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Integrated Deterministic, Numerical, Spatial, And Reliability-Based Evaluation Of Bearing Capacity For Shallow Foundation In Lalitpur Metropolitan City

By

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A THESIS

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The thesis titled "Integrated Deterministic, Numerical, Spatial, and Reliability-Based Evaluation of Bearing Capacity for Shallow Foundation in Lalitpur Metropolitan City" prepared and submitted by Mr. Kaushal Koirala in partial fulfilment of the requirements for the degree of Master of Science (M. Sc.) in Geotechnical Engineering has been examined by us and is accepted for the award of M. Sc. in Geotechnical Engineering by Tribhuvan University.

The undersigned certify that they have read, and recommended to the Institute of Engineering for acceptance, a thesis report entitled "Integrated Deterministic, Numerical, Spatial, and Reliability-Based Evaluation of Bearing Capacity for Shallow Foundation in Lalitpur Metropolitan City" submitted by Mr. Kaushal Koirala in partial fulfillment of the requirements for the degree of Master's in Geotechnical Engineering.

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DECLARATION

I hereby declare that this study, titled "**Integrated Deterministic, Numerical, Spatial, and Reliability-Based Evaluation of Bearing Capacity for Shallow Foundation in Lalitpur Metropolitan City**" is based on my original research work. Related works on the topic by other researchers have been duly acknowledged. I owe all the liabilities relating to the accuracy and authenticity of the data and any other information included hereunder.

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ABSTRACT

Designing a foundation requires an understanding of the soil's carrying capacities. Determining the bearing capability of shallow foundations is crucial in the context of Lalitpur Metropolitan City, Nepal, where the geological conditions are diverse. When an integrated approach is taken into consideration, it aids in determining the carrying capacity of weak foundations by taking a different approach to foundation design. The evaluation of shallow foundation carrying capacity in this situation was done using the traditional theories of Terzaghi, Hansen, Vesic, IS codes, and Bowles.

The results indicate significant variability throughout the location in bearing capacity across the study area. The allowable bearing capacity ranges from approximately 65-173 kN/m^2 at 1.5 m depth and increases up to 356 kN/m^2 at 3 m depth, indicating improved soil strength with depth. Numerical results from PLAXIS 3D show good agreement with analytical methods, with variations within ±20%. Reliability analysis reveals that the probability of failure (P_f) varies from 0.0015 to 0.38, while the reliability index (β) ranges from 0.3 to 3, according to footing size and depth. Larger footing sizes and greater embedment depths show improved performance and reduced failure probability. The developed GIS-based zonation maps effectively identify weak and strong zones, providing a practical tool for preliminary foundation design and urban planning. The study demonstrates that integrating deterministic, numerical, spatial, and probabilistic approaches leads to a more realistic and reliable assessment of foundation performance.

Keywords: Bearing Capacity Zonation, Shallow Foundations, GIS, PLAXIS 3D, Reliability Analysis, Monte Carlo Simulation.

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LIST OF ACRONYMS AND ABBREVIATIONS

3D: Three Dimensional

B: Width of Foundation

c: Cohesion

COV: Coefficient of Variance

D_f: Depth of Foundation

E: Young's Modulus

FEM: Finite Element Method

FOS: Factor of Safety

GIS: Geographical Information System

IS: Indian Standard

MRT: Mandatory Rule of Thumb

N: Measured Penetration Number (SPT-N value)

NBC: National Building Code

NC: Normally Consolidated

OCR: Over Consolidation Ratio

P_f: Probability of Failure

q_u: Unconfined compressive strength

SPT: Standard Penetration Test

β: Reliability index

γ: Bulk Unit Weight

γ_{sat}: Saturated Unit Weight

Φ^{-1}: Inverse Standard Normal Distribution

N_{60}: Corrected SPT number for field conditions

Pa: Atmospheric Pressure

ν: Poisson's ratio

ϕ: Angle of Internal Friction

1 INTRODUCTION

1.1 Background of the Study

Any civil engineering construction's operation and design have a connection to the soil's capacity to resist the loads applied to it. Bearing capacity is the highest pressure that the ground can sustain without experiencing shear failure or excessive soil settlement threatening the structural integrity. In geotechnical engineering, the type of foundation to be used and the structural safety depend heavily on the soil's bearing capacity. This is especially critical in a fast-growing urban center like Lalitpur Metropolitan City, which is experiencing a diverse range of construction projects, such as residential houses, commercial centers, and essential infrastructure.

Bearing capacity zonation is one technique of mapping the change in soil bearing capacity in a geographic location. A zonation map, in contrast to using isolated site investigations, will give a detailed impression of where shallow foundations will be most effective and where the ground may need to be improved or alternative methods of foundation used. This is a very useful tool in urban planning, as it minimizes uncertainty, facilitates economical design, and improves structural safety. With a city as big and densely populated and geologically diverse as Lalitpur, the creation of such a map can be an effective guideline in decision-making among engineers, architects, contractors, and policymakers.

Lalitpur is situated in the Kathmandu Valley's lower region. which is filled with the Quaternary deposits of alluvial, fluvial, and lacustrine nature (Katel et al., 1996). In the south, the soils are mostly alluvial and fine-grained and may be accompanied by a low groundwater table, hence the low bearing capacity and vulnerability to settlement (K. C. & Raj Dahal, 2020). This variability poses serious problems to the foundation design, where shallow foundations that have not been properly investigated can be subjected to different settlements, tilting, or even collapse.

Due to the low cost and the ability to support low to medium-rise buildings with their square footing, Shallow foundations are commonly used in Lalitpur. Their performance, however, is very much reliant on the subsoil conditions and groundwater level. Such a minor alteration in the type of soils or water table levels can lead to great disparities in ultimate and safe bearing capacities. This explains why a bearing capacity zonation map is needed that could be used as a guideline for preliminary design and site selection before a thorough geotechnical investigation is conducted.

The research incorporates various methods to do this. Analytical techniques such as Terzaghi's classical bearing capacity theory, Hansen's method with correction factors, Vesic's revisions, and the IS code are theoretical models that can be used to determine ultimate and safe capacities. These techniques are useful, but tend to provide conservative or generalized outcomes that do not necessarily accurately reflect real soil-structure interaction. Thus, the PLAXIS 3D numerical modeling is also included, allowing to model complicated scenarios, including the existence of nonlinear stress-strain behaviors, and the impact of groundwater. This gives a more realistic understanding of settlement and deformation patterns in a loading manner. Additionally, laboratory and field tests, including the engineering and Index properties of soil to examine the realistic result on the actual site, are used to verify the accuracy of the analytical and numerical results.

Besides these techniques, this research will make use of reliability analysis to measure uncertainty on soil parameters and determine the likelihood of failure. The nature of geotechnical properties is that they are variable, and this can greatly lead to safety and performance of shallow foundations. According to the reliability theory, the parameters are random variables, and the safety measures are the reliability index (β) and the probability of failure (P_f) (Baecher & Christian, 2003).By explicitly taking into consideration the varying characteristics of soil and loading circumstances, the probabilistic approach allows for a more realistic assessment of foundation safety. Combining reliability analysis, including deterministic, numerical, and spatial techniques, the study not only gives the projected bearing capacity but also gives an idea of how confident the design is, allowing the determination of areas where further research or ground improvement might be required.

It is anticipated that the outcome of this study will be a bearing capacity zonation map of Lalitpur Metropolitan City with various zones of shallow foundations. This risk-assessment-reliability-enhanced map will be an invaluable tool in the guidance of urban development, making the designs safer, lowering the costs of construction, and making the land use sustainable. Finally, the combination of classical theories, numerical modeling, GIS-survey-based spatial analysis, and reliability evaluation is a holistic approach to the problem of geotechnical conditions of the Lalitpur Metropolitan city, as it can be used to deal with the complex conditions and give more confidence to the decisions made in the foundation design. The study is anticipated to improve the precision, dependability, and usefulness of the geotechnical evaluations of heterogeneous soils by such a thorough examination of the subject area.

1.2 Problem Statement

The ground conditions of the soils in the Lalitpur Metropolitan City are very heterogeneous since the geological formation of the city is located in the paleo-Kathmandu lakebed, which has a high variation in soil bearing capacity. This variability presents a significant problem in designing shallow foundations in a safe and economical way. Designs that rely on deterministic or estimated soil parameters can cause over settlement, tilting, or shear failure, and the engineer is clueless of the uncertainty around soil qualities due to the lack of probabilistic evaluation. In the absence of a detailed zonation map enhanced with reliability-based analysis, engineers will have to use either an overly conservative design that will raise the cost of construction or under-design foundations, which will reduce the structural safety. The uncoordinated and repetitive geotechnical studies in the city contribute to both the costs and time lag of the project. Deterministic techniques give an estimate of the bearing capacity under baseline conditions, numerical modeling is used to provide realistic soil behavior under complex loading conditions, and GIS-based spatial analysis is used to visualize the variations within the metropolitan area. Combining these strategies with reliability-based evaluation, which measures probability of failure (P_f) and reliability index (β), this study will create a risk-based bearing capacity zonation map of Lalitpur Metropolitan City. This kind of map will furnish spatially continuous, scientifically tested information on allowable bearing pressures, quantify uncertainty and risk, and will be a useful instrument to the engineer, planner, and policymaker to promote safe, efficient, and sustainable urban development.

1.3 Objectives

To come up with a detailed, risk-based judgment of shallow foundation bearing capacity in Lalitpur Metropolitan City through a combination of deterministic calculations, numerical modeling, spatial analysis, and reliability-based analysis to provide a zonation map to inform the safe and effective design of foundations. The specific objectives of this thesis address

a) To calculate the soil's ultimate and safe bearing capacity at the borehole sites of choice through classical deterministic techniques.

b) To simulate the soil behavior during foundation loading through numerical modeling (PLAXIS 3D), settlement, stress distribution, and failure mechanisms, and compare with the classical method.

c) To produce a spatially continuous bearing capacity map of the metropolitan area through GIS-based interpolation and zoning of different areas.

d) To measure uncertainty and safety of bearing capacity estimates, through reliability-based analysis, find the probability of failure (P_f) and the reliability index (β).

1.4 Scope of Study

This study uses a combined approach to evaluate the bearing capability of shallow foundations in the Lalitpur Metropolitan City. The scope includes:

a) Geographical Area:

I. The whole administration of Lalitpur Metropolitan City.

b) Foundation Type:

I. Isolated footings (shallow foundations of 2.25 m^2 and 4 m^2) are generally not more than 1.5-3 meters deep.

c) Data Parameters:

I. Standard Penetration Test (SPT-N) values.
II. Soil classification
III. Parameters of shear strength: cohesion (c) and angle of friction (ϕ).
IV. Unit weight of soil
V. Information on the groundwater table.

d) Approaches:

I. Deterministic Analysis: Classical bearing capacity calculation (Terzaghi, Hansen, Vesic, IS 6403:1981, and Settlement criteria).
II. Numerical Modeling: Finite element simulations (PLAXIS 3D) to estimate the settlements, stress distribution, and failure mechanisms.
III. Spatial Analysis: Interpolation of bearing capacity based on GIS to produce continuous bearing capacity zonation maps
IV. Reliability-Based Assessment: Measurement of uncertainty in the parameters of soils, calculation of probability of failure (P_f) and reliability index (β), to aid risk-based foundation design.

1.5 Limitations

a) The research pays attention to the superficial foundations; deep foundations are not considered.

b) Analysis will be based on available borehole information; thus, site-specific anomalies might not be completely reflected.

c) The layered soils and irregular soils are not considered.

d) The impact of ground water table is taken on the surface due to the inaccessibility of the water table data.
e) Footing under the influence of the static loading only is under consideration.
f) Findings are accurate for the area being studied and might need revision in case new soil research is done.

1.6 Organization of the Thesis

The thesis is organized systematically and coherently.

- Chapter 1: The study's background, problem statement, objectives, scope, significance, and the necessity of an integrated deterministic, numerical, spatial, and reliability-based evaluation of Lalitpur Metropolitan City's bearing capacity are all covered in the Introduction.
- Chapter 2: The literature review examines previous studies on soil carrying capacity estimation, numerical modelling, GIS-based zonation, and reliability analysis. It also highlights research gaps and justifies the combined methodology used in this study.
- Chapter 3: Study Area Description gives the geographical, geological, and soil conditions of Lalitpur with a special focus on the spatial variability and influencing factors on the foundation design.
- Chapter 4: Materials and Methods (Methodology) describes data collection, borehole and laboratory studies, and geotechnical parameters, and discusses the deterministic calculations, numerical simulations (PLAXIS 3D), GIS-based spatial interpolation, and the reliability-based assessment, and the workflow of applying these methods to a full zonation map.
- Chapter 5: Results and Discussions contains the results of deterministic, numerical, spatial, and reliability analysis with tables, figures, and zonation maps that depict the differences in bearing capacity and risk throughout the study area, also compares them, and discusses the consistency of the results and the practical implications to foundation design, urban planning, and risk-informed decision-making
- Chapter 6: Conclusion and Recommendations summarizes the main findings, contributions, and limitations of this study, and offers recommendations on safe and efficient design of foundations and future research on the subject.
- Lastly, the References section indicates all scholarly sources, codes, and technical reports used, and the Appendices contain additional materials, including raw data, calculation sheets, PLAXIS input/output, and GIS layers to support the research.

2 LITERATURE REVIEW

2.1 Theoretical Background

The design of shallow foundations started with the work of (Terzaghi, 1943)who developed the first theoretical model of the design of shallow foundations. His formula took into account the general shear failure in homogeneous soil and was a foundation for further developments in geotechnical engineering. The model by Terzaghi has, over the years, been revised and transformed to include, but not limited to, local conditions, groundwater effects, and heterogeneity of soils. In the design of shallow footings, the designers should be aware of the final bearing capacity of the soil beneath. The ultimate bearing capacity formula that was proposed by (Terzaghi, 1943)is in common usage. (Meyerhof, 1951)followed by (Hansen, 1970)and (Vesić, 1973)gave the formula for the ultimate bearing capacity of a footing, modified by (Bowles, 1977). Terzaghi hypothesized that a uniform surcharge, $q = \gamma \times Df$, would replace the mass of the soil over the footing's bottom when calculating the soil's final bearing capacity. The bearing capacity factors, N_q, N_c, and N_γ, are factors of cohesion, surcharge, and soil unit weight to the final load-bearing capacity. (Skempton, 1951) showed that in case of cohesive soil, the bearing capacity factor N_c in the Terzaghi equation tends to rise with depth.

Empirical techniques based on in-situ tests, like the Standard Penetration Test (SPT), have practical applications, especially in underdeveloped nations, and are extensively used since they are inexpensive and easy to use. Researchers such as (Bowles, 1996) and (Peck et al., 1974) proposed correlations between SPT N-values and soil strength parameters, including cohesion, friction angle, and ultimately the **allowable bearing capacity**. These methods are valuable where direct laboratory testing of soil samples is not feasible due to budget or time constraints.

2.1.1 Principal Modes of Shear Failure

Three sorts of bearing capacity failures have been identified based on the shearing zone pattern, according to (Vesić, 1973):

a. General Shear Failure. It is a kind of failure that is common in soil that exhibits brittle stress-strain behavior. Abrupt, catastrophic failure with tilting are main features. It is typically seen in stiff, tight soil. The settlement abruptly rises at a specific load intensity, q_u. The soil then experiences shear failure at that specific load. Furthermore, the ground surface is included in the failure surface.

b. Punching shear failure: Soils with plastic stress-strain behavior exhibit this kind of failure. The primary features are poorly defined shear planes and significant displacement of wedge-shaped soil beneath the foundation. The foundation fails at a specific load intensity, q_u. The load-displacement curve becomes sharp and, more significantly, linear at this point. The ultimate load is not clearly recognized. This kind is typically found in soft clay and loose sand. No heave occurs, and the ground's surface is not reached by the failure surface. Only the footing's vertical penetration takes place.

c. Local Shear Failure: Soils with relatively flexible stress-strain behavior exhibit this kind of failure. It demonstrates some of the characteristics of the previously mentioned punching and general shear failure. At a certain load intensity, the footing fails at q_u. The footing must be significantly moved in order for failure surfaces to expand to the ground. Following this, there is an additional load and a notable increase in settlement. There needs to be substantial vertical settling for heave to happen. The final load is not shown by the load settling curve. Medium-density sand and medium-consistency clay frequently exhibit this pattern.

2.1.2 General requirements of foundation:

The foundation needs to meet certain general requirements in order to work well. They are:

a. Shear failure

Any foundation must meet the soil rupture requirement or be secure against shear failure. A sufficient factor of safety is often provided by the designer to avoid bearing capacity failure. The range of values is 2 to 4. Equations for bearing capacity can be used analytically to determine the soil's bearing capacity. Alternatively, it can be computed using data from field tests. Codes, particularly construction codes, are used to calculate approximate values. NBC 201 advises against building in areas that are flooded, rock-falling, riverbed, swampy, fill-prone, or prone to landslides. The code also specifies some standards for site investigations. For site exploration, at least two test pits must be dug. If the kind of subsurface soil varies significantly, the number of pits must be increased. According to MRT, the minimum depth of exploration is two meters. However, in hilly regions, exploration must be carried out down to the depth of sound rock, regardless of whether it is more or less than two meters.

b. Settlement criterion

The entire vertical displacement that takes place at the foundation level is known as settlement. More significantly, the foundation's differential settling must to stay within the tolerance range. There might not be any negative consequences on the construction if it settles evenly into the ground. According to IS:1904, permissible total settlement for shallow foundations is generally 40–65 mm for sand and 65–100 mm for clay. Three distinct kinds of settlement can result from the foundation's load:

i. Immediate or elastic settlement, S_i: Less than seven days after the load is placed, it happens right away. It is known as distortion settlement in clay-type soil. This is due to the fact that just the soil's shape is altered; neither its volume nor its water content is altered. It has been discovered to be disregarded because of its low value when compared to long-term consolidation settlement. Granular soils are where it happens. In the end, it occurs during the building phase. It doesn't depend on time.

ii. Primary consolidation settlement or Primary consolidation, S_c: It is caused by the slow release of pore water from the soil's voids, which dissipates surplus pore water pressure and raises effective stress. One to five years or longer is the time frame. It often happens in inorganic clays and depends on time.

iii. Secondary compression settlement, S_s: This kind of settling occurs under constant effective stress, with volume variations brought on by soil particle rearrangement. It depends on time. For organic soils, it is quite important. S_i, S_c, and S_s add up to the entire vertical settlement, represented by S_t.

2.1.3 Shallow Foundation

(Terzaghi, 1943) states that a shallow foundation is positioned at a depth (Df) that is no larger than the foundation's width (B). For example, a Df/B ratio of less than or equal to 1. Additionally, the depth of this kind of foundation is only three meters below the surface. The Df/B ratio for moderately deep foundations is more than 1 and less than 15. In a deep foundation, the load is not entirely transferred to the foundation's base. Both the bearing at the foundation's base and the frictional resistance surrounding it absorb some of the weight. In a shallow foundation, the construction process is completely obvious, but not in a deep one.The soil disturbance is small, but in a deep foundation, it extends to a larger zone along the length.

2.1.4 Standard Penetration Test (SPT)

It is one of the field tests used to determine the soil's geotechnical engineering characteristics. In most nations, this is the most used subsurface exploratory drilling test. Since cohesionless soil types, including gravels, sands, and silts, are difficult to sample for an undisturbed sample, this in-situ test is typically performed on them. The test helps assess cohesionless soil's relative density and shearing resistance angle. The typical split spoon sampler is now lowered to the bottom of the borehole to conduct the test. Soil samples are collected using a split-spoon sampler. A steel tube that has been longitudinally divided into two pieces makes up the tool. The diameter of the widely used tube is around 35 mm on the inside and 51 mm on the outside. A 63.5 kg drop hammer is used to push the sampler into the ground. At a rate of thirty blows per minute, the mass is permitted to descend from a height of 750 mm. IS:2131-1963 provides this rate. It is noted how many blows are needed to push the spoon sampler 150 mm. The figure is commonly referred to as the N value. The SPT is carried out vertically at intervals of 750 mm. If the borehole depth is large, it can be completed up to 1500 mm. It should be noted that the data is not included if the number of blow counts exceeds 50, and 10 consecutive blows result in no advance. This concludes the test. The soil sample is then taken out of the tube when the sampler is removed. For additional soil qualities, the sample is put in the tube and brought to the laboratory. Since many soil properties, including relative density, undrained shear strength, friction angle, elastic modulus, etc., are correlated, the SPT N value is highly helpful and frequently utilized. The SPT is influenced by several variables. The variables include the kind and effectiveness of the hammer, the diameter of the borehole, the sampling technique, and the length of the rod. (Bolton Seed et al., 1985). The equation can be presented as;

$$N_{60} = N \times \eta_H \times \eta_B \times \eta_S \times \eta_R / 60 \qquad (2.1)$$

where; N_{60} = SPT number corrected for different corrections

N= Measured Penetration number

η_H= Hammer efficiency

η_B= Borehole Diameter Correction

η_S=Correction of Sampler

η_R= Correction for Rod length

Additionally, the number should be corrected for overburden and dilatancy before being used in design charts or empirical correlations.

a. Overburden Correction:

The granular soil's penetration resistance "N_{60}" value is influenced by the effective overburden pressure (σ_o'). The granular soil with a higher confining pressure yields a higher value when two granular soils with the same relative density are tested. The confining pressure rises with depth since it is exactly proportional to the overburden pressure. As a result, the N value is overstated at deeper depths and underestimated at shallower ones. For uniformity, the correction is required to reach a standard effective overburden pressure. The revised N is provided by

$$(N_1)_{60}=C_N N_{60} \qquad (2.2)$$

where; N_{60}= Standard Penetration number corrected for field conditions given by

$(N_1)_{60}$=value of N_{60} after being correction

$\sigma_a{'}$ =p_a (≈100kN/m^2)

C_N=Correction factor for overburden pressure

Widely used empirical correlations for C_N are given by (Liao & Whitman, 1986) :

$$C_N= (1/ (\sigma_o{'}/p_a))^{0.5} \qquad (2.3)$$

The values for efficiency are given by (Bolton Seed et al., 1985)

b. Dilatancy Correction:

The volume change we see in granular soils during shear deformations is called dilatancy. If the material expands its volume in response to an increase in shear, we refer to it as dilative. In a similar vein, when shear decreases, the material should compress. Therefore, only when the SPT is carried out in fine or completely saturated sand can this kind of correction be used. Above all, the dilatancy adjustment is used. When $(N_1)_{60}$ value obtained after overburden correction is greater than 15, the (Peck et al., 1974) gives relation as:

$$(N_1)_{60}>15,\ (N_1)'_{60}=15+0.5[(N_1)_{60}-15] \qquad (2.4)$$

where: $(N_1)'_{60}$ =Final corrected value.

If [$(N_1)_{60}\leq 15$, $(N_1)'_{60}=(N_1)_{60}$

2.1.5 Correlation for SPT

a) Correlation between $(N_1)_{60}$ and Angle of Friction (ϕ):

(Peck et al., 1974) have given a correlation between $(N_1)_{60}$ and ϕ;

$$\phi(\text{deg}) = 27.1+0.3(N_1)_{60}-0.00054[(N_1)_{60}]^2 \tag{2.5}$$

b) Correlation between $(N)_{60}$ and Unit weight:

The relation between γ_{sat} and N_{60} given by (Bowles, 1996) for different soils is given below

Saturated Unit Weight for cohesionless soil;

$$\gamma_{sat} = 16 + 0.1 * N_{60}(\text{kN/m}^3) \tag{2.6}$$

Saturated Unit Weight for cohesive soil;

$$\gamma_{sat} = 16.8 + 0.15 * N_{60}(\text{kN/m}^3) \tag{2.7}$$

c) Correlation between N_{60} and unconfined compressive strength (q_u)

Table 2.1: Correlation between SPT value and q_u

N_{60} Range	Consistency	Consistency Index	q_u (kN/m²)
< 2	Very soft	< 0.5	< 25
2 – 8	Soft to medium	0.5 – 0.75	25 – 80
8 – 15	Stiff	0.75 – 1.0	80 – 150
15 – 30	Very stiff	1.0 – 1.5	150 – 400
> 30	Hard	> 1.5	> 400

Table 2.1 provides a general indication of soil consistency and is primarily used for preliminary assessment in engineering practice as suggested by (Das B.M., 2016).

d) Correlation between *N* and unconfined Modulus of Elasticity (Es)

Table 2.2: Correlation between SPT value and Modulus of Elasticity

Soil Type	**SPT Correlation (Es)**
Sand (normally consolidated)	E_s = 500 × (N+15)
	E_s = 7000\√N
	E_s = 6000 × N
	E_s = (15000–22000) \ln N
Sand (saturated)	E_s = 250× (N+15)
Sands (all, normally consolidated)	E_s = (2600–2900) × N
Sand (overconsolidated)	E_s = 40000 + 1050 × N
	E_s(OCR) ≈ ($E_{s,NC}$\√OCR)
Gravelly sand	E_s = 1200 × (N+6)
	E_s = 600 × (N+6), (N ≤ 15)
	E_s= 600 × ((N+6) +2000), (N > 15)
Clayey sand	E_s = 320 × (N+15)

Table 2.2 provides a general estimation of values as provided by (Bowles, 1996) . The correlations are approximate and should be applied with engineering judgement, as the stiffness of soils depends on many factors, including stress and structure.

2.1.6 Development of Terzaghi's Bearing Capacity Theories

The first researcher to develop a comprehensive theory that can be used to calculate the ultimate bearing capacity of rough shallow foundations was Tergazhi. It is a general theory of Bearing capacity. He replaced the soil above the foundation base with a uniform surcharge γD_f. He assumed the Mohr-Coulomb failure criterion for the shear strength of the soil. Furthermore, he assumed that the load on the footing is vertical and uniform. The footing is long, i.e, strip or continuous (L/B = infinity).

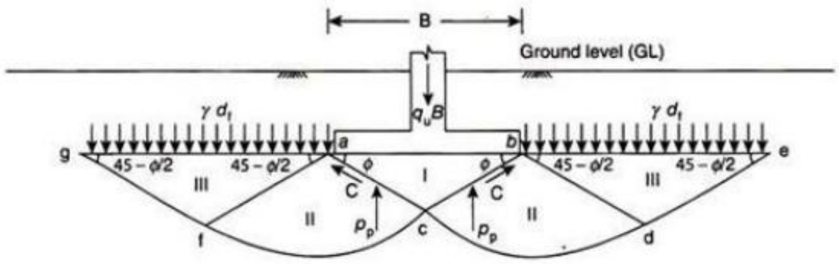

Figure 2.1: Failure surface as per Tergazhi (BM Das)

In Figure 2.1, a wedge-shaped mass of soil, abc, is shown. It does not experience horizontal movement but moves downward as soon as the footing is subjected to pressure qu, which is equivalent to the ultimate bearing capacity of the soil. The sides ca and cb are lifted by an angle ϕ. The mass of soil abc is known as zone I and is in elastic equilibrium. The influence of the soil above the base of the footing is considered as a surcharge γDf. The shear strength of the soil above the footing base is not to be neglected in deep foundations. Otherwise, it will be a blunder. The failure zone above consists of zone II (called the radial shear zone) on both sides of Zone I. The zone III is on the outside of the footing end-to-end with Zone II. The sides ca and cb are vertical retaining walls. The sides thrust the soil in Zone II downwards and outwards. The zones I, II, and III may be called the zone of elastic equilibrium, the zone of radial shear state, and the zone of Rankine passive state, respectively. The equilibrium of wedge abc was taken to find the ultimate bearing capacity of a strip footing. The (Terzaghi, 1943) The bearing capacity equation is written as;

$$q_u = c\,N_c + qN_q + 0.5\,\gamma B N_\gamma \qquad (2.8)$$

Where,

c = Effective Cohesion

γ = Unit weight of soil

q = Overburden Pressure = γD_f

The bearing capacity factors N_c, N_q, and N_γ are the dimensionless numbers and, respectively, the contributions of cohesion, surcharge, and unit weight of soil.(Terzaghi, 1943). These rely only on the soil's angle of shearing resistance (ϕ). It is possible to compute the ultimate carrying capacity in terms of both total stress and effective stress.

To find the qu of circular and square foundations as per (Terzaghi, 1943);

$q_u = 1.3\ cN_c + qN_q + 0.3\gamma BN_\gamma$ (Circular foundation) (2.9)

$q_u = 1.3\ cN_c + qN_q + 0.4\gamma BN_\gamma$ (Square foundation) (2.10)

where B equals the dimension of each side of the foundation.

However, the shear strength parameters c_m *and* ϕ_m are utilized in the equation for local shear failure (Terzaghi, 1943), which suggested empirical changes to shear strength values c and ϕ.

Mobilized cohesion, $c_m = 2/3\times c$ (2.11)

For mobilized angle of shearing resistance, ϕ_m *or* ϕ;

$\tan \phi_m = 2/3\times\tan\phi$ (2.12)

The bearing capacity factors are indicated as N_c, N_q, *and* N_γ corresponding to local shear failure to differentiate from general shear failure.

The ultimate bearing capacity equation can be written as;

$q_u = 2/3\ cN_c + qN_q + 0.5\ \gamma BN_\gamma$ (2.13)

2.1.7 Hansen's Bearing capacity equations

(Hansen, 1970) provided a general bearing capacity equation, which was similar to Meyerhof's. Compared to Meyerhof and Hansen, Terzaghi's results were found to be conservative. For cohesive soils, Hansen's equations give better results compared to Terzaghi's. Hansen's equation can be presented as;

$q_u = cN_c\times s_c\times d_c\times i_c + qN_q\times s_q\times d_q\times i_q + 0.5\gamma BN_\gamma\times s_\gamma\times d_\gamma\times i_\gamma$ (2.14)

where s, d, and i are Hansen's shape, depth, and inclination factors, respectively.

2.1.8 Vesic's Bearing capacity equations

(Vesić, 1973) used the failure surface similar to that used in Terzaghi's theory, except that the slope of the elastic wedge (zone-I) is assumed to be $(45 + \phi/2)$ with horizontal instead of ϕ. The equation can be presented as;

$q_u = cN_c\times s_c\times d_c\times i_c + qN_q\times s_q\times d_q\times i_q + 0.5\gamma BN_\gamma\times s_\gamma\times d_\gamma\times i_\gamma$ (2.15)

where s, d, and i are Vesic's shape, depth, and inclination factors, respectively.

2.1.9 IS Code method (IS 6403:1981)

In India and many South Asian countries, bearing capacity calculations are commonly performed using the Indian Standard Code (Bureau of Indian

Standards, 1981), which is largely based on Terzaghi's theory with modifications... The equation can be presented as;

$$q_u = cN_c \times s_c \times d_c \times i_c + q(N_q\text{-}1) \times s_q \times d_q \times i_q + 0.5\gamma B N_\gamma \times s_\gamma \times d_\gamma \times i_\gamma \quad (2.16)$$

2.1.10 Bowles (1977)

(Bowles, 1977)proposed the modified form of the bearing equation based on Settlement Criteria, which can be expressed as:

For B≤ 1.22 m;

$$qnet_{(all)}\ (kN/m^2) = 19.16(N_1)_{60} \times F_d \times (So/25.4) \quad (2.17)$$

For B>1.22 m;

$$qnet_{(all)}\ (kN/m^2) = 11.98 \times (N_1)_{60} \times \left(\frac{3.28B+1}{3.28B}\right)^2 \times Fd \times (S_o/25.4) \times R_w \quad (2.18)$$

Where,

F_d = depth factor = $1+0.33(D_f/B)$,

B = foundation width, in meters

S_o = settlement, in mm

R_w = Water table correction factor

(Bond & Harris, 2004) recommends that the maximum total settlement be 25 mm for the serviceability limit state.

2.2 Zonation mapping practices

The idea of bearing capacity zonation has been successfully implemented in developed areas to map the changes in soil strength. A zonation map of Kathmandu Valley was prepared by applying GIS techniques on SPT data interpolated by applying spatial analysis to identify weak, moderate, and strong soils. This method allowed urban planners to give priority to site-specific soil-improvement actions. (Rahman et al., 2026) created GIS-based zonation maps of Dhaka Metropolitan City using a combination of borehole data and spatial interpolation, showing that GIS can enable planners and engineers to rapidly locate safe, moderate, and weak areas for shallow foundations. (Hamdani et al., 2025) applied GIS to create the bearing-capacity zonation of CPT in soft-soil areas. (Danai & Acharya, 2018) analyzed bearing capacity and developed zonation maps for the Kathmandu Valley using SPT-based soil data. Analytical methods and PLAXIS 2D modeling were used, along with load-settlement analysis to estimate ultimate capacity. Their results indicated significant spatial variation in bearing capacity with depth, and numerical results differed from analytical methods by about 5–20%.

(Raut & Acharya, 2020) evaluated the bearing capacity of shallow foundations in the urban areas of Dhulikhel and Banepa using borehole SPT data and soil parameters such as cohesion, friction angle, unit weight, modulus of elasticity, and Poisson's ratio. Classical methods, including Terzaghi, Meyerhof, Hansen, Vesic, and Bowles, were applied. The study showed that bearing capacity varies spatially and generally increases with depth, ranging from 83 kPa to 447 kPa. (A. Jasem Al-Maliki et al., 2018) investigated suitable GIS interpolation methods for estimating bearing capacity in An-Najaf City using data from 464 boreholes. Various deterministic and geostatistical techniques were evaluated, and the study identified IDW as the most reliable method, highlighting strong spatial variability in bearing capacity.

2.3 Numerical modeling

Although helpful, empirical methods are not very accurate in complicated or stratified soil profiles. To supplement these techniques, numerical analysis with finite element methods (FEM) is becoming more popular. The widely used one is PLAXIS 3D, which is created by (Brinkgreve et al., 2016) to model the interaction between the soil and structure under applied loading. (Danai & Acharya, 2018) , modeled shallow footings in Kathmandu Valley using PLAXIS 2D based on the Mohr-Coulomb constitutive model to obtain site-specific bearing capacities that were subsequently interpolated in GIS to create zonation maps. On the same note, (Raut & Acharya, 2020) did numerical modeling using PLAXIS 2D and GIS-based zonation mapping in Banepa and Dhulikhel. Similar studies have integrated numerical modeling with GIS to develop bearing capacity maps. For example, a study by (Prasad Tiwari et al., 2022) combined PLAXIS 3D simulation with GIS interpolation, showing that hybrid approaches improve the reliability and spatial representation of soil bearing capacity for foundation design.

2.4 Reliability concept

The reliability analysis has turned out to be a significant instrument in the engineering design since it offers a sensible framework of quantifying the uncertainties of loads, material characteristics, and modeling assumptions. In traditional deterministic design methods, uncertainty is typically considered by a global factor of safety. Nonetheless, this method does not directly measure the failure probability and can result in either overly conservative design or inadequate safety. Reliability based methods overcome this constraint by using probability and statistics in the analysis of engineering (Baecher & Christian, 2003).

When performing a reliability analysis, a limit state function is generally used to characterize the performance of the system, making a distinction between safe and failure states. The limit state function generally takes the form of

$$g(X)=R-Q \tag{2.19}$$

where R is the resistance, and Q is the load. When load is greater than the resistance (when $g(X)<0$), failure occurs (Melchers & Beck, 1974). This formulation enables uncertainties in loads and resistance to be modeled explicitly using random variables with the following statistical properties mean, variance, and coefficient of variation (COV).

Reliability analysis is based on the concept of random variables. Several parameters of geotechnical engineering, e.g., the cohesion of the soil, the angle of friction, the unit weight or the loads applied, are considered as random variables due to their variability. Geological processes and sampling constraints in particular mean that soil properties vary significantly spatially. (Phoon & Kulhawy, 1999) suggest there are three primary factors of soil variability namely, inherent variability, measurement error, and transformation uncertainty. Such uncertainties can greatly affect the dependability of geotechnical structures, and as such, they should be factored in during probabilistic analysis.

The other significant concept of reliability analysis is the index of reliability which is usually represented by β. The index of reliability gives a quantitative estimate of the system safety by giving the correlation between the mean and the standard deviation of the limit state function. A greater reliability index is associated with a low chance of failure, which means that it is a safer system (Ditlevsen & Madsen, 1996). Failure is likely to be the result of:

$$P_f=\Phi(-\beta) \tag{2.20}$$

In which Φ is the cumulative distribution of the standard normal distribution.

Various analytical and numerical tools have been created to measure reliability. The First Order Second Moment (FOSM) method is one of the first and estimates the reliability of a limit state function in terms of the mean and variance. The FOSM approach is broadly employed due to its simplicity and computational efficiency but it only makes the linear approximation of limit state function around the mean values of the variables (Ang, 2007)

Another more developed method is the First Order Reliability Method (FORM), which was coined by (AM & Lind, 1974). FORM is an enhancement of FOSM, which converts the random variables to a standardized normal space and finds the most likely point of failure on the limit state surface. This approach offers a better approximation of the index of reliability, especially when the performance functions are nonlinear.

Monte Carlo simulation is another method of reliability analysis that is commonly applied. The process in this approach is to create number of random positions of the input variables and to compute the limit state function at every realization. The likelihood of failure is then found from the number of cases of failure to the sum of the number of simulations. The fact that Monte Carlo simulation is not based on linear approximations is also beneficial, as it can be applied to complex nonlinear problems (Melchers & Beck, 1974).

The reliability analysis has been extensively used in geotechnical engineering to determine the performance of foundations, slopes, and retaining structures. (Baecher & Christian, 2003) emphasized that probabilistic methods are particularly relevant in geotechnical engineering due to large variation and uncertainty of soil properties. (Fenton & Griffiths, 2010) showed that the more the variability in the parameters of soil strength, the more likely it is to fail, and the lower the index of reliability. These probabilistic techniques give a more realistic estimate of the system performance than the classical design techniques that are deterministic.

2.5 Review of previous paper

Bearing capacity for shallow foundations has been widely investigated by a combination of empirical, geostatistical, and numerical methods, each of which has its own merits and drawbacks. (A. Jasem Al-Maliki et al., 2018) explored the possibility to predict soil bearing capacity based on a large dataset of boreholes and tested the different GIS-based interpolation methods, including Inverse Distance weighting (IDW), Local Polynomial Interpolation (LPI), Radial Basis Function (RBF), and Global Polynomial Interpolation (GPI), and geostatistical methods, The research showed that interpolation procedures could be useful in producing spatial distribution maps of bear capacity but the accuracy of these processes relies heavily on the kind and distribution of input data. The authors found that there is no universal best interpolation method and highlighted the need to apply statistical validation as a means to select the most suitable one based on parameters like RMSE and coefficient of determination (R^2). Likewise, (Hussaini & Hozeh, 2021) were interested in creating the bearing capacity zonation maps of Kabul city based on the (Bowles, 1996) empirical equation, as it includes the corrected SPT-N value along with the settlement criterion of 25 mm. The paper employed GIS-based interpolation of the IDW to generate spatial maps and indicated that the permissible bearing capacity is very diverse and depends on the soil type and subsurface environment. Although the method was quite practical and useful in preliminary design, it was restricted by the fact that it used empirical correlations and was not validated by more complex numerical methods. (Rahman et al., 2026) built on such GIS-based methods, including corrections of the groundwater conditions, unit weight, and parameters of the field in the values of the SPT-N to produce thematic maps of bearing capacity at various depths. The research paper pointed out that bearing capacity tends to increase with depth and that GIS models can be useful in conducting regional analysis at large scales and initial foundation design. Nonetheless, the approach was still based on simplified empirical relations and failed to consider uncertainty and variability in soil properties, which can impact on the performance in the real world. (Raut & Acharya, 2020) thoroughly developed the investigation in the Nepalese setting by combining the classical theories of bearing capacity, such as those by Terzaghi, Meyerhof, Hansen, and Vesic, settlement-based criteria, and numerical analysis with PLAXIS 2D. The analysis used borehole data and soil parameters that included cohesion, angle of internal friction, unit weight, and modulus of elasticity to come up with bearing capacity zonation maps of Dhulikhel and Banepa. The findings indicated that the values of bearing capacity are different according to the technique applied and that settlement factors are very important in the establishment of allowable capacity. Moreover, numerical modeling gave a more realistic portrayal of the soil behavior than the analytical approaches; nevertheless, the work was based on two-dimensional modeling and failed to include probabilistic analysis and three-dimensional influences. Similarly, (Danai & Acharya, 2018) used PLAXIS 2D with finite element analysis to assess the bearing capacity of shallow foundations in Kathmandu valley. The experiment involved the Mohr-Coulomb failure criterion and examined the influence of different parameters, including depth to width ratio (D_f/B), cohesion, friction angle, and unit weight, on bearing capacity. The findings demonstrated that the bearing capacity is proportional to depth and that load-settlement curves are useful in determining ultimate bearing capacity. Although the study offered more realistic observations than the traditional techniques, the application of two-dimensional modeling and simplified assumptions of soils limited the study, as they might not entirely reflect real-life conditions in the field. In general, the literature reviewed suggests that although there have been great advances in estimating bearing capacity through GIS-based techniques, empirical correlations, and numerical modeling, most literature is confined to two dimensions and deterministic models. It has not integrated the use of advanced probabilistic methods and 3D modeling, which are crucial in capturing soil variability and real foundation behavior, especially when dealing with complex issues, such as square footings and settlement-based design.

2.6 Research gaps

Although a large number of studies have been performed on the estimation of bearing capacity by employing empirical techniques, zonation through GIS, and zonation by numerical modeling, there are various gaps that have been left in the literature. The majority of prior research involved simplified methods of analysis or two-dimensional numerical modeling with PLAXIS 2D, with little regard for realistic three-dimensional soil-structure interaction. Moreover, most researchers have extracted much on using SPT-based empirical correlations without considering the uncertainty and variability of soil properties that could considerably influence the foundation performance. Probabilistic analysis or Monte Carlo analysis. Reliability-based methods, which include probabilistic analysis or Monte Carlo analysis, have hardly been applied to bearing capacity estimation, especially in the context of shallow foundations.

Additionally, the literature has mostly assumed standard or non-specified size of footings, whereas the performance of various sizes of footings, including 1.5 m × 1.5 m square footing and 2 m × 2 m square footing, which is popular in real-life engineering works, has not been compared. Lack of research on footing size effect on bearing capacity and settlement behaviour under realistic field conditions. Also, zonation research has been done on most areas, like Kathmandu, Banepa, and Dhulikhel, whereas the Lalitpur area does not have a zonation bearing capacity map that has been done based on analytical, numerical, and probabilistic studies.

Hence, detailed research that integrates PLAXIS 3D numerical simulation, reliability assessment, and GIS-supported zonation mapping to assess bearing capacity more precisely is required. The study must consider the uncertain parameters in the soil properties and the performance of the various footing sizes, 1.5 m x 1.5 m and 2 m x 2 m, in a realistic situation.

3 STUDY AREA DESCRIPTION

3.1 Location of Lalitpur Metropolitan City

Lalitpur Metropolitan City is in the heart of Nepal in Kathmandu Valley, Bagmati Province, and is the seat of Lalitpur District. The city is directly South of Kathmandu, and the Bagmati River separates the two cities, but the expansion of the urban areas has merged the cities in practice. One of the three major historic urban centers of the valley is Lalitpur, historically known as Patan, along with Kathmandu and Bhaktapur. The city is at an average height of approximately 1,350-1,400 above sea level, with the geographical latitude and longitude of 27.66° N and 85.32° E, respectively. Kathmandu Valley has traditionally been a significant cultural and urban settlement since it is a fertile basin with a strategic location, and in recent decades, it has grown in space and population considerably (Shrestha et al., 2024). Lalitpur is especially associated with the rich cultural heritage, traditional Newari architecture, and historic monuments, including Patan Durbar Square, which is a UNESCO World Heritage site (UNESCO, 2019). It is also suggested by recent research that Lalitpur Metropolitan City has experienced a swift urbanization, land-use alteration, and infrastructure development, which are more general trends of urban transformation in the Kathmandu Valley (Central Bureau of Statistics, 2021).

Figure 3.1: Location Area with Borehole

3.2 Geographic setting

Lalitpur Metropolitan City is described by its geographic location of the city, situated in the fertile Kathmandu Valley basin in central Nepal. The city is located in Bagmati Province at an average altitude of 1,350-1,400 meters above sea level. Geographically, Lalitpur is situated to the south of Kathmandu and is mostly divided by the Bagmati River, which is a key natural boundary in the valley. The valley is in turn enclosed by hills, like Phulchoki, Shivapuri, and Chandragiri, forming a bowl-shaped topography, which affects the climate, drainage and urbanization of the place. It is said that the Kathmandu Valley is a bowl-shaped intermontane basin that consists of fluvial and lacustrine sediments that highly regulate its geomorphology and settlement pattern (Ishtiaque et al., 2017).

The Lalitpur Metropolitan City terrain is mostly flat to gently rolling, comprising alluvial and lacustrine deposits, which were deposited by the ancient lake that once occupied the Kathmandu Valley. Such fertile soils have long been at the heart of agriculture and early settlement, and as a result, Lalitpur has become one of the most important cultural and urban centers in history, formerly called Patan. Alluvial plains and river terraces mostly comprise the valley floor, which can be inhabited and utilized for agriculture (Ishtiaque et al., 2017). In the metropolitan area, the city is fed by a few small tributaries of the Bagmati River, including the Nakkhu Khola, which passes through the southern part of the city. The Kathmandu Valley, and Lalitpur in particular, due to the topography of its basin, is a subtropical highland, whose climate is moderately hot, with monsoon-dominated precipitation, and most of the rainfall falls in summer (Ishtiaque et al., 2017).

Geographical and urban research indicates that the special basin form, rich soil, and water are some of the factors that have influenced both settlement patterns and land use and urban growth in Lalitpur Metropolitan City. From a geotechnical perspective, shallow foundations are widely used in Lalitpur because they are economical and suitable for most low-rise buildings. However, soil conditions vary significantly across the valley from soft, compressible soils to dense materials leading to differences in bearing capacity. If there is construction in weaker soil areas, increasing the risk of settlement and failure if shallow foundations are not properly designed.

The prior research by (Thapa & Murayama, 2012) points out that the topography, accessibility, and land availability of the Kathmandu Valley, as well as the socioeconomic factors of migration and the establishment of infrastructure, are key factors that have significant impacts on the urban growth in the valley.

Nevertheless, recent studies have shown that the accelerated urbanization and population growth have resulted in a drastic change in land use, agricultural land loss, and strain on environmental resources, which makes the idea of sustainable urban planning a primary concern for the future progression of Lalitpur Metropolitan City (Ishtiaque et al., 2017).

3.3 Geological conditions

The Lalitpur Metropolitan City geological conditions have a strong relation with the Kathmandu Valley geological formation, which is an intermontane basin, filled with thick Quaternary sediments. There was an old lake that occupied the valley, and over the years, it was filled with lacustrine, fluvio-lacustrine, and alluvial deposits that are composed of clay, silt, sand, and gravel, which are the major subsurface materials of the valley today (Yoshida & Igarashi, 1984). These sediments are very varied in their thickness, with some sections of the Kathmandu Valley having a depth of approximately 200 m to as much as 600 m of the sediments overriding the older basement rocks of the hill surrounding area (Paudel & Sakai, 1970).

The Kalimati Formation, Patan Formation, and Thimi Formation are the geological formations that make up the bulk of Lalitpur and are the sedimentary sequence of Kathmandu Basin. The Kalimati Formation is mainly black clay and organic-rich lacustrine formations, whereas the Patan Formation is mainly sand, silt, and gravel deposited by old river systems. These structures govern the engineering properties of soils, such as bearing capacity, permeability, and settlement properties, of importance in foundation design (Sakai, 2001). The ground conditions in Lalitpur are prone to settlement due to the existence of soft clay layers and loose sediments in certain regions, which may enhance the seismic waves during an earthquake.

3.4 Soil profile

The sedimentary rocks of the Kathmandu Valley highly impact the soil profile of Lalitpur Metropolitan City; these are mainly composed of the lacustrine and fluvio-lacustrine sediments that resulted from the lake that existed in the basin. The profile of the subsurface is generally formed of alternating bands of clay, silt, sand, and gravel, with clayey layers prevailing in most regions of the valley floor. The topmost layer is usually soft to medium-stiff silty clay or clayey silt and may be characterized by organic materials since material has been accumulated for a long duration, and also by human activities. The medium dense sand and sandy silt are often met with below this layer, and deep below that is the dense sand or gravel, which are often competent bearing strata of foundations (Yoshida & Igarashi, 1984)

In most of Lalitpur, the soil profile is related to the sedimentary units of the Kathmandu Basin, especially the Kalimati and Patan Formations. The Kalimati Formation is primarily made of black or gray organic clay, which is very compressive, has a fairly low shear strength, and is thus not suitable to support heavy loads without ground improvement. The Patan Formation, on the other hand, contains sand, silt, and gravel deposits, which have better drainage characteristics and greater bearing capacity (Sakai, 2001). The depth of these layers is spatially variable, and at certain places the depth of the sediment reaches several hundred meters before getting to the bedrock.

The engineering design of Lalitpur is critical due to the soil profile of the geotechnical aspect. Settlement, differential settlement, and seismic amplification during earthquakes may occur when there is the presence of soft clay and loose sediments. Research in the Kathmandu Valley has shown that these soil conditions have a major effect on the foundation performance and structural stability, especially during earthquakes like the 2015 Gorkha Earthquake (Thapa & Murayama, 2012). Thus, safe and effective foundation design in Lalitpur requires detailed geotechnical studies, such as borehole exploration, as well as laboratory tests.

4 MATERIALS AND METHODS (METHODOLOGY)

The methodology integrates field data, analytical methods, numerical modeling, GIS analysis, and reliability assessment to analyze the bearing capacity of shallow foundations. Borehole data, including soil stratification and SPT values, were used to develop soil profiles and estimate engineering parameters through empirical correlations. Deterministic analysis was performed using classical methods such as Terzaghi, IS, Hansen, Vesic, and Bowles to obtain baseline bearing capacity values.

Numerical modeling was conducted using PLAXIS 3D with the Mohr–Coulomb model, where appropriate boundary conditions and loading were applied, and allowable bearing capacity was determined based on a 25 mm settlement criterion. GIS-based analysis using ArcGIS was carried out to map spatial variability through IDW interpolation and classify foundation suitability zones. Finally, reliability analysis using Monte Carlo simulation was performed to calculate the probability of failure and reliability index, and the results were validated with field data and literature. The overall methodological workflow is presented in Figure 4-1, and the coordination schema is summarized in Table 4-1:

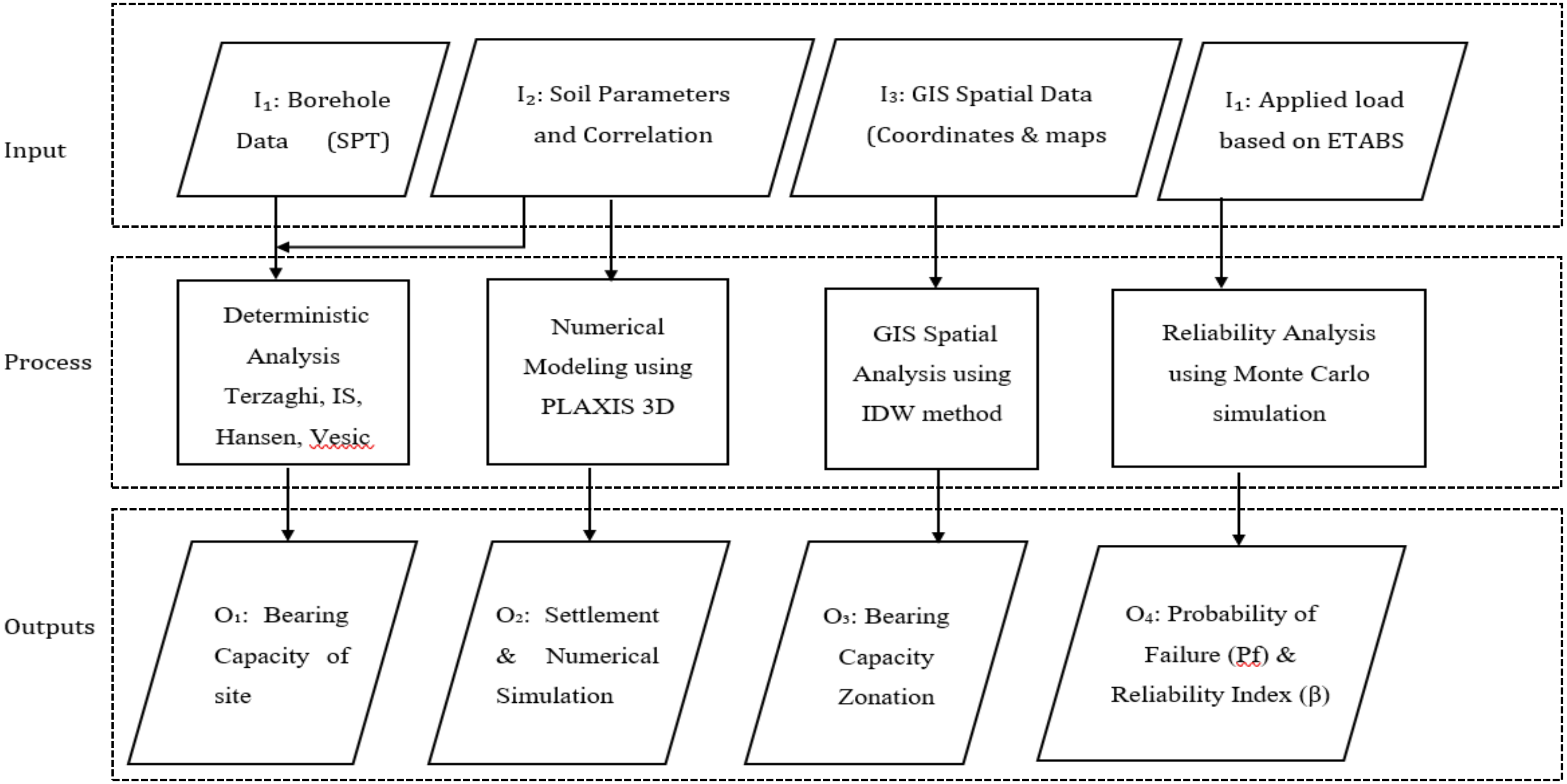

Figure 4.1: Methodological Framework

Table 4.1: Coordination Schema

Problem Statement	Objective	Method	Data/Tools	Output
Lack of spatial understanding of bearing capacity in Lalitpur	Objective 1: Estimate bearing capacity	Deterministic analysis (Terzaghi, IS, Hansen, Vesic, and Bowles)	Borehole data, SPT values, empirical correlations	Allowable & ultimate bearing capacity
Need for realistic soil behavior simulation	Objective 2: Numerical modeling	PLAXIS 3D (Mohr-Coulomb model)	Soil parameters (c, φ, γ, E, ν), geometry, boundary conditions	Settlement, stress distribution, failure mechanism
Lack of regional zonation	Objective 3: Spatial mapping	GIS interpolation (IDW/Kriging)	ArcGIS, borehole coordinates, BC values	Bearing capacity zonation maps
Uncertainty in soil properties and loading	Objective 4: Reliability analysis	Monte Carlo Simulation	Excel	Probability of failure (P_f), Reliability index (β)

4.1 Data Collection

Accurate and adequate data collection plays a crucial role in a viable geotechnical study and quantitative analysis. In this study, geotechnical investigation reports that were carried out in the region of study were utilized to obtain subsurface soil information and engineering parameters. The numerical modeling, bearing capacity estimation, and reliability-based analyses were the primary products of the collected data.

4.1.1 Borehole data

The information gathered in the boreholes was used to get an idea about the subsurface soil profile and stratification of the region analyzed. These values were derived based on borehole logs, which were prepared in the field during investigations. Borehole logs provide vertical information on the layers of soil that are observed at different depths and therefore assist to find the type of soil, the layer's thickness, and the variation of soil properties.

The borehole records were composed of:

a) Soil stratification: Elucidation of various layers of soil, such as clay, silty clay, sand, and gravel, and how deep each layer is found.

b) Layer thickness: The depth of each soil layer that is required to calculate soil geometry in numerical models.

c) Standard Penetration Test (SPT) values: The values of field resistance at depths of intervals to identify the density and strength characteristics of soils.

According to these borehole descriptions, representative soil profiles, which are founded on numerical simulations, were created. The stratification of the soil helped establish the geometry of the soil layers in the finite element model, and the SPT values were used to forecast the engineering properties using empirical correlations. The foundation design is based on the average SPT N-value that is taken to represent the weakest layer of the soil in the influence depth (approximately 1 to 1.5 times footing width). The soil may often be non-homogeneous and therefore weaker layers may be present, and a minimum of the way will provide a safe and conservative design with limited chances of settlement or bearing capacity failure.

4.1.2 Soil parameters

Numerical modeling, analytical calculations, and probabilistic analysis require inputs of engineering soil parameters. These parameters define the mechanical and physical properties of soil when it is loaded.

a) Empirical Correlation:

Where laboratory test results were not available at some depths, SPT values and soil classifications were used to correlate different soil parameters for the given site. Such correlations are common in geotechnical engineering to provide estimates of the behavior of soil. Section 2.1 includes different relations based on SPT value that is required to analyze the bearing capacity of soil.

The key soil parameters in this study are:

- Saturated Unit weight (γ_{sat}): This is needed to compute the pressure of overburden and stress distribution.
- Cohesion (c): This is a component of shear strength in cohesive soils.
- Angle of internal friction (ϕ): The angle of inter-particle resistance.
- Young modulus (E): This is a measurement of the stiffness and deformation of soil.

- Poisson ratio (ν): Characterizes behavior of lateral deformation under loading.

The input variables of the finite element software PLAXIS 3D to model soil-foundation interaction were these parameters. Moreover, statistical dispersion of the major parameters, like cohesion, friction angle, and unit weight, was included. All parameters were thoroughly checked based on the normal practice of geotechnical.

4.2 Deterministic analysis

Deterministic analysis was performed to determine foundation performance based on single representative values of soil parameters based on site investigations. This methodology presumes that the soil properties do not vary, but rather are constant, which overlooks the inherent variability and uncertainty. The analysis gives a baseline measure of bearing capacity and settlement, which is subsequently compared with the numerical and probabilistic outcomes.

First, typical soil parameters including unit weight, cohesion, angle of internal friction and modulus of elasticity were identified based on borehole data and empirical correlations. The average values of the soils were used as representative values in each of the layers to make the calculation easier. For shear failure criteria, (Terzaghi, 1943), IS, (Hansen, 1970) and (Vesić, 1973) were used while, (Bowles, 1977) methods was employed have been utilized as settlement criteria.

Dividing the ultimate bearing capacity by the allowable bearing pressure gives Factor of Safety. This gave an indication of the stability of the foundation when subjected to a static load. The determined outcomes of the deterministic analysis are used as a benchmark in the numerical modeling and reliability-grounded probabilistic evaluation, as well as the production of zonation maps.

4.3 Numerical modeling

Simulation of soil-foundation interaction was done using numerical modeling, and realistic estimates of bearing capacity were obtained. PLAXIS 3D was used because it is appropriate to analyze geotechnical problems, which have complicated soil behavior and boundary conditions.

Based on the borehole data, a representative soil profile was created and the soil layer was modeled based on its thickness and engineering characteristics. The Mohr-Coulomb constitutive model has been employed to model soil behavior because it has few parameters to be used and gives credible results when used in foundation analysis.

The development of the modeling process was conducted through the development of geometry, assigning material properties, developing a finite element mesh, defining boundary conditions, and loading. The groundwater conditions were established with a worst-case scenario at the surface. The foundation loads were applied progressively to record stress-strain behavior and failure mechanism. According to the suggestion of the (Bond & Harris, 2004), The settlement of the shallow foundation, which is tolerable, is normally 25mm, and therefore the specified displacement of 25mm is applied to the footing plate at the depths of 1.5m and 3m of the soil.

Allowable bearing capacity, settlement, stress distribution and deformation patterns were computed using the numerical model. Findings of the simulations were checked against deterministic calculations.

4.4 GIS Spatial analysis

A Geographic Information System (GIS) analysis was conducted to assess the geographical difference of bearing capacity and foundation suitability in the study area. Integration of geotechnical information with spatial mapping techniques was performed with the help of the software ArcGIS.

Co-ordinate data was used to geo-reference the borehole locations, spatial databases were prepared including soil properties, bears value, and settlement outcomes. Continuous spatial distribution maps were generated using interpolation methods like Inverse Distance Weighting (IDW).

Thematic maps were made to categorize zones according to the ranges of bearing capacity that were calculated. The maps are useful in visualizing safe and unsafe areas to build foundations and aid in making decisions during site planning. GIS analysis gave a regional view of soil behavior, which cannot be done by using point-based deterministic calculations.

4.5 Reliability analysis

A Monte Carlo simulation in Microsoft Excel was used to conduct a reliability-based analysis of shallow foundation bearing capacity in this study. The aim was to test the likelihood of failure and the index of reliability of various footing setups in the presence of uncertainty in bearing capacity and bearing load. The ultimate bearing capacity q_u was treated as a random variable and assumed to follow a normal distribution. The normal distribution was selected because bearing capacity is always positive and typically shows skewed variability in geotechnical data. The parameters of the normal distribution were calculated from the mean and the coefficient of variation of the bearing capacity.

The applied structural load was also considered as a random variable and modeled using a normal distribution with a mean load of 458.52 kN and a coefficient of variation of 0.39.

Monte Carlo simulation was then performed with 10,000 simulation trials for each footing configuration. In each simulation:

1. A random value of bearing capacity was generated from the normal distribution.
2. A random value of applied load was generated from the normal distribution.
3. The foundation resistance was calculated as

   $$R = q_u \times A \tag{4.1}$$

   where A is the footing area.

4. The limit state function was defined as

   $$g = R - Q \tag{4.2}$$

   where
   R=foundation resistance
   Q= applied load.

Failure was assumed to occur when $g \leq 0$

The probability of failure, P_f, was estimated as the ratio of the number of failed simulations to the total number of simulations:

$$P_f = N_f / N \tag{4.3}$$

Where,
N_f =number of failure cases
N = total simulations.

The reliability index β was calculated using the inverse standard normal distribution:

$$\beta = -\Phi^{-1}(P_f) \tag{4.4}$$

For different footing sizes and depths varying the coefficient of variation of bearing capacity from 0.2 to 1.0, the analysis was performed to determine the influence of uncertainty on foundation reliability. The results were then summarized and compared to evaluate how variability in bearing capacity affects the probability of failure and the reliability index of shallow foundations.

4.6 Validation method

Research work was validated by.

- Identification of about 3-5 representative sites in the study area. Field and laboratory studies will be carried out on a thorough basis at every chosen location to identify the engineering properties of soil. The tests will consist of grain-size distribution tests, Atterberg limits, unit-weight tests, and shear-strength parameters, including cohesion and angle of internal friction. These are parameters that are necessary in assessing the bearing capacity and behavior of the soil.
- By applying a literature paper depending on the bearing capacity ranges and geological conditions in Lalitpur metropolitan city.

5 RESULTS AND DISCUSSION

5.1 Results

5.1.1 Deterministic analysis

Table 5.1:Values of bearing capacity for 1.5 m× 1.5 m foundation

Borehole location	Shear Failure Criteria				Settlement Criteria	Minimum value of bearing capacity
	Tergazhi kN/m^2	Vesic kN/m^2	IS kN/m^2	Hansen kN/m^2	Bowles kN/m^2	
Bagdole	88.57	144.54	94.09	119.56	106.15	88.57
	157.25	398.59	197.80	289.24	149.05	149.05
Bhakundole	78.06	97.14	88.92	97.47	119.42	78.06
	106.92	167.68	118.56	169.05	182.17	106.92
Balkumari	95.04	160.05	101.00	128.44	92.88	92.88
	171.39	322.18	159.75	233.59	132.48	132.48
Bhaisipati	90.46	112.84	103.74	113.18	132.68	90.46
	106.92	167.75	118.56	169.14	182.17	106.92
Dhapakhel	96.72	160.79	101.66	128.99	132.68	96.72
	163.95	360.86	179.24	262.10	165.61	163.95
Dhobighat	95.04	139.88	88.27	112.25	92.88	88.27
	69.46	107.10	74.10	108.42	115.92	69.46
Harisiddhi	78.01	97.10	88.92	97.43	106.15	78.01
	94.47	147.58	103.74	148.95	165.61	94.47
Hattiban	96.72	159.17	100.63	127.68	132.68	96.72
	181.82	395.96	197.02	288.13	198.73	181.82
Jwalakhel	86.79	169.33	106.92	135.86	106.15	86.79
	155.68	342.75	170.09	248.71	149.05	149.05
Jhamsikhel	78.01	97.66	88.92	98.04	106.15	78.01

	69.46	108.21	74.10	109.75	115.92	69.46
Khumaltar	95.04	137.02	86.47	109.96	92.88	86.47
	181.04	335.83	166.65	243.69	149.05	149.05
Kupondole	89.75	133.98	84.50	107.53	79.61	79.61
	153.56	287.34	142.23	207.99	99.36	99.36
Sanepa	65.55	81.97	74.10	82.35	79.61	65.55
	81.61	128.11	88.92	129.61	82.80	81.61
Manbhawan	133.27	206.51	130.80	165.61	171.39	130.80
	81.92	128.31	88.92	129.85	132.48	81.92
Sanepa 1	137.94	195.71	124.00	156.94	178.02	124.00
	94.47	147.58	103.74	148.95	165.61	94.47
Talchikhel	140.12	175.58	163.02	175.94	184.66	140.12
	119.57	188.05	133.38	189.48	221.44	119.57
Bhaisipati 1	179.84	246.05	156.30	197.23	229.89	156.30
	329.13	595.61	298.31	436.51	311.02	298.31
Bhaisipati 2	109.92	158.55	100.21	127.19	126.65	100.21
	145.74	320.94	159.09	232.63	127.97	127.97
Bhaisipati 3	81.41	158.41	99.96	127.13	90.47	81.41
	171.04	439.97	218.69	319.80	179.16	171.04
Hariharbhawan	152.68	191.30	177.84	186.10	211.80	152.68
	192.93	309.39	222.30	292.89	285.43	192.93
Gwarko	90.44	113.21	103.74	113.59	126.65	90.44
	131.92	209.42	148.20	211.11	221.44	131.92
Harisiddhi 1	90.44	112.81	103.74	113.15	126.65	90.44
	94.56	147.63	103.74	149.02	179.16	94.56
Hattiban 1	102.93	128.58	118.56	128.94	162.84	102.93
	156.94	248.13	177.84	249.52	247.04	156.94
Hattiban 2	120.76	201.90	127.96	161.89	184.66	120.76

	358.54	673.39	338.93	496.09	387.80	338.93
Sunakothi	156.18	250.28	159.06	200.61	238.94	156.18
	185.27	407.39	202.78	296.56	204.75	185.27
Jwalakhel 1	152.61	191.16	177.84	191.52	202.75	152.61
	169.45	268.66	192.66	270.13	259.83	169.45
Khumaltar 1	581.13	977.66	664.29	536.09	126.65	126.65
	1305.13	2844.2	1472.7	1528.22	259.83	259.83
Kupondole 1	101.59	170.44	107.81	136.71	144.75	101.59
	157.89	347.60	172.54	252.30	153.56	153.56
Bhaisipati 4	94.37	152.03	96.09	121.96	126.65	94.37
	272.20	531.44	266.17	389.48	311.02	266.17
Kupondole 2	131.63	219.69	139.36	176.14	202.75	131.63
	196.20	431.17	214.81	314.19	221.44	196.20
Bishal Bazar Lalitpur	75.61	125.65	79.22	100.86	72.37	72.37
	134.51	296.24	146.66	214.46	102.37	102.37
Nakkhu	78.02	97.04	88.92	97.36	108.56	78.02
	119.42	188.05	133.38	189.48	204.75	119.42
Khokana	102.93	128.58	118.56	128.94	162.84	102.93
	119.42	187.94	133.38	189.35	204.75	119.42
Sanepa 2	101.69	143.36	90.53	115.03	108.56	90.53
	185.27	382.80	190.54	278.66	204.75	185.27
Sanepa Sangrila Housing	123.93	218.68	138.45	175.38	162.84	123.93
	185.27	487.50	242.65	354.88	204.75	185.27

Terzaghi, Vesic, IS code, Hansen, and Bowles methods were applied to determine the bearing capacity of soil at different locations and depths of boreholes (1.5 m and 3 m), and the lowest value of these was used as the safe bearing capacity. The findings show that there is a significant difference in soil strength across different locations.

Minimum bearing capacity at Bagdole varies with depth of 1.5 m and 3 m to 88.57 kN/m as and 149.05 kN/m^2 respectively, and at Bhakundole it varies to 78.06 kN/m as and 106.92 kN/m^2 respectively. Similarly, Balkumari shows an increase from 92.88 kN/m^2 to 132.48 kN/m^2, and Bhaisepati from 90.46 kN/m^2 to 106.92 kN/m^2. The values increase to 163.95 kN/m^{2}at Dhapakhel, which is much better than 96.72 kN/m^{2,} indicating a considerable increase in soil strength with depth. Dhobighat, conversely, exhibits comparatively fewer values, with 88.27 kN/m^2 at 1.5 m and 69.46 kN/m^2 at 3 m, indicating weaker subsurface conditions. Harisiddhi and Hattiban are moderate to high with Hattiban reaching up to 181.82 kN/m^2 at 3 m depth. Jawalakhel and Jhamsikhel have moderate values, with a range of between approximately 78.01 kN/m^2 and 149.05 kN/m^2. Khumaltar experiences a large range, with measurements of 86.47 kN/m 2 at the shallow depth and as high as kN/m^2 at the deeper levels, and in yet another case it is very high in some layers, reaching 259.83 kN/m^2.

In addition, Kupundole varies between 79.61 kN/m^2 and 196.20 kN/m^2 with respect to depth and location. The values of sanepa at shallow depths like 65.55 kN/m^2 are relatively low with a high value of about 81.61 kN/m^2 at 3 m, which is a weak soil. Manbhawan and Talchikhel have moderate to high capacities with Talchikhel having approximate capacities of 221.44 kN/m^2. In certain instances, Bhaisepati1 has very high values up to 298.31 kN/m^2 at 3 m depth. Hariharbhawan registers a value of between 152.68 kN/m^2 to 192.93 kN/m^2 whereas Gwarko and Harisiddhi 1 register values of approximately 90.44 kN/m^2 and 179.16 kN/m^2 respectively. The values in Sunakothi are relatively high (156.18 kN/m^2 to 185.27 kN/m^2). In contrast, Bishal Bazar (Lalitpur) shows lower values, from 72.37 kN/m^2 at 1.5 m to 102.37 kN/m^2 at 3 m, and Nakkhu ranges from 78.02 kN/m^2 to 119.42 kN/m^2. The Ranibhavvan Mars and the Sanepa Sangrila Housing have moderate values of between 90.53 kN/m^2 and 204.75 kN/m^2. In general, the findings indicate that the bearing capacity increases with depth owing to better soil conditions, but there are irregular patterns at certain sites owing to the heterogeneity of the soil. Minimal values help a safe and careful design solution and the bearing capacity of the site might demand soil enhancement or alternative foundation solutions.

Table 5.2: Values of bearing capacity for 2 m× 2 m foundation

Borehole location	Shear Failure Criteria				Settlement Criteria	Minimum value of bearing capacity
	Tergazhi kN/m^2	Vesic kN/m^2	IS kN/m^2	Hansen kN/m^2	Bowles kN/m^2	
Bagdole	95.39	144.98	99.89	120.43	97.37	95.39
	164.87	373.84	207.31	284.89	136.72	136.72
Bhakundole	78.06	90.44	88.92	88.84	109.54	78.06
	106.92	149.81	118.56	143.68	167.11	106.92
Balkumari	104.33	160.43	109.33	131.96	85.20	85.20
	181.35	301.98	167.34	230.03	121.53	121.53
Bhaisipati	90.46	105.02	103.74	103.15	121.71	90.46
	106.92	149.88	118.56	143.75	167.11	106.92
Dhapakhel	106.54	161.49	110.28	132.67	121.71	106.54
	173.78	338.68	187.95	258.21	151.91	151.91
Dhobighat	104.33	140.21	95.55	115.33	85.20	85.20
	69.46	95.94	74.10	92.10	106.34	69.46
Harisiddhi	78.01	90.40	88.92	88.80	97.37	78.01
	94.47	131.94	103.74	126.58	151.91	94.47
Hattiban	106.54	159.86	109.16	131.33	121.71	106.54
	193.05	372.13	206.80	283.98	182.30	182.30
Jwalakhel	95.39	169.84	115.82	139.64	97.37	95.39
	164.87	321.46	178.27	244.97	136.72	136.72
Jhamsikhel	78.01	90.96	88.92	89.36	97.37	78.01
	69.46	97.05	74.10	93.21	106.34	69.46
Khumaltar	104.33	137.35	93.60	112.98	85.20	85.20
	191.73	314.97	174.67	240.03	136.72	136.72

Kupondole	98.40	134.21	91.40	110.45	73.03	73.03
	162.20	268.98	148.85	204.73	91.15	91.15
Sanepa	65.55	76.39	74.10	75.05	73.03	65.55
	81.61	114.71	88.92	110.11	75.96	75.96
Manbhawan	147.32	207.82	142.20	170.51	157.22	142.20
	81.92	114.91	88.92	110.31	121.53	81.92
Sanepa 1	152.57	197.02	134.85	161.61	163.31	134.85
	94.47	131.94	103.74	126.58	151.91	94.47
Talchikhel	140.12	163.29	163.02	160.36	169.39	140.12
	119.57	167.95	133.38	161.05	203.13	119.57
Bhaisipati 1	199.82	248.37	170.50	203.38	210.89	170.50
	351.52	562.54	314.25	430.82	285.31	285.31
Bhaisipati 2	121.02	159.19	108.67	130.80	116.18	108.67
	154.17	300.76	166.64	229.08	117.39	117.39
Bhaisipati 3	89.34	158.77	108.19	130.61	82.99	82.99
	181.42	413.15	229.41	315.11	164.34	164.34
Hariharbhawan	152.68	177.90	177.84	174.70	194.29	152.68
	192.93	275.89	222.30	264.39	261.83	192.93
Gwarko	90.44	105.39	103.74	103.53	116.18	90.44
	131.92	187.09	148.20	179.42	203.13	131.92
Harisiddhi 1	90.44	104.99	103.74	103.13	116.18	90.44
	94.56	132.00	103.74	126.63	164.34	94.56
Hattiban 1	102.93	119.65	118.56	117.52	149.38	102.93
	156.94	221.33	177.84	212.13	226.61	156.94
Hattiban 2	133.64	203.32	139.22	166.74	169.39	133.64
	384.50	638.30	357.98	490.11	355.74	355.74
Sunakothi	173.68	252.76	173.61	206.92	219.18	173.61
	196.78	382.97	212.88	292.30	187.82	187.82

Jwalakhel 1	152.61	177.76	177.84	174.56	185.99	152.61
	169.45	239.63	192.66	229.66	238.35	169.45
Khumaltar 1	592.24	919.63	672.64	692.08	116.18	116.18
	1322.69	2552.7	1484.6	1759.58	238.35	238.35
Kupondole 1	112.03	171.29	117.04	140.65	132.78	112.03
	167.25	326.07	180.86	248.52	140.87	140.87
Bhaisipati 4	103.90	152.65	104.20	125.42	116.18	103.90
	290.72	501.94	280.39	384.41	285.31	280.39
Kupondole 2	145.91	221.44	151.78	181.50	185.99	145.91
	208.56	405.62	225.64	309.74	203.13	203.13
Bishal Bazar Lalitpur	82.84	125.82	85.66	103.57	66.39	66.39
	142.11	277.34	153.50	211.11	93.91	93.91
Nakkhu	78.02	90.34	88.92	88.74	99.58	78.02
	119.42	167.95	133.38	161.05	187.82	119.42
Khokana	102.93	119.65	118.56	117.52	149.38	102.93
	119.42	167.84	133.38	160.94	187.82	119.42
Sanepa 2	111.78	143.81	98.08	118.23	99.58	98.08
	196.78	359.85	200.03	274.66	187.82	187.82
Sanepa Sangrila Housing	247.14	435.96	302.15	355.20	149.38	149.38
	196.78	458.28	254.75	349.78	187.82	187.82

Terzaghi, Vesic, IS code, Hansen, and Bowles methods were applied to determine the bearing capacity of soil at different locations and depths of boreholes (1.5 m and 3 m) and the lowest of these was taken as the safe bearing capacity. The findings indicate that there is a discernible difference in places due to disparities in subsurface soil conditions.

The bearing capacity at Bagdole is 95.39 kN/m^2 at a depth of 1.5 m to 136.72 kN/m^{2}at 3 m. Similarly, Bhakundole shows an increase from 78.06 kN/m^2 to 106.92 kN/m^2, and Balkumari from 85.20 kN/m^2 to 121.53 kN/m^2. The values of

Bhaisipati are typically moderate to high, i.e., 90.46 kN/m^2 at low depth and 106.92 kN/m^2 at 3 m, and at some points it is as high as 285.31 kN/m^2 at deeper depths. The value of Dhapakhel at 106.54 kN/m^2 increases to 151.91 kN/m^2 with depth, which shows that soil strength is improved with depth. On the contrary, Dhobighat has relatively less values, which go down to 85.20 kN/m^2 at 1.5 m and 69.46 kN/m^2 at 3 m, indicating weak soil strata.

Harisiddhi and Hattiban exhibit moderate to high bearing capacities, where Hattiban has the maximum bearing capacity of kN/m^2 and even 355.74 kN/m^2 at greater depths, which is a sign of dense soil. Jawalakhel has bearing capacity of 95.39 kN/m^2 at 1.5 m and 136.72 kN/m^2. Khumaltar is very variable, ranging between approximately 85.20 kN/m^2 at shallow depth, 136.72 kN/m^2 at moderate depth, and in one case to very high values of 238.35 kN/m^2 and even higher (IS method) a pointer of very dense strata at depth. Kupundole varies between rather low levels of 73.03 kN/m^2 and high levels of 203.13 kN/m^2.

Sanepa demonstrates relatively low bearing capacities, with values such as 65.55 kN/m^2 at 1.5 m and 75.96 kN/m^2 at 3 m, showing weak soil that might need ground improvement. The values of Manbhawan and Talchikhel are moderate, with Talchikhel attaining approximately 119.32 kN/m^2. Hariharbhawan results in an improvement of 152.68 kN/m^2 to 192.93 kN/m^2, whereas Gwarko and Harisiddhi 1 are between about 90.44 kN/m^2 and 164.34 kN/m^2. The values of Sunakothi are comparatively high, ranging from 173.61 kN/m^2 to 187.82 kN/m^2. Conversely, Bishal Bazar (Lalitpur) demonstrates lower figures, with 66.39 kN/m^2 at shallow depth, to 93.91 kN/m^2 at 3 meters, and Nakkhu with 78.02 kN/m^2 to 119.42 kN/m^2. Ranibhavvan Mars and Sanepa Sangrila Housing have moderate to high values, as the values are between approximately 98.08 kN/m^2. and 187.82 kN/m^2.

In general, the findings reveal that bearing capacity tends to increase with depth through enhanced soil properties, but there are inconsistent patterns at certain places, caused by the soil heterogeneity. The use of the minimum of all procedures will be a safe and conservative design method, and sites with low bearing capacity should be thought over for the improvement of the soil or other forms of foundations.

5.1.2 Numerical modeling

Shear failure criteria of narrow footing width regulate the permissible bearing capacity of a footing. According to the suggestion of Eurocode 7, the acceptable settlement of a shallow foundation is 25mm in general; hence, the 25mm of displacement is applied to the footing plate, which is at a depth of 1.5m beneath the soil surface. The materials used are assumed to behave in a drained condition during the bearing capacity calculations. The standard boundary conditions are imposed when the standard model of simulation is developed with all material sets included in it. This will automatically create a complete fixity at the bottom of the geometry ($u_x = u_y = u_z = 0$) and roller boundaries at the vertical sides of the YZ planes ($u_x = 0$; u_y and u_z are free) and XZ- planes ($u_y = 0$; u_x and u_z are free). (Hakro et al., 2022)explains that models should be sufficiently large to ensure that soil deformation doesn't reach the boundaries. Similarly, other models using PLAXIS software have used sufficient sizes (e.g., 10x10x10 m) to avoid any boundary influence (Raheem, 2022). For this research, a lateral length of 5 times the width of the footing from the center and a depth of 6 m to ensure the stresses below the footing were captured, which is (1.5-2) times the footing depth. The model of the soil and the footings is done using 15-noded axisymmetric elements in FEM. In this case, settlement criteria were used to assess the value of the PLAXIS 3D, and compared to the Bowles formula.

Table 5.3: PLAXIS 3D parameters

Name	**Soil**	**Footing**	**Unit**
Material model	Mohr coulomb	Linear elastic	**-**
Drainage type	Drained	Non porous	-
Unit weight	16.69	24	kN/m^3
Young's Modulus	4200	2.35×10^4	kN/m^2
Poisson's Ratio	0.3	0.2	-
Cohesion	2	-	kN/m^2
Friction angle	29.86	-	Degree
Mesh type	Fine	Fine	-
K0 Determination	Automatic	Automatic	-

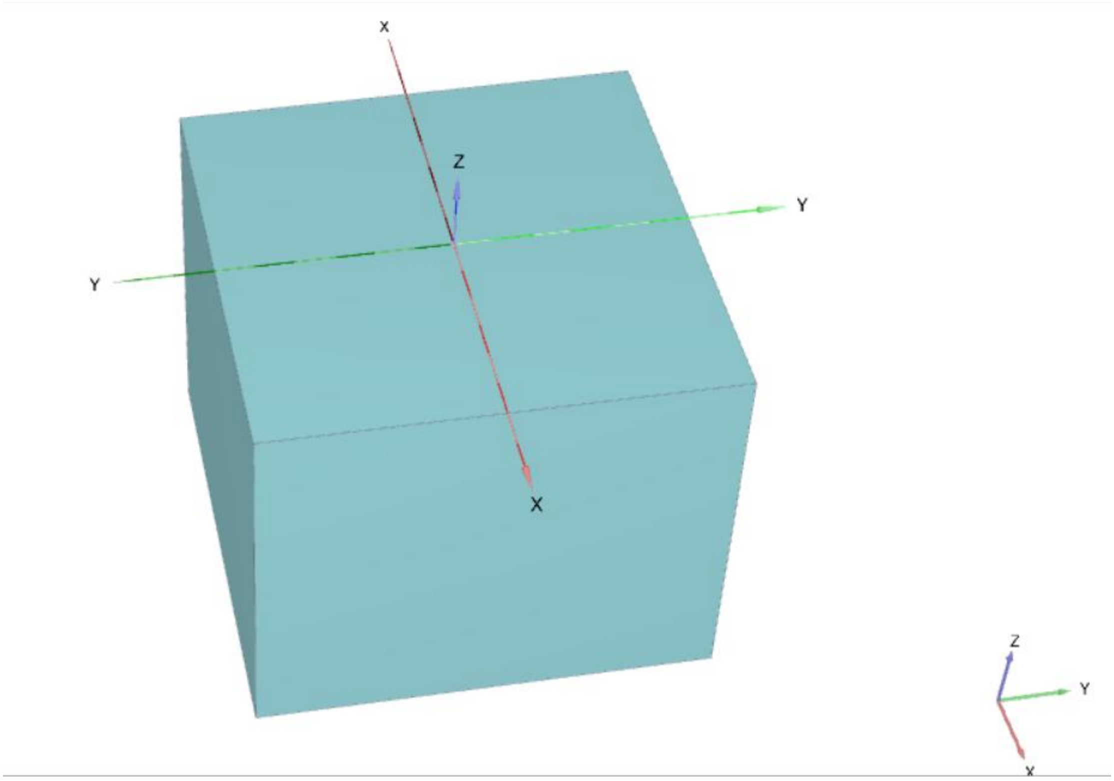

Figure 5.1: Soil modelling

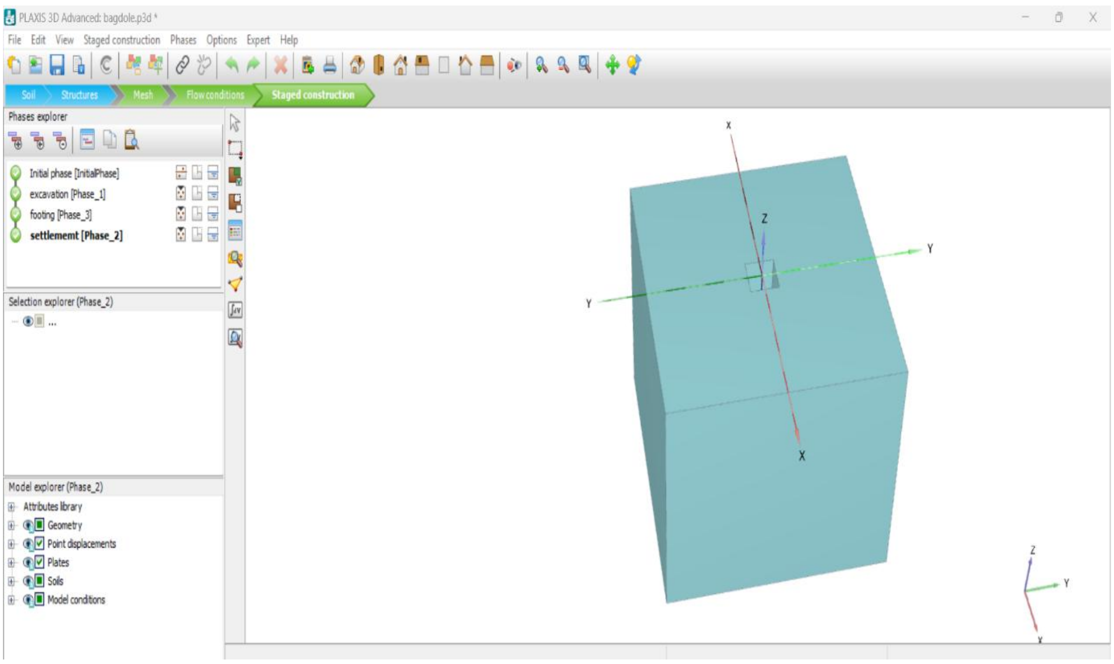

Figure 5.2: Staged construction

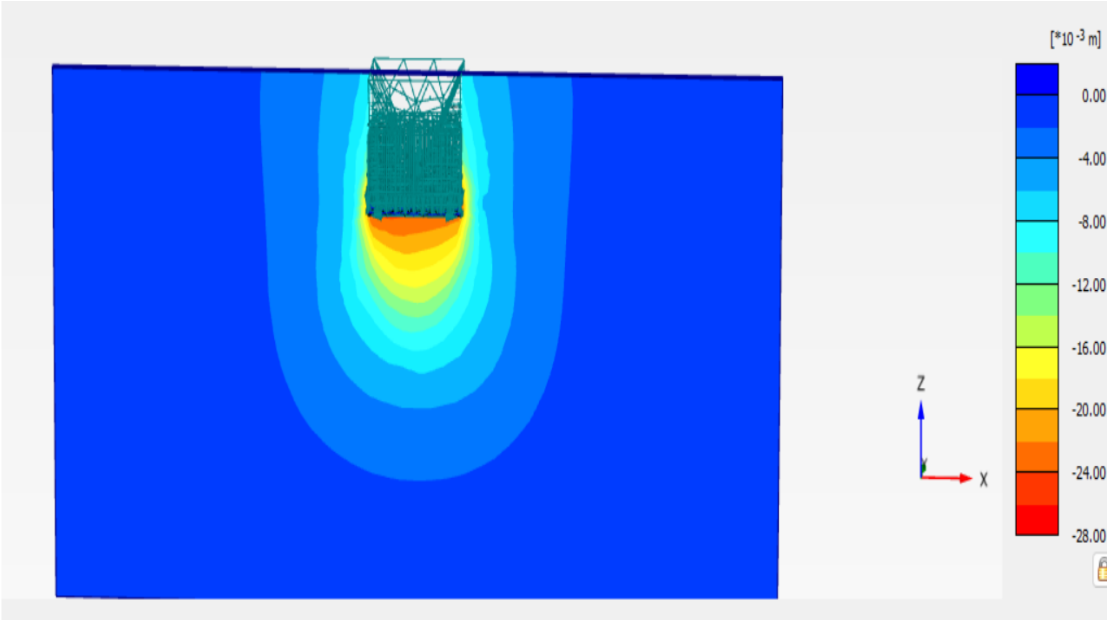

Figure 5.3: Displacement of footing

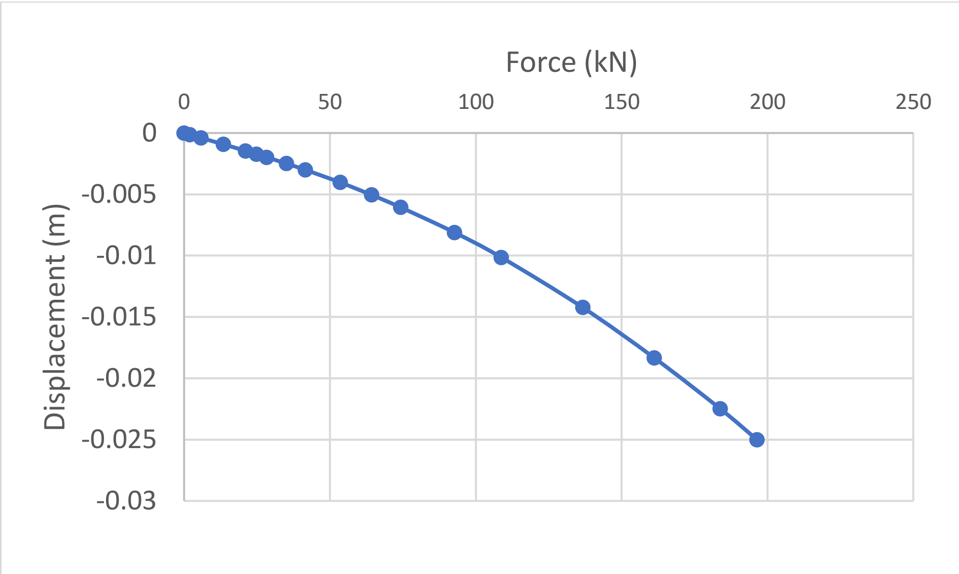

Figure 5.4: Load Displacement curve

For 140 points, bearing capacity was found and compared with the bearing capacity given by Bowles, which is shown below:

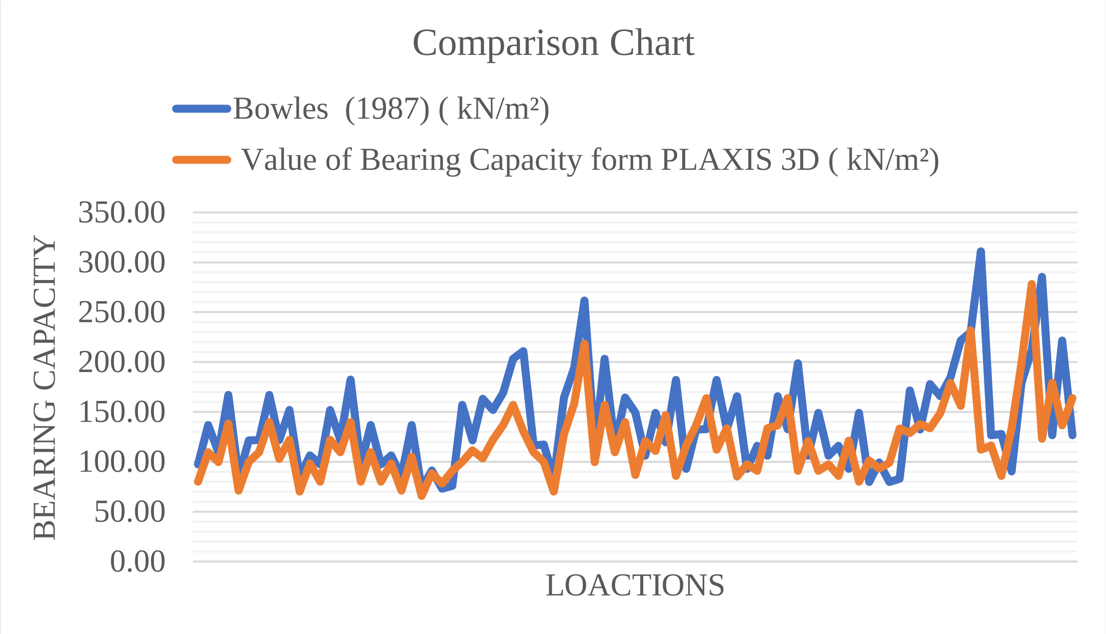

Figure 5.5: Comparison between PLAXIS 3D and Bowles' formula

Here, the PLAXIS 3D value was evaluated using settlement criteria, and it was compared with the Bowles' formula. The tentative PLAXIS 3D graph shows a pattern similar to the Bowles equation. The tentative difference between these values was in the range of 20%.

5.1.3 GIS Spatial Analysis

According to the analytical model, GIS map was made with ArcGIS. The bearing capacity map was created by the IDW method to visualize the spatial variation of bearing capacity in Lalitpur Metropolitan City. Footing size and depth were used to produce different maps. Bearing capacity values obtained through analysis by SPT were spatially combined with the help of GIS to create a thematic map showing the difference of soil bearing capacity throughout the research area. The resulting GIS map is a clear demonstration of the spatial distribution of bearing capacity, indicating locations with various bearing capacities.

According to the results, the higher the SPT N-value, the higher the bearing capacity zone and the lower the N-values, the lower the load-bearing capacity. Shifts in soil properties including density, friction angle, and depth are the main factors that affect the change in bearing capacity. Areas that have relatively thick deposits of soil and also have higher values of the friction angle are more resistant to shear failure and hence can be used in constructing foundations.

The visualization created by GIS makes it easy to identify critical areas where the bearing capacity is not adequate, which can be improved either by ground or special foundation design. On the other hand, field with greater bearing capacity is appropriate to sustain heavier structural loads without high settlement or failure rates.

All in all, GIS map can be a useful tool in spatial decision-making, allowing the engineers and planners to evaluate the suitability of the site and optimize the design of the foundation according to the localized soil conditions. The combination of GIS and geotechnical data makes the analysis more efficient and more accurate.

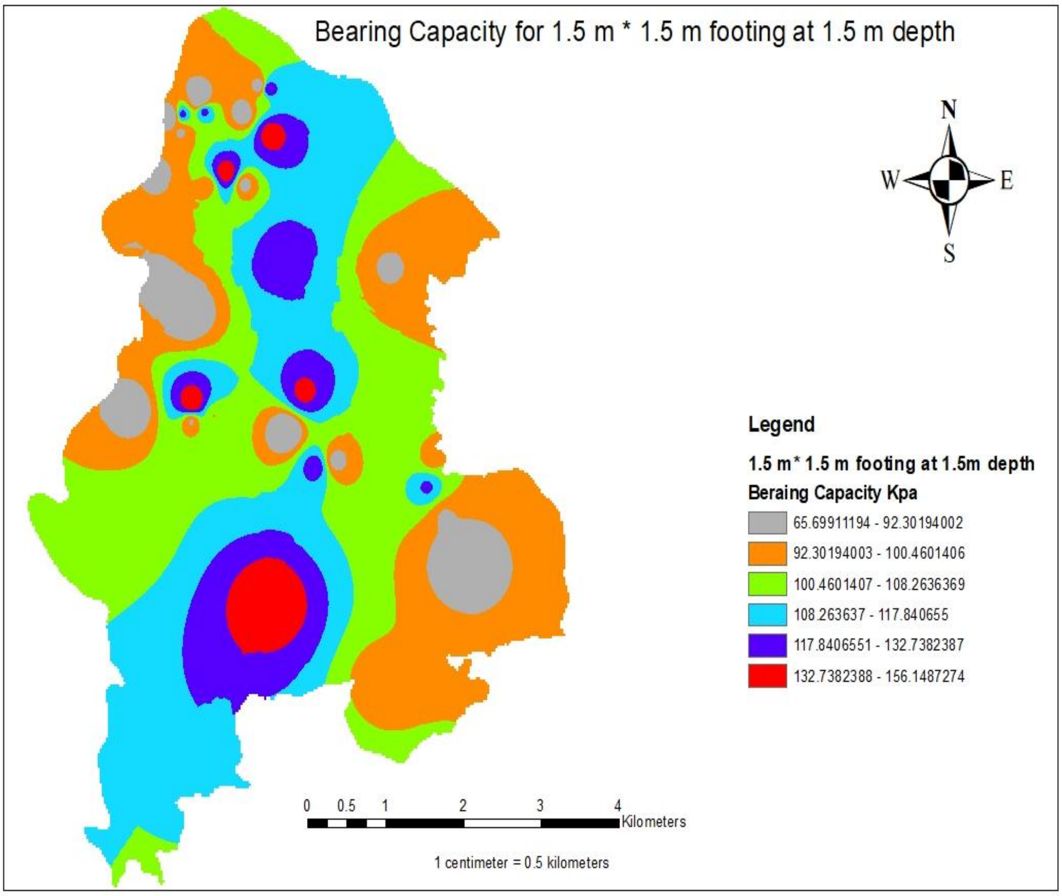

Figure 5.6: Bearing capacity map for 1.5 m × 1.5 m at 1.5 m

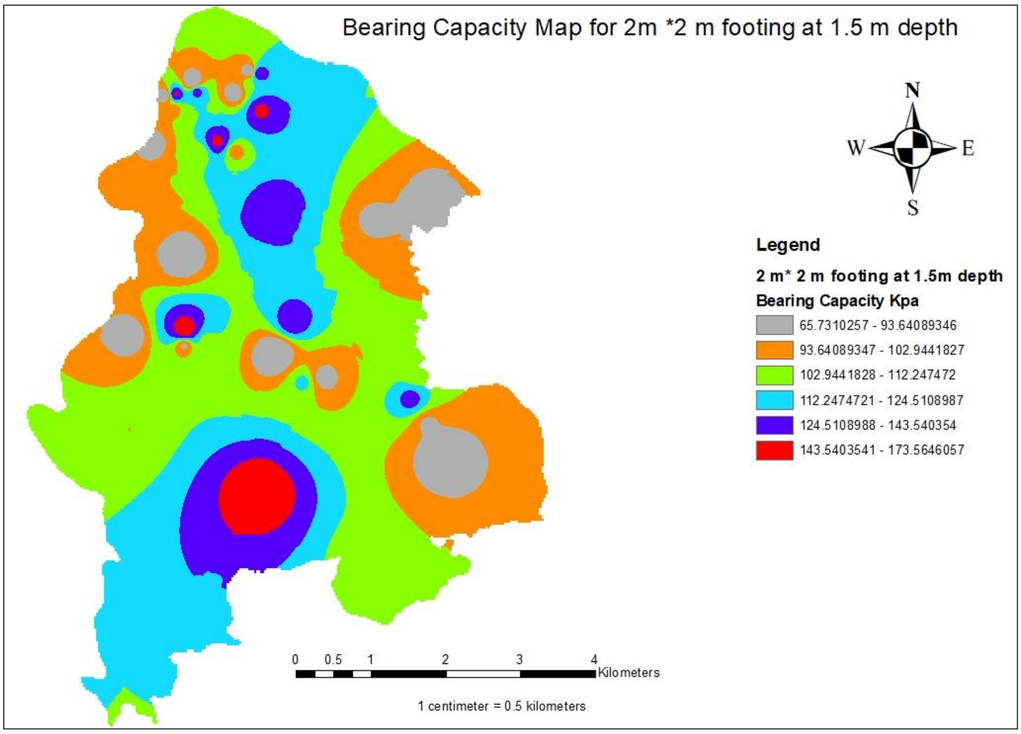

Figure 5.8: Bearing capacity Map for 2 m × 2 m at 1.5 m

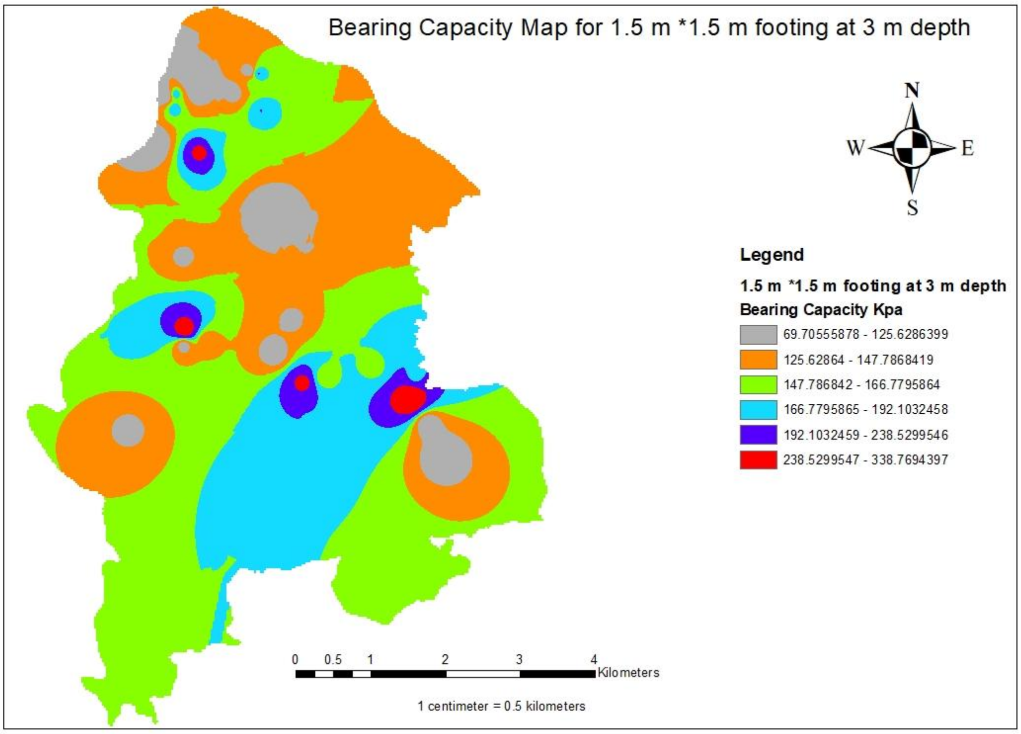

Figure 5.7: Bearing capacity map for 1.5 m × 1.5 m at 3 m

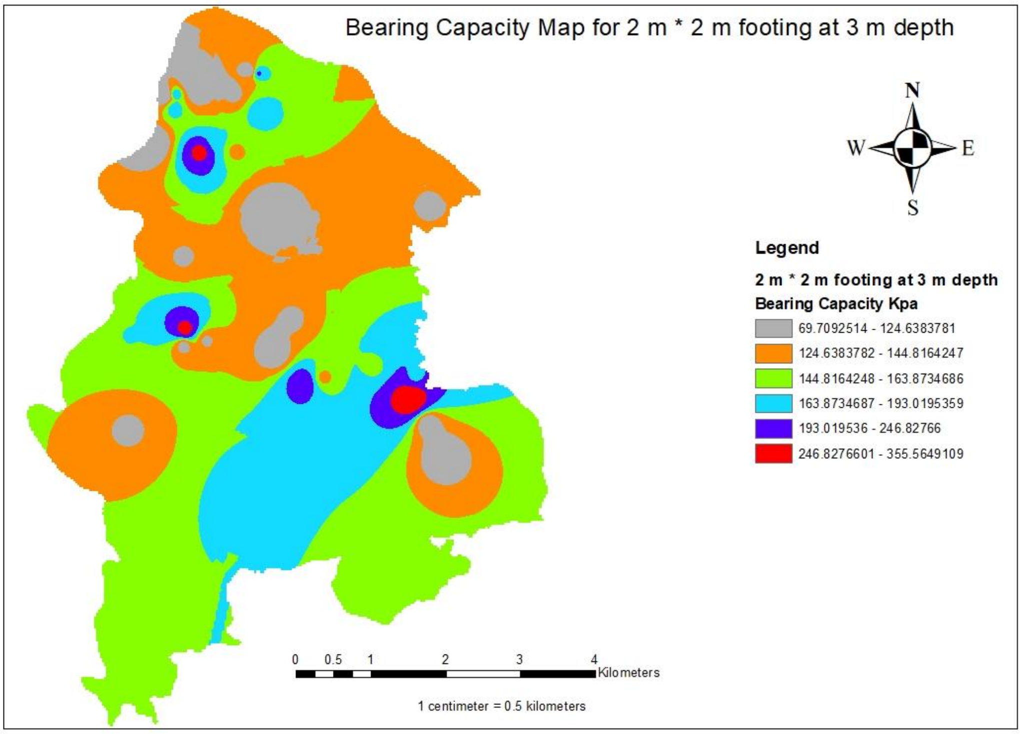

Figure 5.9: Bearing capacity map for 2 m × 2 m at 3 m

The prepared bearing capacity zonation maps that are based on different footing sizes (1.5 m x 1.5 m and 2 m x 2 m) and the embedment depths (1.5 m and 3 m) show the spatial distribution of the allowable bearing capacity in the study area. The findings indicate a decisive impact of both depth and footing size, as well as regional differences in soil conditions.

The zone bearing capacity map of the 1.5 m x 1.5 m footing at an embedment depth of 1.5 m reflects the spatial distribution of the maximum allowable bearing capacity of the study area, which would indicate the effects of shallow soil conditions on foundation performance. The low bearing capacity zones primarily dominate the northern part of the city, such as Sanepa, Kupundole, Jhamsikhel, and Dhobighat. In the meantime, the central region, which includes Jawalakhel, Manbhawan, Gwarko, and Talchikhel, has moderate values of bearing capacity, showing fairly better soil conditions. The southern and southwestern regions, such as Khumaltar, Sunakothi, and sections of Bhaisipati, have higher values of bearing capacity, which means that the soils are denser; but these are seen to be of localized pockets and not continuous zones, which is a sign of spatial variability in the conditions of the subsurface.

The zonation map of the bearing capacity of the 2 m x 2 m footing, at 1.5 m embedding depth, shows how the allowable bearing capacity varies spatially throughout the study region, and how the size of the footing affects the soil response. The northern zone, which comprises Sanepa, Kupundole, Jhamsikhel, and Dhobighat, like in the smaller footing case, is predominantly low bearing capacity, indicating the weaker near-surface soils. The main part, which consists of Jawalakhel, Manbhawan, Gwarko, and Talchikhel, has moderate values of bearing capacity. Southern and southeastern areas such as Khumaltar, Hattiban, Harisiddhi, and portions of Sunakothi exhibit relatively higher values of bearing capacity and are denser and more competent soils, but are not continuous, as they are spatially heterogeneous.

The spatial variation of allowable bearing capacity in the study area is depicted on the bearing capacity zonation map of the 1.5 m × 1.5 m footing at an embedment depth of 3 m, which shows the influence of an increase in depth on the soil behavior. The values of bearing capacity in the northern region, such as Sanepa, Kupundole, Jhamsikhel, and Dhobighat, are relatively lower, but there is an improvement compared to shallow depth as the confinement is greater. The central region, which includes Jawalakhel, Manbhawan, Gwarko, and Talchikhel, has moderate to high bearing capacity values, which implies the existence of more compact layers of soil at depth. The southern and southeastern parts of the country, such as Khumaltar, Sunakothi, Hattiban, and Harisiddhi, give high bearing capacity values that are often over 200 kPa, indicating the more compact and stable nature of the subsurface strata, but the high bearing capacity values are still observed to be localized, similar to the variability in the soil conditions. The findings indicate that the bearing capacity is considerably increased with the depth of embedment throughout the study region. The integration of geological data with the bearing capacity zonation maps provides a clear understanding of the spatial variability of allowable bearing capacity across the study area. The results indicate that the distribution of bearing capacity is strongly controlled by the underlying geological formations, reflecting variations in lithology, soil type, and degree of compaction.

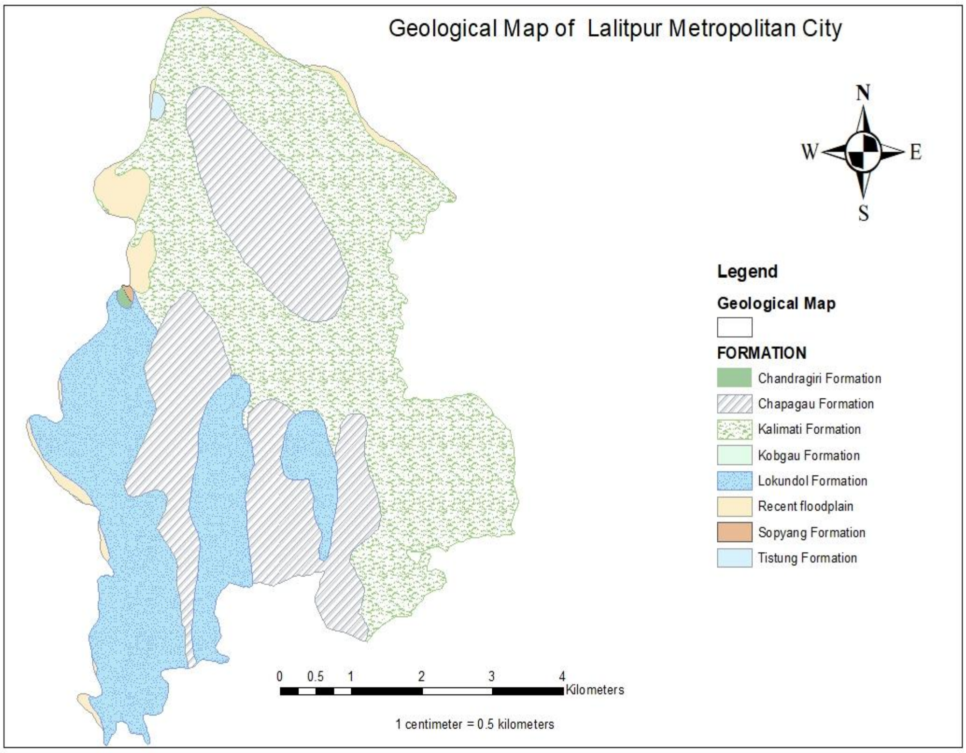

Figure 5.10: Geological map of Lalitpur Metropolitan City

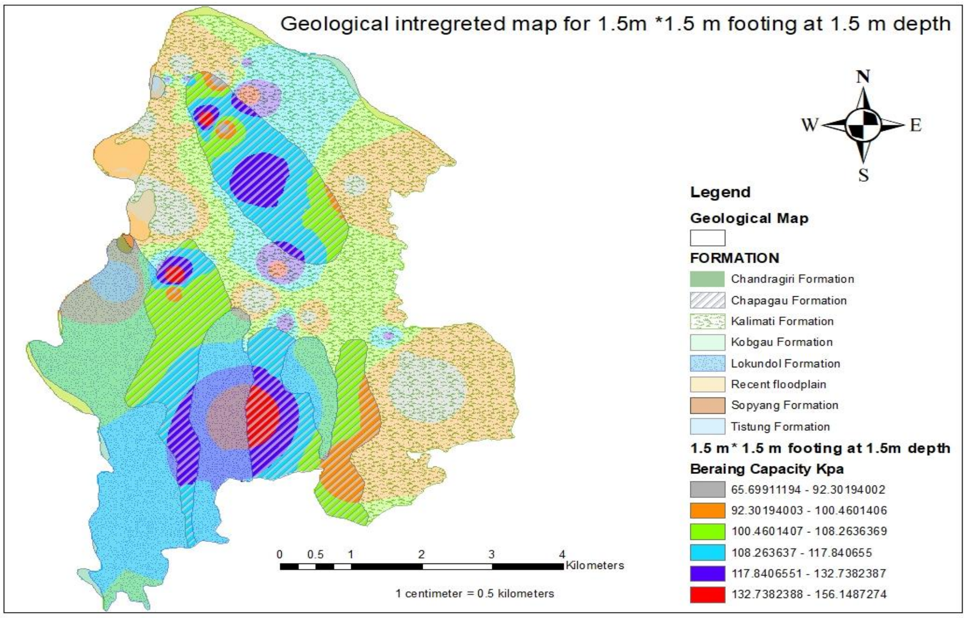

Figure 5.11: Geological integrated map for 1.5 m× 1.5 m footing at 1.5 m depth

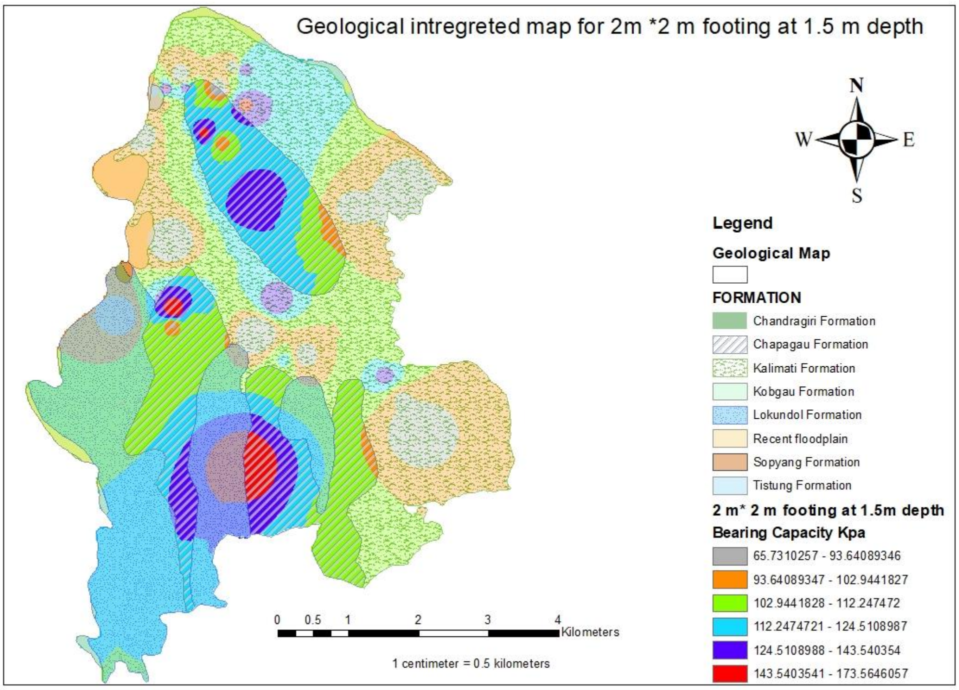

Figure 5.12: Geological integrated map for 2 m× 2 m footing at 1.5 m depth

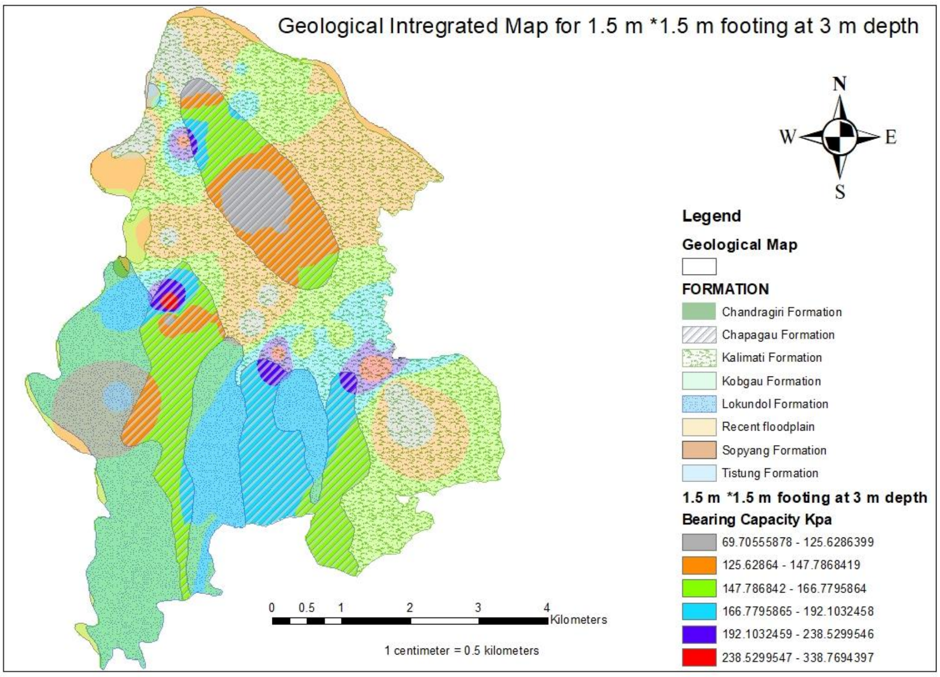

Figure 5.13: Geological integrated map for 1.5 m× 1.5 m footing at 3 m depth

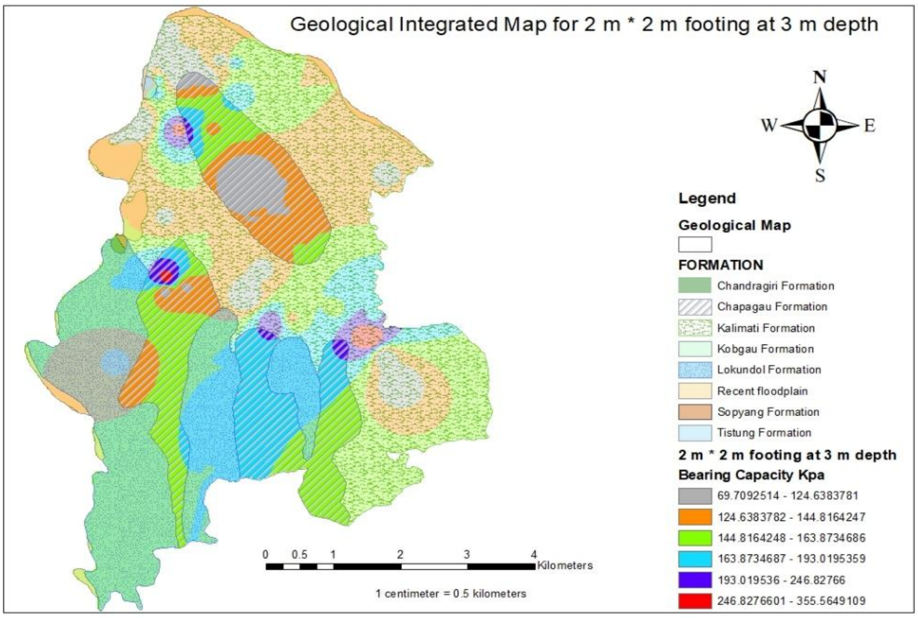

Figure 5.14: Geological integrated map for 2 m× 2 m footing at 3 m depth

Geologic map of Lalitpur Metropolitan City indicates that geological formation of the study area consists of heterogeneous lithology fluvio-lacustrine and rock formations, such as the Lukundol Formation, Chapagaon Formation, Kalimati Formation, Tistung Formation, Chandragiri Formation, Sopayang Formation, and recent floodplain deposits, and this lithological variability The bearing capacity zonation plots constructed on various foundation sizes and embedment depths when plotted alongside geological map of Lalitpur Metropolitan City clearly show the combined effect of foundation geometry and lithological variation on soil performance. In this instance, the lower bearing capacity areas are mainly related to the Lukundol Formation, Kalimati Formation, and recent floodplain deposits, which are fine-grained silty clay and alluvial deposits with high compressibility and low shear strength. Conversely, there are moderate to rather higher values in regions that are related to the Chapagaon Formation and localized Tistung and Chandragiri Formations, as indications of coarse or more competent materials.

When the same footing size is used, but the depth is greater (1.5 m × 1.5 m at 3 m depth), the bearing capacity is much more enhanced, with values of about 338 kPa, showing that soil conditions are better at greater depths. This rise is especially pronounced in locations that are likely to be covered with Chapagaon terrace deposits because coarse-grained gravel and sand layers are more likely to be stronger, and even in the areas dominated by Lukundol, the deeper layers are characterized by a relatively better bearing capacity because of the enhanced compaction and consolidation.

As the size of the footing is changed to 2 m x 2 m at a depth of 1.5m, the bearing capacity of the footing is slightly better than the smaller footing. This is explained by the fact that the load is distributed in a more extensive area, yet the spatial distribution is similar, with smaller values in fine-grained formations (Lukundol, Kalimati, and floodplain deposits) and greater values in coarse-grained Chapagaon deposits and stronger formations.

Maximum values of bearing capacity can be seen in a 2 m x 2 m footing at 3 m. Here, much of the study area is located in high bearing capacity areas, especially those related to Chapagaon Formation and competent formations like Tistung and Chandragiri, where the combination of greater footing size and deep embedment leads to the optimum load-bearing conditions. The deeper layers of soil have a comparatively greater resistance than the shallow conditions even where the soils are underlain by finer deposits.

In general, the findings strongly suggest that bearing capacity increases as footing size and depth, mainly determined by geological structures. Compressible and fine-grained formations are always associated with low bearing capacity, and coarse-grained terrace deposits and rock competence formations are always associated with high bearing capacity, supporting the prevailing role of lithology in the performance of foundation in the study area.

5.1.4 Reliability analysis

The variability and uncertainty in bearing capacity and the effects of the variability on the foundation reliability were evaluated by the probabilistic analysis based on the Monte Carlo simulation technique. To introduce randomness in soil parameters, the simulation used various values of the coefficient of variation (COV) of bearing capacity, ranging between 0.2 and 1.0, and the load applied was considered to be normally distributed with a mean of 458.52 kN and a coefficient of variation of 0.39. Each borehole location and footing condition was simulated 10,000 times. The chosen limit state function was $g=q_u \times A-Q$, with failure at $g=0$.

The analysis was carried out for four footing cases: 1.5 m depth with 2.25 m^2 footing area [1.5(2.25)], 1.5 m depth with 4 m^2 footing area [1.5(4)], 3 m depth with 2.25 m^2 footing area [3(2.25)], and 3 m depth with 4 m^2 footing area [3(4)]. Results are given in terms of probability of failure (P_f) and reliability index(β).

For 1.5(2.25), the outcomes show that the reliability performance is not so good. The average probability of failure increases from 0.1705 at COV = 0.2 to 0.3622 at COV = 1.0. In line with this, the reliability index goes down to 0.3606 as compared to 1.3291. This high degree of decrease in reliability as the variability increases underscores the weakness of shallow foundations in smaller footing areas in uncertain soil conditions. The cases of failure are also widely varied, which proves the outcome of the spatial unevenness in the various locations of boreholes.

However, 1.5(4) case has better reliability because of the increased area of the footing. The probability of failure has an average of 0.0108 at COV = 0.2 to 0.2562 at COV = 1.0, and the reliability index is lowered to 2.7905 to 0.6596. The bigger footing area enhances the resisting capacity thus minimizing chances of failure. Nonetheless, the effect of growing variability is observable, with reliability declining as the COV values increase.

In case 3(2.25) of a deeper foundation, the finding is that there is a distinct improvement in the outcome of the case in comparison to the shallow depth of the same footing size. The mean probability of failure is 0.0469 at COV = 0.2 and 0.3083 at COV = 1.0, and the reliability index is 1.8967 and 0.5034, respectively. This shows that the deeper the embedment, the higher the mobilization of soil strength and the better the foundation performance, but the system is sensitive to variability.

The case of 3(4) has the highest performance compared to all other scenarios. The probability of failure is very low at the average of 0.0015 when COV = 0.2, and it reaches 0.2251 when COV = 1.0. The reliability index in turn, drops to 0.7572. This configuration offers the highest resistance and the lowest probability of failure, and is therefore the most reliable under uncertain soil conditions due to the combination of increased depth and increased area of footing.

On the whole, the findings indicate that there is a steady trend when the coefficient of variation of the bearing capacity is large; the probability of failure is large, and the index of reliability is low. This is a direct indication of the effect of soil uncertainty on foundation performance. Moreover, the size of footing and the depth of embedment have a great impact on reliability, as large and deep foundations perform better.

The findings also indicate the existence of spatial variation in the properties of the soil because certain locations of boreholes always have lower reliability than others. This underlines the need to have probabilistic approaches in geotechnical design to take more care of uncertainty and provide safer and more cost-effective foundation solutions.

Overall, the Monte Carlo simulation indicates that foundation reliability is very sensitive to soil variability, and the expansion of footing size and depth is a viable approach to alleviate the risk of failure. The 3 m depth with the 4 m2 footing area is the most reliable condition analyzed, and the depth of 1.5 m with 2.25 m 2 footing area is the most critical case.

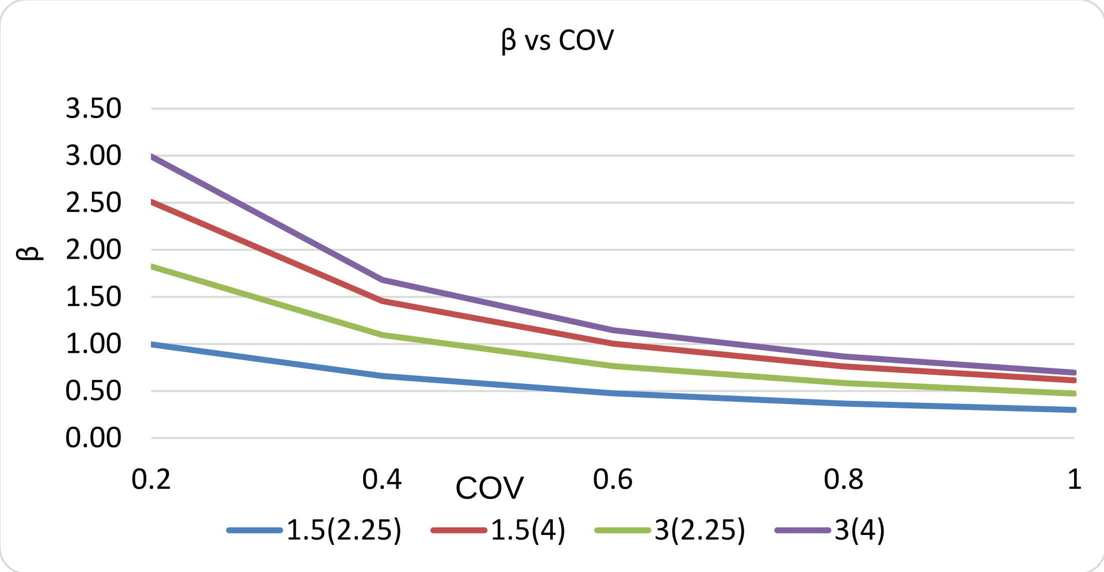

Figure 5.15: β vs COV

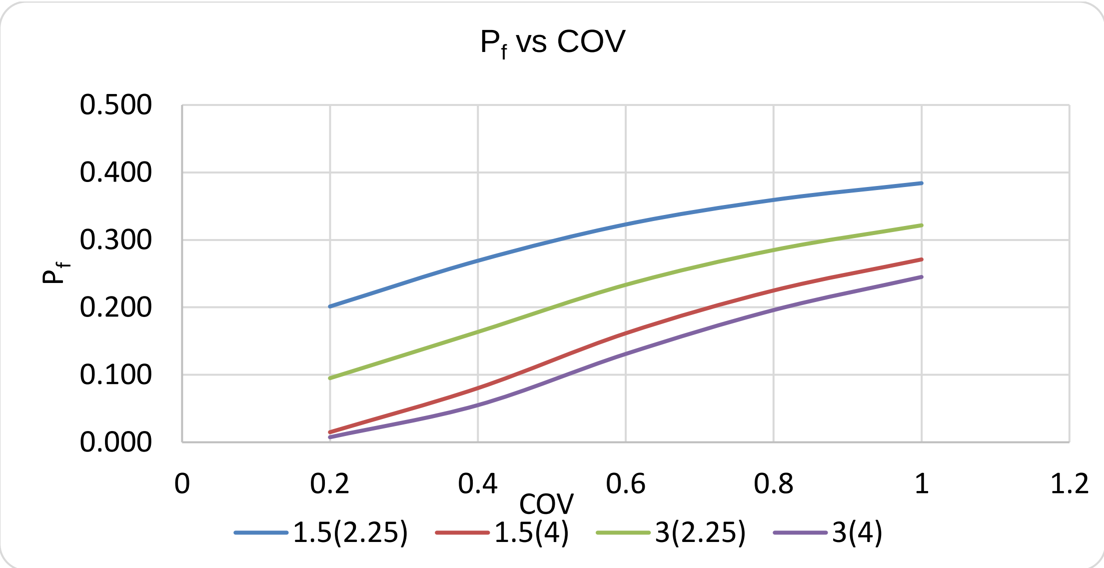

Figure 5.16: P_f vs COV

5.1.5 Validation of data

5.1.5.1 Validation by laboratory test

In order to measure the extent of the accuracy of the bearing capacity zonation map developed, a validating study was conducted with the help of laboratory test data that includes Index and Engineering properties of soil collected in three representative sites of Lalitpur Metropolitan City: Chakupat, Kumaripati, and Jawalakhel. These sites were chosen to depict various soil conditions in the study area.

On the site, soil samples were subjected to laboratory testing to obtain the most important engineering parameters, such as unit weight, Atterberg limits, and shear strength parameters (cohesion and angle of internal friction). Shear strength of cohesive soil at Chakupat was calculated based on the test of unconfined compressive strength (UCS), and at Kumaripati and Jawalakhel, the direct shear test. UCS test was done for cohesive soil, and the Direct Shear Test was done for cohesionless soils. The classical bearing capacity equations, including Tergazhi, Hansen, Vesic, and IS, were then used to calculate the minimum bearing capacity based on these parameters. The values of the minimum bearing capacity calculated were compared to the values calculated in the GIS-based zonation map at the corresponding points. According to the IS classification, the soil at Chakupat is MH, and the soils at Jawalakhel and Kumaripati is classified as SM, meaning that the soils are silty sand with low plastic fines.

Table 5.4: Soil Parameters from lab test

	Jawalakhel	Kumaripati	Chakupat
Cohesion (kN/m2)	0.27	1.74	39.8
Angle of friction (Degree)	29.25	28.4	-
Bulk Density (kN/m3)	18.34	17.46	17.79
Water Content (%)	21.47	25	26.6
Specific Gravity	2.59	2.51	2.49
Liquid Limit (%)	28.5	28.3	52
Plastic Limit (%)	23.75	23.4	31.27
Plasticity Index (%)	4.75	4.9	20.73

Table 5.5: Bearing Capacity % difference for 2.25 m^2 footing at 1.5 m depth

Location	Minimum BC (kpa)	Predicted from GIS (kpa)	Difference in %
Chakupat	102.67	113.27	10.31
Kumaripati	114.34	122.91	7.49
Jawalkhel	96.31	108.94	13.10

The minimum bearing capacities of the 1.5 m x 1.5 m footing were 102.52 kPa (Chakupat), 114.34 kPa (Kumaripati), and 96.31 kPa as determined in the laboratory, whereas the GIS predicted values were 113.27 kPa, 122.91 kPa, and 108.94 kPa. The variations in percentages were between about 7.5 % to 13.1%.

Table 5.6: Bearing Capacity % difference for 4 m^2 footing at 1.5 m depth

Location	Minimum BC (kpa)	Predicted from GIS (kpa)	Difference in %
Chakupat	102.67	117.57	14.50
Kumaripati	123.13	131.6	6.87
Jawalkhel	105.30	115.71	9.88

Likewise, in the case of the 2 m × 2 m footing, the values obtained in the lab were 102.52 kPa, 123.13 kPa, and 105.3 kPa, whereas the GIS predicted values were 117.57 kPa, 131.60 kPa, and 115.71 kPa. The variations in percentages were between about 6.87 % to 14.5%.

In general, the difference between the laboratory-determined and the GIS-predicted values of the bearing capacity is within a range of -3.2% to 15, which means that there is good agreement between the two methods. This difference is explained by the natural heterogeneity of soil and disparities between point measurements in laboratories and spatial interpolation methods. Laboratory tests are conditions at a given point, whereas GIS interpolation techniques are estimates of values on a continuous surface, which may tend to smooth out local variations.

This is what one should expect in geotechnical engineering practice. It has been demonstrated in the past that the characteristics of soils do vary significantly in space and that the models used to predict are normally characterized by uncertainty, owing to the small sample size and the natural heterogeneity of soils (Phoon & Kulhawy, 1999). Equally, the classical geotechnical literature explains that bearing capacity values obtained by various means can be different because of the analysis assumptions and differences in soil conditions (Bowles, 1996).

Hence, the observed difference of about 15 percent between the values obtained in the laboratory and those obtained in GIS is acceptable and within the range that is often experienced in geotechnical engineering practice. This validates the fact that the GIS-based bearing capacity zonation map is a good representation of spatial variation in bearing capacity within the study area.

5.1.5.2 Validation by Literature

The precision of the constructed bearing capacity zonation map was determined by comparing the obtained data with previous studies conducted in the Kathmandu Valley and with observed values for the geological formations of Lalitpur Metropolitan City.

The values of bearing capacity generated in the present study are highly comparable with those of the past study by (K. C. & Raj Dahal, 2020) and (Danai & Acharya, 2018). The range of shallow bearing capacity (65 to 173 kPa) obtained in the given research is close to the range of the previous ones, i.e., 61 to 151 kPa (K. C. & Raj Dahal, 2020) and 50 to 150 kPa (Danai & Acharya, 2018). This fact of similarity proves that the current work gives a realistic representation of near-surface foundation conditions.

Regarding depth, the bearing capacity increases dramatically in the current study, as high as 355 kPa at 3 m depth. The trend is in line with the results of (Danai & Acharya, 2018), who found up to 200 300 kPa of bearing capacity at deeper levels via analytical and numerical (PLAXIS) techniques. The increase with depth observed is an indication of better soil density and strength at deeper layers, and this is an established geotechnical behaviour.

The zonation in terms of space as evident in this paper using GIS also follows the past (K. C. & Raj Dahal, 2020) documented the difference between bearing capacity in the valley, determined through SPT data, and (Danai & Acharya, 2018) detected the zonation patterns by both analytical and numerical methods. Likewise, the current research indicates that there is a considerable difference throughout the study area, which proves that the distribution of bearing capacity is not uniform.

Additional validation was done by overlaying the zonation maps on the geological structure of the study area. The findings show that low bearing capacity areas are largely related to the Kalimati Formation, Lukundol Formation and the recent floodplain deposits that are composed of fine-grained soils such as silty clay and alluvial sediments. It is known that these soils are highly compressible and low in shear strength, resulting in reduced bearing capacity(Rizal & Paudyal, 2023); (Paudel & Sakai, 1970).

Conversely, the regions of higher bearing capacity are associated with Chapagaon, Tistung, and Chandragiri Formations that are made of coarse-grained rocks, including sand, gravel, and cobbles. Previous geological research has reported that these materials have better engineering properties and higher load-bearing capacity (Yoshida & Igarashi, 1984).

In general, the high correlation between the current study and the past studies, as well as the geological conditions, indicates that the bearing capacity zonation map developed is realistic and reliable in depicting the condition of the subsurface in Lalitpur Metropolitan City.

5.2 Discussion

The present study integrates deterministic bearing capacity analysis, spatial zonation mapping, and probabilistic reliability assessment to evaluate the geotechnical performance of shallow foundations across the study area. The combined interpretation of these approaches provides a comprehensive understanding of soil behavior, spatial variability, and associated risks.

The results of this study clearly indicate that the bearing capacity of soil within Lalitpur Metropolitan City varies significantly due to heterogeneous subsurface conditions. The deterministic analysis shows that bearing capacity ranges from approximately 65 kPa in weaker zones to values exceeding 300 kPa in stronger zones, reflecting the diverse nature of soil deposits across the study area. A consistent trend observed throughout the analysis is the increase in bearing capacity with depth, as most locations show higher values at 3 m depth compared to 1.5 m depth. This behavior can be attributed to the presence of denser and more compact soil layers at greater depths, which provide higher resistance to shear failure. However, some locations exhibit irregular trends, indicating the presence of layered or variable soil profiles, highlighting the limitations of assuming uniform soil conditions.

The comparison between deterministic methods and numerical modeling using PLAXIS 3D shows good agreement, with differences generally within about 20%, which is acceptable in geotechnical engineering practice. The numerical modeling approach, based on settlement criteria, provides a more realistic representation of soil behavior, especially for larger footing sizes where settlement governs the design. This demonstrates that while analytical methods provide a reliable baseline, numerical modeling enhances the understanding of soil–structure interaction.

The GIS-based zonation maps effectively illustrate the spatial variation of bearing capacity across the study area by transforming discrete borehole data into continuous surfaces. The maps identify low bearing capacity zones in areas such as Sanepa, Kupundole, Jhamsikhel, and Dhobighat, moderate zones in central areas like Jawalakhel, Gwarko, and Manbhawan, and higher bearing capacity zones in southern regions, including Khumaltar, Sunakothi, and Hattiban. This spatial pattern remains consistent across different footing sizes and depths, although the magnitude of bearing capacity increases with depth.

The integration of geological data further strengthens the interpretation of the zonation maps. Low bearing capacity zones correspond to the Kalimati Formation, Lukundol Formation, and recent floodplain deposits, which consist mainly of fine-grained soils such as silty clay and alluvial sediments. These soils are characterized by low shear strength and high compressibility, resulting in reduced load-bearing capacity. In contrast, higher bearing capacity zones are associated with the Chapagaon, Tistung, and Chandragiri Formations, which are composed of coarse-grained materials such as sand, gravel, and cobbles, providing better strength and stability. This strong correlation confirms that the spatial distribution of bearing capacity is largely controlled by geological formations.

Validation of the results using laboratory data and previous studies shows good agreement, with differences generally within ±15%, which is acceptable considering the inherent variability of soil properties. The consistency with previous studies conducted in the Kathmandu Valley further confirms that the obtained bearing capacity ranges and trends are realistic. The reliability analysis highlights the importance of considering uncertainty in soil properties, showing that the probability of failure increases with increasing variability, while reliability improves with greater footing size and depth. This indicates that shallow foundations with smaller dimensions are more critical, whereas deeper and larger foundations provide safer and more stable performance.

Overall, the study demonstrates that bearing capacity is influenced by multiple factors, including soil variability, depth, footing size, and geological conditions, and that an integrated approach combining deterministic, numerical, spatial, and probabilistic methods provides a more comprehensive and reliable assessment of foundation performance.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This study presents an integrated evaluation of the bearing capacity of shallow foundations in Lalitpur Metropolitan City using deterministic analysis, numerical modeling, GIS-based spatial mapping, and reliability-based assessment. The results demonstrate that bearing capacity varies significantly across the study area due to heterogeneous subsurface conditions, with values ranging from approximately 65 kPa to more than 300 kPa depending on location, depth, and soil type.

A consistent trend observed throughout the study is that bearing capacity increases with depth, with higher values obtained at 3 m depth compared to 1.5 m, indicating improved soil strength and compaction at deeper layers. Additionally, the analysis shows that larger footing sizes provide slightly improved bearing capacity due to better load distribution, although the influence becomes less significant at greater depths.

The GIS-based zonation maps effectively represent the spatial variation of bearing capacity, identifying weak, moderate, and strong zones across Lalitpur. The zonation results show a clear pattern, with low bearing capacity zones in northern regions, moderate zones in central areas, and higher bearing capacity zones in southern regions. These spatial variations are strongly correlated with geological formations, where fine-grained lacustrine deposits (Kalimati and Lukundol formations) correspond to lower bearing capacity, and coarse-grained formations (Chapagaon, Tistung, and Chandragiri) correspond to higher bearing capacity.

The comparison between deterministic and numerical (PLAXIS 3D) results shows good agreement, with variations within acceptable limits, confirming the reliability of the analytical methods. Validation using laboratory data and previous studies further confirms the accuracy of the results, with differences generally within ±15%, which is acceptable in geotechnical engineering practice.

The reliability analysis highlights the importance of considering uncertainty in soil properties. The results show that the probability of failure increases with increasing variability, while reliability improves with increasing footing size and depth. Among the analyzed cases, shallow foundations with smaller footing areas were found to be more critical, whereas deeper and larger foundations provided the most reliable performance.

Overall, the study demonstrates that the integration of deterministic, numerical, spatial, and probabilistic approaches provides a more realistic and reliable assessment of bearing capacity compared to conventional single-method approaches. The developed bearing capacity zonation map serves as a valuable tool for preliminary foundation design, site selection, and urban planning in Lalitpur Metropolitan City.

6.2 Recommendations

1. The bearing capacity zonation map should be used as a decision-support tool for urban planning and preliminary foundation design, especially in data-scarce areas.
2. In zones identified with low bearing capacity, site-specific investigation and appropriate ground improvement or alternative foundation systems (e.g., raft or pile) should be prioritized.
3. For safe and economical design, it is recommended to optimize both footing depth and size, as deeper and larger foundations significantly improve performance.
4. Integration of GIS and probabilistic methods should be encouraged in routine geotechnical practice to better capture spatial variability and uncertainty.
5. Future work should focus on high-resolution data collection and advanced modeling (e.g., geostatistics or AI/ML) to enhance prediction accuracy and applicability.

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APPENDIX A:

Project	:	Soil Investigation of Proposed Building Site
Location	:	Kupondole, Lalitpur
Client	:	-
Bore Hole No	:	**1**
Diameter of BH, mm	:	100 mm
RL of GWT	:	1.40 m
Date	:	22nd Sep, 2016 to 23rd Sep, 2016
Logged By	:	Gajendra Prasad Deo
Prepared By	:	Manoj Subedi
Checked By	:	Sandeep Kr. Jha
Certified By	:	Dr. R. K. Poudel

Scale 1=50cm Each	Depth m	Thickness m	Sampiling Depth m	Sampiling Type	Soil Classification	Group Symbol	Soil Symbol	SPT (Field Record) 15 cm	SPT (Field Record) 30 cm	SPT (Field Record) 45 cm	Value N
	0.00										
		3.00	1.50	SPT	Filling materials including gravel mix sandy clayey silt with pieces of bricks	FM		4	7	6	**13**
	3.00		3.00	SPT				3	4	5	**9**
			4.50	SPT				4	4	6	**10**
			5.00	UDS							
			6.00	SPT				6	8	8	**16**
		8.50	7.50	SPT	Gray to brown moist clayey silt of low plasticity with some cut layer of sandy silt	ML		7	7	8	**15**
			9.00	SPT				5	7	9	**16**
			10.50	SPT				6	8	7	**15**
	11.50										
			12.00	SPT				2	3	4	**7**
		3.50	13.50	SPT	Dark gray moist clayey silt of medium plasticity	MI		3	4	3	**7**
	15.00		15.00	SPT				2	3	3	**6**

Table A.6: Sample of hydrometer analysis

Time (min)	Actual Hydrometer Reading	Reading corrected for miniscus	Tempera ture	Corrected hydrometer reading	Effective length L (cm)	k factor	k=k^0.5	(L/t)^0.5	d (mm)	% finer (P)	adjusted % fines
									0.075		39.700
1.00	1.030	1.031	22.000	30.399	11.213	0.000172	0.013103	3.349	0.044	0.705	27.975
2.00	1.029	1.030	22.000	29.399	11.377	0.000172	0.013103	2.385	0.031	0.681	27.054
3.00	1.029	1.030	22.000	29.399	11.377	0.000172	0.013103	1.947	0.026	0.681	27.054
4.00	1.028	1.029	22.000	28.399	11.541	0.000172	0.013103	1.699	0.022	0.658	26.134
8.00	1.027	1.028	22.000	27.399	11.705	0.000172	0.013103	1.210	0.016	0.635	25.214
16.00	1.026	1.027	22.000	26.399	11.869	0.000172	0.013103	0.861	0.011	0.612	24.294
30.00	1.024	1.025	22.000	24.399	12.198	0.000172	0.013103	0.638	0.008	0.566	22.453
60.00	1.021	1.022	24.000	21.999	12.690	0.000164	0.012799	0.460	0.006	0.510	20.244
120.00	1.019	1.020	24.000	19.999	13.018	0.000168	0.012952	0.329	0.004	0.464	18.404
180.00	1.015	1.016	24.000	15.999	13.674	0.000168	0.012952	0.276	0.004	0.371	14.723

240.00	1.011	1.012	24.000	11.999	14.331	0.000168	0.012952	0.244	0.003	0.278	11.042
300.00	1.009	1.010	25.000	10.299	14.659	0.000160	0.012658	0.221	0.003	0.239	9.478
1,440.00	1.005	1.006	21.000	5.199	15.315	0.000176	0.013260	0.103	0.001	0.121	4.784

corrected hydrometer reading=[(r-1) × 1000]-zero correction+Temperature Correction

Miniscus correction=0.001

Zero correction=0.001

Specific Gravity=2.708

mass of soil taken=50gms

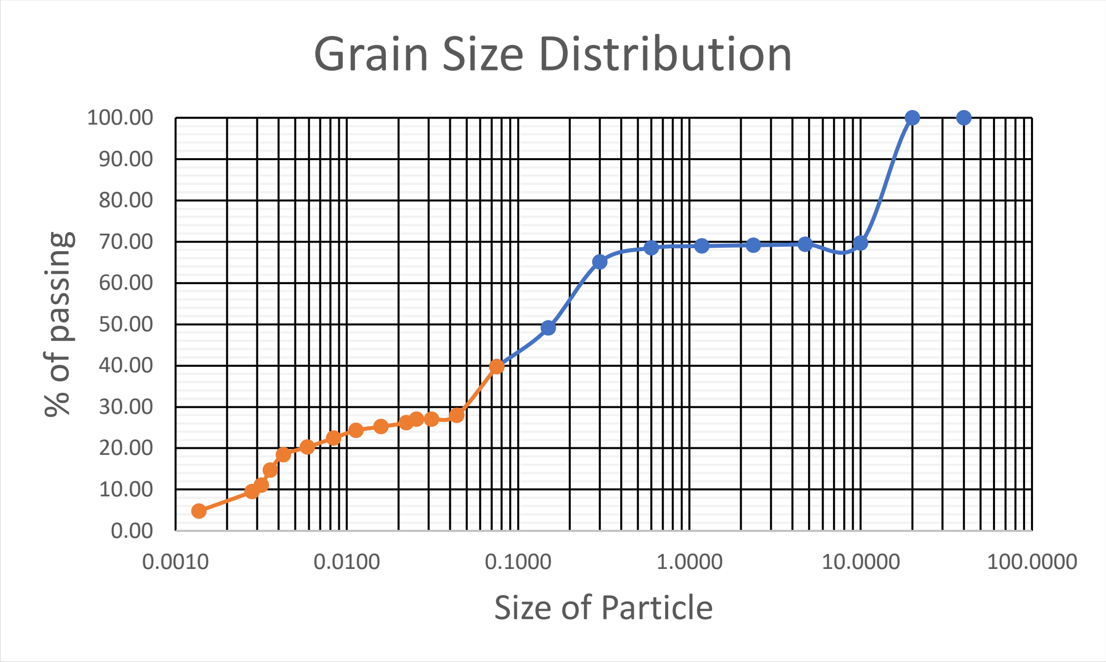

Figure A.2: Grain size distribution

Table A.7: Sample of Atterberg Limits

Particular		Liquid Limit			Plastic Limit	
Specimen No.		1	2	3	1	2
Container No.		A	B	C	D	E
No. of Blows		8	22	32		
Wt. of Container + Wet Soil	gms	25.60	39.30	19.20	31.60	26.80
Wt. of Container + Dry Soil	gms	21.50	34.20	16.10	28.80	24.80
Wt. of Water Present	gms	4.10	5.10	3.10	2.80	2.00
Wt. of Empty Container	gms	8.30	16.80	4.50	16.90	16.20
Wt. of Dry Soil	gms	13.20	17.40	11.60	11.90	8.60
Moisture Content	%	31.06	29.31	26.72	23.53	23.26

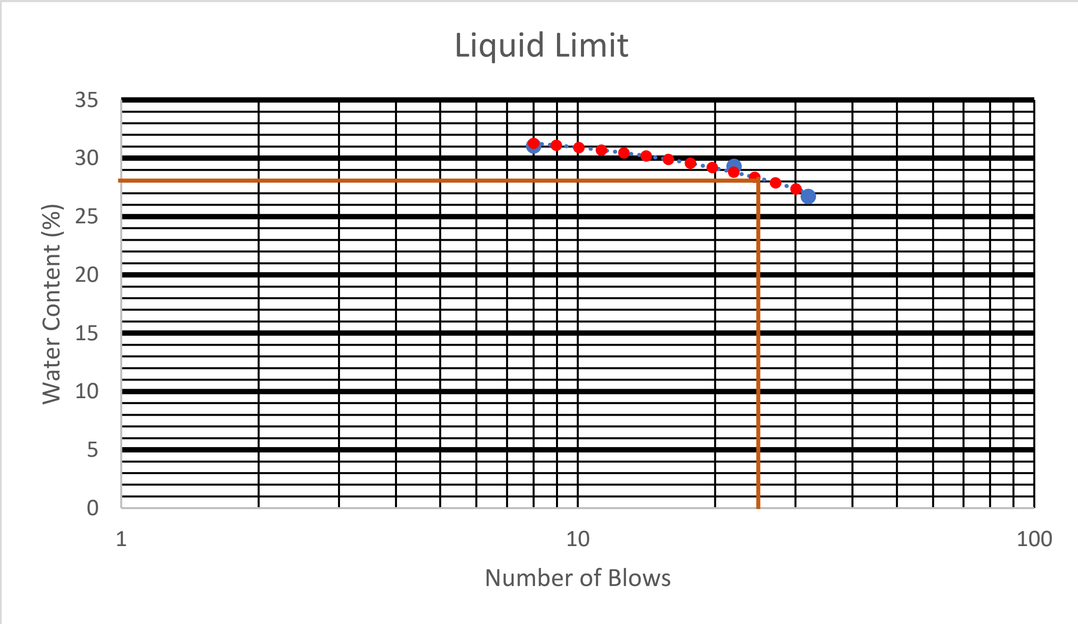

Figure A.3: Liquid Limit

Table A.8: Sample of Moisture Content calculation

Sample	Depth (m)	Wt. of wet soil + can (gm)	Wt. of dry soil + can (gm)	Wt. of can (gm)	Wt. of dry soil (gm)	Wt. of water (gm)	Moisture Content (%)
Kumaripati	1.5	28.90	24.90	8.90	16.00	4.00	25.00

Table A.9: Sample of Bulk Density calculation

Sample	Depth (m)	Can wt.	Wt. of wet soil + can (gm)	Moisture Content (%)	Wt. of wet soil (gm)	Bulk density kN/m^3	Dry density kN/m^3
Kumaripati	1.5	115.50	260.70	25.00	145.20	17.46	13.97

Table A.10: Sample of Direct Shear Test calculation

Moisture Content (%)		25.00
Provision ring factor (kN/div)		0.00145
Area of mould(m^2)		0.0036
Vol^m of mould (m^3)		0.00009
Wt. of mould(kg)		0.142
Test	Normal stress (kPa)	Peak shear stress (kPa)
1	10.78	6.85
2	21.57	14.90
3	32.36	18.53

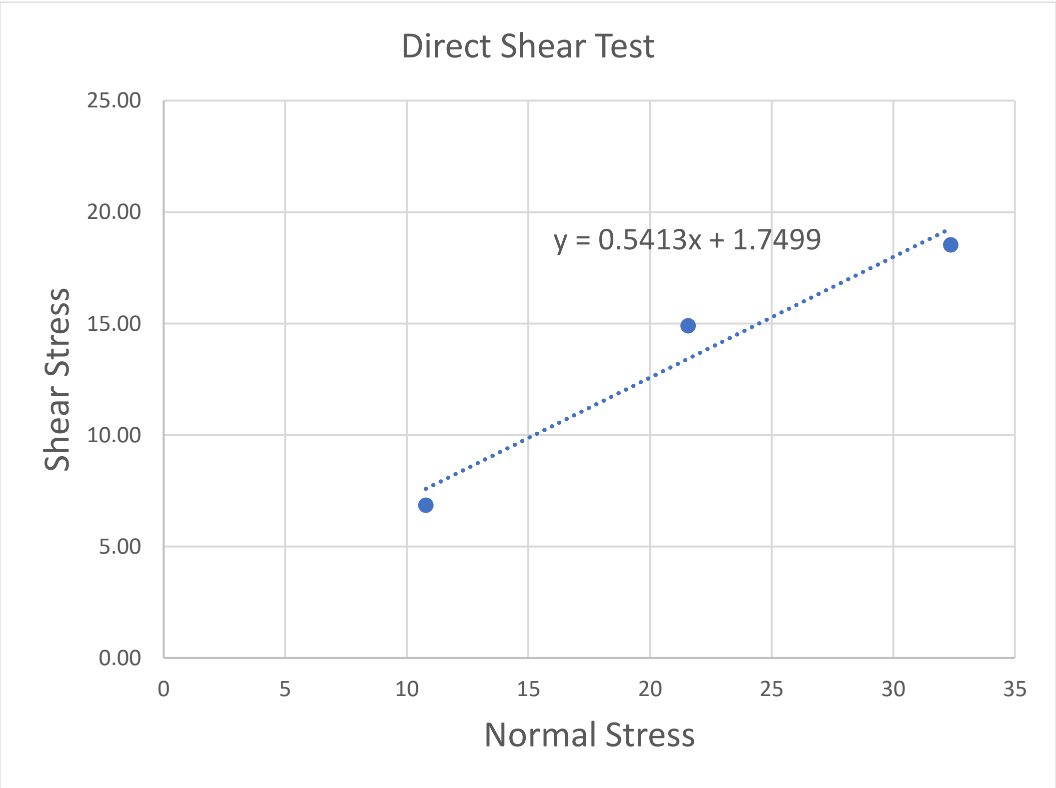

Figure A.4: Direct Shear Test results

Table A.11: Sample of Specific Gravity calculation

Depth (m)		1.5
Room temperature	C	25°
Flask No.	C	
Wt. of empty flask (A)	gm	89.801
Wt. of empty flask + Oven dry sample (B)	gm	128.801
Wt. of flask + Full water + sample (C)	gm	219.911
Wt. of flak + Full water (D)	gm	196.454
Wt. of Oven-dry sample (B-A)	gm	39.000
Specific Gravity of Sample		2.509

Table A.12: Soil Classification according to IS Code

Location	**% Passing 75 µm**	**Soil Type**	**LL%**	**PI %**	**A-line PI = 0.73(LL−20)**	**Soil Type**
Chakupat	62.59	Fine-grained	52	20.63	23.36	MH
Jawalakhel	19.41	Coarse-grained	28.5	4.75	6.21	SM
Kumaripati	39.7	Coarse-grained	28.3	4.91	6.06	SM

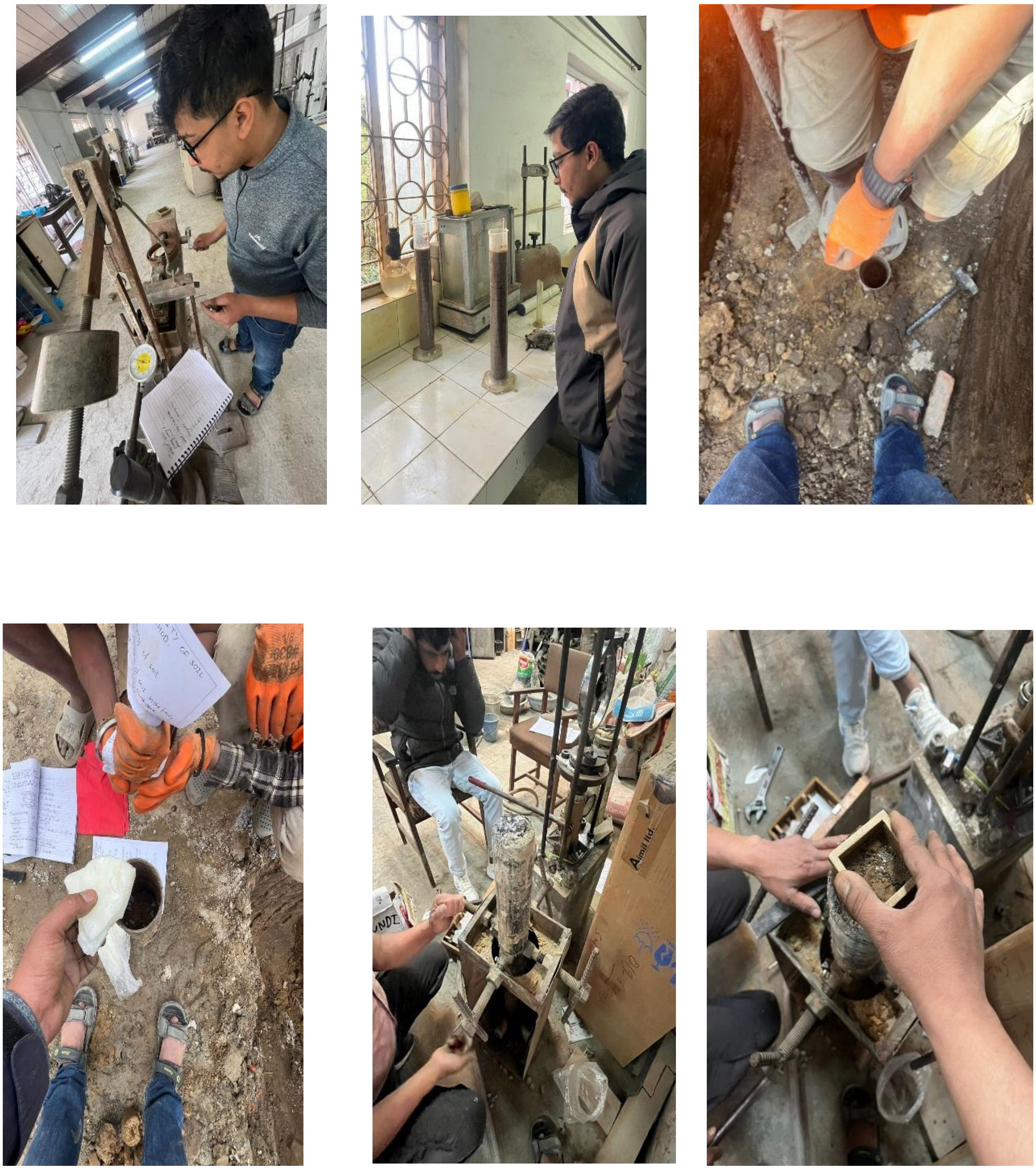

Figure A.5: Pictures of soil collection and test

APPENDIX B

i. Sample Calculation for Deterministic Analysis:

Location: Bagdole

Coordinates: 27.6666138 N, 85.301176 E

Depth of Foundation: 1.5 m

Width of Foundation: 1.5m

$C = 0\ kN/m^2$

Angle of Internal Friction = 29.89°

Effective Unit Weight = $16.69\ kN/m^3$

Bearing Capacity Calculation by Terzaghi (1943) approach:

N_c =29.8

N_q =18.1

N_γ = 15.3

$q = 10.32\ kN/m^2$

$q_u = (1.3*0*29.8)+(10.32*18.1)+(0.5*(16.69-9.81)*1.5*15.3) = 265.70\ kN/m^2$

$q_a = 265.70/3 = 88.57\ kN/m^2$

Bearing Capacity Calculation by IS approach:

N_c =29.8

N_q =18.11

N_γ = 17.08

$s_c = 1.3$, $s_q = 1.2$, $s_\gamma = 0.8$

$d_c = 1.2$, $d_q = 1.0$, $d_\gamma = 1.0$

$i_q=i_\gamma=i_c=1$

$q_u = (0*29.8*1.3*1.2*1) + (10.32*(18.11-1)*1.2*1.0*1.0) + (0.5*(16.69-9.81)*0.8*17.08*1.5*1.17*1) = 282.27\ kN/m^2$

$q_a = 282.27/3 = 94.09\ kN/m^2$

Bearing Capacity Calculation by Hansen (1970) approach:

N_c =29.8

N_q =18.11

N_γ = 16.45

s_c = 1.61, s_q = 1.5, s_γ = 0.60

d_c = 1.4, d_q = 1.1, d_γ = 1

$i_q=i_\gamma=i_c=1$

q_u = (1*29.8*1.61*1.4*1) + (10.32*18.11*1.5*1.1*1) + (0.5*(16.69-9.81)*1.5*17.95*0.60*1*1) =358.69 kN/m^2

q_a = 358.69/3 = 119.56 kN/m^2

Bearing Capacity Calculation by Vesic (1973) approach:

N_c =29.8

N_q =18.11

N_γ = 21.94

s_c = 1.61, s_q = 1.6, s_γ = 0.60

d_c = 1.4, d_q = 1.29, d_γ = 1

$i_q=i_\gamma=i_c=1$

q_u = (1*29.8*1.61*1.4*1) + (10.32*18.11*1.60*1.29*1) + (0.5*(16.69-9.8)*1.5*21.94*0.60*1*1) = 447.08 kN/m^2

q_a = 447.08/3 = 149.01 kN/m^2

Bearing Capacity Calculation by Bowles ' (1977) approach:

q_a = 11.98*9*((3.28*1.5+1)/(3.28*1.5))2*1+0.33(1.5/1.52)*(25/25.4)*0.5

= 106.15 kN/m^2

ii. Sample Calculation for Reliability Analysis in Excel:

Depth (m)=1.500

Area (m^2)= 4.000

Mean q_u (kPa)= 286.16

COV q_u= 20.0%

Mean load Q (kN)= 458.52COV Q= 39.0%

Number of simulations = 10000

Sigma q_u (kPa)= 57.232

Sigma Q (kN)= 178.823

N_f=ROUND (NORM.S. DIST ((((area*mean q_u)-mean load/SQRT ((area*sigma q_u) ^2+(sigma Q) ^2)), TRUE) *10000, 0)

P_f =N_f/10000

β=-NORM.S.INV (P_f)