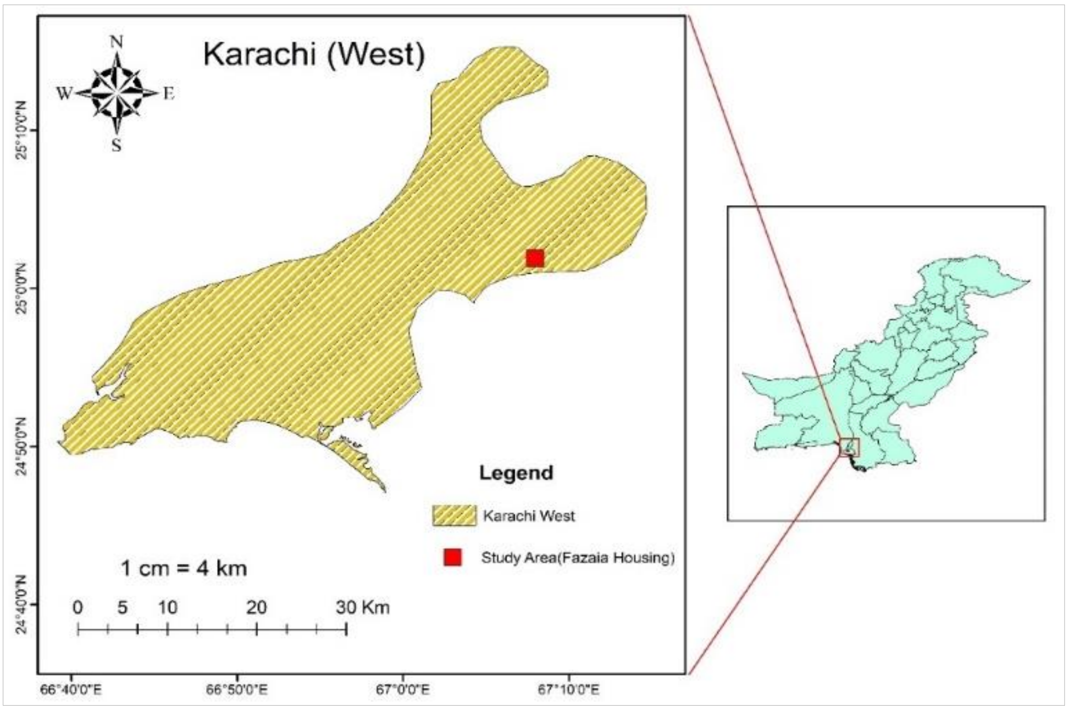

Figure 1: Location map of the Fazaia Housing Scheme Sector-A Zone-11, Karachi, Pakistan.

3.1 Nari Formation

This formation is mainly marine, although it represents the start of a massive inflow of sandstone and shale, causing the sea to withdraw to the southwest. The Nari formation primarily consists of soft sandstone and shale bedded with small proportions of limestone, siltstone, and conglomerate. The limestone is found mainly in the underlying layers, not in all areas of this formation (Shah, 1977).

3.2 Gaj Formation

Most of the rocks in the Gaj formation are composed of limestone, with small portions of sandstone, clay, and gravel serving as subordinates. Gravels are deposited on various deteriorated bedrocks, including shales, sandstones, and limestones. In contrast to the bedrocks, the gravel deposits indicate the lateral planted depositional surface formed all over the eroded bedrocks by a free-flowing and ever-shifting river system. The limestone in the Gaj formation ranges in color from light brown to reddish-brown, while the sandstone is greyish and brown, while clay can be found in various colors (Kazmi and Jan, 1997).

3.3 Manchar Formation

This formation comprises mainly sandstone and shale, with minor quantities of the conglomerate in the upper portion. The bottom part is dominated by sandstone, while the higher half is dominated by shale, which rises closer to the shore. Sandstone is a grittier, crumblier, and more softly bedded rock. Pleistocene conglomerates can be seen in an unconformable state on the emerging surface of the Manchar formation (Kazmi, 1979).

3.4 Quaternary Deposits

Quaternary sediments are mainly comprised of sand, silty sand, sandy silt, and recent deltaic, shoreline, and eruptive mud deposits with small clay contents, most likely due to coastal geographical control and dominance of coastal layers from the shore. It has a high degree of tectonic stability. It contributes to forming the platform cover in the Indus basin and filling the valley in the intermountain bay (Maldonado et al., 2011).

4.1 Materials and Methods

4.1 Site Investigation

In this study, an extensive geological and geotechnical examination of the project area was carried out to determine the subsurface stratification and groundwater conditions and the geotechnical features of the soil. The geotechnical investigations have consisted primarily of field explorations and surveys, in-situ testing, laboratory testing, desk studies, data processing, and data analysis.

4.1.1 Drilling

Vertical borings (Figure 3) were conducted on-site to ascertain geotechnical and hydrological conditions to collect soil and water samples for laboratory examination. Nine (09) boreholes (BH-01 to BH-09) were drilled at well-spaced locations along the intended tower construction alignment to a maximum depth of 25 m to 30 m. Disturbed soil samples were gathered during the boring process, and standard penetration tests (SPT) were performed. Disturbed and undisturbed soil samples were packed and transported under established protocols. The groundwater table (GWT) was not encountered in all boreholes up to the depth of exploration.

4.1.2 Standard Penetration Test (SPT)

The standard penetration test (SPT) is a commonly used in situ dynamic testing technique for determining the geotechnical engineering parameters of subsurface soils. It is performed under controlled conditions (Standard, 1990). The SPT is a simple and affordable technique that estimates soil relative density and shear strength parameters (Clayton, 1995). The findings of this test can be used to estimate the density, bearing capacity, and settling of granular soil (Hussain et al., 2022). The statistics may also be utilized to determine the estimated strength of cohesive soil based on their correlation (Bazaraa, 1967). The test is conducted in boreholes using a thick-walled sample tube with an exterior diameter of 50.8 mm, an internal diameter of 35 mm, and a length of about 650 mm. The sampler is penetrated 150 mm in soil by a 63.5 kg hammer repeatedly dropped from 750 mm height. The procedure is repeated twice, and the number of blows required until the tube reaches a depth of 450 mm is recorded. The SPT blow count value, commonly termed "Standard Penetration Resistance" or the N-value, is the number of blows necessary for second and third 150 mm penetration. The N-value indicates the relative density of subsurface soil used in statistical geotechnical correlations to determine soil's approximate shear strength parameters (Clayton, 1995).

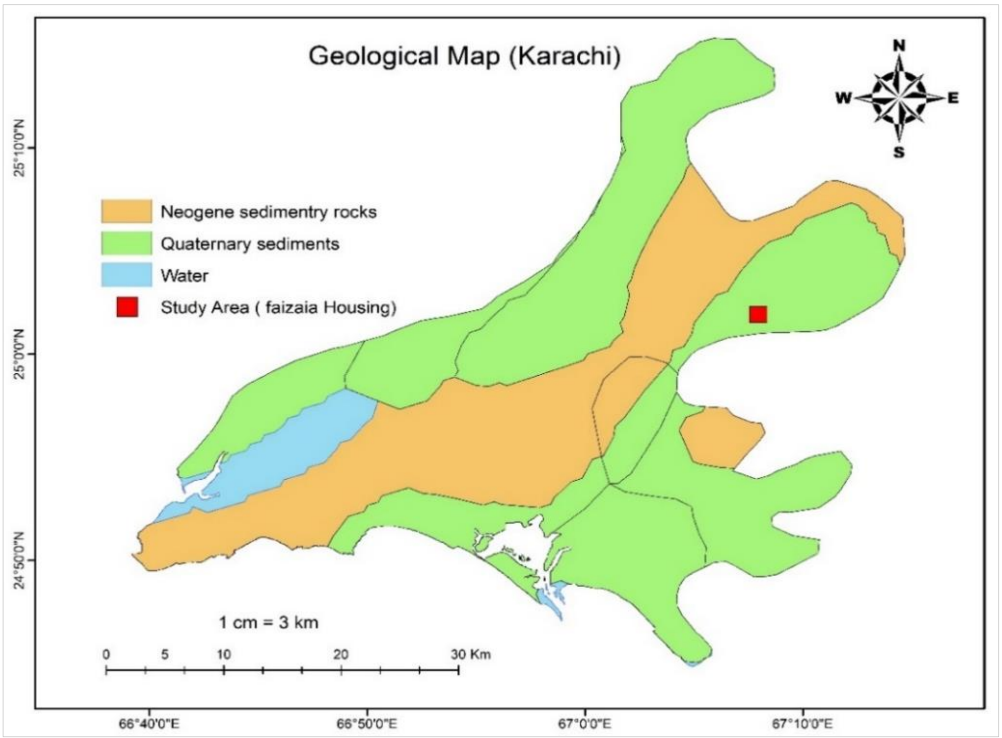

Figure 2: Geological map of the Fazaia Housing Scheme Sector-A Zone-11, Karachi, Pakistan.

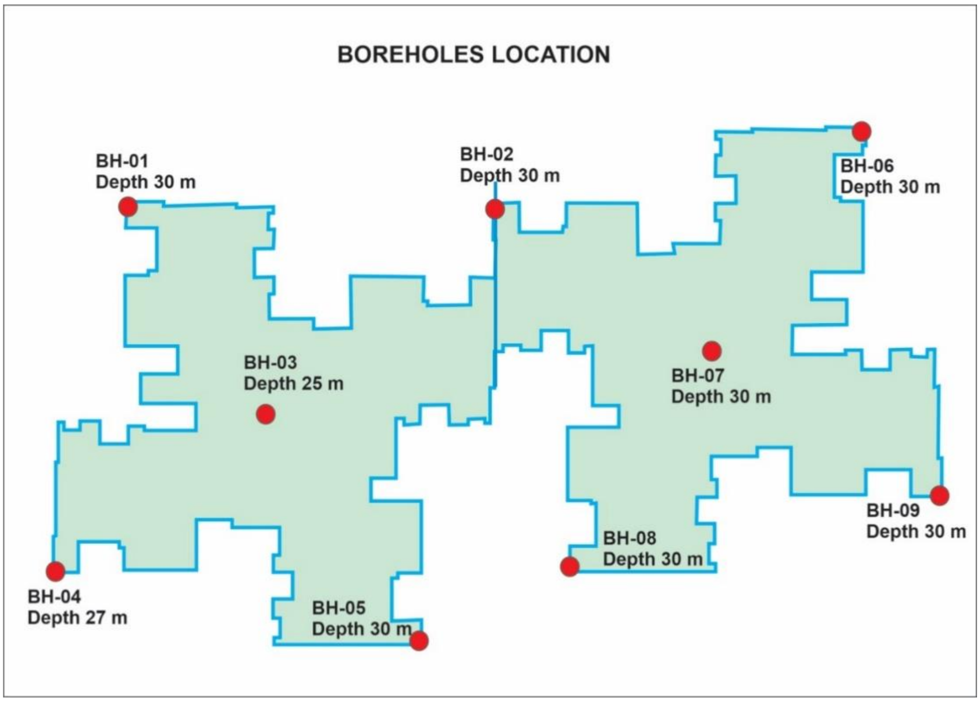

Figure 3: Locations of boreholes in Zone-11 Sector-A.

4.2 Laboratory Assessment of the Soil

Different soil types physical properties were obtained from in-situ and laboratory studies. The laboratory tests that were done on core samples following geotechnical standards include:

- Particle size analysis (PSA) (Astm 2005).
- Natural moisture content (NMC) (Standard 2005).
- Density (ASTM 2009).
- Specific gravity (AASHTO 85AD).
- Atterberg limits (Astm 2003).
- Direct shear test (ASTM 2007)
- Unconfined compressive strength (UCS) (ASTM 2005).
- Elastic modulus ((D638) 2003)
- Poisson's Ratio (ASTM 1998)
- Compression Index (ASTM 2003).
- Swell Potential (ASTM n.d.)
- Free Swell Index (Standard 2014)
- Collapse Potential (ASTM n.d.).
- Chemical testing (ASTM 2001)

These tests were conducted to determine the uniformity, gradation, strength, and settlement parameters of the area's subsoils, which would affect design considerations and foundation selection.

5. Results and Discussion

5.1 Soil Characterization of Zone-11, Sector-A

The soil on the site has mainly consisted of yellowish-brown, highly weathered & fractured, very thickly bedded, loosely cemented, friable sandstone, as shown in (Figure 4). Thin layers of poorly graded sand (SP) and poorly graded sand/silt (SP-SM) with occasional beds of silty clay layers (ML-CL) sandwiched. Most subsurface soils consisted of non-plastic soil except for periodic clayey layers having little effect on the overall geology of the site. The samples were fragile and easily disturbed during the boring and sampling process. Some soil samples collapsed when submerged for saturation due to the loose cementation of sand grains. The so-called sandstone cores crumble into sand grains under a mallet hammer blow rather than breaking into stone chips; therefore, they were classified as SP, SM, and SP- SM.

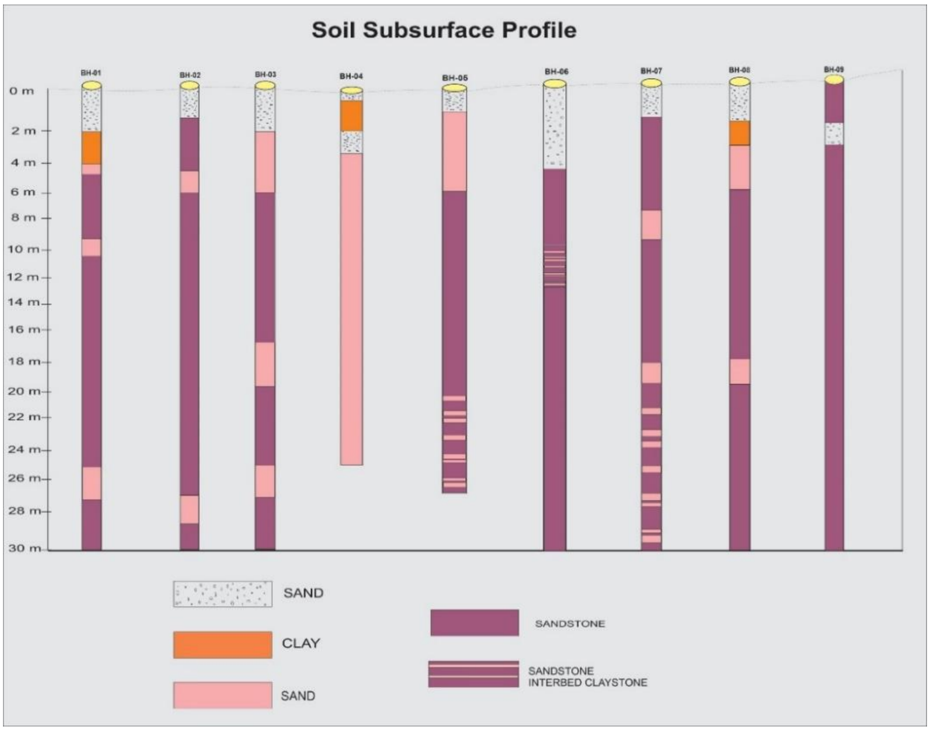

Figure 4: Subsurface soil profile, Pak Fazaia, Sector-A, Zone-11, Karachi, Pakistan.

5.2 Standard Penetration Test (SPT)

The SPT results are shown in (Table 1). It is observed that the value of the SPT blow count varies with drilling depth. It was also found that each borehole had different N-values. The SPT results revealed that the N-values were high above 40, and a refusal was seen with depth where N-values were reached 50. The SPT results indicate that the site's soil (sand/clay) is very dense/hard.

Table 1: SPT Results with Description Conducted in the Study Area.

S. No.	Depth (m)	No of Blows	USCS Symbol	Description
BH-01	1.5 -1.72, 3.0-3.07, 10.5-10.52	48, 50, 40	SP-SM, ML-CL, SP-SM	Very dense fine-grained sand, hard silty clay
BH-02	1.5-2.0	50	SP	Very dense fine-grained sand
BH-03	1.5-1.95, 3.0-3.12, 4.5-4.60	50, 50, 50	SP-SM, CL, ML-CL	Hard silty clay, dense fine-grained sand
BH-04	1.5-1.95, 30-3.05, 7.5-7.57, 25-25.07	50, 50, 50, 50	ML-CL, CL, SM, SP-SM	Hard silty clay, dense sand
BH-05	1.5-1.55, 3.0-3.12	50, 50	SM, ML-CL	Very dense fine-grained sand
BH-06	1.5-1.57, 3.0-3.05, 4.5-4.60	50, 49, 50	SM, SM, SM	Very dense fine to coarse grained sand
BH-07	1.5-1.55,	50	SP-SM	Very dense fine-grained sand
BH-08	1.5-1.95, 3.0-3.12, 4.5-4.60	50, 50, 50	SP-SM, CL, ML-CL	Dense fine grained silty sand, hard silty clay
BH-09	1.5-1.72, 3.0-3.03	49, 50		Very dense fine to coarse-grained sand

5.3 Physical Tests

5.3.1 Particle Size Analysis (PSA)

A sieve analysis or gradation test determines the distribution of aggregate particles by size within a particular soil sample. The uniformity coefficient (Cu) and curvature (Cc) coefficient will be determined to identify whether the soil sample is poorly, gapped, or well graded. A well-graded soil has a higher Cu value (more than 3), while a poorly graded soil has a low Cu value (less than 3). The results show that Cu ranges from 1.05 to 6.61, Cc ranges from 1.29 to 292, and the mean values range from 0.016 to 1.43. These values indicate that the soil at the site is poorly graded. The soil in the area mainly consists of fine-grained sand with little gravel at places (SP-SM) and silty clays (ML-CL).

5.3.2 Natural Moisture Content (NMC)

The NMC determinations for typical soil samples vary from 11.1% to a maximum of 14.3%. These results indicate that the region's soil water content is somewhat lower in its natural condition. It is essential to determine the soil's in-situ moisture level before using it for construction work. NMC enables the in-situ conditions of the soil samples to be determined and the quantity of water to be added or the extent of drying necessary to get the soil samples to their maximum dry unit weight (Haines, 1923).

5.3.3 Density

The saturated unit weight (γ_{sat}) of selected soil samples ranges from 18.0 kN/m^3 to 24.2 kN/m^3, respectively. The bulk unit weight (γb) values range from 15.4 kN/m^3 to 18.6 kN/m^3. On the other hand, dry unit weight (γ_d) values vary from 14.3 kN/m^3 to 18.8 kN/m^3, respectively. Higher soil dry density is frequently related to higher strength, lower permeability, and better volume stability (Hussain et al., 2022). Higher soil densities are always considered best for structural support (Bell, 1993). The density result combined with SPT indicates dense soil (Terzaghi et al., 1996).

5.3.4 Specific Gravity

Specific gravity results show that the values vary between 2.60 and 2.70. The greater the specific value of soil, the greater its appropriateness. Soil with a specific gravity higher than 2.55 is suitable for large-scale building projects (Hussain, 2022). Specific gravity measurements may determine whether the soil at the location is stable enough to sustain a building and allow for appropriate drainage (Horpibulsuk et al., 2012).

5.3.5 Atterberg Limits

The research area's consistency was assessed by determining liquid and plastic limits and plasticity indices. The Atterberg limits results show that the liquid limit ranges from 20.11% to 39.89%, plastic limits vary between 15.68% to 33.73%, respectively, and the plasticity index ranges from 2.54% to 14.9%. Soil samples in the study area show a low to medium plasticity range. Soil with higher plasticity remolds and shows related swelling potential (Chen and Baladi, 1985).

5.4 Mechanical Tests

5.4.1 Direct Shear Test

The strength parameters, cohesion (C), and angle of internal friction (ϕ) of soil were studied by direct shear test. The recorded cohesion (C) values vary between 2.80 kN/m^2 to 5.10 kN/m^2. Similarly, the angle of internal friction (ϕ) values ranges from 30.0 °C to 32.7 °C. Soil has a low to high shearing resistance and loads sustainability at different depths depending on soil density (Abu-Farsakh et al., 2007).

5.4.2 Unconfined Compressive Strength (UCS)

The compressive strength parameters of the undisturbed soil samples were obtained through the UCS test. The soil samples natural UCS values range from 217 kPa to 1238 kPa. Similarly, saturated samples' UCS values varied between 0 kPa to 972 kPa. The results show that the UCS of soil samples is relatively low because the core samples collected were highly brittle. The stability of structures against loads can be evaluated by UCS (Shah et al., 2022).

5.4.3 Elastic Modulus

Elastic parameters of the selected soil samples were obtained through the soil's stress (σ) and strain (ε) behavior. The results indicated that the values range from 6.50 MPa to 26.8 MPa. Elastic parameter of different soil depends on their texture and grain size (0.5 MPa to 320 MPa for organic lays and well-graded sands) (Kézdi and Réthái, 1974; Obrzud, 2010).

5.4.4 Poisson's Ratio

Deformation properties of the study's area soil samples were studied through Poisson's ratio (μ). The minimum and maximum values range from 0.300 to 0.380. Soil with a Poisson ratio of 0.1–0.25 suggests that rocks fracture easily, while a high Poisson ratio, such as 0.35–0.45, indicates rocks fracture more difficultly (Gercek, 2007).

5.4.5 Compression Index

Deformation parameters of the study area were studied by an Oedometer test carried out by applying various loads to soil samples. Based on the findings of these experiments, it is possible to determine how a soil would deform in the field in response to changes in applied stress (Alzabeebee et al., 2021). The results show soil samples' low compression index (Cc), ranging from 0.04 to 0.30.

5.4.6 Swell Potential

Swell potential refers to how soil shrinks or expands in response to variations in soil moisture content. The swell potential of the study area ranges from 6.4% to 9.4%. The soil's relatively low swell potential will not cause settlement or subsidence (Schafer and Singer, 1976).

5.4.7 Free Swell Index

Free swell tests are often used to detect expansive clays and estimate the possibility of swelling. The free swell results of the tested soil samples indicated a variation of 9.32% to 13.0%. As a result, most of the soil samples under analysis are non-expansive. Soils with a free swell index of less than 50% are considered to have a lower degree of expansion (Rao et al., 2004).

5.4.8 Collapse Potential

The collapse potential is related to the deterioration of cementing within soil particles whenever the soil is subjected to moist and loaded conditions (Pells et al., 1975). The laboratory results of soil samples range from 0.001% to 2.10%, indicating a low to moderate degree of severity of collapse potential.

5.4.9 Chemical Tests of Soil

Chemical testing on soil is required due to the negative impact of sulfates and chlorides on the strength of concrete (Cuisinier et al., 2011). These tests aid in assessing the concrete's predicted exposure to these contaminants and developing preventive measures to protect the concrete's durability (Ann et al., 2009). The chemical properties of soil were determined through chloride, sulfate content, and pH values. The results are summarized in (Table 2). Tests on soil samples obtained from the boreholes indicate 'negligible' exposure to sulfate and chloride (Hussain et al., 2022).

Table 2: Chemical properties of the Soil

BH No.	pH	Sulfate content (%)	Chloride content (%)
BH-No.01 to BH-No. 09	6.5-7.5	0.01-0.06	0.2-0.3

6. Bearing Capacity

Bearing capacity refers to the soil's ability to withstand the pressures of objects applied to the ground's surface above. The bearing capacities for isolated foundations were determined by applying the Terzaghi method based on shear criteria, Meyerhof Bearing capacity based on limit settlement criteria using SPT values, and Bowles method, and Teng's approach (Terzaghi n.d.; Meyerhof, 1974; Bowles, 1988; Teng, 1962). The bearing capacity for isolated foundations may be considered based on shear failure criteria applying the Terzaghi method. In contrast, the bearing capacity for raft foundations may be based on the limiting settlement criteria while considering Meyerhof and Teng's conservative design methods and Bowel's general design considerations. The tolerable settlement limit may be considered as 25-50 mm for isolated and raft foundations, respectively. The results are summarized in (Table 3).

6.1 Terzaghi Method

The Terzaghi Bearing Capacity for Zone-11 Sector-A ranges from 2.8 tsf to 6.1 tsf for isolated foundations, as shown in (Figure 5).

Table 3: Bearing Capacity and Coefficient of Subgrade Reaction for Isolated and Raft Foundations.

Location	Bearing Capacity Isolated Foundation (Terzaghi) (tsf)	Bearing Capacity Raft Foundation (Bowels) (tsf)	Bearing Capacity Raft Foundation (Meyerhof) (tsf)	Coefficient of Subgrade Reaction Isolated Foundation (tsf)	Coefficient of Subgrade Reaction Raft Foundation (tsf)
BH-49 Z-11 Sector-A	4.6	7.1	4.7	24.8	13.6
Bh-50 Z-11 Sector A	6.1	8.0	5.4	24.8	15.4
BH-51 Z-11 Sector-A	4.2	7.1	4.7	24.8	13.6
BH-52 Z-11 Sector-A	4.6	7.5	5.0	26.1	14.3
BH-53 Z-11 Sector-A	3.9	7.1	4.7	24.8	13.6
BH-54 Z-11 Sector-A	2.8	7.5	4.7	26.1	14.3
BH-55 Z-11 Sector-A	4.6	7.5	5.0	26.1	14.3
BH-56 Z-11 Sector-A	4.2	7.1	4.7	24.8	13.6
BH-57 Z-11 Sector-A	6.1	7.1	4.7	24.8	13.6

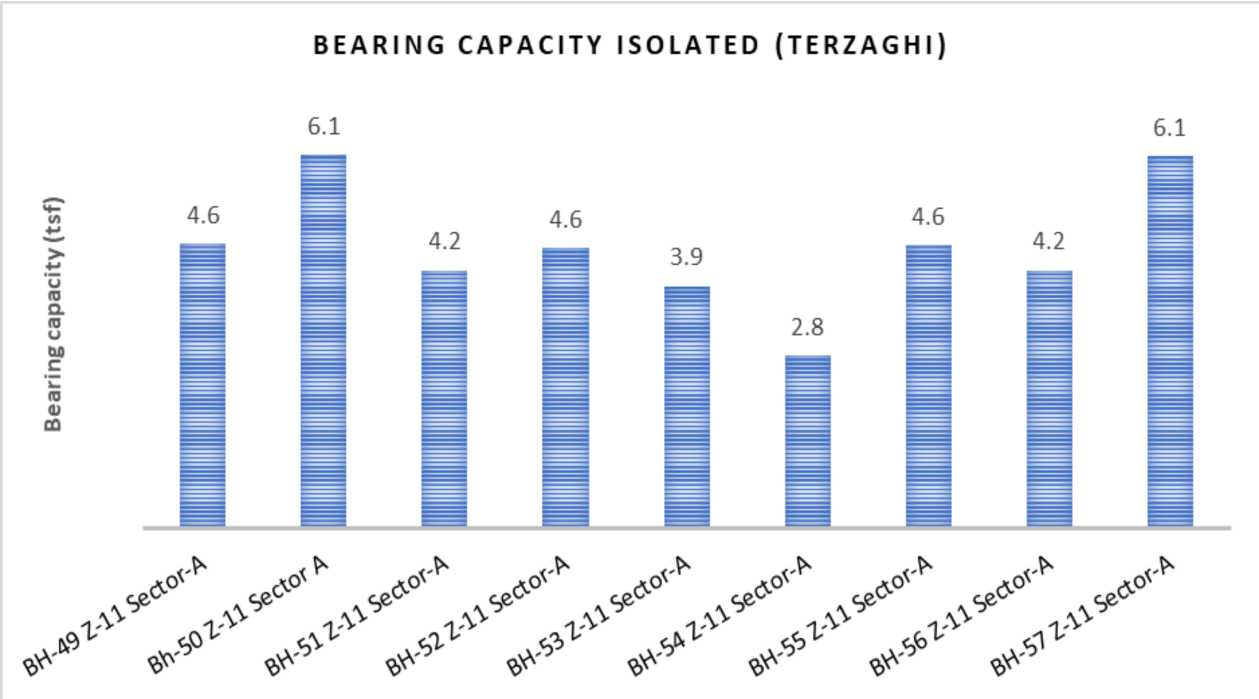

Figure 5: Terzaghi bearing Capacity based on shear criteria-Zone-11.

6.2 Bowel's Method

The Bowel's Bearing Capacity (Figure 6) for Zone-11 Sector-A ranges from 7.1 tsf to 8.0 tsf for Raft foundations based on Bowel's method.

6.3 Meyerhof's Method

The Meyerhof's Bearing Capacity (Figure 7) for Zone-11 Sector-A ranges from 4.7 tsf to 5.4 tsf for Raft foundations based on Meyerhof's conservative approach for raft foundation design.

6.4 Coefficient of Subgrade Reaction

The coefficient of subgrade reaction (K_s) may be defined as the load-to-horizontal-surface area ratio of a soil mass divided by the equivalent surface settlement. The coefficient of subgrade response is a parameter that applies to the contact between soil and foundation. This parameter is influenced by the rigidity of the ground, the foundation's rigidity, and the foundation's size. Terzaghi, Winkler, and Horvath stated that the soil stiffness, characterized as the proportion of contact pressures to related vertical displacement, is linear and can be calculated using the coefficient of subgrade response (Horvath, 1983). The exact value of K_s can be determined through Plate Load Test (PLT) (Naeini, 2014).

$$K_s = \frac{\Delta\sigma}{\Delta\delta}$$

K_s= Coefficient of subgrade reaction, q = contact pressure/bearing capacity, δ = Settlement

6.5 Coefficient of Subgrade Reaction (Isolated Foundation)

For an isolated foundation with a limiting settlement of 25 mm, the calculated coefficient of subgrade reaction varies from 24.8 MN/m^3 to 26.1 MN/m^3, as shown in (Figure 8).

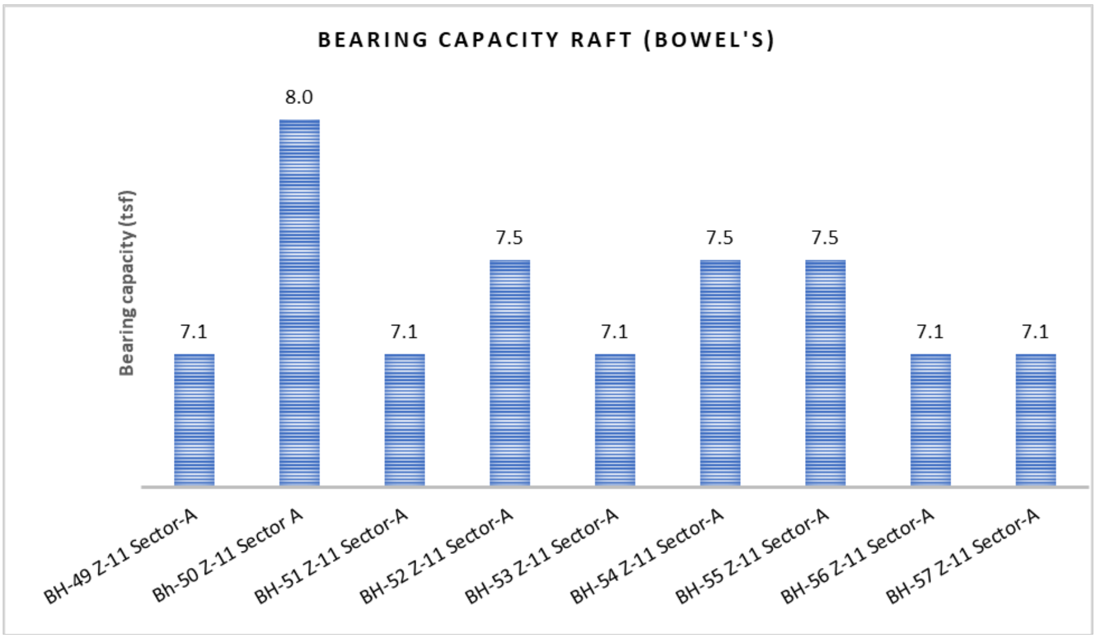

Figure 6: Bowel's bearing Capacity based on limiting settlement-Zone-11.

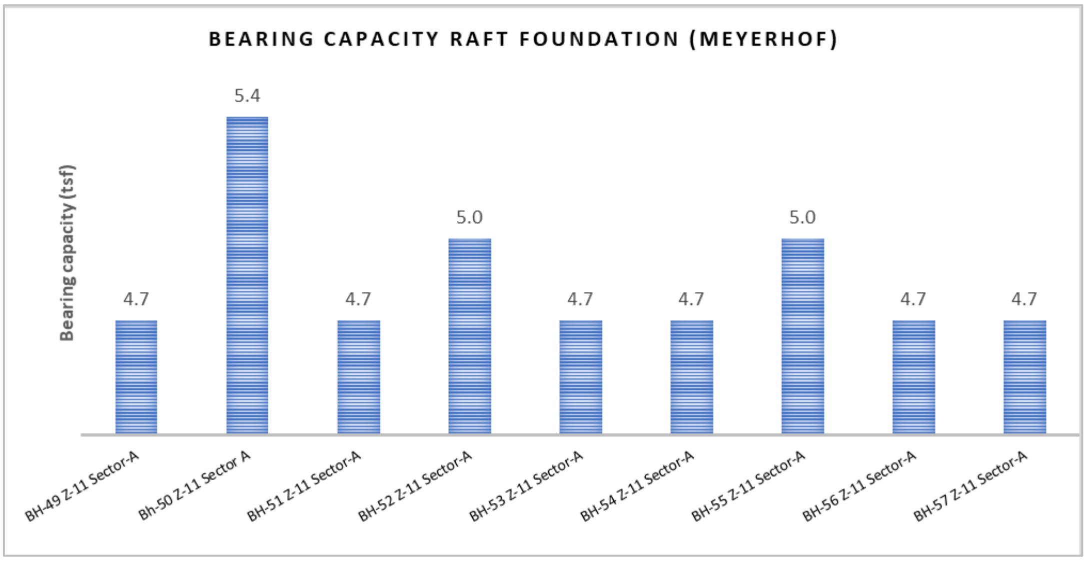

Figure 7: Meyerhof bearing capacity based on limiting settlement-Zone-11.

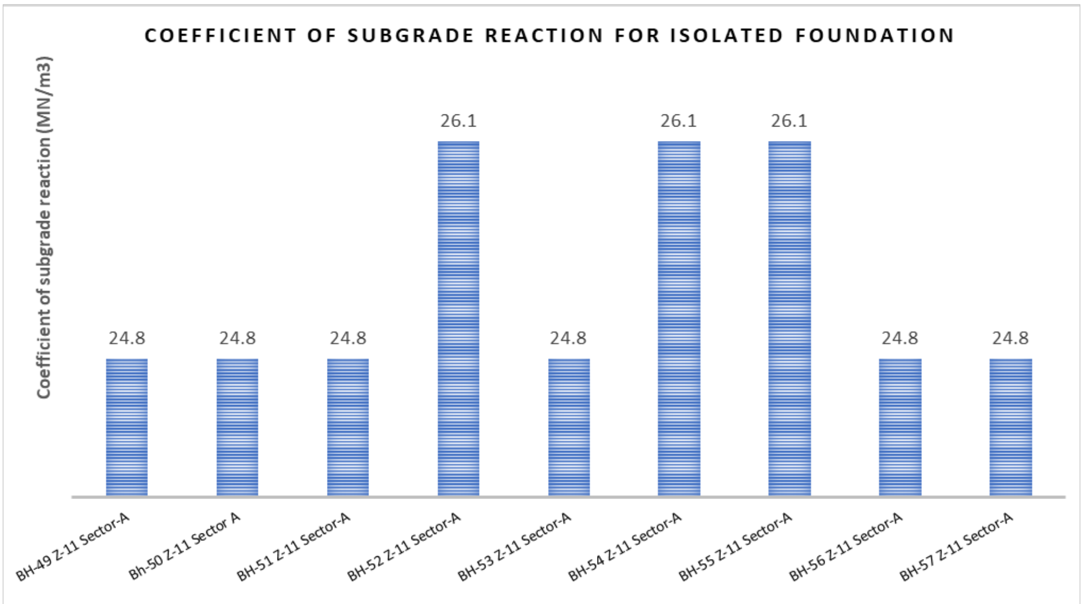

Figure 8: Estimated Ks based on Bowel's bearing capacity (25 mm settlement) -Zone-11.

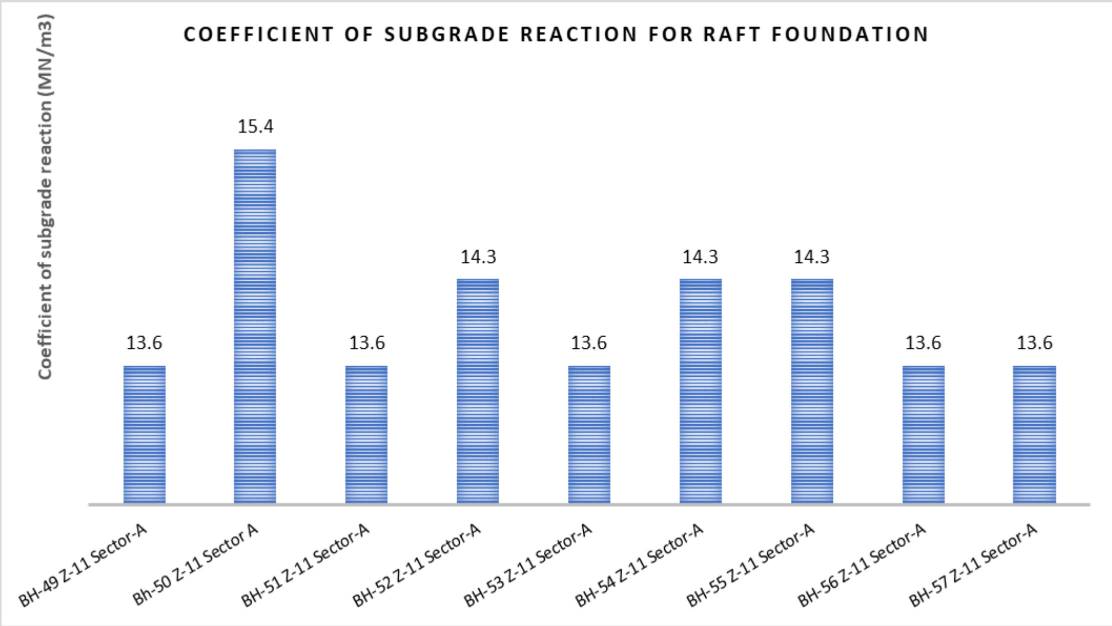

Figure 9: Estimated Ks based on Bowel's bearing capacity (50 mm settlement) -Zone-11.

6.6 Coefficient of Subgrade Reaction (Raft Foundation)

For an isolated foundation with a limiting settlement of 25 mm, the calculated coefficient of subgrade reaction varies from 13.6 MN/m³ to 15.4 MN/m³, as shown in (Figure 9).

7. Conclusion

- The soil stratigraphy of Fazaia Housing Scheme Sector-A Zone-11 has been determined through deep soil boring and laboratory results conducted on soil samples.
- The entire soil profiles consisted of yellowish-brown, highly weathered & fractured, very thickly bedded, loosely cemented, friable sandstone consisting of poorly graded sand (SP) and poorly graded sand/silt (SP-SM) with occasional beds of silty clay layers (ML-CL) sandwiched.
- Most subsurface soils consisted of non-plastic soil except for occasional clayey layers having little effect on the overall geology of the site.
- The samples were fragile and easily disturbed during the boring and sampling process.
- Some samples collapsed when submerged for saturation due to the loose cementation of sand grains.
- The so-called sandstone cores crumble into sand grains under a mallet hammer blow rather than breaking into stone chips; therefore, they were classified as SP, SM, and SP- SM.
- The uniaxial compressive strength has been found relatively low.
- The soil consists of weakly cemented sand; therefore, the foundation type may be a raft/mat foundation.
- It may be concluded that the bearing capacity of the raft foundation may be based on limiting settlement criteria rather than the shear failure criteria.
- The depth of the raft foundation may be decided based on the structural load. The weight of excavated soil should be equal to or greater than the estimated structural load.
- The Terzaghi method would be adequate for isolated foundation design based on shear failure criteria.
- For raft foundations, Terzaghi Method overestimates the bearing capacity, and Meyerhof's Methods give conservative values for the bearing capacity of raft foundations. Bowel's method is supposed to be reasonable. However, Teng and Meyerhof's Methods would suit a conservative design approach.
- The estimated coefficient of subgrade reaction was determined through bearing capacity.

Acknowledgments

The authors thank Dr. Altaf Hussain, Wajid Hussain, Sadam Hussain, and Hadi Hussain for their review and valuable suggestions in this article.

Conflicts of Interest

The authors declare no conflict of interest regarding the publication of this article.

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