Suitable cost-effective foundation selection approach of medium height buildings on sandy silt and silt clay soils in Adama city

Shimelis Regassa Guranda

A Thesis Submitted to

The department of Civil Engineering

School of Civil Engineering & Architecture

Presented in Partial Fulfillment of the Requirement for the degree of Mater's of Science in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

May, 2016

Adama, Ethiopia

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Advisor: Dr. Argaw Asha

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I/we, the advisors of the thesis entitled "Suitable cost-effective foundation selection approach of medium height buildings on sandy silt and silt clay soils in Adama city" and developed by Shimelis Regassa, here by certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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I hereby declare that this Master Thesis entitled “Suitable cost-effective foundation selection approach of medium height buildings on sandy silt and silt clay soils in Adama city” is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation

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LIST OF ACRONYMS

f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{ctk}	Characteristic axial tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
f_{cd}	Design value of concrete compressive strength
f_{tk}	Characteristic tensile strength of reinforcement
f_{cd}	Design value of concrete compressive strength
f_{yk}	Characteristic yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement
f_y	Yield strength of reinforcement
f_{ctd}	Compressive strain in the concrete
E_s	Modulus of elasticity
E_s	Modulus of elasticity of the footing
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
ε_{Ud}	Strain of reinforcement or prestressing steel at maximum load
γ_I	Importance factor
γ_c	Partial factor for concrete
γ_s	Partial factor for reinforcing or prestressing steel
γ_E	Partial factor for the effect of an action
γ_F	Partial factor for an action
γ	Unit weight
c_u	undrained Shear strength
a_g	Design ground acceleration
a_{gR}	Reference peak ground acceleration
α_u/α_1	Multiplication factor
N	Below count
ψ_0	Combination values of variable actions
ψ_1	Frequent values of variable actions
ψ_2	Quasi-permanent values of variable actions
φ	Dynamic magnification factor
q_o	Basic value of the behavior factor

T_1	Fundamental period of the building in the horizontal direction of interest
T_c	Upper limit of the constant acceleration region of the elastic spectrum
G_k	Characteristic value of permanent action
Q_k	Characteristic value of single variable action
W_k	Characteristic value of wind action
E_{xy}	Earthquake load in x and y direction
F_{bk}	Base shear force
$S_d (T_k)$	Ordinate of the design spectrum at period T_k
m_k	Effective modal mass corresponding to a mode k
k	Number of modes taken into account
T_k	Period of vibration of mode k
θ	Inter-story drift sensitivity coefficient
P_{tot}	Total gravity load at and above the story
d_r	Design inter-story drift
V_{tot}	Total seismic story shear; and
h	Inter-story height.
d_s	Displacement of a point of the structural system
q_d	Displacement behavior factor
D_f	Depth of foundation below ground level
B	Width of foundation
L	Length
q_u	Ultimate Bearing Capacity
q_{nu}	Net Ultimate Bearing Capacity
q_{ult}	Ultimate gross bearing capacity
q_{all}	Allowable bearing capacity
q'_o	Effective overburden pressure
FS	Factor of safety
N	Statistical blow count
N_{avg}	Average Blow count
Q_p	Load-carrying capacity of the pile point
q_p	Ultimate Point Resistance
f_{av}	Average unit frictional resistance

Q_s	Frictional or skin resistance of a pile
N_{55}	Corrected SPT blow count for 55% hammer efficiency
N_{60}	Corrected SPT blow count for 60% hammer efficiency
N_{70}	Corrected SPT blow count for 70% hammer efficiency
D	Piles diameter
$(Q_u)_g$	Ultimate load-bearing capacity of the group pile
Q_u	Ultimate load-bearing capacity of each pile without the group effect
s	Center to center spacing of piles
n	Number of rows in the group
m	Number of columns in the group
α	Empirical adhesion factor
S_e	Settlement
S_g	Settlement of a pile group
v_s	Poisson's ratio of the soil
I_f	Moment of inertia of the cross section
z_s	Static deflection
k_s	Coefficient of subgrade reaction
k_{sv}	Unit subgrade spring coefficient,
A	Cross-sectional area of a pile
k_{sr}	Rotational spring constant
C_u	Uniformity coefficient,
D_{10}	Effective grain size corresponds to 10 percent finer particles
D_{30}	Size of particle at 30 percent finer on the gradation curve
μ	Absolute viscosity of liquid
G_s	Specific gravity
R_a	Actual hydrometer reading
R_c	Hydrometer reading correction
C_o	Zero correction
C_T	Temperature correction
C_{sg}	Correction for specific gravity different from 2.65
M_s	Mass of soil used in the suspension
I_P	Plasticity index

W_p	Plastic limit
W	Water content
LL	Liquid limit
γ_{moist}	Moist unit weight
W_m	Mass of water
γ_d	Dry unit weight
C_N	Overburden pressure
η_g	Group efficiency
η_H	Hammer Efficiency Correction
η_R	Rod length correction factor
η_{S-}	Sampler correction factor
η_B	Bore hole diameter correction factor
D_r	Relative Density
P_a	Atmospheric pressure (P_a= 100 KN/m^2)
C_w	Groundwater correction factor
z	Depth to the groundwater table
Φ'	Effective friction angle
G_{Max}	Maximum shear modulus
R^2	Coefficient of multiple determination
R_a^2	Adjust coefficient of multiple determination
f	Rejection region for a level test
SSE	Error sum of squares
SST	Total sum of squares
SSR	Regression sum of squares
VIF	Variance inflation factor
Std. Error	Standard error of the regression coefficient
y	Predicted value of dependent variables
T-Value	T-test value for testing the hypothesis
F- Ratio	F-statistic for testing the null hypothesis
CV	Coefficient of Variation

LIST OF ABBREVIATIONS

CSI	Computers and Structures, Inc
ETABS	Extended 3D Analysis of Building Systems
ES EN	Ethiopian standard euro norm
SPT	Standard penetration test
DCM	Ductile class of medium
DCH	Ductile class of High
ASTM	American Society for Testing and Materials
ASCE	American Society of Civil Engineers
PGA	The peak (maximum) ground acceleration
AASHTO	American Association of State Highway and Transportation officials
NaPO3	Sodium hexametaphosphate
USCS	Unified Soil Classification System
SST	Total sum of squares
SSR	Regressions sum of squares
RLLF	reduced live load factors

UNITS AND THEIR MULTIPLES

Force	KN
Moment	KN m
Mass density	KN/m3
Weight density	KN/m3
Stress, pressure, strength and stiffness	kPa
Speed	m/s

Abstract

Predicting a suitable cost-effective foundation type during the early stages of foundation design is a difficult task for geotechnical engineers as the cost is known only after finishing the design of foundations. This study presents the influence and prediction method of suitable cost-effective foundation type during early stages of foundation design for medium rise (G+10) buildings on typical sandy clay and sandy silt soils stratum in Adama city. Analysis of super structure to generate forces and moments, and sub structure foundation design done by CSI ETABS and Safe software respectively. Geotechnical property of soils like Atterberg limits test, Particle size distribution, standard proctor compaction and standard penetration test (SPT) were identified using laboratory and field tests. Overall costs of foundation were calculated from quantities and corresponding unit rate of structural and nonstructural foundation elements. Finally, an attempt is made to develop suitable method by multiple regression through computer program SPSS software. From the comparative analysis that has been made in this study, it was generally found that, design engineers have got an optional solution in foundation type selection at the same time saving design time.

***Key words**: Bearing capacity, cost-effective foundation type selection, Modulus of Subgrade Reaction, Cost estimation*

CHAPTER ONE

INTRODUCTION

1.1.Statement of the problem

Predicting suitable cost-effective foundation type during early stages of foundation design is a difficult task for geotechnical engineers because the cost is known after finishing the design of foundations. The available foundation type selection is governed by considering load of super structure, bearing capacity of soil conditions, availability of materials and equipment, the qualifications and experience of local contractors, environmental limitations on construction, size of development proposals, timescale and cost limitations.

Developing suitable cost estimation method during early stages of different foundation design plays a very important role by reducing the economy conflict in foundation construction and extra cost due to inappropriate selection of foundation type and the speed up the design period of foundation.

Reinforced concrete frame of medium height buildings G^{+2}, G^{+4}, G^{+6} G^{+8} and G^{+10} to be constructed in Adama city was intended to modeled and analyzed. The combatable shallow foundations for each category of building such as isolated, combined, mat foundation and bored pile of deep foundation type were assumed to be placed on sandy silt and silt clay soil. Modeling, analysis and design of reinforced concrete foundation of isolated footing, combined footing, mat foundations and bored pile foundation subject to static loading conditions is done by CSI SAFE software. Bill off quantities for each type of footing for different buildings are estimated.

The main objective of this dissertation is to develop suitable cost-effective foundation type selection approach during early stages of different foundation design for foundations rest on sandy silt and silt clay soils in Adama city for low and medium high buildings through multiple regression by SPSS computer program.

1.2. Objectives

1.2.1. General objective

The main objective of this study is to develop cost-effective foundation type selection approach for foundations rest sandy silt and silt clay soils for low and medium high buildings.

1.2.2. Specific objective

Specific objective of the present study is

1. To model and analysis reinforced concrete of G^{+2}, G^{+4}, G^{+6} G^{+8} and G^{+10} buildings for generating reaction force and moments at the base of column and design isolated, combined, mat and end bearing pile foundation by CSI ETABS and CSI SAFE software according to new ESEN code.
2. To identify engineering property of soil and bearing capacity of foundation after processing of laboratory and field tests respectively.
3. To develop cost-effective foundation type selection equation during early stages of foundation design for foundations rest on sandy silt and silt clay soils in Adama city after designing foundation and estimating bill off quantities for low and medium high buildings through multiple regression by SPSS computer program.

1.3. Hypotheses

H_1: There is no a significant impact of bearing capacity and height of building on cost of foundation.

1.4. Research questions

The central questions for this research were as follows:

1. What is impact of bearing capacity of soil and height of building on cost of foundation?
2. How to develop suitable cost-effective foundation type selection method during early stages of foundation design?

1.5. Significance of the study

Cost of foundation plays a very important role in selecting a proper foundation type for reducing design period and economy conflict in foundation construction. Hence developing suitable cost estimation approaches during early stages of foundation design with considerations of different soil condition, scale of development and dynamic load effects is essential. The study is expected to have a great significance for variety of users. Developers take the leading role in reduce paying of extra cost due to inappropriate selection of foundation. Also, the study would provide additional knowledge to designers in scheming new suitable cost estimation approaches. Academician undertaking similar studies will also find this study a useful reference material as regards empirical literature. The study will also contribute to the theoretical debate around recruitment practices and teacher retention. The findings of this study have relevance both to the recruiting literature and to school practices.

1.6.Delimitations and limitations of the study

This research is limited to modeling, analysis and designing of super structure by ETABS software and sub structures by Safe software and spread sheet programs. The bill off quantities and cost estimation is done for isolated footing, combined footing, mat foundations and end bearing pile foundation rests on silt, fine sand and silty clays soils for G+2, G+4, G+6 G+8 and G+10 buildings which assumed to be built in Adama city. The allowable bearing capacities are ended based on empirical methods using test data from field tests of the standard penetration test results.

CHAPTER TWO

LITERATURE REVIEW

2. Introduction

This chapter reviews both theoretical and empirical literature relating to the study problem. The theoretical literature focuses on the theories and models behind the study, the empirical literature reviews previous scholarly work in relation to the present study's research objectives. The contributions of previous scholars as well as the gaps thereof are also explored.

2.1.Super structure

2.1.1. Modeling base of super structure

ESEN code states that the design of concrete structures shall be in accordance with the geometry of the building and the general rules given in ES EN 1990, 1991, 1992 and 1998:2015 codes.

2.1.2. Design working life

ESEN 1990:2015 code section 2.3. table 2.1 (Ministry of Construction, 2015) define that the life time of a building is dependent on its durability and construction material quality and the life period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary assumed. Indicative design working life of building structures and other common structures are 50 years.

2.1.3. Description of structural materials

ES EN 1992-1-1:2015 code section 3.1.3. table 3.1. (Ministry of Construction, 2015) describe the characteristic of reinforced concrete C-25/30 and 30/37 and ES EN 1992-1-1:2015 code ANNEX C (Ministry of Construction, 2015) describe the property of bar with characteristic yield strength 500.

2.1.4. Reinforcement Cover

ES EN 1992-1-1:2015 code section 4.2 to 4.4 (Ministry of Construction, 2015) describes that the choice of concrete class depends upon durability requirement related to exposure conditions. Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions. The project is building in neither an offshore structure susceptible to corrosion induced by salt and chloride nor an industrial building exposed to aggressive chemical. Hence corrosion induced by carbonation is the only fitting characteristic used.

2.1.5. Loading of the Structural Model

2.1.5.1.Dead Load

ES EN 1991-1-1:2015 code annex A-1 to Annex A-11 (Ministry of Construction, 2015) recommended the unit weights of material for determining loading on structure. the following weights of material have been used:

Concrete………25KN/m3 ESEN1991-1 Table-A.1

Screed……….23KN/m3 ESEN1991-1 Table-A.1

Ceramic………27KN /m3 ESEN1991-1 Table-A.7

Plastering…….23KN/m3 ESEN199-1 Table-A.1

Steel…………78.5KN /m3 ESEN1991-1 Table-A.4

HCB……….....14 KN /m3 ESEN1991-1 Table-A.8

2.1.5.2.Live Load

ES EN 1991-1-1:2015 code section 6.3 table 6.1 to 6.11 (Ministry of Construction, 2015) recommended the live load on each floor. For residential buildings-imposed loads on floor is $2KN/m^2$ and on stair is $2KN/m^2$.

2.1.5.3.Earth quake loads

Earth quake forces constitute to both vertical and horizontal forces on the building. The total vibration caused by earth quake may be resolved in to three manually perpendicular directions, usually taken as vertical and two horizontal directions. The seismic load in a building is assigned based the ground type, expected ground acceleration, the available ductility of the structure, the seismic zone, the natural period of vibration of the structure and regularity in the structural system among other factors. The following portion of this report summarizes these properties for the specific project under design.

i) Seismic zone

ES EN 1998-1:2015 code annex D (Ministry of Construction, 2015) determine the specific zone of the site of the building under design. According to this code Adama belongs to zone 4 with the value of the coefficient bedrock acceleration ratio (α_o) is 0.15.

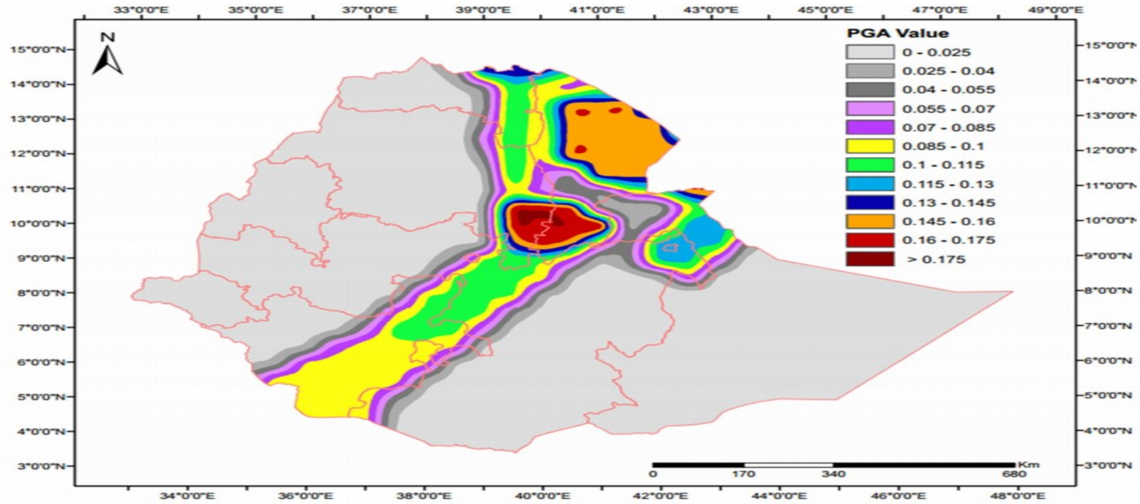


FIGURE 2.1: Ethiopia's Seismic hazard map in terms of peak ground acceleration

Source: ESEN 1998, Design of Structures for Earthquake Resistance" Part 1, 2015

ii) Ductility class

ES EN 1998-1:2015 code section 5.4 (Ministry of Construction, 2015) states earthquake resistant concrete buildings shall be designed to provide energy dissipation capacity and an overall ductile behavior. These concrete buildings are classified in two ductility classes DCM (medium ductility) and DCH (high ductility), depending on their hysteretic dissipation capacity. Ductility class choice depends upon the peak ground acceleration value. For areas with PGA value greater than 0.1, this code recommends adopting ductility class medium design procedure. That is under the assumption that to take into account the ductility capacity of the structural members of the building in addition to the elastic strength of the members would most likely result in an effective design.

iii) Ground type

ES EN 1998-1:2015 code section 3.1.2. table 3.1 (Ministry of Construction, 2015) define by the stratigraphic profiles and parameters of the soil condition. The ground type shall be determined based on the standard penetration test result which is the widely practiced method of test during geotechnical investigation. The standard penetration test result for ground type 'A' is 15 to 50 and less than 15 for ground type 'D'.

iv) Importance class

ES EN 1998-1:2015 code section 4.2.5 (Ministry of Construction, 2015) classifies buildings in to 4 importance classes depending on the consequences of collapse for human life. Building of high importance class are those that will provide civil protection in the immediate post-earthquake period. Ordinary buildings, not belonging in the other categories classifies in to importance class II with importance factor 1.

v) Response spectrum function

ES EN 1998-1:2015 code section 3.2.2.2 and table able 3.2 and 3.3 (Ministry of Construction, 2015) states that the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, called an "elastic response spectrum". The elastic response spectrum is the same for both no-collapse and damage limitation requirement. Determining the response spectrum for a specific location demands detail geotechnical study. Even though we don't have a site-specific spectrum function for project, considering the fundamental period, we expect in our building, the governing spectrum type from the two options provided in the ES EN 1998-1:2015 code have been adopted. As mentioned on code spectra type 1 is for areas of high seismicity (defined as $M_s > 5.5$) and Spectra type 2 is for areas of moderate seismicity ($M_s \leq 5.5$).

vi) Mass source

ES EN 1998-1:2015 code section 4.2.4 and table able 4.2 (Ministry of Construction, 2015) states that the weight of the structure used in the calculation of automatic seismic loads is based on the specified mass of the structure and is termed mass source in ETABS. Mass source participation factor for residential buildings is associated with all gravity loads and about 25% of live load.

vii) Behavior factor

ES EN 1998-1:2015 code section 7 table 7.1 (Ministry of Construction, 2015) states that behavior factor (q_o) is a factor used for design purpose to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures. The value of the behavior factor for systems regular in elevation of frame system, dual system, coupled wall system of medium ductility class structure is $3\alpha_u/\alpha_1$. For buildings which are not regular in elevation, the value of q_o should be reduced by 20%. For multistorey, multi-bay frames $\alpha_u/\alpha_1 = 1.3$

where

α_u/α_1 -multiplication factor

viii) Accounting for Effect of Cracking

ES EN 1991-1-2:2015 code (Ministry of Construction, 2015) requires that the stiffness of all load bearing elements must account for both their flexural, shear flexibility and their axial flexibility effect of cracking. This code also states that unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete elements may be taken to be equal to one half of the gross section uncracked elastic stiffness. To satisfy the above requirements, stiffness properties of slabs with shell properties, beams, columns, and walls has to be reduced to 50%.

2.1.5.4. Evaluation of Structural Regularity

ES EN 1998-1:2015 code section 4.3.6.3.1 and 4.3.6.3.2 (Ministry of Construction, 2015) define the criteria for regularity in plan and in elevation respectively.

Table 2.1 Consequences of structural regularity on seismic analysis and design

Regularity		Allowed Simplification		Behavior factor
Plan	Elevation	Model	Linear-elastic Analysis	(For linear analysis)
Yes	Yes	Planar	Lateral force	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial	Lateral force	Reference value
No	No	Spatial	Modal	Decreased value

Source: ESEN 1998, Design of Structures for Earthquake Resistance” Part 1, 2015

2.1.6. Load combinations

Ultimate limit state load combinations defined in ES EN 1990:2015 code section 6.4.3 (Ministry of Construction, 2015) and serviceability limit states stated in ES EN 1990:2015 code section 6.5.3 (Ministry of Construction, 2015) and seismic design situation stated in ES EN 1998-1:2015 code section 4.3.3.5. (Ministry of Construction, 2015)

1. In persistent and transient situations

   $\sum 1.35G_{k,j} + 1.5Q_{k,1} + 1.05W_k$ (type ADD) (2.1)

2. Serviceability (function, including damage to structural and non-structural elements, e.g., partition walls)

   $\sum G_{k,1} + Q_{k,1} + \sum 0.7Q_{k,i}\ \ (j \geq 1, i > 1)$ (type ADD) (2.2)

3. In Seismic design situation

   $1G_{k,j} + 0.3Q_{k,1} \pm E_{xy}$ (type ADD) (2.3)

4. Envelope $\text{COMB1} + \text{COMB2} + \cdots \text{COMB7}$ (type ENV) (2.4)

   Where

   G_k-Characteristic value of permanent action

   Q_k-Characteristic value of single variable action

W_k-Characteristic value of wind action

E_{xy}- Earthquake load in x and y direction

2.1.7. Methods of analysis

Depending on the structural characteristics of the building one of the following two types of linear-elastic analysis may be used:

2.1.7.1. The "lateral force method of analysis''

This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. This requirement is deemed to be satisfied in buildings which fulfil both of the two following conditions.

i) They have fundamental periods of vibration T_1 in the two main directions which are smaller than the following values

$$T_1 \leq \begin{cases} 4.T_c \\ 2.0s \end{cases}$$

Where

T_c – is given in table 3.2 or table 3.3 ES EN 1998-1:2015 code (Ministry of Construction, 2015)

ii) They meet the criteria for regularity in elevation

2.1.7.2. Modal response spectrum analysis

Modal Analysis results give modal frequencies/period and mode shapes that are primarily used in Response Spectrum Analysis in ETABS. Modal analysis type can be either Eigen vector or Ritz vector. The "modal response spectrum analysis" shall be applied to buildings which do not satisfy the conditions for applying the lateral force method of analysis. The response of all modes of vibration contributing significantly to the global response shall be taken into account.

i) The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;

ii) All modes with effective modal masses greater than 5% of the total mass are taken into account.

The base shear force acting in the direction of application of the seismic action, may be expressed as

$$F_{bk} = S_d(T_k) * m_k \qquad (2.5)$$

Where

F_{bk}- base shear force

S_d (T_k) -is the ordinate of the design spectrum at period T_k

m_k- effective modal mass corresponding to a mode k

It can be shown that the sum of the effective modal masses (for all modes and a given direction) is equal to the mass of the structure. The minimum number k of modes to be taken into account in a spatial analysis should satisfy both the two following conditions:

$$k \geq 3.\sqrt{n} \qquad (2.6)$$

and

$$T_k \leq 0.20\ (s) \qquad (2.7)$$

where

k- is the number of modes taken into account;

n- is the number of stores above the foundation or the top of a rigid basement;

T_k- is the period of vibration of mode k.

2.1.8. Inter-story Drift

ES EN 1998-1:2015 code (Ministry of Construction, 2015) states that second-order effects (P-Δ effects) need not be taken into account if the following condition is fulfilled in all stores

$$\Theta = \frac{P_{tot} * d_r}{V_{tot} h} \leq 0.10 \qquad (2.8)$$

$$d_s = q_d d_e \qquad (2.9)$$

Where

θ = the inter-story drift sensitivity coefficient;

P_{tot} = is the total gravity load at and above the story considered in the seismic design situation;

d_r = the design inter-story drift, evaluated as the difference of the average lateral displacement d_s at the top and bottom of the story under consideration

V_{tot} = the total seismic story shear; and

h = the inter-story height.

d_s = the displacement of a point of the structural system induced by the design seismic action;

q_d = the displacement behavior factor, assumed equal to q unless otherwise specified;

d_e = the displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum in accordance with

ES EN 1998-1:2015 code (Ministry of Construction, 2015) also states that If $0.1 < \theta \leq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$. Additionally, the value of the coefficient θ shall not exceed 0.3.

2.1.9. Inter-story Drift Limitation

ES EN 1998-1:2015 code (Ministry of Construction, 2015) also states unless otherwise specified the following limits shall be observed:

a) For buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

   $d_r v \leq 0.010h$

Where

d_r - is the design inter-story drift

h - is the story height;

v - is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. The recommended values of v are 0.4 for importance classes III and IV and v = 0.5 for importance classes I and II.

2.2. Sub structure

2.2.1. Foundations: Categories

(Ming, 2015) states that foundations are divided into two major categories: shallow foundations and deep foundations, depending on the depth of embedment of the foundation.

2.2.2. Shallow Foundations: Definition

As description made by (Muni, 2011) shallow foundation is one in which the ratio of the embedment depth to the minimum plan dimension, which is usually the width (B), is $D_f/B < 2.5$. As explanation made by (Ming, 2015) if the ratio of the embedment to the width of a foundation (D/B) is larger than 3, then the foundation is considered a deep foundation; otherwise, the foundation is considered a shallow foundation. (Ahmed S, 2015) also specifies that foundations are considered to be shallow. If $[D_f \leq (3 \rightarrow 4)\ B]$ otherwise, the foundation is deep.

Where

D_f -depth of foundation below ground level, and

B-width of foundation (least dimension)

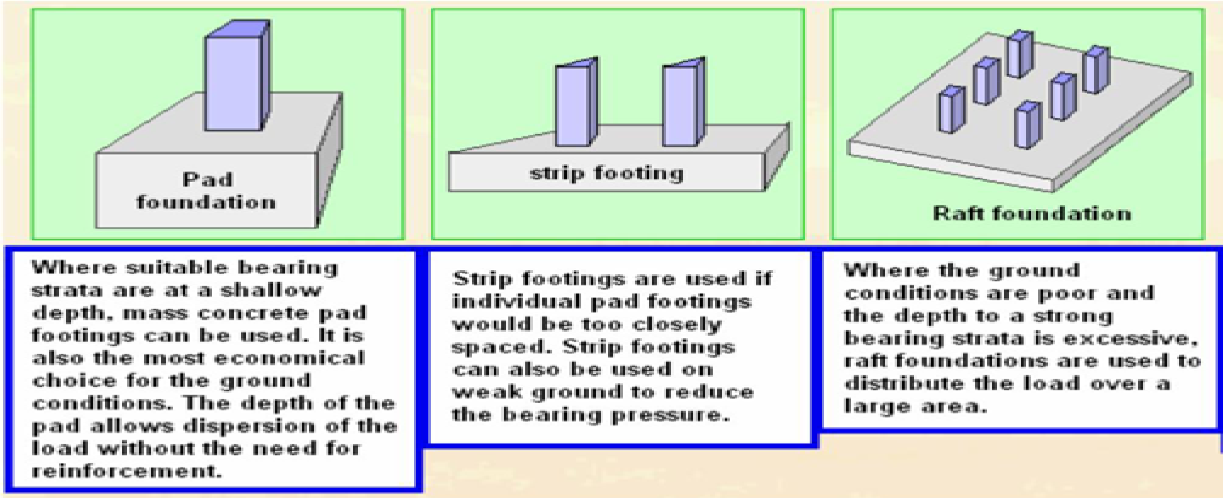


FIGURE 2.2 Shallow Foundation

Source: A S M Fahad Hossain, Geotechnical Engineering Sessional II, Ehsanullah University of Science and Technology, 2017

2.2.3. Deep foundations: Definition

(Ming, 2015) states that Deep foundations typically include piles and drilled shafts. Piles are driven into the subsoil, while drilled shafts (also known as cast-in-drilled-hole) are of larger diameter (>1.0m) and are cast in predrilled holes with reinforcement casing. (Ming, 2015) express that depending on load transfer mechanisms piles can be categorized as friction pile, end bearing pile, and combination. It is convenient to consider that end Bearing Piles will transmit load into the firm soil layer of the ground such as rock, gravel, very dense sand whereas friction Piles transmit the load from the structure to the penetrable soil by means of skin friction or cohesion between the soil & the embedded surface of the pile.

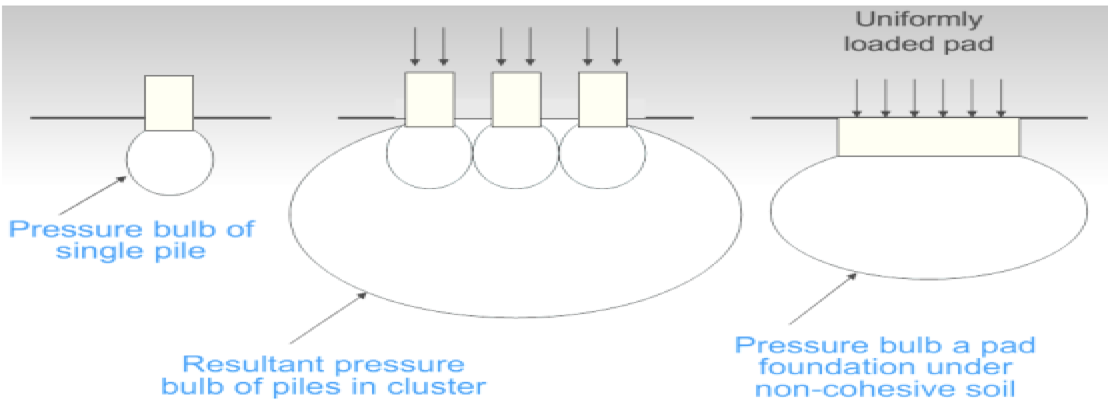


FIGURE 2.3 Loading condition of pile foundation

Source: A S M Fahad Hossain, Geotechnical Engineering Sessional II, Ehsanullah University of Science and Technology, 2017

2.2.4. Foundation selection

Foundation selections is governed by many factors such as the availability of materials and equipment, the qualifications and experience of local contractors, environmental limitations on construction access, size of development proposals, timescale, and cost limitations few of them. Other factors are present in detail as follow.

The size of the contract can have also significant effect on the economics of the solution. For example, the effects of fixed costs such as those for getting piling rigs on and off the site can be very small when spread over a large number of piles; on the other hand, they can prove to be the major cost when a small number of piles are to be driven.

On the other hand, high costs can be generated by complex shuttering details to foundations. These costs can be reduced significantly if details are simplified, for example, concrete can be cast against hand packed hardcore in raft construction. Two stage concrete pours for raft edges can use earth faces for shuttering to the first pour. Brickwork built off the raft edge can act as shuttering to the second pour.

The arrangement of the superstructure supports is additional factor and very critical to the foundation economy. For example, the design of the superstructure should not be made completely independently of the foundation economy. In the same way the foundation economy should not be considered independently of the superstructure. A typical example of this kind of problem is the use of fixed feet on portal frames which often create greater additional costs on the foundation than they do savings on the superstructure.

If the initial proposed scheme is not suitable for identified ground conditions, the proposals should be modified for the successful functioning of the building and give significant savings on foundations. For example, where piling is necessary for a single story building the economic span for ground beams, etc., often produces loads in the piles which do not fully exploit their load-carrying capacity. In such situations consideration should be given to changing the building form from single-story to multi-story.

Is the investigation sufficient to design a safe and economic foundation? For example, for ground investigation undertaken, if predicted a piled solution may have lacked detailed investigation of the upper strata, it would be necessary to consider a raft as an alternative. An alternative example would be a ground investigation based upon boreholes and sample collection or testing at shallow depth predicting surface spread foundations: this would not provide the information necessary to consider a piled solution. If a piled foundation solution is subsequently found to be necessary, important sub-soil information would not be known. In some cases the engineer is only called in after the initial site investigation has been completed and the alternative foundation solution is only appreciated at that stage, making further sub-soil investigation necessary in order to design the most economic foundation.

If the site previously used for buildings such residential, mining, industrial, or other structures, then obstruction such as ground contamination can hinder immediate excavation works. Such considerations need to be taken into account while the type of foundation is selected.

Water table is another significant criterion that affects the foundation selection. Foundation should not be placed on soil that undergoes expansion and contraction due to water table fluctuation. So, it should be construction on fully dry soil or on fully wet soil. If the groundwater table is located below the formation level of the foundation, then a shallow footing like isolated or combined footing should be chosen. Moreover, for higher water table, raft/mat footing should be proposed. This is to counteract uplift pressure and counter the effect caused by water to avoid any overturning moments at the early stage of construction. If this option cannot be executed, then deep foundations like micro piles or bored piles should be considered to provide the necessary uplift resistance.

If the foundations of an adjoining structure are very near to the target foundation, it may affect the choice as the safety of adjoining structure is of paramount importance. The selection of proper foundation type would be more challenging it the neighboring structure is high rise building or an old property.

If the area has a history of severe natural events or extreme weather, then these parameters should be selected taken these parameters into considerations. The past record or data of natural disasters and extreme weather should be employed as a base for foundation selection.

The best foundation selection solution for a specific soil stratigraphy is based on the sub-soil characteristics and loads. As per description of (Ahmed S, 2015) isolated Footings are usually used when the loads on the columns are relatively small and the bearing capacity of the soil supporting the foundations is large. Combined Footings are used when the loads on the columns are heavy and the distance between these columns is relatively small (less than 30 cm) and when as an alternative to neighbor footing which is an eccentrically loaded footing and it’s danger if used when the load on the column is heavy. Mat Foundations are used if the area of isolated and combined footing > 50% of the structure area, if the bearing capacity of the soil is small (usually < 150 KN/m^2) and if the soil supporting the structure classified as (bad soils) such as: Expansive Soil, Compressible soil and Collapsible soil. For these shallow foundations suitable bearing strata may be at 1.5 m and greater below ground surface. (Omer, 2003) states that pile foundations are the most suitable foundations to combat the high swelling soil and to resist unexpected differential movement in order to transfer superstructure loads safety to deep layers, and to provide an anchor sufficiently for uplift pressure upon the active zone area.

Whether it is for shallow or deep foundations, it is recommended that foundation support cost be defined as the total cost of the foundation system divided by the load the foundation supports in tons.

2.2.5. Economic foundation selection

When there are number of feasible foundation choices for the given project site, economical factor may influence the selection of the foundation. Nonetheless, choosing an economical foundation should not compromise the safety, workmanship, strength, and durability of the foundation. Whether it is for shallow or deep foundations, it is recommended that foundation support cost be defined as the total cost of the foundation system divided by the load the foundation supports in tons.

2.2.6. Why Cost Evaluation is Necessary?

(Wright S, Ditullio L and Mcgowan J, 2006) assessed different stratigraphy of poor soil under various loads where different deep piling solutions were chosen. He calculated the average total deep piling foundation cost as 27.8% of the wind turbine costs in poor soil stratigraphy. On the other hand, on a number of usual relatively healthy soil stratigraphy (Johansson, J., Hassanzadeh, M., Engström, S., 2010) showed that the average cost for shallow foundations is about 5% of total cost. Therefore, in the relevant literature mentioned before one can see how the cost of deep foundations is considered very high in poor soil conditions and thus in this situation usually shallow foundations are chosen.

An example of this situation was found on three on shore wind farms with a very weak first soil layer in East Anglia, where the contractor could not deliver the chosen shallow foundations solution due to high costs stemming from the weak first layer. He reported that the dimensions of the shallow foundation had to be significantly increased and the cost of shallow foundations would be also raised. He proposed a solution including three different kinds and dimensions of densely positioned precast friction piles for each of the different stratigraphy and was therefore able to reduce the cost and fulfill the contract. The contractor also relied on his expertise from previous wind turbine foundations in the area that allowed him to secure at a low cost the needed piling equipment (Handley, 2006).

For major projects, if the estimated costs of alternative foundation systems during the design stage are within 15 percent of the total cost, then alternate foundation designs should be considered for inclusion in contract documents. If alternate designs are included in the contract documents, both designs should be adequately detailed. If not, material costs and installed foundation cost will not likely determine the low bid. Use of alternate foundation designs will generally provide the most cost-effective foundation system. Additionally, the foundation form, depth and dimensions are chosen to reduce the cost, the time, and the difficulty of construction.

2.2.7. Design of foundation

According to (Ming, 2015) the design of foundations mainly includes three primary aspects

1) Bearing capacity (ultimate limit state)
2) Settlement (serviceability limit state)
3) Structural design (ultimate and serviceability limit states)

2.2.8. Bearing capacity

According to (Ming, 2015) bearing capacity is the capability of the soil beneath a foundation to support a superstructure load. It is convenient to consider that bearing capacity is the general term used to describe the load carrying capacity of foundation soil or rock in term of average pressure that enables it to bear and transmit loads from a structure.

i) The Ultimate Bearing Capacity of Soil (q_u)

Ultimate bearing capacity of soil is the maximum bearing capacity of soil at which the soil fails by shear (maximum value the soil can bear it). As stated by (Joseph, 1997) if the contact ground stress imposed by the structural load exceeds the ultimate bearing capacity, the shear stresses induced in the ground would cause plastic shear deformation within the foundation’s influence zone. So, we must design a foundation for a bearing capacity less than the ultimate bearing capacity to prevent shear failure in the soil.

ii) Ultimate gross bearing capacity (q_{ult})

As defined by (Muni, 2011) gross ultimate bearing capacity is the sum of the ultimate net bearing capacity and the overburden pressure above the footing base. Ultimate gross bearing capacity (or ultimate gross soil pressure), gross $\mathbf{q_{ult}}$, is the minimum value of the gross effective contact pressure at which the supporting material fails in shear.

iii) Net Ultimate Bearing Capacity (q_{nu})

(V.N.S., 2001) states that the net ultimate bearing capacity (q_{nu}) is the pressure at the base level of the foundation in excess of the effective overburden pressure. It is the minimum value of the excess effective contact pressure at which the supporting material fails in shear, expressed as

$$q_{nu} = q_{ult} - \delta'_o \tag{2.10}$$

$$\delta'_o = \gamma \mathrm{Df} \tag{2.11}$$

Where

q_{nu}- Net Ultimate Bearing Capacity

q_{ult}- Ultimate gross bearing capacity

δ'_o- effective overburden pressure

γ- unit weight

D_f-depth of foundation

iv) Allowable bearing capacity or safe bearing capacity (q_{all})

(Muni, 2011) indicate that allowable bearing capacity or safe bearing capacity (q_{all}) is the working pressure that would ensure a margin of safety against collapse of the structure from shear failure. The allowable bearing capacity is usually a fraction of the ultimate net bearing capacity. As description given by (Ming, 2015) the allowable bearing capacity is used to compare with the maximum stress due to the superstructure. The allowable bearing capacity q_{all} is the ultimate bearing capacity q_{ult} divided by an appropriate factor of safety (FS).

$$q_{all} = \frac{q_{ult}}{FS} \tag{2.12}$$

Where

q_{ult} = ultimate bearing capacity,

q_{all} = allowable bearing capacity,

FS = factor of safety. In foundation design, the acceptable factor of safety is generally not less than 3.0.

2.2.9. Factor of safety (FS)

As stated by (Muni, 2011) factor of safety is the ratio of the ultimate net bearing capacity to the allowable net bearing capacity or to the applied maximum net vertical stress. In geotechnical engineering, a factor of safety between 2 and 5 is used to calculate the allowable bearing capacity.

Table 2.2 Typical Factors of Safety as U.S. Army Corps of Engineers (U.S. Army Corps of Engineers, 1992)

Structure	FS
Footings	3
Mats	>3
Groups	3

Source: Engineering and Design bearing capacity of soils manual, U.S. Army Corps of Engineers Washington, DC, 1992

2.2.10. Allowable bearing capacity of foundation from SPT

2.2.10.1. Design N Values

As stated by (Joseph, 1997) in the boring an average N value is used in the zone of major stressing. For example, for a spread footing the zone of interest is from about one-half the footing width B above the estimated base location to a depth of about 2B below. Weighted averaging using depth increment * N may be preferable to an ordinary arithmetic average; that is,

$$N_{avg} = \sum N * \frac{Z_i}{\sum Z_i} \quad and\ not \frac{\sum N_i}{i} \tag{2.13}$$

N_{avg}=Blow count average in range of 0.5B and 2B

For pile foundations there may be merit in the simple averaging of blow count N for any stratum unless it is very thick. Here it may be better to subdivide the thick stratum into several "strata" and average the N count for each subdivision. The average corrected N'_{70} (or other base value) can then be computed from the average field-measured N and stratum data, or individual corrected N'_{70} values can be computed and then averaged.

2.2.10.2. Factors affecting the SPT blow count

In a field standard penetration test, the blow count depends on many factors, including the hammer efficiency, borehole diameter, type of samplers and sampling depths.

a) Hammer Efficiency Correction, (η_H)

The hammer efficiency correction is assigned based on countries in which it is fabricated, hammer type and hammer release. The hammer efficiency of China's Donut type rope and pully hammer is assigned to be 60.

b) Drill rod length correction factor (η_R)

The drill rod length correction factor depends on length of the rod. For rod length greater than 10 meter it is assigned as 1 and for 4 to 10 meter it is as 0.85 to 0.95.

c) Sampler correction factor, (η_S)

The sampler correction factor for standard sampler without liner assigned as 1.2.

d) Bore hole diameter correction factor (ηB)

Table 2.6 Bore hole correction

The bore hole diameter correction factor depends on bore hole diameter. For bore hole diameter 60 to 120mm it is assigned as 1.

e) Correction Factor for Overburden Pressure in Granular Soils, CN

There are a number of empirical relations proposed for correcting N values for overburden pressure (C_N). However, the most commonly used relationship is as follows

$$C_N = \left(\frac{P_a}{q'_o}\right)^{0.5} \leq 1.7 \qquad (2.14)$$

Where

C_N - overburden pressure

q'_o - Effective overburden pressure at the location of the average N_{55} count (KNm^2)

$q'_o = \gamma' * D_f$

γ - unit weight of soil

D_f- embedment depth of foundation,

Pa –Atmospheric pressure (P_a= 100 KN/m^2)

f) Correction factor for groundwater level (η_w)

If groundwater level is within a depth B below the base of the footing. The groundwater correction factor is

$$C_w = \frac{1}{2} + \left(\frac{z}{2(D_f + B)}\right) \qquad (2.15)$$

Where

Z-depth to the groundwater table

D_f- Footing depth

B-Footing width

If the depth of the groundwater level is beyond B from the bottom of the footing base C_w=1

g) The standard blow count (N60)

The standard blow count N_{60} can be computed from the measured N as follows

$$N_{60} = N * \eta_H * \eta_R * \eta_S * \eta_B \qquad (2.16)$$

Where

N- average for influence zone of about 0.5B above footing base to at least 2B below

η_H- Hammer Efficiency Correction

η_R- rod length correction factor

η_S- Sampler correction factor

η_B- Bore hole diameter correction factor

Standard blow count N_{60} Corrected for granular soil ($N_{1(60)}$) and groundwater

$$N_{1(60)} = N_{60} * C_N * C_W \qquad (2.17)$$

The field-corrected standard penetration number N_{60} can be used to derive certain soil characteristics.

2.2.10.3. Allowable bearing capacity of isolated and combined footing

$$q_{all} = (\frac{N}{F_1})K_d \qquad B \le F_4 \qquad (2.18)$$

$$q_{all} = \left(\frac{N}{F_2}\right)\left(\frac{(B + F_3)}{B}\right)^2 K_d \qquad B > F_4 \qquad (2.19)$$

Table 2.3 F factors

	N55	N70
F1	0.05	0.04
F2	0.08	0.06
F3	0.3	0.3
F4	1.2	1.2

Where

$$K_d = 1 + 0.33\frac{D_f}{B} \quad \le 1.33$$

N_{55} and N_{70} is the corrected SPT blow count corresponding to a 55% and 70% hammer efficiency respectively

q_{all}- allowable bearing capacity (in kPa) for ΔHo = 25mm settlement

B- is the width in m

D_f-depth of foundation in m

N is the statistical average value for the footing influence zone of about 0.5B above footing base to at least 2B below

2.2.10.4. Allowable bearing capacity of mat foundation

(Joseph, 1997) also provided the following expression for the allowable bearing capacity of mat foundations for assumed 25-mm and for any settlement.

$$q_{all} = \frac{N}{F_2} K_d \qquad (2.20)$$

For any settlement

$$q'_{all} = \frac{\Delta H_i}{\Delta H_o} q_{all} \qquad (2.21)$$

where

$$K_d = 1 + 0.33 \frac{D_f}{B} \quad \leq 1.33$$

q'_{all}- allowable bearing capacity of mat (in kPa) for any settlement

ΔH_i= any settlement (in millimeters)

ΔH_o= 25mm settlement

2.2.10.5. Estimation of ultimate bearing pressure of single piles from SPT data

The following formulas were recommended by (Joseph, 1997) to find the ultimate load-carrying capacity of single piles.

$$Q_u = Q_p + Q_s \qquad (2.22)$$

Where

Q_p-load-carrying capacity of the pile point

Q_s- frictional resistance

Again, he recommended the following formula to find the ultimate point resistance q_p in homogeneous granular soil

$$q_P = (40N)_{60} \frac{L}{D} \leq (400N_{60}) \quad (KN) \qquad (2.23)$$

where

q_p-Ultimate Point Resistance

N_{60} = statistical average of the SPT N_{60} numbers in a zone of about 8B above to 3B below the pile point

D= diameter of pile point

L = length of the pile

Additionally, he extends his recommendation as follows to find the average unit frictional resistance for high displacement driven piles

$$f_{av} = 2N_{60} \quad \left(\frac{KN}{m2}\right) \qquad (2.24)$$

ii) for low displacement driven piles

$$f_{av} = N_{60} \quad \left(\frac{KN}{m2}\right) \qquad (2.25)$$

Thus,

$$Q_S = PLf_{av} \qquad (2.26)$$

where

f_{av}-average unit frictional resistance

N_{60} = statistical average of the SPT N_{60} numbers in a zone of about 10B above to 4B below the pile point

D = perimeter

L- Length

Q_s-frictional or skin resistance of a pile

2.2.10.6. Estimation of ultimate bearing pressure of group piles from SPT data

(Joseph, 1997) also recommended equation for ultimate bearing pressure of group piles from the standard penetration test (SPT) data as follows:

$$Q_{ult} = \frac{N_{55}D}{0.0318}1.33x1.5 = 62.7N_{55}D \qquad (kpa) \qquad (2.27)$$

where

N_{55} SPT blow count corresponding to a 55% hammer efficiency

D- pile diameter in m

Q_{ult} = ultimate bearing pressure of group piles

2.2.10.7. Vertical Bearing capacity of pile groups

As stated by (Ming, 2015) in a pile group, the minimum center-to-center spacing of 3b (b = individual pile diameter or width) is recommended to optimize the group capacity and minimize installation problems. Ultimate bearing capacity of a pile group may be different from the sum of the ultimate bearing capacity of individual piles. The efficiency of a pile group is defined as:

$$\eta_g = \frac{(Q_u)_g}{nQ_u} \qquad (2.28)$$

Where

η_g- group efficiency

$(Q_u)_g$- ultimate load-bearing capacity of the group pile

Q_u- ultimate load-bearing capacity of each pile without the group effect

n- number of piles in the pile group,

As stated by (V.N.S., 2001) the group efficiency of a pile group is defined as follows:

$$\eta g = 1 - \theta \left[\frac{(n-1)m + (m-1)n}{90mn}\right] \qquad (2.29)$$

Where

η_g - group efficiency

$\theta = \tan^{-1}$ (d/s) in degrees

d = diameter of pile,

s = spacing of piles center to center.

n- is the number of rows in the group

m- is the number of columns in the group

2.2.11. Settlement of Foundation (serviceability limit state)

Often times, a foundation failure is not due to inadequate bearing capacity, but due to excessive settlement. Therefore, settlement is another important factor that controls foundation design. Allowable or limiting settlement of a building structure will depend on the nature of structure, the foundation and the soil. Different types of structures have varying degree of tolerance to settlement and distortions. These variations depend on the type of construction, use of the structure, rigidity of the structure and the presence of sensitive finishes. (Braja, 2010) demonstrate that a total settlement of 25mm and a differential settlement of 20mm between columns in most buildings shall be considered safe for building on isolated pad footings on sand for working load (un factored). A total settlement of 40mm and a differential settlement of 20mm between columns shall be considered safe for building on isolated pad footings on clay soil for working load. (Ahmed S, 2015) suggest that allowable settlement of shallow foundations may control the allowable bearing capacity. The allowable settlement itself may be controlled by local building codes. For example; the maximum allowable settlement for mat foundation is 50 mm, and 25 mm for isolated footing (Ministry of Construction, 2015). (Muni, 2011) stated that practically it is impossible to prevent settlement of shallow foundations. At least, elastic settlement will occur. As a geotechnical engineer one's duty is to prevent the foundation system from reaching a serviceability limit state. Based on settlement consideration (Joseph, 1997) recommended the following equation for net allowable bearing pressure of shallow foundations from the standard penetration test data in sand soil.

$$q_{net} = q_{all} - \Upsilon D_f \tag{2.30}$$

$$q_{net} = (\frac{N_{60}}{0.05}) K_d (\frac{S_e}{25}) \qquad B \leq 1.22 \tag{2.31}$$

$$q_{net} = \left(\frac{N_{60}}{0.08}\right)\left(\frac{(B+0.3)}{B}\right)^2 K_d (\frac{S_e}{25}) \qquad B > 1.22 \tag{2.32}$$

where

$K_d = 1 + 0.33 \frac{D_f}{B} \quad \leq 1.33$

Se = tolerable settlement (mm)

q_{net} = net allowable bearing pressure

B = width (m)

N_{60} = statistical average of the SPT value for depth of 2B to 3B, measured from the bottom of the foundation.

The net allowable bearing capacity for mats constructed over granular soil deposits can be adequately determined from the standard penetration resistance numbers.

$$q_{net} = \frac{N_{60}}{0.08} K_d \left(\frac{S_e}{25}\right) \leq 16.63 N_{60} \left[\frac{S_e}{25}\right] \tag{2.33}$$

$$q = \frac{Q}{A} - \Upsilon D_f \tag{2.34}$$

$$\frac{Q}{A} \leq q_{net} \tag{2.35}$$

where

q_{net}- The net ultimate bearing capacity

q- The net pressure applied on a foundation

Q - dead weight of the structure and the live load

A -area of the raft

Hence, in all cases, q should be less than or equal to q_{net}

$$S_e = \frac{1.25 q_{net}}{N_{60} K_d} \qquad (B \leq 1.22) \tag{2.36}$$

$$S_e = \frac{2 q_{net}}{N_{60} K_d} (\frac{B}{B+0.3})^2 \quad B > 1.22 \tag{2.37}$$

Where

$\mathrm{K_d} = \text{depth factor} = 1 + 0.33 \frac{\mathrm{D_f}}{\mathrm{B}} \quad \leq 1.33$

B- foundation width (m)

S_e-settlement (mm)

q_{net}- The net ultimate bearing capacity

N_{60}- standard penetration resistance between the bottom of the foundation and 2B below the bottom.

2.2.12. Modulus of Subgrade Reaction

(Joseph, 1997) states that the modulus of subgrade reaction is a conceptual relationship between soil pressure and deflection that is widely used in the structural analysis of foundation members. It is used for continuous footings, mats, and various types of pilings. The modulus of subgrade reaction method is preferable owing to its greater ease of use and to the substantial savings in computer computation time. As definition made by (Braja, 2010) a foundation of width B is subjected to a load per unit area of q, it will undergo a settlement Δ. The coefficient of subgrade modulus can be defined as

$$k = \frac{q}{\Delta} \qquad (2.38)$$

$$q = \frac{W}{A} \qquad (2.39)$$

Where

W-foundation weight

Δ- Settlement

k- coefficient of subgrade reaction

A- Area of the foundation3

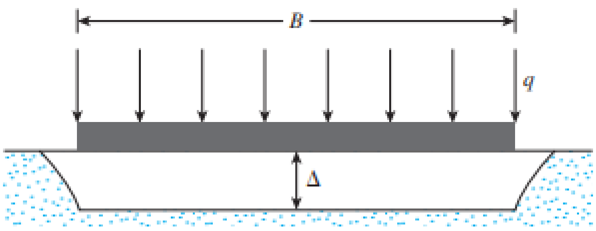


FIGURE 2.4 Definition of coefficient of subgrade reaction, k

Source: Braja M. Das, Principles of Soil Dynamics, United States of America, 2011.

As stated by (N. S. V. , 2011) the value of modulus of subgrade reaction (k_s) formulated as follows

$$k_s = \frac{1}{B(mm)}\left(0.65 \sqrt[12]{\frac{E_s B^4}{E_f I_f}}\right)\frac{E_s}{1-v_s^2} \approx \frac{E_s}{B(1-v_s^2)} \quad (2.40)$$

Where

B, I_f, E_f = width, moment of inertia of the cross section and modulus of elasticity of the footing respectively

E_s, v_s = modulus of elasticity and Poisson's ratio of the soil

2.2.13. Relative Density

As stated by (Braja, 2010) the correlation between N_{60} and D_r is as follows

$$\mathrm{D_r} = \left(\frac{\mathrm{N_{60}}}{(17 + (24 * (\mathrm{q_o}/\mathrm{P_a})))}\right)^{0.5} \quad (2.41)$$

Where

Dr-Relative Density

q'o- Effective overburden pressure

Pa –Atmospheric pressure (P_a= 100 KN/m^2)

N_{60}- Standard blow count

2.2.14. Effective friction angle

As stated by (Ming, 2015) the correlation between N_{60} and Effective friction angle is as follows

$$\Phi' = \tan^{-1}\left(\frac{\mathrm{N_{60}}}{(12.2 + (20.3 * (\mathrm{q'}/\mathrm{P_a})))}\right)^{0.34} \quad (2.42)$$

Where

Φ - Effective friction angle

q'o- Effective overburden pressure

Pa –Atmospheric pressure (P_a= 100 KN/m^2)

N_{60}- Standard blow count

As stated by (Joseph, 1997) the equation for buildings from SPT N-value and angle of friction is as follows,

$$\Phi = 4.5 * N_{70} * +20 \quad (2.43)$$

Φ – Angle of friction

N_{70}*- Is the average SPT value standardize N_{70}

Again, (Joseph, 1997) states based on N_{55} angle of friction can calculated as follows

$$\Phi = 25 + 28 * \left(\frac{N_{55}}{q'_o}\right)^{0.5} \quad (2.44)$$

Where

N_{55} – Is the average SPT value at a depth about 0.75B below the proposed base of the footing and corrected using C_N

q'_o – effective overburden pressure at location of average N_{55} count

2.2.15. Maximum shear modulus (G_{Max})

As stated by (Braja, 2010) the correlation between N_{60} and Maximum shear modulus is as follows

$$G_{Max} = 35 * 161.5 * N_{60}{}^{0.34} * q'^{0.4}_o \qquad (2.45)$$

Where

G_{Max} - Maximum shear modulus (kpa)

q'_o- Effective overburden pressure (kpa)

N_{60}- Standard blow count

2.2.16. Equations for stress-strain modulus E_s

$$\text{For Clayey sand soil} \qquad Es = 320 * (N + 15) \qquad (2.46)$$

$$\textit{For Silts, Sandy silt, or Clayey silt soil} \qquad Es = 300 * (N + 6) \qquad (2.47)$$

Where

N- The values estimated as N_{55}

E_s- stress-strain modulus

2.3. Modeling and analysis of foundation

2.3.1. Modeling and analysis of shallow foundation

Based on relative stiffness of the foundation structure and the supporting soil, (American Society of Civil Engineers, 2000) calculate the foundation stiffness using one of the following three methods.

a) Method 1

Shallow footings can be idealized using three uncoupled springs (FIGURE 2.5 (b)). The uncoupled spring model is limited to those applications when the shallow footing is considered to be rigid with respect to the supporting soils.

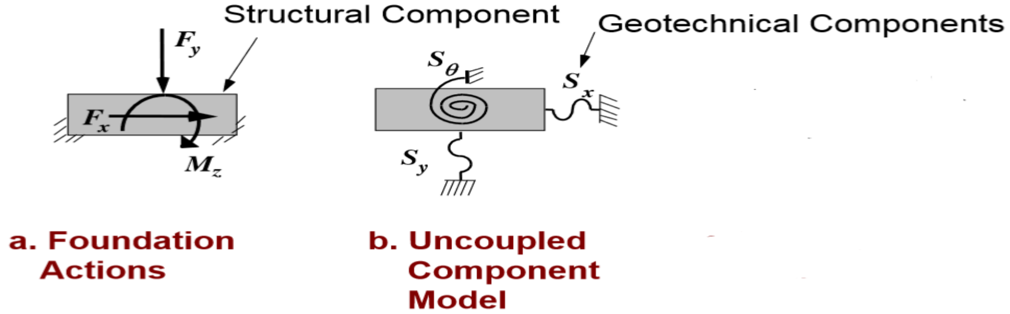
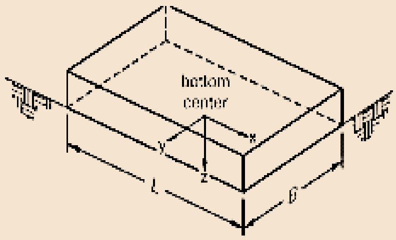


FIGURE 2.5 Rigid with respect to the supporting soil, an uncoupled spring model

Source: American Society of Civil Engineers, Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, Virginia, 2000.

(American Society of Civil Engineers, 2000) reports that spring stiffness solutions that are applicable to any solid basement shape on the surface of, or partially or fully embedded in, a homogenous half space has two step calculations as in next FIGURE.2.6 1st the stiffness terms are calculated for foundation at the surface, then an embedment correction factor is calculated for each stiffness term.

Degree of Freedom	Stiffness of Foundation at Surface	Note
Translation along x-axis	$K_{x,sur} = \frac{GB}{2-v}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$	
Translation along y-axis	$K_{y,sur} = \frac{GB}{2-v}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$	
Translation along z-axis	$K_{z,sur} = \frac{GB}{1-v}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$	
Rocking about x-axis	$K_{xx,sur} = \frac{GB^3}{1-v}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]$	
Rocking about y-axis	$K_{yy,sur} = \frac{GB^3}{1-v}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$	Orient axes such that $L \geq B$
Torsion about z-axis	$K_{zz,sur} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$	



Degree of Freedom	Correction Factor for Embedment	Note
Translation along x-axis	$\beta_x = \left(1 + 0.21\sqrt{\frac{D}{B}}\right) \cdot \left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$	
Translation along y-axis	$\beta_y = \beta_x$	
Translation along z-axis	$\beta_z = \left[1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right] \cdot \left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$	d = height of effective sidewall contact (may be less than total foundation height)
Rocking about x-axis	$\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$	h = depth to centroid of effective sidewall contact
Rocking about y-axis	$\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$	For each degree of freedom, calculate $K_{emb} = \beta K_{sur}$
Torsion about z-axis	$\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$	

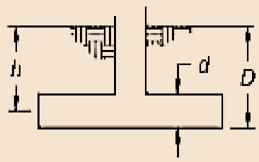


FIGURE 2.6 Elastic solutions for rigid footing spring constants

Source: American Society of Civil Engineers, Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, Virginia, 2000.

b) Method 2

Again, (American Society of Civil Engineers, 2000) describe that for shallow bearing foundations that are not rigid with respect to the supporting soils; a finite element representation of linear or nonlinear foundation behavior using Winkler models shall be used. Distributed vertical stiffness properties shall be calculated by dividing the total vertical stiffness by the area. Uniformly distributed rotational stiffness properties shall be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. Vertical and rotational stiffness shall be decoupled for a Winkler model. It shall be permitted to use the procedure illustrated in next FIGURE to decouple this stiffness. The stiffness per unit length in these end zones is based on the vertical stiffness of a BXB/6 isolated footing. The stiffness per unit length in the middle zone is equivalent to that of an infinitely long strip footing.

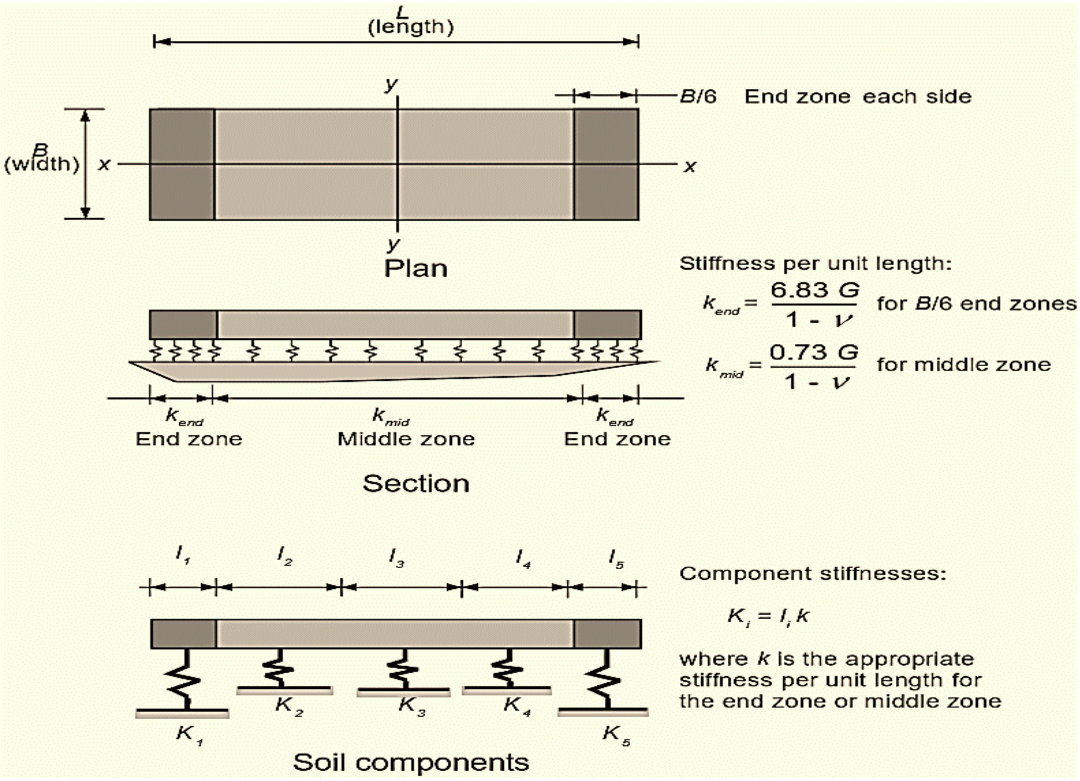


FIGURE 2.7 Vertical stiffness modeling for shallow bearing footing

Source: American Society of Civil Engineers, Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, Virginia, 2000.

c) Method 3

For shallow bearing foundations that are flexible relative to the supporting soil, based on approved theoretical solutions for beams or plates on elastic supports, the foundation stiffness shall be permitted to be calculated by a decoupled Winkler model using a unit subgrade spring coefficient. For flexible foundation systems, the unit subgrade spring coefficient, k_{sv}, shall be calculated by the following Equation.

$$k_{sv} = \frac{1.3G}{B(1-v)} \qquad (2.48)$$

Where

G- Shear modulus

B- Width of footing

v-Poisson's ratio

2.3.2. Modeling and analysis of Pile Foundations

A pile foundation shall be defined as a deep foundation system composed of one or more driven or cast-in-place piles and a pile cap cast-in-place over the piles, which together form a pile group supporting one or more loadbearing columns, or a linear sequence of pile groups supporting a shear wall. The requirements of this section shall apply to piles less than or equal to 60 cm in diameter.

2.3.3. Stiffness Parameters

The uncoupled spring model shown in FIGURE 2.7 shall be used to represent the stiffness of a pile foundation where the footing in the FIGURE represents the pile cap. As per described by (American Society of Civil Engineers, 2000) in calculating the vertical and rocking springs, the contribution of the soil immediately beneath the pile cap shall be neglected. The total lateral stiffness of a pile group shall include the contributions of the piles (with an appropriate modification for group effects) and the passive resistance of the pile cap. The lateral stiffness of piles shall be based on classical methods or on analytical solutions using approved beam-column pile models. The (Winkler, 1876) utilizes a series of unconnected linear springs with stiffness (E_s) to model the soil, with this approach the behavior of a single pile can be analyzed using the equation of an elastic beam supported on an elastic foundation. Another term utilized usually instead of E_s is the soil lateral subgrade reaction modulus (k_h) which is expressed in units of force per unit volume. The relationship between them is given by $E_s = k_h D$,

Where D is the pile diameter

Pile group axial spring stiffness values, k_{sv}, shall be calculated using the following equation.

$$k_{sv} = \sum_{n=1}^{N} \frac{AE}{L} \qquad (2.49)$$

Where

A- Cross-sectional area of a pile

E- Modulus of elasticity of piles

L-Length of piles

N- Number of piles in group

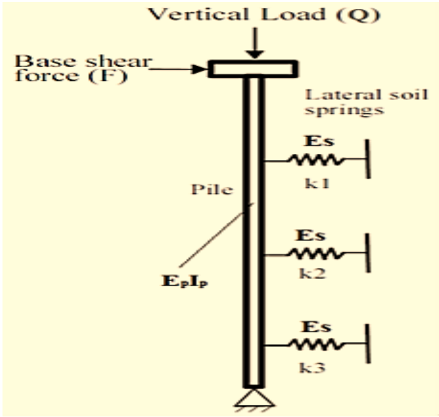


FIGURE 2.8 Pile modeling base on Winkler method

The rocking spring stiffness values about each horizontal pile cap axis shall be computed by modeling each pile axial spring as a discrete Winkler spring. The rotational spring constant, k_{sr}, (moment per unit rotation) shall be calculated using the following equation.

$$k_{sr} = \sum_{k=1}^{n} k_{vn} S_n^2 \qquad (2.50)$$

Where

K_{vn}- Axial stiffness of the n^{th} pile

S_n-Distance between n^{th} pile and axis of rotation

2.4. Geotechnical Investigation

2.4.1. Regional Geology

The Ethiopian rift system which is part of the East African Rift System may be subdivided into three main sections. There are: the south western rift zone, the main Ethiopian rift, and afar. Adama is part of the central Rift Valley and dominated by flatlands stretching between the escarpments bordering the eastern and western sides of the Rift Valley. Adama is found within the Wonji fault belt, which is one of the main structural systems in the Ethiopian rift valley. Its physiographic condition is, therefore, mainly the result of volcano-tectonic activities that occurred in the past and also partly the result of the deposition of sediments, which are considered largely of fluvial and lacustrine origin.

2.4.2. Approximate Soil Identification

It is sometimes useful to be able to make a rapid field identification of the site soil for some purpose. This can be done approximately as follows:

i) Differentiate gravel and sand by visual inspection.

ii) Differentiate fine sand and silt by placing a spoonful of the soil in a deep jar (or test tube) and shaking it to make a suspension. Sand settles out in 11/2 minutes or less whereas silt may take 5 or more minutes. This test may also be used for clay, which takes usually more than 10 minutes. The relative quantities of materials can be obtained by observing the depths of the several materials in the bottom sediment.

iii) Differentiate between silt and clay as follows:

a) Clay lumps are more difficult to crush using the fingers than silt.

b) Moisten a spot on the soil lump and rub your finger across it. If it is smooth, it is clay; if marginally streaked it is clay with silt; if rough it is silt.

c) Form a plastic ball of the soil material and shake it horizontally by jarring your hand. If the material becomes shiny from water coming to the surface it is silt.

iv) Differentiate between organic and inorganic soils by visual inspection for organic material or a smell test for wood or plant decay odor.

2.4.3. Field Exploration

2.4.3.1.Spacing and Depth of Exploratory Borings

Required number of boring layouts at a particular site depend on the nature of the soil, the size and importance of the project. As stated by (Ming, 2015) required number of boring layouts shown on Table 2.8 as follows:

Table 2.4 Required number of boring layouts

No	Project type	Required number of boring
1	Large structure with separate closely spaced footings	Space borings approximately 15m apart in both directions
2	Development of site on soft compressible strata	Space borings 30 to 60 m apart
3	Isolated rigid foundation, less than 930 m2 in area	Minimum of two borings at opposite corners

As stated by (Ming, 2015) all borings should extend no less than 10m below the lowest part of foundation unless rock is encountered for Isolated rigid foundations. Again (Muni, 2011) states that for shallow foundation the depth will be about 1 to 3 times the width of the foundation from the foundation level.

2.4.3.2.Split-spoon sampling

As per (ASTM D1586-11, 2014) standard when a borehole is extended to a predetermined depth, the drill tools are removed and the sampler is lowered to the bottom of the hole. The sampler is driven into the soil by hammer blows to the top of the drill rod. The number of blows required for a spoon penetration of three 152.4-mm intervals are recorded. The number of blows required for the last two intervals are added to give the standard penetration number, N, at that depth. This number is generally referred to as the N value. The sampler is then withdrawn, and the shoe and coupling are removed. Finally, the soil sample recovered from the tube is placed in a glass bottle and transported to the laboratory. Hence, these samples are highly disturbed.

2.5. Laboratory Testing

The method of packing, storing and transportation have to follow technical specifications (ASTM D1586-11, 2014) to ensure minimum damage and disturbance. The laboratory testing for disturbed soil samples sieves analyses, Atterberg Limits and compaction tests help to classify each material type according to unified and the AASHTO soil classification system.

2.5.1. Sieve Analysis

As per (ASTM D2487-17, 2018) Standards a sieve analysis is conducted by taking a measured amount of dry, well-pulverized soil and passing it through a stack of progressively finer sieves with a pan at the bottom. The amount of soil retained on each sieve is measured, and the cumulative percentage of soil passing through each is determined. This percentage is generally referred to as percent finer. The sieve analysis is carried out by sieving a known dry mass of sample through the nest of sieves placed one below the other so that the openings decrease in size from the top sieve downwards, with a pan at the bottom of the stack. The whole nest of sieves is given a horizontal shaking for about 10 minutes (if required, more) till the mass of soil remaining on each sieve reaches a constant value (the shaking can be done using a mechanical shaker).

By determining the mass of soil sample left on each sieve the following calculation can be made.

i) Percentage retained on any sieve = (mass of soil retained/ total soil mass) *100

ii) Cumulative % retained on sieve = Sum of percentage retained on all coarser sieve

iii) Percentage finer than any sieve size =100 % minus cumulative percentage retained

The results may be plotted in the form of a graph on semi-log paper with the percentage finer on the arithmetic scale and the particle diameter on the log scale. The grain diameter, D, is plotted on the logarithmic scale and the percent finer is plotted on the arithmetic scale.

Table 2.5 Soil-Separate Size Limits

Classification system	Grain size (mm)
Unified	Gravel: 75 mm to 4.75 mm
	Sand: 4.75 mm to 0.075 mm
	Silt and clay (fines): < 0.075mm
AASHTO	Gravel: 75 mm to 2 mm
	Sand: 2 mm to 0.05 mm
	Silt: 0.05 mm to 0.002 mm
	Clay: < 0.002mm

Source: ASTM D2487 – 17 Standards

AS stated by (V.N.S., 2001) uniformly graded or poorly graded soils are soils possess particles of almost the same diameter. A well graded soil possesses a wide range of particle sizes ranging from gravel to clay size particles. A gap graded soil has some of the sizes of particles missing. To determine whether a material is uniformly graded or well graded, Hazen proposed the following equation:

$$Cu = \frac{D_{60}}{D_{10}} \qquad (2.51)$$

Where

C_u- The uniformity coefficient,

D_{60}- The diameter of the particle at 60 per cent finer on the grain size distribution curve

D_{10}- The effective grain size corresponds to 10 percent finer particles

For all practical purposes we can consider the following values for granular soils.

Cu > 4 for well graded gravel

Cu > 6 for well graded sand

Cu < 4 for uniformly graded soil containing particle of the same size

There is another step in the procedure to determine the gradation of particles. This is based on the term called the coefficient of curvature which is expressed as

$$C_c = \frac{{D_{30}}^2}{D_{10}} * D_{60} \qquad (2.52)$$

Where

C_c- The coefficient of curvature

D_{30}- The size of particle at 30 percent finer on the gradation curve

The soil is said to be well graded if C_c lies between 1 and 3 for gravels and sands. Two samples of soils are said to be similarly graded if their grain size distribution curves are almost parallel to each other on a semilogarithmic plot. When the curves are almost parallel to each other the ratios of their diameters at any percentage finer approximately remain constant.

2.5.2. Hydrometer Analysis

The common laboratory method used to determine the size distribution of fine-grained soils is a hydrometer test. The hydrometer test involves mixing a small amount of soil into a suspension and observing how the suspension settles in time. Larger particles will settle quickly, followed by smaller particles. When the hydrometer is lowered into the suspension, it will sink into the suspension until the buoyancy force is sufficient to balance the weight of the hydrometer. As per (ASTM D422-63, 2003) standards test procedure is as follows

a. Take 50gm dry sample from the soil passing through the No. 200 sieve

b. Mix this sample with 125 mL of a 4% of $NaPO_3$ solution in a small evaporating dish

c. Allow the soil mixture to stand for about 1 hour. At the end of the soaking period transfer the mixture to a dispersion cup and add distilled water until the cup is about two-thirds full. Mix for about 2 min.

d. After mixing, transfer all the contents of the dispersion cup to the sedimentation cylinder, being careful not to lose any material Now add temperature-stabilized water to fill the cylinder to the 1000 mL mark.

e. Mix the suspension well by placing the palm of the hand over the open end and turning the cylinder upside down and back for a period of 1 min. Set the cylinder down on a table.

f. Start the timer immediately after setting the cylinder. Insert the hydrometer into the suspension just about 20 seconds before the elapsed time of 2 min. and take the first reading at 2 min. Take the temperature reading. Remove the hydrometer and the thermometer and place both of them in the control jar.

g. The control jar contains 1000 mL of temperature-stabilized distilled water mixed with 125 mL of the same 4% solution of $NaPO_3$.

h. The hydrometer readings are taken at the top level of the meniscus in both the sedimentation and control jars.

i. Steps 6 through 8 are repeated by taking hydrometer and temperature readings at elapsed times of 4, 8, 16, 30, 60 min. and 2, 4, 8, 16, 32, 64 and 96 hr.

j. Necessary computations can be made with the data collected to obtain the grain distribution curve.

The diameter D (cm) of the particle at time t D (seconds) is calculated from Stokes's law as

$$\Upsilon_s = G_s \Upsilon_w \qquad (2.53)$$

$$D = \sqrt{\frac{18\mu}{(G_s - 1)\Upsilon_w}} \sqrt{\frac{L}{t}} \qquad (2.54)$$

Where

γ_s = unit weight of solid sphere

γ_w = unit weight of liquid (g/cm^3)

μ = absolute viscosity of liquid (g-sec)/cm^2

D = diameter of sphere (mm)

G_s= specific gravity

L= distance (cm)

t= time (min)

The hydrometer is used for the determination of unit weight of suspensions at different depths and particular intervals of time. A unit volume of soil suspension at a depth L and at any time / contains particles finer than a particular diameter D.

The reading R on the stem of the hydrometer (corrected for meniscus) may be expressed as

$$\mathrm{Ra} = 1.6061 * 10^3(\rho_f - 1) \qquad (2.55)$$

Where

R_a- reading on the stem of the hydrometer

P_f- density of suspension per unit volume = specific gravity of the suspension.

The ASTM 152 H hydrometer gives the distance of any reading R on the stem to the center of volume and is designated as L. The distance L varies linearly with the reading R. An expression for L may be written as follows:

$$L = L_1 + L_2/2 \qquad (2.56)$$

Where

L_1- distance from reading R to the top of the bulb

L_2- length of hydrometer bulb = 14 cm for ASTM 152 H hydrometer

For an ASTM 152 H hydrometer, the value of L for any reading R (corrected for meniscus) may be obtained from

$$L = 16.3 - 0.1641 * R \qquad (2.57)$$

2.5.3. Determination of Percent Finer

As per (ASTM D422-63, 2003) standards hydrometer is calibrated to read from 0 to 60g of soil in a 1000 mL suspension with the limitation that the soil has a specific gravity G = 2.65. The reading is, of course, directly related to the specific gravity of the suspension. Lesser the specific gravity of the suspension, the deeper the hydrometer will sink into the suspension. It must also be remembered here, that the specific gravity of water decreases as the temperature rises from 4° C. The use of a dispersing agent also affects the hydrometer reading. Corrections for this can be obtained by using a sedimentation cylinder of water from the same source and with the same quantity of dispersing agent as that used in the soil-water suspension to obtain a zero correction. If the temperature during the test is quite high, the density of water will be equally less and hydrometer will sink too deep. The values of temperature correlation are tabulated as follow:

Table 2.6 Temperature correction factors C_T

Temp oC	CT	Temp oC	CT
15	-1.10	23	+0.70
16	-0.90	24	+1.00
17	-0.70	25	+1.30
18	-0.50	26	+1.65
19	-0.30	27	+2.00
20	0.00	28	+2.50
21	+0.20	29	+3.05
22	+0.40	30	+3.80

The actual hydrometer reading R_a has to be corrected as follows

1.correction for meniscus C_m only for use in Eq. (3.24)

2.zero correction C_o and temperature correction C_r for obtaining percent finer.

Reading for obtaining percent finer

$$R_c = R_a - C_o + C_T \qquad (2.58)$$

Where

R_a - Actual hydrometer reading

R_c - Hydrometer reading correction

C_o- Zero correction

C_T- Temperature correction

The 152 H hydrometer is calibrated for a suspension with a specific gravity of solids G_s = 2.65. If the specific gravity of solids used in the suspension is different from 2.65, the percent finer has to be corrected by the factor C_{sg} expressed as

$$C_{sg} = 1.65 * \frac{G_s}{2.65 * (G_s - 1)} \qquad (2.59)$$

Where

C_{sg} -correction for specific gravity different from 2.65

G_s- Specific gravity

The percentage of particles finer than a particle diameter D in the mass of soil M_s used in the suspension with the correction factor C_s may be expressed as

$$P' = \frac{C_{sg} * R_c}{M_s} * 100 \qquad (2.60)$$

Where

P'- Percent of finer

C_{sg}- correction for specific gravity different from 2.65

R_c- Hydrometer reading correction

M_s - Mass of soil used in the suspension in gms (not more than 60 gm)

If M_p is the mass of soil particles passing through 75micron sieve and M the total mass taken for the combined sieve and hydrometer analysis, the percent finer for the entire sample which gives points for plotting a grain size distribution curve may be expressed as

$$P = P'\% * \left(\frac{M_p}{M}\right) \qquad (2.61)$$

Where

P- Corrected percent

Mp- Mass of soil particles passing through 75micron sieve

M- Total mass taken for the combined sieve and hydrometer analysis

2.5.4. Atterberg Limits

(ASTM D4318-00, 2000) standard states that the water contents corresponding to the transition from one state to another are termed as Atterberg Limits and the tests required to determine the limits are the Atterberg Limit Tests. The transition state from the liquid state to a plastic state is called the liquid limit, w_l. At this stage all soils possess a certain small shear strength. This arbitrarily chosen shear strength is probably the smallest value that is feasible to measure in a standardized procedure. The transition from the plastic state to the semisolid state is termed the plastic limit, w_P. At this state the soil rolled into threads of about 3 mm diameter just crumbles. Further decrease of the water contents of the same will lead finally to the point where the sample can decrease in volume no further. Further decrease in water in the voids occurs without change in the void volume. The color of the soil begins to change from dark to light. This water content is called the shrinkage limit, w_s. The range of water content between the liquid and plastic limits, which is an important measure of plastic behavior, is called the plasticity index, I_p.

$$I_P = W_l - W_P \qquad (2.62)$$

Where

I_P- plasticity index

W_l- liquid limit

W_P- plastic limit

Where sampling operations have preserved the natural stratification of a sample, the various strata must be kept separated and tests performed on the particular stratum of interest with as little contamination as possible from other strata. Specimen-Obtain a representative portion from the total sample sufficient to provide 150 to 200 g of material passing the 425-µm (No. 40) sieve.

i) Liquid Limit

As per (ASTM D4318-00, 2000) standard Casagrande liquid limit device used for determining the liquid limits of soils. The device contains a brass cup which can be raised and allowed to fall on a hard rubber base operated by either a hand crank or electric motor. The cup is raised by one cm. The limits are determined on that portion of soil finer than a No. 40 sieve or 425micron sieve. About 100g of soil is mixed thoroughly with distilled water into a uniform paste. A portion of the paste is placed in the cup and leveled to a maximum depth of 10 mm. A channel of the dimensions of 11 mm width and 8 mm depth is cut through the sample along the symmetrical axis of the cup. The grooving tool should always be held normal to the cup at the point of contact. The handle is turned at a rate of about two revolutions per second and the number of blows necessary to close the groove along the bottom for a distance of 12.5 mm is counted. The groove should be closed by a flow of the soil and not by slippage between the soil and the cup. The water content of the soil in the cup is altered and the tests repeated. At least four tests should be carried out by adjusting the water contents in such a way that the number of blows required to close the groove may fall within the range of 5 to 40. Within the range of 5 to 40 blows, the plotted points lie almost on a straight line. The curve so obtained is known as a 'flow curve'. The liquid limit is defined as the water content at which the groove cut into the soil will close over a distance of 12.5 mm following 25 blows. The results are presented in a plot of water content (ordinate, arithmetic scale) versus terminal blows (abscissa, logarithmic scale). The equation of the flow curve can be written as

$$W = -I_f \log N + C \qquad (2.63)$$

Where

W- water content

If- slope of the flow curve, termed as flow index

N-number of blows

C-a constant

ii) Liquid Limit by One-Point Method

The determination of liquid limit as explained earlier requires a considerable amount of time and labor. We can use what is termed the 'one-point method' if an approximate value of the limit is required. The formula used for this purpose is

$$LL = W(N/25)^n \qquad (2.64)$$

Where

LL-Liquid limit

W- water content

N-one-point liquid limit for given trial, %,

n-An index whose value equals 0.068 to 0.121. An average value of 0.104 may be useful for all practical purposes.

iii) Plastic Limit

About 15 gm of soil, passing through a No. 40 sieve, is mixed thoroughly. The soil is rolled on a glass plate with the hand, until it is about 3 mm in diameter. This procedure of mixing and rolling is repeated till the soil shows signs of crumbling. The water content of the crumbled portion of the thread is determined. This is called the plastic limit.

iv) Water content

(ASTM D2216-10, 2011) tests were undertaken to test the optimum moisture content and corresponding maximum dry unit weight of selected samples. Water content of the soil is defined as the percentage of water present in soil mass by its weight.

Procedure

1) Determine the mass of the empty can (W_1)
2) Determine the combined mass of moist soil and the can (W_2)
3) Determine combined mass of dry soil after enough drying in oven for 24 hours (W_3)

Calculation

$$\text{Mass of moisture} = (W_2 - W_3) \quad (2.65)$$

$$\text{Mass of dry soil} = (W_3 - W_1) \quad (2.66)$$

$$\text{Water content (W\%)} = \frac{W_2 - W_3}{W_3 - W_1} * 100 \quad (2.67)$$

2.5.5. Soil Classification

Soil classification is used to specify a certain soil type that is best suited for a given application. Each classification was devised for a specific use. For example, the American Association of State Highway and Transportation officials (AASHTO) developed one scheme that classifies soils according to their usefulness in roads and highways, while the Unified Soil Classification System (USCS) was originally developed for use in airfield construction but was later modified for general use.

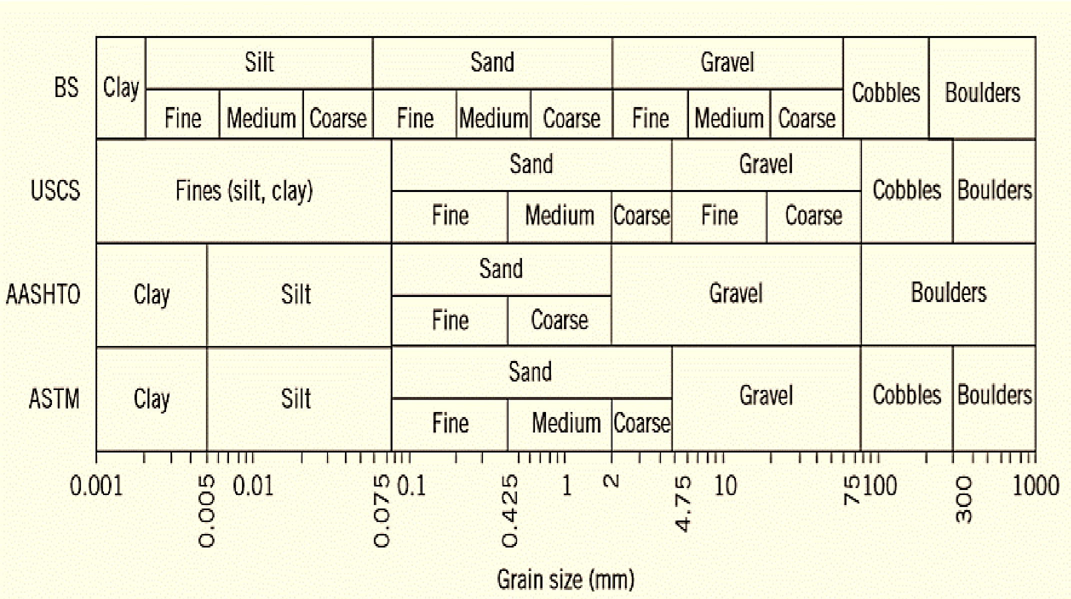


FIGURE-2.9 Comparison of four systems for describing soils based on particle size.

Organic soils are defined as soil deposits that contain a mixture of soil particles and organic (peat) matter. They may be identified by observation of peat-type materials, a dark color, and/or a woody odor. (ASTM D2487-00, 2000) currently suggests that the organic classification (OL, OH) be obtained by performing the liquid limit on the natural soil, then oven-drying the sample overnight and performing a second liquid limit test on the oven-dry material. If the liquid limit test after oven drying is less than 75 percent of that obtained from the undried soil, the soil is "organic.

Fine-grained soils have poor load-bearing capacities compared with coarse-grained soils. Fine grained soils are practically impermeable, change volume and strength with variations in moisture conditions, and are susceptible to frost. The engineering properties of coarse-grained soils are controlled mainly by the grain size of the particles and their structural arrangement. The engineering properties of fine-grained soils are controlled by mineralogical factors rather than grain size. Thin layers of fine-grained soils, even within thick deposits of coarse-grained soils, have been responsible for many geotechnical failures, and therefore you need to pay special attention to fine-grained soils. (Joseph, 1997) states that the Unified Soil Classification System (USCS) is much used in foundation work.

i) Classification of Soils for Engineering Purposes (Unified Soil Classification System)

(ASTM D2487-00, 2000) standard requirement lists the following procedure for Classification of Fine-Grained Soils

1.Preliminary Classification Procedure

i) Class the soil as fine-grained if 50 % or more by dry weight of the test specimen passes the No. 200 (75-µm) sieve

ii) Class the soil as coarse-grained if more than 50 % by dry weight of the test specimen is retained on the No. 200 (75-µm) sieve

ii) Procedure for Classification of Fine-Grained Soils

1) The soil is an inorganic clay if the position of the plasticity index versus liquid limit plot, falls on or above the "A" line, the plasticity index is greater than 4, and the presence of organic matter does not influence the liquid limit
2) Classify the soil as a lean clay, CL, if the liquid limit is less than 50.
3) Classify the soil as a fat clay, CH, if the liquid limit is 50 or greater.
4) Classify the soil as a silty clay, CL-ML, if the position of the plasticity index versus liquid limit plot falls on or above the "A" line and the plasticity index is in the range of 4 to 7.
5) The soil is an inorganic silt if the position of the plasticity index versus liquid limit plot, falls below the "A" line or the plasticity index is less than 4, and presence of organic matter does not influence the liquid limit
6) Classify the soil as a silt, ML, if the liquid limit is less than 50.
7) Classify the soil as an elastic silt, MH, if the liquid limit is 50 or greater.
8) The soil is an organic silt or clay if organic matter is present in sufficient amounts to influence the liquid limit
9) If the soil has a dark color and an organic odor when moist and warm, a second liquid limit test shall be performed on a test specimen which has been oven dried at 110 ± 5°C to a constant weight, typically overnight.
10) The soil is an organic silt or organic clay if the liquid limit after oven drying is less than 75 % of the liquid limit of the original specimen determined before oven drying
11) Classify the soil as an organic silt or organic clay, OL, if the liquid limit (not oven dried) is less than 50 %. Classify the soil as an organic silt, OL, if the plasticity index is less than 4, or the position of the plasticity index versus liquid limit plot falls below the "A" line. Classify the soil as an organic clay, OL, if the

plasticity index is 4 or greater and the position of the plasticity index versus liquid limit plot falls on or above the “A” line.

12) Classify the soil as an organic clay or organic silt, OH, if the liquid limit (not oven dried) is 50 or greater. Classify the soil as an organic silt, OH, if the position of the plasticity index versus liquid limit plot falls below the “A” line. Classify the soil as an organic clay, OH, if the position of the plasticity index versus liquid-limit plot falls on or above the “A” line.

13) If less than 30 % but 15 % or more of the test specimen is retained on the No. 200 (75-µm) sieve, the words “with sand” or “with gravel” (whichever is predominant) shall be added to the group name. For example, lean clay with sand, CL; silt with gravel, ML. If the percent of sand is equal to the percent of gravel, use “with sand.”

14) If 30 % or more of the test specimen is retained on the No. 200 (75-µm) sieve, the words “sandy” or “gravelly” shall be added to the group name. Add the word “sandy” if 30 % or more of the test specimen is retained on the No. 200 (75-µm) sieve and the coarse-grained portion is predominantly sand. Add the word “gravelly” if 30 % or more of the test specimen is retained on the No. 200 (75-µm) sieve and the coarse-grained portion is predominantly gravel. For example, sandy lean clay, CL; gravelly fat clay, CH; sandy silt, ML. If the percent of sand is equal to the percent of gravel, use “sandy.”

2.5.6. Soil Compaction

(ASTM D 698-07, 2007) standard states that compaction is the densification of soils by removal of air through the application of mechanical energy. The degree of compaction is measured in terms of its dry unit weight.

The compaction test Procedure is as follows:

1. Prepare at least four the soil specimens for testing in accordance with ASTM standard requirement.
2. Determine the weight of the soil sample as well as the weight of the compaction mold with its base (without the collar) by using the balance and record the weights.
3. Compute the amount of initial water to add by the following method:
   a) Assume water content for the first test to be 8 percent.
   b) Compute water to add from the following equation:
4. $\text{water to add (in ml)} = \frac{(\text{soil mass in grams})8}{100}$ (2.68)

5. Where “water to add” and the “soil mass” are in grams. gram of water is equal to approximately one milliliter of water.
6. Measure out the water, add it to the soil, and then mix it thoroughly into the soil using the trowel until the soil gets a uniform color.
7. Assemble the compaction mold to the base, place some soil in the mold and compact each layer with 25 blows for 101.6mm mold. The drops should be applied at a uniform rate not exceedingly around 1.5 seconds per drop, and the rammer should provide uniform coverage of the specimen surface.
8. The soil should completely fill the cylinder and the last compacted layer must extend slightly above the collar joint. If the soil is below the collar joint at the completion of the drops, the test point must be repeated.
9. Carefully remove the collar and trim off the compacted soil so that it is completely even with the top of the mold using the trowel. Replace small bits of soil that may fall out during the trimming process.
10. Weigh the compacted soil while it’s in the mold and to the base, and record the mass. Determine the wet mass of the soil by subtracting the weight of the mold and base.
11. Remove the soil from the mold using a mechanical extruder and take soil moisture content samples from the top and bottom of the specimen. Fill the moisture cans with soil and determine the water content.
12. Place the soil specimen in the large tray and break up the soil until it appears visually as if it will pass through the # 4 sieve, add 2 percent more water based on the original sample mass, and re-mix as in step 4. Repeat steps 5 through 9 until, based on wet mass, a peak value is reached followed by two slightly lesser compacted soil masses.

2.5.7. Calculations and Plotting of Compaction Curve

1.Calculate the moisture content of each compacted soil specimen by using the average of the two water contents.

$$W_s = \left((\text{Mass of compacted soil} + \text{mold}) - \text{Mass of mould}\right) * 9.81 \quad (2.80)$$

2.Compute the wet density in grams per cm3 of the compacted soil sample by dividing the wet mass by the volume of the mold used.

$$\Upsilon_{moist} = \frac{W_s}{V} \qquad (2.69)$$

Where:

W_s -weight of wet soil

Y_{moist} - moist unit weight

V-volume

3.Calculate the molding water content of each compacted specimen as explained below.

$$W_t = \frac{W_m}{(mass\ of\ dry\ soil + can - mass\ of\ can)} \quad (2.70)$$

Where

W_t- water content

W_m-mass of water

4.Compute the dry density using the wet density and the water content by using the following formula:

$$Y_d = \frac{Y_{moist}}{(1 + (w_t/100))} \quad (2.71)$$

Where

Y_{moist}- moist unit weight

Y_d- dry unit weight

W_t- water content

5.To calculate points for plotting the 100 % saturation curve or zero air voids curve, select values of dry unit weight, calculate corresponding values of water content corresponding to the condition of 100 % saturation as follows:

$$W_{sat} = \frac{Y_w * G_s - Y_d}{Y_d * G_s} * 100 \quad (2.72)$$

where:

w_{sat} = water content for complete saturation, nearest 0.1 %,

Y_w = unit weight of water (9.789 kN/m3) at 20°C,

Y_d = dry unit weight of soil, (kN/m3), three significant digits, and

Gs = specific gravity of soil (estimated or measured), to nearest 0.01 value,

6. Plot the dry density values on the y-axis and the moisture contents on the x-axis. Draw a smooth curve connecting the plotted points.

2.6. Data analysis

2.6.1. Regression

Regression is used to assess the strength of a relationship between one dependent and independent variable(s). It helps to predicting value of a dependent variable from one or more independent variable. Regression analysis helps in predicting how much variance is being accounted in a single response (dependent variable) by a set of independent variables.

A) Bivariate regression

Two variables are involved in the bivariate regression. One is the dependent variable that is to be predicted, the other is independent variable that explains the variance in the dependent variable. Bivariate regression is similar to bivariate correlation, because both are design for situation in which there are just two variables. The purpose of this kind of regression can be either prediction or explanation; however, bivariate regression is most frequently used to see how well scores on the dependent variable can be predicted from data on the independent variable.

B) Multiple regression

In multiple regression, however, there are two or more independent variables. Stated differently, multiple regression involves just one y variable but two, three, or more X variables. The different kinds of regression are like correlation, since they are utilized to evaluate the relationship among variables. This make us to think regression is another way to measure correlation, however there are difference in correlation and regression in their purpose, labeling of variable and kind of inferential tests applied.

Table 2.7 Differences between correlation and regression

Characteristics	Correlation	Regression
Purpose of technique	Association/connection between two variables	Understanding link i-e prediction and explanation
Labels attached	No clear labels	Clear distinction of dependent and independent variable
Inferential tests	Correlation coefficient	Regression coefficient, intercept, t-statics, change in regression coefficient.

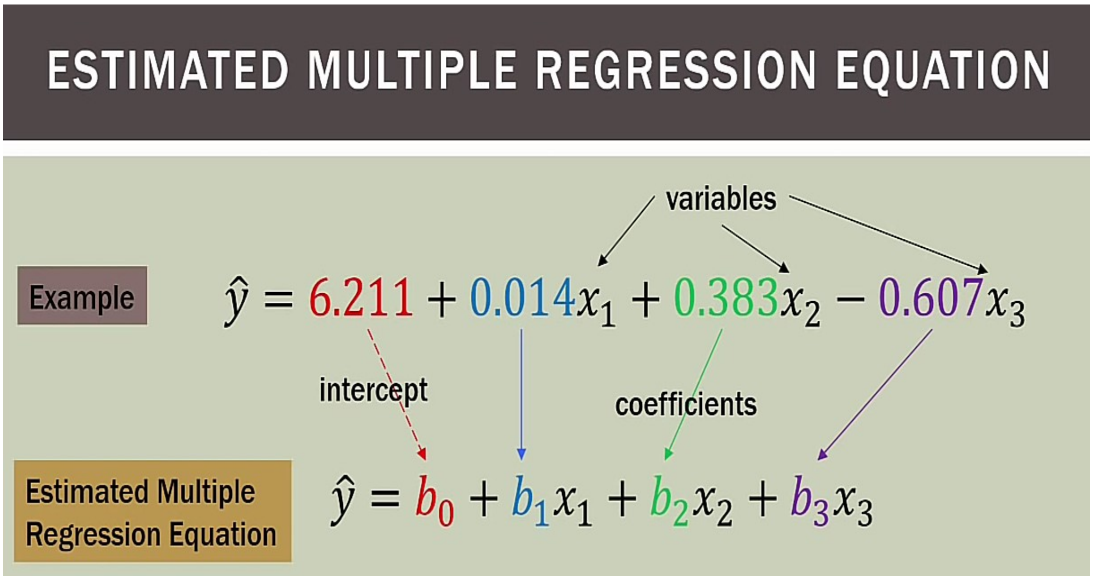
Both bivariate and multiple regression almost always involve inferential tests and measure of the extent to which variability among the scores on the denpendent variable has been explained or accounted for.

C) Regression coefficient

Regression coefficient is a measure of how strongly each independent also known as predictor variable predicts the dependent variable. There are two types of regression coefficient, unstandardized coefficients and standardized coefficient, also known as beta values.

D) unstandardized coefficients

An unstandardized coefficient is used in the equation as coefficients of different independent variables along with the constant terms to predict the value of dependent variable. Standardized coefficient is, however, measured in standard deviations.



To estimate the parameters β_0, β_1..., β_p using the principle of least squares, form the sum of squared deviations of the observed y_j 's from the regression line:

$$\sum_{i=1}^{n} \varepsilon_i^2 = \sum_{i=1}^{n} (Y_i - (\beta_0 - \beta_1 - \cdots - \beta_k x_{pi}))^2 \qquad (2.73)$$

The least squares estimates are those values of the β_is that minimize the equation. You could do this by taking the partial derivative with respect to each parameter, and then solving the k+1 unknowns using the k+1 equations.

E) R values

Regression analysis would provide us with two different R values. A simple R value represents the correlation between the observed values and predicted values (based on the regression equation obtained) of the dependent variable. The other R values is referred to as R square, it is the square of R and gives the proportion of variance in the dependent variables accounted for by the set of independent variables chosen for the model.

R square is used to find out how well the independent variables are able to predict the dependent variable. However, the R square value tend to be a bit inflated when the number of independent variables is more or when the number of cases is large. The adjusted R square takes into account these things and gives more accurate information about the fitness of the model. For example, R square value of 0.70 would mean that the independent variables in the model can predict 70% of the variance in the dependent variable.

Just as with simple regression, the error sum of squares is

$$SSE = \sum (y_i - \bar{Y}_i)^2 \qquad (2.74)$$

It is again interpreted as a measure of how much variation in the observed y values is not explained by (not attributed to) the model relationship. The number of d_f associated with SSE is n–(p+1) because p+1 d_f are lost in estimating the p+1 β coefficients. Just as before, the total sum of squares is

$$SST = \sum (y_i - \bar{Y})^2 \qquad (2.75)$$

And the regression sum of squares is:

$$SSR = \sum (\bar{Y}_i - \bar{Y})^2 = SST - SSE \qquad (2.76)$$

Then the coefficient of multiple determination

$$R^2 = 1 - \frac{SSE}{SST} = \frac{SSR}{SST} \qquad (2.77)$$

The objective in multiple regression is not simply to explain most of the observed y variation, but to do so using a model with relatively few predictors that are easily interpreted. It is thus desirable to adjust R^2 to take account of the size of the model:

$$R_a^2 = 1 - \frac{\frac{SSE}{(n-(p+1))}}{\frac{SST}{(n-1)}} = 1 - \frac{n-1}{n-(p+1)} * \frac{SSE}{SST} \qquad (2.78)$$

Because the ratio in front of SSE/SST exceeds 1, is smaller than R^2. Furthermore, the larger the number of predictors P relative to the sample size n, the smaller will be relative to R_2. Adjusted R^2 can even be negative, whereas R^2 itself must be between 0 and 1. A value of that is substantially smaller than R^2 itself is a warning that the model may contain too many predictors. SSE is still the basis for estimating the remaining model parameter

$$\tilde{\sigma}^2 = S^2 = \frac{SSE}{n-(p+1)} \quad (2.79)$$

F) A Model Utility Test

The model utility test in simple linear regression involves the null hypothesis H_0: $\beta_1 = 0$, according to which there is no useful linear relation between Y and the predictor X. In MLR we test the hypothesis H_0: $\beta_1 = 0$, $\beta_2 = 0$..., $\beta_p = 0$, which says that there is no useful linear relationship between Y and any of the P predictors. If at least one of these β's is not 0, the model is deemed useful. We could test each β separately, but that would take time and be very conservative. A better test is a joint test, and is based on a statistic that has an f distribution when H_0 is true.

Null hypothesis: H_0: $\beta 1 = \beta 2 = \ldots = \beta p = 0$

Alternative hypothesis: Ha: at least one $\beta i \neq 0$ (i = 1..., p)

Test statistic value:

$$f = \frac{\frac{SSR}{p}}{\frac{SSE}{(n-(p+1))}} \quad (2.80)$$

Rejection region for a level α test: $f \geq F \geq \alpha, p, n-(p+1)$

CHAPTER THREE

MATERIALS AND METHODS

3.1.Materials employed in the research

3.1.1. Geotechnical Site Investigation

3.1.1.1.Project Description

On the basis of the agreement made with the Yejoka general business group and ZTY Engineering plc, geotechnical investigation were carryout for designing of medium height buildings of B+G+8 reinforced concrete. Two (2) test pits are drilled to a depth of 15m below the natural ground level. The geotechnical site investigation was carried out by auger drilling and the necessary soil samples are collected for laboratory testing. The report generally covers the investigation of subsurface soil formation and the bearing capacity of the foundation layers.

3.1.1.2.Scope of geotechnical investigation Services

The scopes of the geotechnical investigation include core drilling, in situ testing, collection of representative samples and subsequent laboratory tests to determine the engineering properties and ground water measurements. The purposes of the investigation are:

1) Determine the type and extent of geological layers;
2) Investigate the presence of ground water and identify its level if encountered;
3) Determine the engineering properties of the geotechnical layers constituting the sub-surface geology of the site;
4) Develop engineering recommendations to guide design and construction of the project by Performing laboratory tests on selected representative soil samples from the boreholes to evaluate pertinent engineering properties and preparing geotechnical investigation report.

3.1.1.3.Site Location

The site is located in Adama city, Bedatu kebele, Oromia Regional State, Ethiopia. The site was owned by Yejoka general business group plc., for the construction of B+G+8 Building. Residential building is currently existed on site.

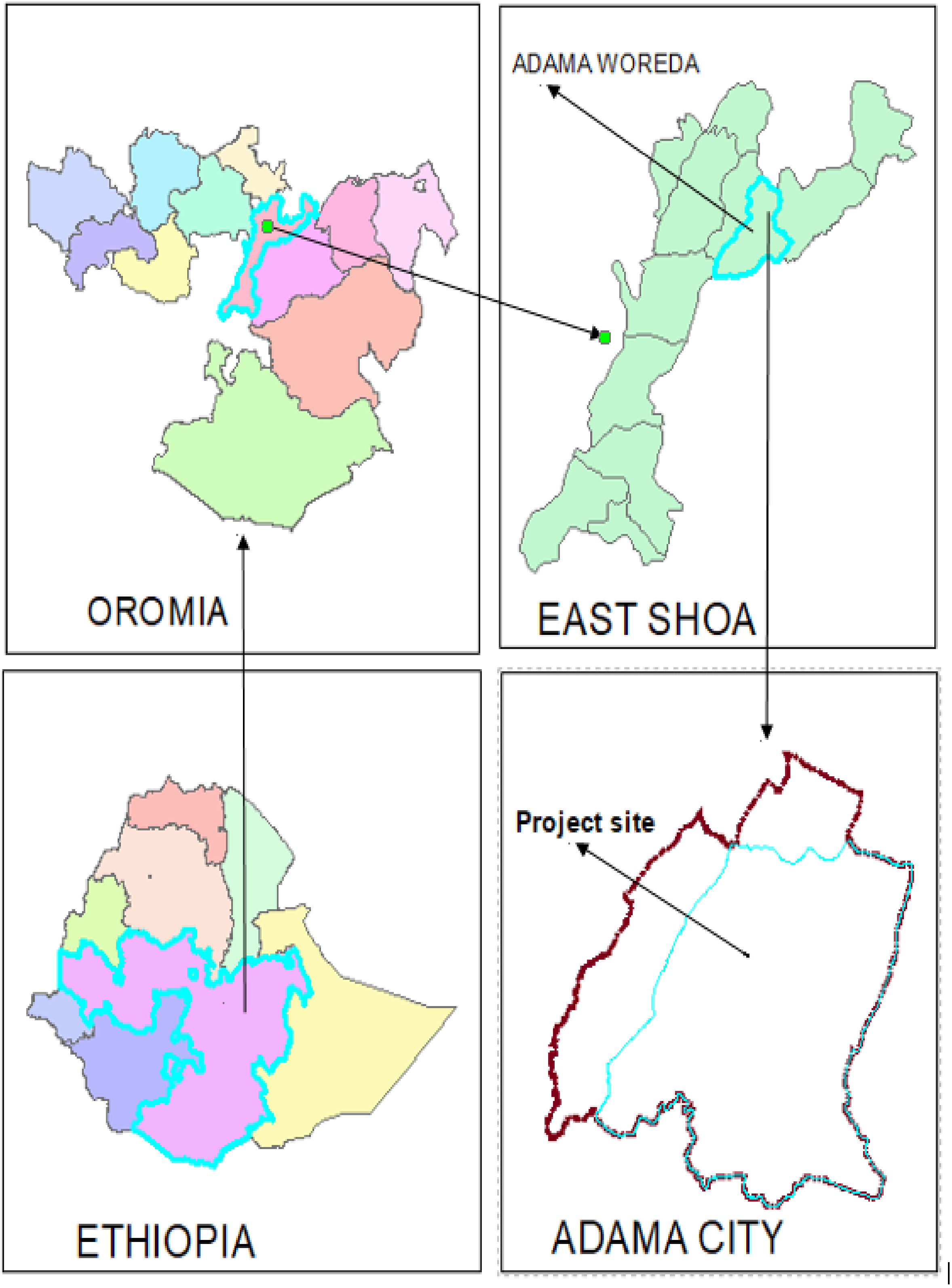


FIGURE 3.1 Location map of project site

3.1.1.4.Existing Condition of Building Site

The project area of the proposed site for B+G+8 Building is generally characterized by flat topographical feature and slopes gently to the south. The following photos show the existing condition of the site.

FIGURE 3.2 Partial View of the Project Site

3.2. Methods, Tools and techniques employed in the research

At first the forces and moments coming from the structure above the ground modeled and analyzed by (CSI) software, ETABS v18.0.1. according to ESEN code. The values of forces and moments generated from the structure different from each other due to number of stories above it.

Representative soil samples collected from the boreholes were taken to ASTU Soil laboratory subjected to different kind of quantitative and qualitative tests. The method of packing, storing and transportation was in accordance with the technical specifications ASTM to ensure minimum damage and disturbance. The laboratory testing for disturbed soil samples sieve analyses, Atterberg Limits and compaction tests on selected soil samples, were used to help classify each material type according to unified and the AASHTO soil classification system.

The results of standard penetration test have been correlated with engineering properties of soils, bearing capacity, and settlement of foundations as suggested by many investigators.

The pressure against the footings or the bearing capacity of foundation soil was evaluated by using standard penetration test. Blow count are corrected after calculation of average N value in the zone of major stressing by considering overburden stress and instrumental influencing factors. Allowable bearing pressures are computed for isolated and combined footings placed at a depth of 3m below the lowest natural ground level for immediate settlement limited to 25mm and allowable bearing pressures for mat foundations are computed at a depth of 3m below the lowest natural ground level for immediate settlement limited to 50mm. Again, allowable bearing pressures are computed for Pile foundation placed at different depth of safety factor 3.

After modeling and analyzing the super structure by ETABSv18.0.1 software and generating the values of forces and moments from super structure, these forces and moments transport to SAFE v16.0.1. software for modeling foundation. Based on relative stiffness of the foundation structure and the supporting soils isolated, combined, mat and pile foundations were modeled, analyze and structural foundation design was done and finally checked by manually prepared hand calculations excel sheet to produce the most suitable design solution. Different parameters, such as settlements, bending moments and shear forces, that can impact on both foundation design is evaluated using both approaches.

The quantity and cost of modeling of building foundations includes the quantity and cost of non-structural elements and structural elements.

a) For buildings with isolated footings, the quantity of non-structural elements such as earthwork items and stone masonry under grade beams expressed in terms of quantities per m^3 and form work in terms of quantities per m^2. Structural elements such as foundations, foundation columns, grade beams expressed in terms of quantities per m^3 and reinforcement rebar in terms of quantities per kg.

b) For buildings with combined footings, the quantity of non-structural elements such as earthwork items and stone masonry under grade beams expressed in terms of quantities per m^3 and form work in terms of quantities per m^2. Structural elements such as foundations, foundation columns, grade beams expressed in terms of quantities per m^3 and reinforcement rebar in terms of quantities per kg.
c) For buildings with raft foundations, the quantity of non-structural elements such as earthwork items expressed in terms of quantities per m^3 and form work in terms of quantities per m^2. Structural elements such as slab of uniform thickness covering the entire footprint of the building and foundation columns expressed in terms of quantities per m^3 and reinforcement rebar in terms of quantities per kg.
d) For buildings with pile foundations, the quantity of non-structural elements such as earthwork items expressed in terms of quantities per m^3 and form work in terms of quantities per m^2. Structural elements such as piles, pile caps and the tie beams connecting the pile caps expressed in terms of quantities per m^3 and reinforcement rebar in terms of quantities per kg.

Finally, Multiple regression is used to develop suitable cost estimation approaches for selection of foundation. Multiple regression analysis helps in predicting how much variance is being accounted in a single response (dependent variable) by a set of independent variables. IBM SPSS software tool able to calculate the strength of a relationship, clear distinction and change in regression coefficient between one dependent (bearing capacity) and independent variables (different building height and foundation cost).

The proposed cost model for the four types of foundations considered takes into account two important parameters; number of stories and allowable bearing pressure of soils and to brings out suitable cost estimation approaches for selection of foundation during early stages of different foundation design which is constructed on Silty clay and Sandy silt soils.

3.3.Procedure employed in the research

To meet the objectives of the study, procedure of the study is summarized on chart 3.1.

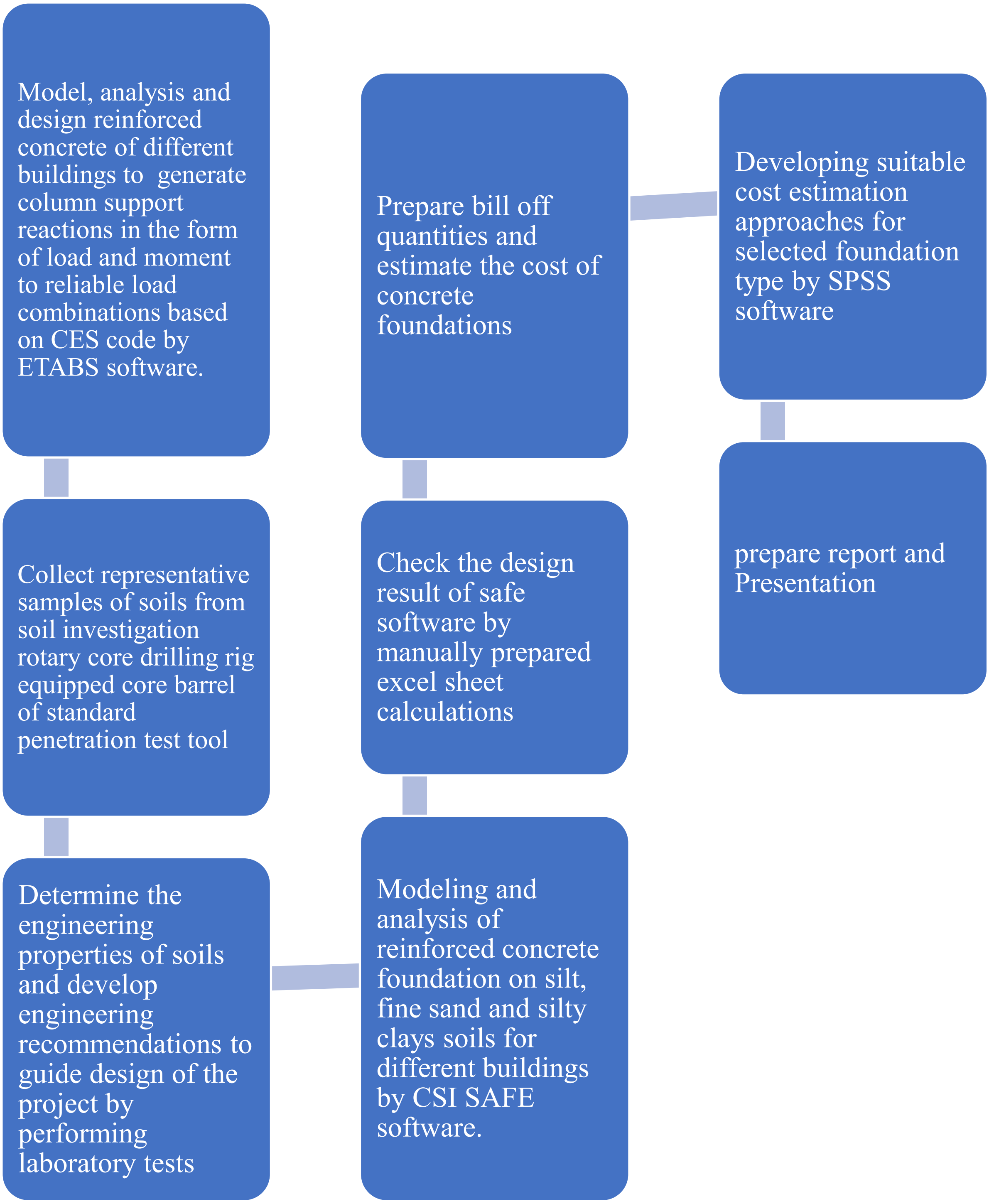


FIGURE 3.3 Flow chart of study procedure

3.4. Data organization and scoring procedure

This thesis is systematically arranged to cover the overall view of developing suitable cost estimation approaches for selection of foundation and is comprised of five chapters starting with introductory chapter. The second chapter comprehensively review the literature of the available related topics to this study such as general and specific criteria for modeling, analysis and design of super structure, sub structure and geotechnical site investigation and laboratory testing. Concise assessment, modeling, analysis, design and cost of Isolated, Combined, Mat and Pile foundation done and briefly concluded with developing suitable cost estimation equation between dependent variable and a set of independent variables. The third chapter states tools and techniques employed in the research as well as procedure of the study have been employed. The fourth chapter fully describe how to develop the model that predicts cost-effective foundation type selection during early stages of foundation design after checking its reliability of two important parameters; number of stories and allowable bearing pressure of soils. In the Chapter five, the conclusions and recommendations for construction and future work are presented.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1.Introduction

Different buildings G^{+2}, G^{+4}, G^{+8} and G^{+10} is modeled as per ESEN code and approaches described in chapter 3 for generating values of forces and moments from super structure. Again, based on relative stiffness of the foundation structure and the supporting soils isolated, combined, mat and pile foundations were modeled, analyze and structural foundation design was done and checked by manually prepared excel sheet hand calculations. The analysis considers the effect of increasing the applied loads over the footings on the underlying soil, maximum vertical, horizontal stresses and settlements. The quantities and costs of steel reinforcement, reinforced concrete, excavation and form works for the foundation under this study were also considered. The proposed cost model for the four types of foundations considered takes into account two important parameters; number of stories and allowable bearing pressure of soils and brings out the cost implications for designing the foundations in sandy clay and sandy silt soils. To create a model that predicts the specific outcome being researched multiple linear regression were used in SPSS software program. This allow to assess the impact of one factor when another is taken into account.

4.2.Geotechnical Site Investigation Findings

4.2.1 Local Geology

Specific conditions encountered at each boring location are indicated on the individual boring logs. Stratification boundaries on the boring logs represent the approximate location of changes in soil types in-situ, the transition between materials may be gradual. Based on the results of the borings, subsurface conditions on each borehole shows more or less the same geology. Based on the result of the borings, subsurface conditions on the project site were generalized into major layers as follows.

1) The top part of the project site covered by light dark organic poor graded sandy clay; low stiff layer encountered underlying the project area. This layer soil extends to maximum of 2.to 2.5m.
2) Light brown and brown poorly graded, medium stiff, low plastic silty sand layer is encountered underlying the project area form 2.50m to 9m.
3) Light Gray, low plastic silty and fractured pumice rock layer is encountered underlying extends from 9m to 10m.

4) Brown and light brown medium stiff, inorganic silty and fine sand encountered underlying the project area form 9m to 15m in both bore holes.

4.2.2 Spacing and Depth of Exploratory Borings

Since the soil is uniform over the whole and the site is small in areas only 2 bore holes were done. The depth will be about 1times the width of the foundation from the foundation level. The site investigation was carried out by auger drilling and the necessary soil samples are collected for laboratory testing.

4.2.3 Split-spoon sampling and bore logs

The standard split tube of inside diameter of 34.93 mm and an outside diameter of 50.8 mm is used in the field. Split-spoon samples generally are taken at intervals of about 1.5 m. A total of 2 borings were drilled varying in depth from 14 and 15 meters. Finally, the soil sample recovered from the tube is placed in a well secured plastic and transported to the laboratory. Detailed logs of borings are shown on FIGURE 4.1 and 4.2. The borehole log includes important information such as description of strata, Run length, SPT–N value and sampling depth. All information's are shown on annexed Borehole log diagram.

BORING LOG: BH1					
Project		Apartment			
Client		Joca General Business Sh.C			
Location		Adama city Badatu kebele			
Comment		Starting date			
		Finishing date			
Split spoon		Hammer		Ground data	
Internal dim.	34.93mm	Type	Donut rope & pully	Water depth	None
External dim.	50.80mm	Weight	622.72N	Casing depth	5m
Length	457.20mm	Fall	762mm	Hole depth	15m
Sampling method	Sampler without liner	Drill Rod length	Depends on depth		
		Bore hole diameter	108mm		

Depth	Profil	Symbol	Depth From	Depth To	Blows 0–152.4mm	Blows 152–304mm	Blows 304–457mm	N Value	Visual material classification
1 2 3		⊠	2	2.5	4	6	6	12	Light dark organic poorly graded sandy clay
4 5		⊠	4.5	5	5	7	7	14	Light brown poorly graded sandy clay
6 7 8 9		⊠	5.5	6	5	8	8	16	Brown inorganic silt and fine sand
		⊠	8.5	9	4	9	10	19	
10									Dark brown inorganic silt and fine sand
11 12 13		⊠	11.5	12	6	11	13	24	Brown inorganic silt and fine sand
14 15		⊠	13	13.5	8	14	13	27	Dark brown inorganic silt and fine sand

FIGURE 4.1 Test pit logs for bole hole one

BORING LOG: BH2					
Project			Apartment		
Client			Joca General Business Sh.C		
Location			Adama city Badatu kebele		
Comment			Starting date		
			Finishing date		
Split spoon		Hammer		Ground data	
Internal dim.	34.93mm	Type	Donut rope & pully	Water depth	None
External dim.	50.80mm	Weight	622.72N	Casing depth	5m
Length	457.20mm	Fall	762mm	Hole depth	15m
Sampling method	Sampler without liner	Drill Rod length	Depends on depth		
		Bore hole diameter	105mm		

Depth	Profil	Symbol	Depth From	Depth To	Blows 0–152.4mm	Blows 152–304mm	Blows 304–457mm	N Value	Visual material classification
1, 2		⊠	1.5	2	3	6	6	12	Light dark organic poorly graded sandy clay
3, 4, 5, 6		⊠	3.5	4	4	7	7	14	Light brown poorly graded sandy clay
		⊠	5	5.5	5	8	8	16	
7, 8, 9		⊠	7.5	8	4	9	10	19	Brown inorganic silt and fine sand
10		⊠	9.5	10	4	11	13	24	Dark brown inorganic silt and fine sand
11, 12, 13									Brown inorganic silt and fine sand
14, 15		⊠	12	12.45	7	14	13	27	Dark brown inorganic silt and fine sand

FIGURE 4.2. Test pit logs for bole hole two

4.2.4 Groundwater Conditions

During the course of drilling ground water was not encountered in both boreholes. However, variation in the location of the long-term water table may occur as a result of changes in precipitation, evaporation, seepage and other factors not immediately apparent at the time of this exploration.

4.3. Laboratory Testing and Engineering Property of soils

After taking representative soil samples to ASTU Soil laboratory and doing sieve analyses, Atterberg Limits and compaction tests, material types classify according to unified and the AASHTO soil classification system. The results of laboratory tests are presented on annex-B.

4.3.1. Particle size distribution

After conducting sieve operation and determining the mass of each fraction on a balance and calculating the diameter of a particle corresponding to the percentage indicated by a hydrometer reading that conforming the requirements of ASTM Standards, the results plotted in the form of a graph on semi-log paper with the grain diameter, D, is plotted on the logarithmic scale and the percent finer is plotted on the arithmetic scale.

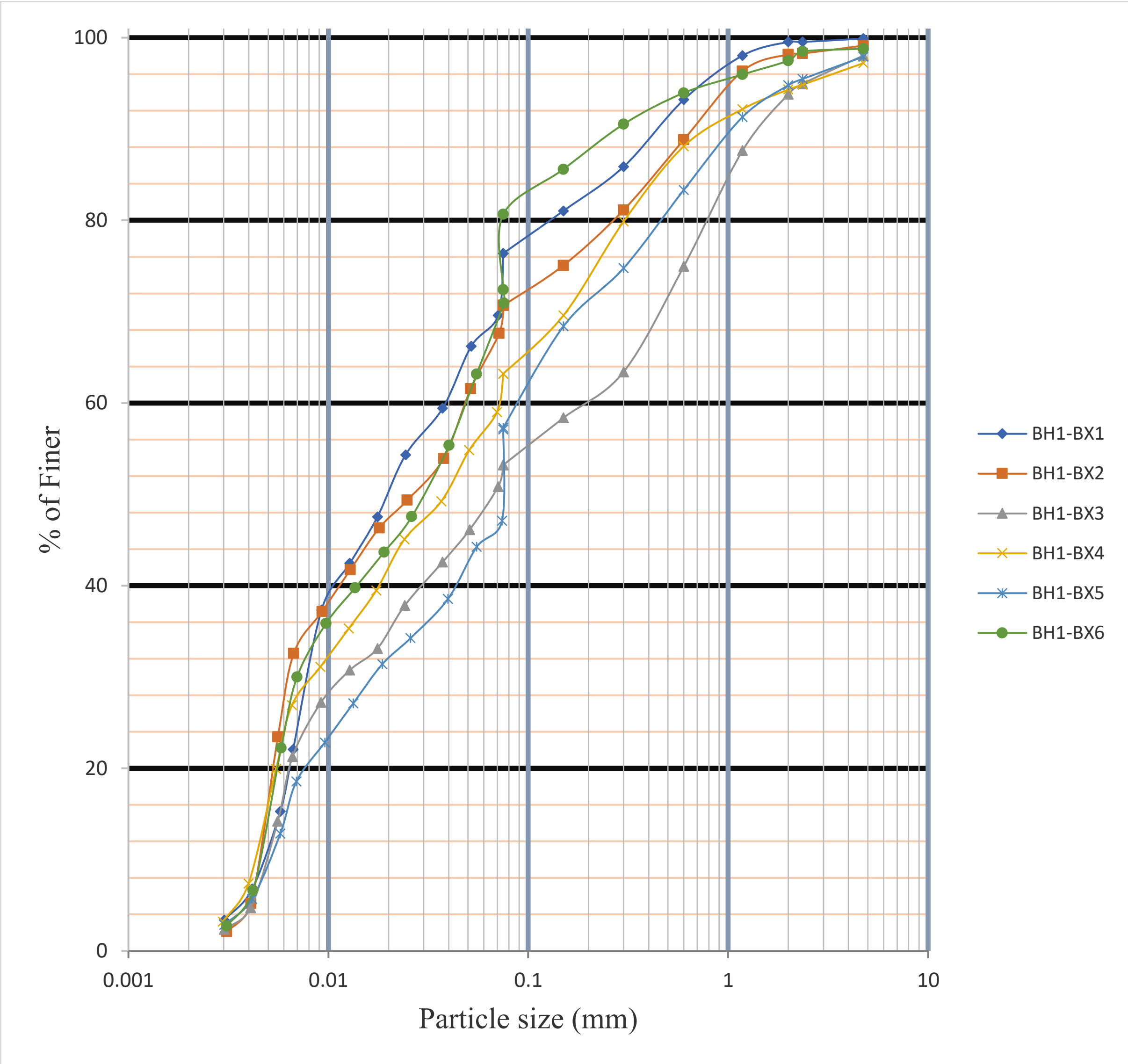


FIGURE 4.3 Grain size distribution of bore hole 1

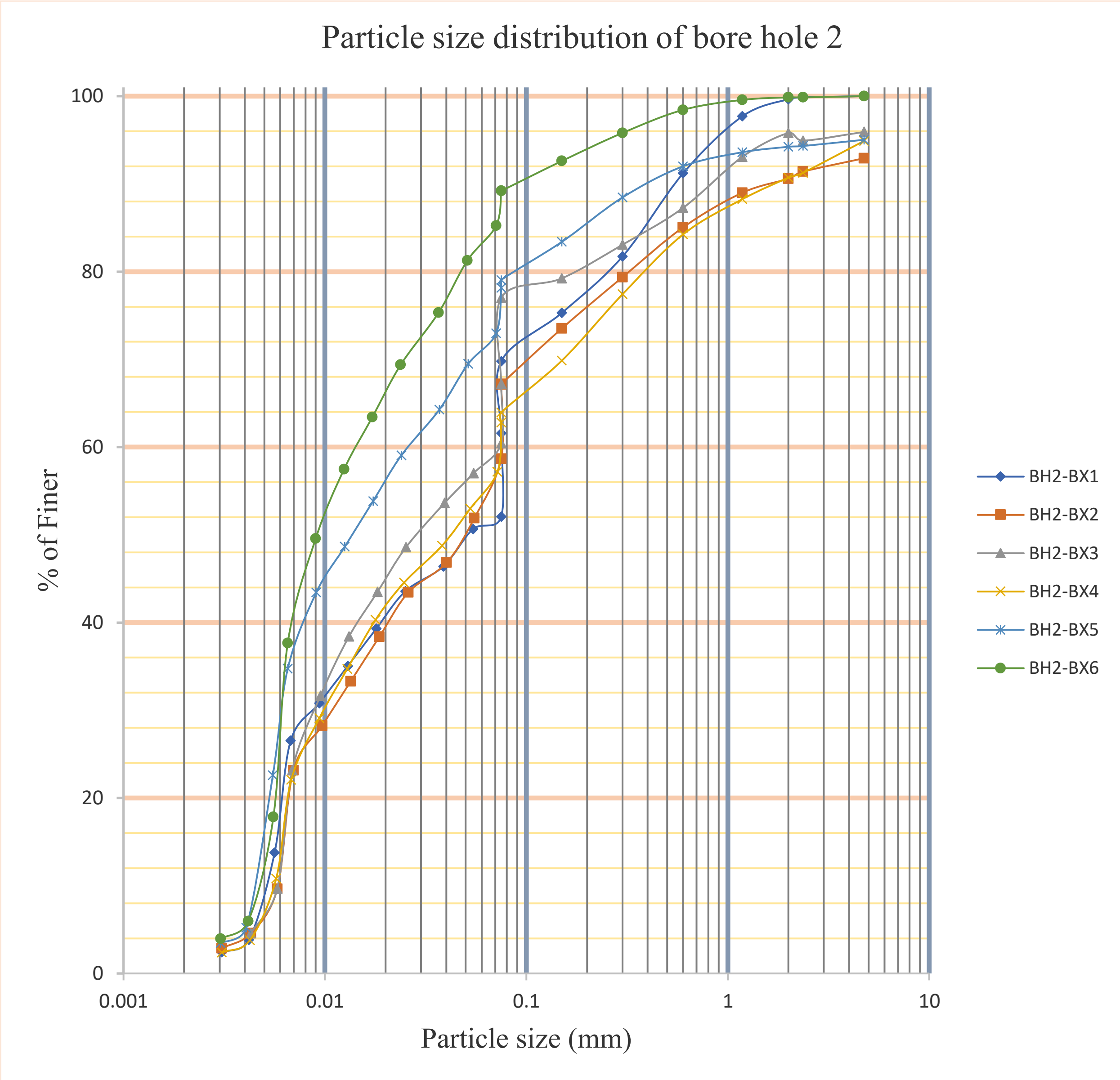


FIGURE 4.4 Grain size distribution of bore hole 2

As one can see from graph maximum size of particles is 5mm. From both boreholes percentage of soil retained on sieve No. 200 is less than 50%. This show that the soil is classify as fine-grained soil. As per AASHTO soil classification the soil has nearly 7% clay, 54% silt, 31% sand and 8% gravel soil. Whereas per USCS soil classification the soil has nearly 61% fine (clay, silt), 21% fine sand, 10% medium sand and 8% Coarse sand.

4.3.2. Water content of the soil

Water content of the soil was measured and calculated for all boreholes. The Water content values of the soil were calculated as per ASTM Standards requirements. Table 4.1 summarizes the average Moisture Content of the soil for each borehole.

Table 4.1 Average Moisture Content of the soil for all bore holes

Samples of (BH1)	Moisture Content(W)	Samples of (BH2)	Moisture Content(W)
BH1-BX1	22.22	BH2-BX1	24.72
BH1-BX2	32.21	BH2-BX2	36.16
BH1-BX3	28.01	BH2-BX3	36.56
BH1-BX4	35.98	BH2-BX4	33.47
BH1-BX5	27.17	BH2-BX5	37.49
BH1-BX6	36.55	BH2-BX6	40.18

4.3.3. Liquid Limit and Plastic limit

Liquid limit for each water content was measured and calculated. The results are presented in a plot of water content (ordinate, arithmetic scale) versus terminal blows (abscissa, logarithmic scale). Again, plastic limit for average of the two water contents from each sample was measured and calculated. Liquid limit and Plastic limit values of soil were calculated as per ASTM Standards requirements. As per ASTM standard if liquid Limit and plastic limit could not be determined the soil is classify as non-plastic soil. From this point of view from bore hole 1 (BH_1) sample BX_3 and BX_4 is non plastic soil. And from bore hole 2 (BH_2) sample BX_4 is non-plastic soil. FIGURE 4.5 summarizes the Liquid Limit and plastic limit of the soil for each borehole.

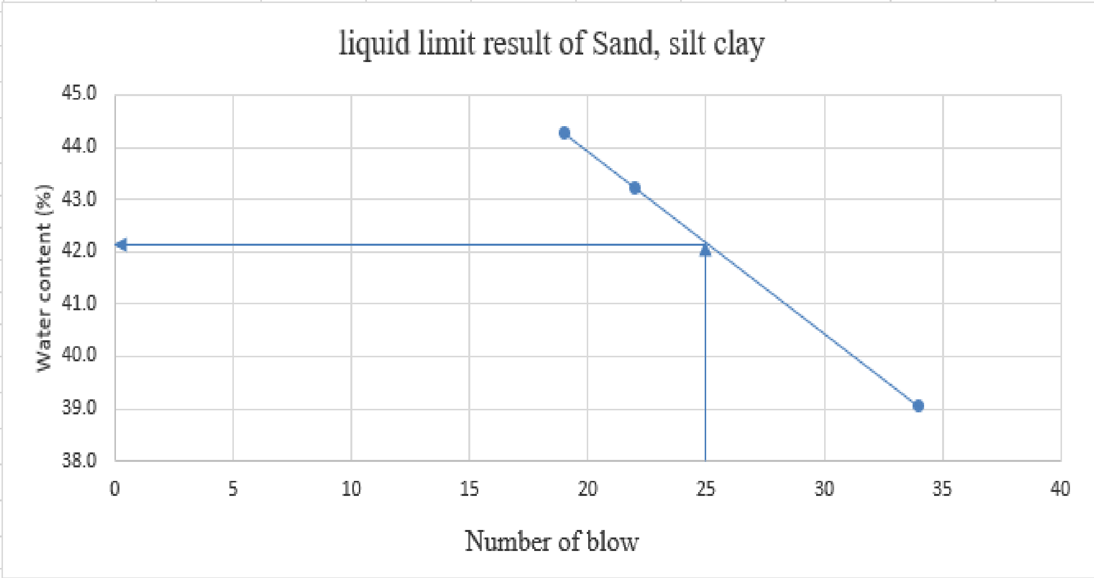


FIGURE 4.5 Liquid limit of sandy, silt clay

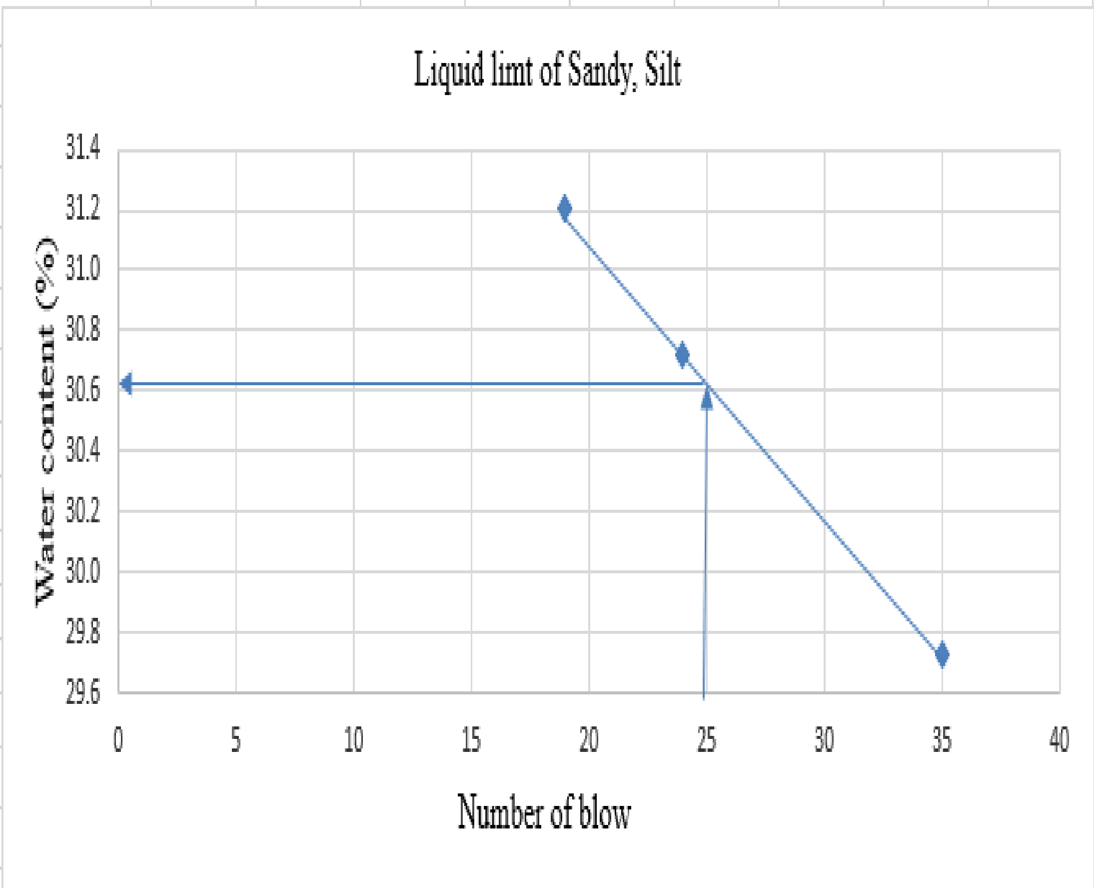


FIGURE 4.6 Liquid limit of sandy, silt

4.3.4. Plasticity Index

The value of the plasticity index PI can be computed from liquid and plastic limits as per ASTM Standards requirements. Table 4.2 summarizes the Plastic Index of the soil for each borehole.

Table 4.2 Plastic Index of the soil for each borehole.

Samples of (BH1)	Plastic Index (PI)	Samples of (BH2)	Plastic Index (PI)
BH1-BX1	18	BH2-BX1	19
BH1-BX2	15	BH2-BX2	20
BH1-BX3	36	BH2-BX3	26
BH1-BX4	0	BH2-BX4	33
BH1-BX5	14	BH2-BX5	14
BH1-BX6	12	BH2-BX6	24

PI can be used in soil classification and in correlations with some geotechnical soil properties.

4.3.5. Liquidity index

The value of the liquidity index LI can be computed from liquid and plastic limits as per ASTM Standards requirements. Table 4.3 summarizes the liquidity index of the soil for each borehole.

Table 4.3 Liquidity index of the soil for each borehole.

Samples of (BH1)	Liquidity index (LI)	Samples of (BH2)	Liquidity index (LI)
BH1-BX1	-0.04	BH2-BX1	0.07
BH1-BX2	0.74	BH2-BX2	1.26
BH1-BX3	0.78	BH2-BX3	0.60
BH1-BX4	1.16	BH2-BX4	1.01
BH1-BX5	-1.93	BH2-BX5	0.73
BH1-BX6	-2.50	BH2-BX6	-1.00

Liquidity index can be used to represent soil consistency and in correlations with some geotechnical properties.

4.3.6. Plasticity Chart

As per classification of ASTM standards after laboratory processing of sieve analysis, liquid limit and plastic limit of soils the plot of the cumulative particle-size distribution curve has been done in classifying the soil. FIGURE 4.6 and 4.7 show the plasticity chart for Bore hole 1 and 2.

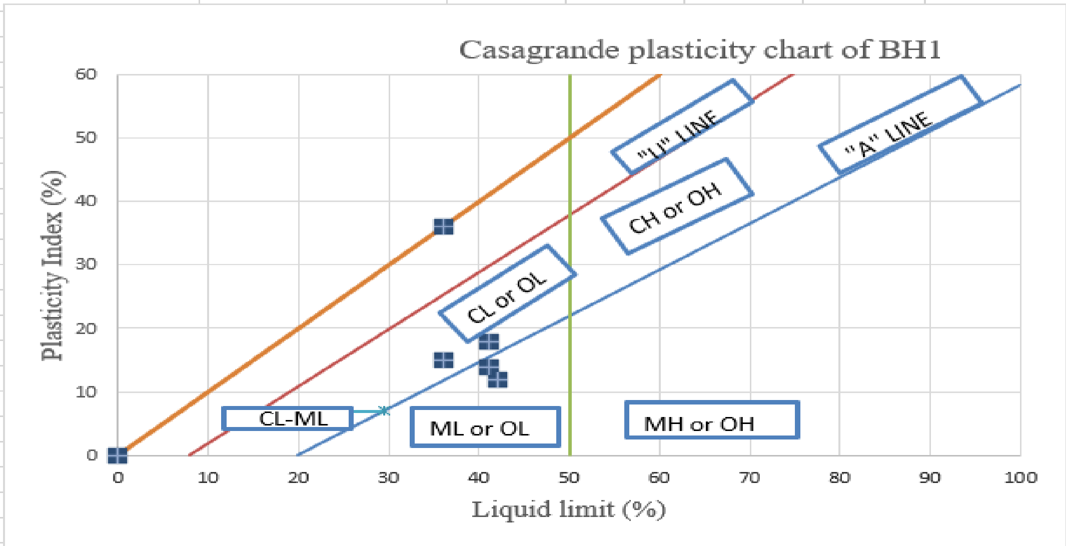


FIGURE 4.7 Plasticity Chart for Bore hole 1 (BH1)

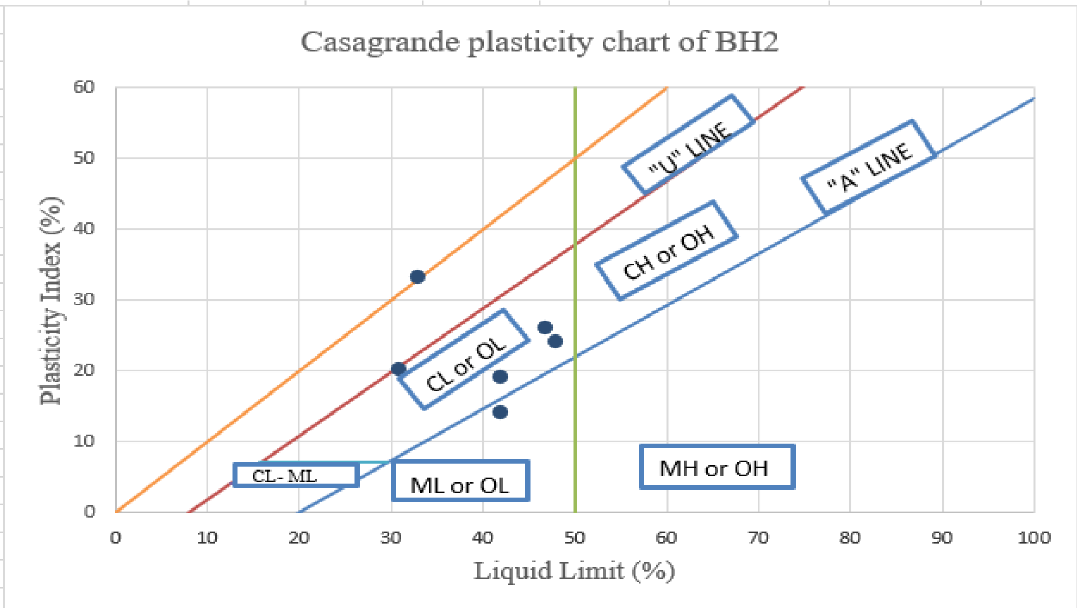


FIGURE 4.8 Plasticity Chart for Bore hole 1 (BH2)

4.3.7. Soil Classification

As per ASTM standard classification requirement depending on laboratory tests result such as percentage of finer retained on number 200 sieve, Liquid Limit and plastic limit, soil description and classification has been done. Table 4.4 summarizes laboratory tests result.

Table 4.4 Percentage of soil retained, Liquid Limit, plastic limit and soil classification

Samples	% Retained on No. 200 sieve	Liquid Limit (LL)	Plastic limit (PL)	Conditions		Soil Classification
				%Retained, LL, PI	PI vs LL plot falls	
BH1-BX1	23.61	41	23	%R<30, LL<50, PI>4	Above line 'A'	Sand, Silty clay, CL-ML
BH1-BX2	29.31	36	21	%R<30, LL<50, PI>4	Above line 'A'	Sand, Silty clay, CL-ML
BH1-BX3	46.78	36	0	%R>30, LL<50, PI>4	Above line 'A'	Sandy, Silty clay, CL-ML
BH1-BX4	36.80	ND	0	%R>30, LL<50, PI>4	Above line 'A'	Sandy, Silty clay, CL-ML
BH1-BX5	42.71	41	27	%R>30, LL<50, PI>4	Below line 'A'	Sandy, silt, ML
BH1-BX6	19.34	42	30	%<30, LL<50, PI>4	Below line 'A'	Sand, silt, ML
BH2-BX1	30.22	42	23	%R>30, LL<50, PI>4	Above line 'A'	sandy Silty clay, CL-ML
BH2-BX2	32.82	31	11	%R>30, LL<50, PI>4	Above line 'A'	sandy Silty clay, CL-ML
BH2-BX3	22.99	47	2	%<30, LL<50, PI>4	Above line 'A'	sand Silty clay, CL-ML
BH2-BX4	36.07	33	0	%R>30, LL<50, PI>4	Above line 'A'	sandy Silty clay, CL-ML
BH2-BX5	20.96	42	28	%<30, LL<50, PI>4	Below line 'A'	Sand, silt, ML
BH2-BX6	10.80	48	24	LL<50, PI>4	Above line 'A'	sand Silty clay, CL-ML

ND- not determined

4.3.8. Plotting Compaction Curve

As per ASTM Standards requirements of water-filling method by calculating the values of dry unit weight and molding water content, saturation curve has plotted.

FIGURE 4.9 and 4.10 shows the saturation curve of Sandy silt (ML) and sand Silty clay (CL-ML) soils respectively.

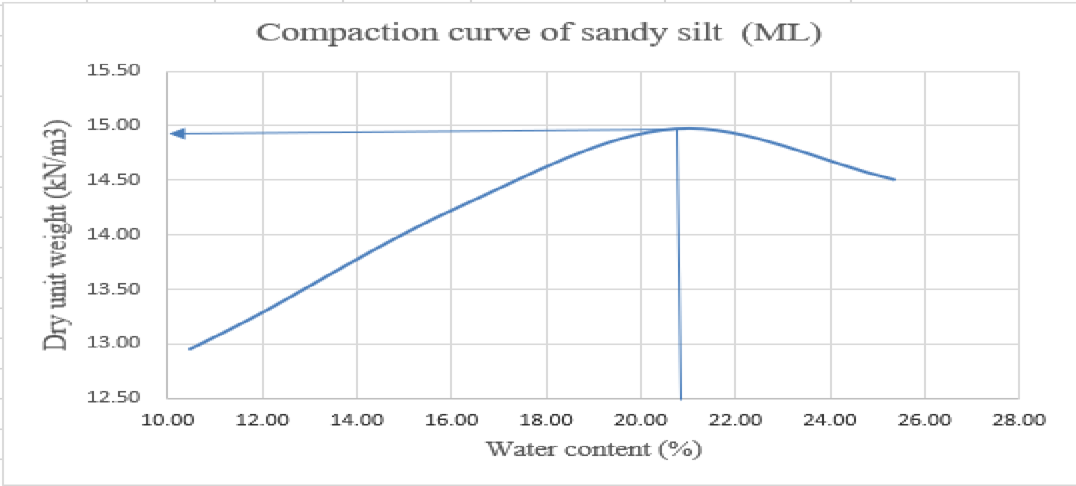


FIGURE 4.9 Saturation curve of Sandy silt (ML) soil

As one can see from graph maximum dry density of the soil is 14.95 g/cm^3 and optimum moisture content of the soli is 20.73%.

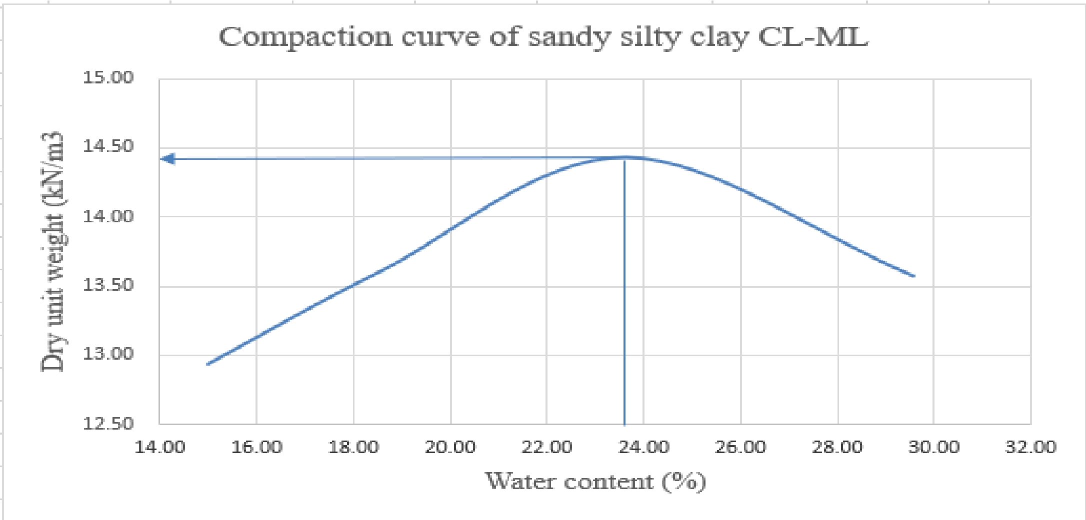


FIGURE 4.10 Saturation curve of sand Silty clay (CL-ML) soil

As one can see from graph maximum dry density of the soil is 14.43 g/cm^3 and optimum moisture content of the soli is 23.63%.

4.4.Bearing capacity of soil from SPT

4.4.1. Standard blow count (N60) Correction

Blow counts are corrected after calculation of average N value in the zone of major stressing by considering overburden stress and instrumental influencing factors.

The depths at which the SPT values taken, standard blow count (N_{60}) corrected for overburden pressure are shown on Table 4.5.

Table 4.5 SPT-N values taken from field and corrected for overburden pressure

The standard blow count (N_{60}) correced for overburden pressure											
η_H=	Donut, Rope and pully- China	0.8			η_B=	60-120	1		η_S=	Standard sampler without liner	1.2
Depth (z	Blows count			source (from)	Sum of the last two blow counts	η_R=	Correction to over burden pressure				$(N_1)_{60}=C_N*N_{60}\leq2*N_{60}$
	1st	2nd	3rd		N (2nd +3rd)	Rod length	N_{60}	Y	$P'_o=Yz$	$C_N=0.77\log(2000/P'_o)$ C_N	
2	3	5	6	BH_2	11	0.8	7.9	14.4	28.9	1.4	11.2
3	4	6	6	BH_1	12	0.8	8.6	14.4	43.3	1.3	11.1
4	4	6	7	BH_2	13	0.8	9.4	14.4	57.7	1.2	11.1
5	5	6	8	BH_1	14	0.9	11.4	14.4	72.2	1.1	12.7
6	5	7	8	BH_2	15	0.9	12.2	15.0	89.7	1.0	12.7
7	4	6	8	BH_1	14	1.0	12.8	15.0	104.7	1.0	12.6
8	4	6	9	BH_2	15	1.0	13.7	15.0	119.6	0.9	12.9
9	4	7	9	BH_1	16	1.0	14.6	15.0	134.6	0.9	13.2
10	4	7	12	BH_2	19	1.0	17.3	15.0	149.5	0.9	15.0
11	6	9	13	BH_1	22	1.0	21.1	15.0	164.5	0.8	17.6
12	5	10	13	BH_2	23	1.0	22.1	15.0	179.4	0.8	17.8
13	7	14	14	BH_1	28	1.0	26.9	15.0	194.6	0.8	20.9
14	7	14	15	BH_2	29	1.0	27.8	15.0	209.6	0.8	21.0

4.4.2. Bearing capacity values for isolated and combined footings

Allowable bearing pressures are computed for isolated and combined footings placed at a depth of 3m below the lowest natural ground level for immediate settlement limited to 25mm. Footing width is a significant parameter and considered at range from 2m to 5m. Since a large foundation width will affect the soil to a greater depth and strains integrated over a greater depth will produce a larger settlement. Annex B show the bearing capacity values of isolated footings at a depth of 3m below the natural ground level for immediate settlement limited to 25mm. From the above analyses, one can see that allowable bearing pressure obtained using bearing capacity equations from SPT test for width ranging from 2m to 5m varies from 172.8 kPa to 249.3 kPa around both boreholes.

4.4.2.1.Bearing capacity values for Mat foundation

The following allowable bearing pressures are computed for mat foundation placed at a depth of 3m below the lowest natural ground level for immediate settlement limited to 50mm. Annex B show the bearing capacity values of mat foundation at a depth of 3m below the natural ground level for immediate settlement limited to 50mm. From analyses, one can see that allowable bearing pressure obtained using bearing capacity equations from SPT test for mat foundation varies from 298kPa to 335kPa around both boreholes. This bearing capacity is calculated by considering 50mm settlement.

4.4.2.2.Bearing capacity values for Pile foundation

The allowable bearing pressures are computed for Pile foundation placed at different depth. Annex B show Bearing capacity of piles foundation based on the SPT-N value. Allowable bearing pressure obtained using bearing capacity equations from SPT test for pile foundation varies depending on weight of super structure, number, with and depth of each pile.

Generally, the foundation types for the case study allocated due to building loads and computed allowable bearing pressure of soils are shown in Table 4.6.

Table 4.6 Foundation type and bearing capacity of the building

Foundation type		Bearing Capacity	Building Height
Isolated	I1	189	2
	I2	189	4
	I3	183	6
	I4	177	8
Combined	C1	184	4
	C2	179	6
	C3	175	8
	C4	172	10
Mat	M1	301	4
	M2	300	6
	M3	299	8
	M4	297	10
Pile	P1	14,847	4
	P2	16,655	6
	P3	30,447	8
	P4	42,668	10

4.5.Modeling and Analysis of Super structure

The proposed quantity and cost model are evolved for the foundations of typical medium height buildings G^{+2}, G^{+4}, G^{+6} G^{+8} and G^{+10} constructed from reinforced concrete with moment resisting frames. The structural grid consists of (3m x 5m) bay frames spaced at 6m center to center along shorter side and 5m center to center along longer side a commonly adopted economical structural system for residential buildings in Adama. In this study the building heights are varied from two to ten stories. While the solid slab panels are of 150mm thickness for all the case study buildings, the column sizes adopted varies according to height of the building from 350 x 350mm for two storied buildings and up to 900 x 900mm for ten storied buildings. The beam sizes are in the range of 250 x 500mm to 350 x 600mm. The 28 days characteristic compressive strength of concrete 25N/mm^2 for all slab and beam and 30N/mm^2 for all column and footing are used. The yield strength of steel reinforcement adopted is 450 N/mm^2 for all superstructure and foundation elements. Corrosion induced by carbonation is the only fitting characteristic conditions to these building, so 50mm for all foundation, 35mm for all column and beam and 20mm for all slab reinforcement cover is used.

While doing model, analysis of reinforced concrete of super structure by ETABS v18.0.1 software the following values have been used.

i) The estimation of various super imposed dead loads acting on structure calculated properly. 1.5KN/m^2 super imposed dead on each single slab, 4.3KN/m^2 staircase flight, 8.5KN/m exterior single beam and 6.5KN/m interior single beam have been loaded. The imposed live loads considered are taken from ESEN -2015 cods according to the function of the rooms. 2KN/m^2 live loads on each single slab and 4KN/m^2 staircase flight have been loaded.

ii) Earth quake force is other load acting on the building. The seismic load in a building is assigned based on spectrum type, the ground type, expected ground acceleration, seismic zone of the site and regularity in the structural system among other factors. Based on the geotechnical investigation the ground type is categorized under category C. and belongs to zone 4 with 0.15 as the coefficient of ground acceleration and spectra type 1. The buildings are multistorey, multi-bay frames and regular in elevation as well as in plan with 3.9 behavior factor.

iii) Stiffness properties of slabs with shell properties, beams, columns and walls has been reduced to 50% to consider elastic flexural and shear stiffness properties of concrete elements uncracked elastic stiffness.

iv) Load combinations are considered as per ESEN code to perform the verification of structure design at ultimate limit state, serviceability limit states and seismic situations.

v) The weight of the structure used in the calculation of automatic seismic loads is based on mass source in ETABS. All gravity loads and 0.25 has been considered as the coefficient mass source participation factor for live loads.

vi) Lateral force method of analysis has been used at the consequences of structural regularity on seismic analysis and design.

vii) The most severe drift limit (α =0.005, for buildings having non-structural elements of brittle materials attached to the structure) has been checked and not exceeded in any story.

ETABS software is used to determine the vibration modes of a structure in modal analysis. These modes are useful to understand the behavior of the structure. While using ETABS v18.0.1 software for modeling and analysis of reinforced concrete of super the following process and norms have been used to produce the most suitable solution.

a) When defining a modal load case eigenvector modal analysis was used which is always linear and based on the stiffness of the full unstressed structure.
b) Linear static load cases applied to the structure followed by linear static analysis in which geometric and material nonlinearity are not considered, except for the P-Delta effect.
c) Loads defined as separate load patterns such as dead load, live load, static earthquake load and assign with specific load values.
d) Linear add types of combinations was used to include results from one or more load cases and enveloped results were used for analysis.
e) Modeling types of floor systems is rigid floor diaphragm
f) Element was meshed to ignore mismatched mesh geometries between adjacent objects by appropriate or relatively lower number of meshes.

The values of forces and moments generated from the super structure at the base of column is presented on annexed-A.

4.6.Model, Analysis and structural design of foundations

After generating forces and moments from super structure by ETABS v18.0.1 software and transferring the values, modeling and structural design analysis of foundations have done by SAFE v16.0.1. software. The allowable bearing pressure of soils considered for the shallow foundations are 172 to 335 KN/m^2 at a depth of 3m below the ground level. The allowable bearing pressure of soils considered for pile foundations are 113 to 13013 KN/m^2 per single piles at different depth, width and number of piles of foundation piles.

Coefficient of subgrade reaction against the footing or mat and the settlement at a given point measured and calculated from equation 2.48. Pile group axial spring stiffness values was measured and calculated using equation 2.49.

4.6.1. procedure for modeling, analysis and design of foundation

While using SAFE software for modeling, analysis and design of reinforced concrete of foundation the general steps required to analyze and design a structure using SAFE software are as follows:

i) Create or modify a model that numerically defines the geometry, properties, loading, and analysis parameters for the structure

i) Perform an analysis of the model

ii) Review the results of the analysis

iii) Check and optimize the design of the structure whether:

iv) Deformed shape less than allowable settlement of structure

v) Soil pressure less than allowable bearing capacity

vi) Punching shear design less than 1.

This is usually an iterative process that may involve several cycles of the above sequence of steps. The result of structural design of foundation is presented on Annexed-D.

4.6.2. Structural design of foundations

Structural design is usually an iterative process that may involve several cycles of sequence required to analyze and design a structure steps. All of these steps can be per formed seamlessly using the SAFE graphical user interfaces and finally checked by manually prepared hand calculations excel sheet. Different parameters, such as settlements, bending moments and shear forces, that can impact on both foundation design is evaluated using both approaches. Manually prepared hand calculations excel sheet shown on annex-C and the structural design of different foundation shown annex-C.

4.7.Quantities and Cost estimation of Foundation

The quantity and cost of modeling of building foundations includes the quantity and cost of non-structural elements and structural elements. The corresponding non-structural and structural unit price of different foundation's items is done as per current price of Adama construction Authority unit rates. Total cost of the foundation is calculated by summing up the total cost incurred per unit for these items of work done by multiplying the volume or area of different items and unit price of these items. The detail cost of different foundation shown annex A-E.

Table 4.7 Foundations types with its cost

Foundation type		Cost	Building Height
Isolated	I1	2,992,147	2
	I2	4,595,465	4
	I3	6,507,300	6
	I4	8,419,135	8
Combined	C1	4,916,329	4
	C2	5,716,796	6
	C3	7,387,984	8
	C4	9,059,171	10
Mat	M1	3,152,446	4
	M2	5,995,850	6
	M3	8,839,253	8
	M4	9,607,226	10
Pile	P1	4,283,050	4
	P2	6,465,890	6
	P3	8,648,730	8
	P4	9,635,149	10

4.8.Assumptions for modeling

Before using multiple regression, the following 7 assumptions have to be checked and the out puts were itemized as follow.

1) All variables are continuous, so all variables changed to continuous
2) The relationships between the dependent variables and the independent variables are linear, both for each independent variable and globally. As one can see from graph of analysis output, normal probability plot is a graphical technique for

assessing whether or not a data set is approximately normal distributed. The data are plotted against a theoretical normal distribution in such a way that the points should form an approximate straight line. So, our data satisfy this argument.

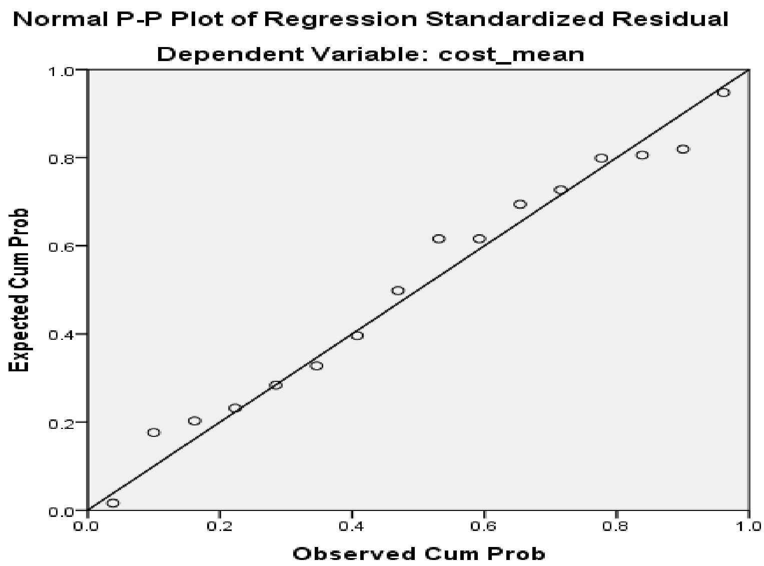


FIGURE 4.11 normal probability plot

1.There is independence of errors

If the Durbin-Watson value is between 1.5-2.5 there is no relationship between the residual variable and the independent variable' so, as it is shown on the table 4.8 the assumptions "there is no relationship between the independent variable and the residuals variable" is satisfied.

Table 4.8 Model Summary b

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	0.970a	0.940	0.931	591291.43	1.314
a. Predictors: (Constant), height, bearing capacity					

2.The residual variable is approximately normally distributed

The result obtained from the Shapiro Wilk test which is shown on table 4.12 indicate that all the variables had a p-value greater than (0.05), meaning that the variables involved in the study follow a normal distribution: therefore, it can be concluded that the residual values is normally distributed so that the regression analysis procedures have been fulfilled.

Table 4.9 Tests of Normality						
	Kolmogorov-Smirnova			Shapiro-Wilk		
	Statistic	df	Sig.	Statistic	df	Sig.
Studentized Residual	0.126	16	0.200*	0.969	16	0.819
*. This is a lower bound of the true significance.						
a. Lilliefors Significance Correction						

3.The independent variable is not strongly correlated with one another (we don't have important multicollinearity)

As one can see from table 4.10 all the VIF (variance inflation factor) column values are less than 10, and tolerance values are greater than 10% respectively, indicating that there is no multi collinearity influence between the explanatory variables.

Table 4.10 Collinearity statistics					
Model		Collinearity statistics			
	(Constant)				
1				Tolerance	VIF
	Bearing capacity			0.904	1.106
	Building height			0.904	1.106
a. Dependent Variable: cost					

4.There are no significant outliers in the series

If there are no circles or asterisks on either end of the box plot, this is an indication that no outliers are present.

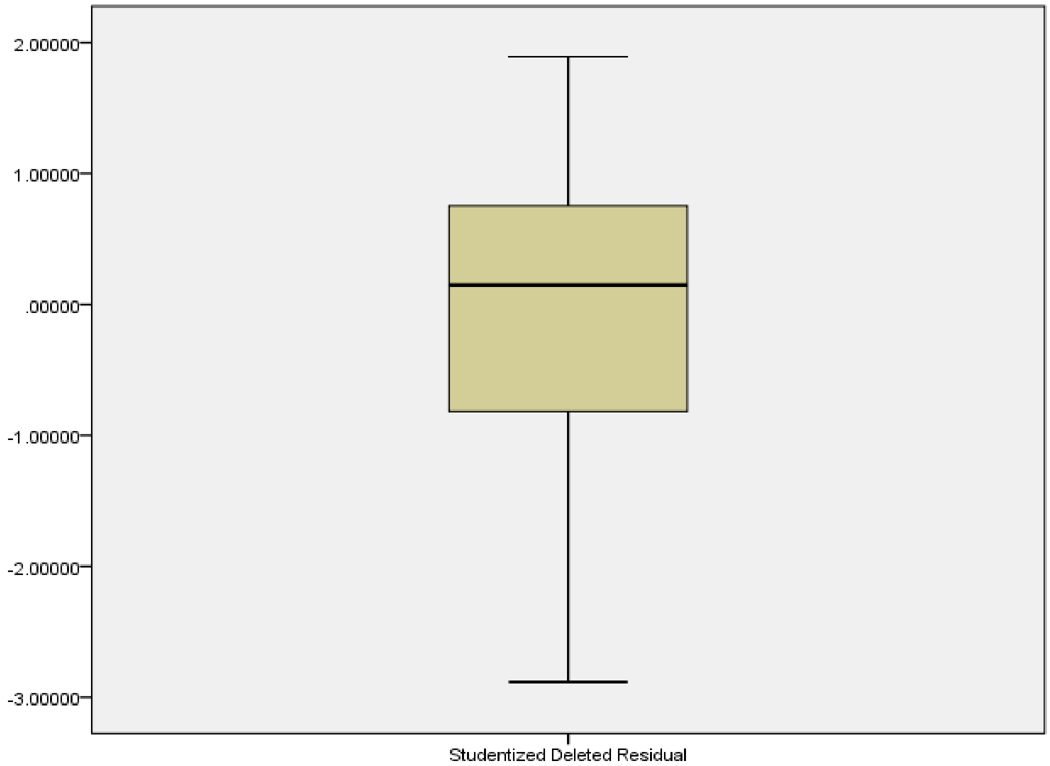


FIGURE 4.12 Outlier's plot

5.The dependent variable has the same variance for all the values of the independent variable (Test for heteroscedasticity)

Based on the scatterplot output shown on FIGURE 4.1 below, it appears that the spots are diffused and do not form a clear specific pattern, so it can be concluded that the regression model does not occur heteroskedasticity problem.

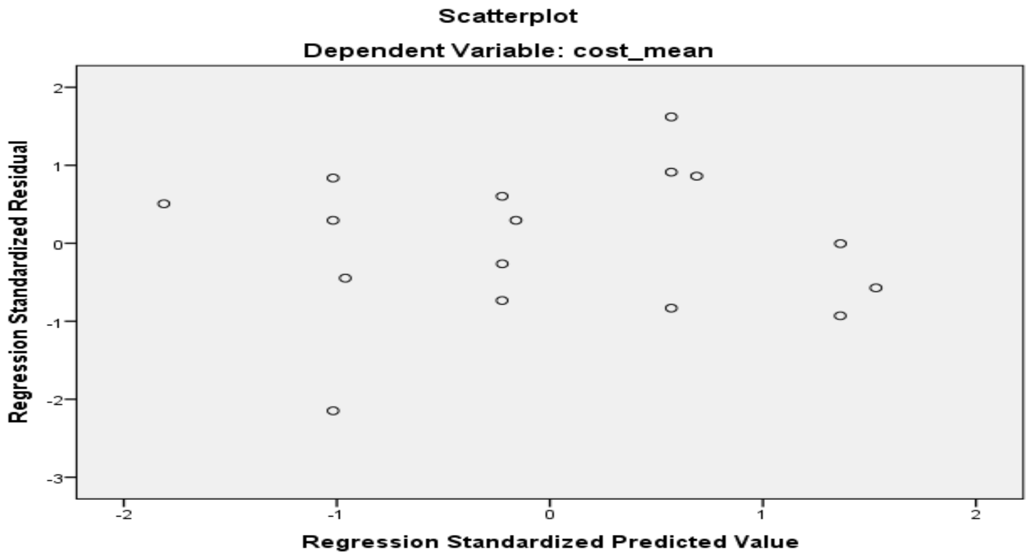


FIGURE 4.13 Test for heteroscedasticity

Before conducting regression analysis, all relevant tests of regression such as sample size requirement, multicollinearity, outliers, normality and heteroscedasticity are satisfied. Therefore, regression analysis can be continued and predictable results brings out the cost implications for designing the foundations.

4.9.Hypothesis

H_1: There is no a significant impact of bearing capacity and height of building on cost of foundation

The hypothesis tests if bearing capacity and height of building significant impact on cost of foundation. The dependent variable cost of foundation was regressed on predicting variables the bearing capacity and height of building to test the hypothesis H_1. As indicated in table 4.11 below, bearing capacity of soil and building height significantly predicted foundation cost $F=66.83$, $P< 0.005$, which indicates that the Bearing capacity of soil and building height can play a significant role in cost of foundation.

we can see that R-square value is 0.931, which means that our independent variable i.e., bearing capacity and building height cause 93.1% change in the dependent variable i.e., cost. However, 6.9% of the variance is expected by other factors not covered by in this study. Additionally, these results clearly show the positive effect of the bearing capacity of soil and height of building on cost of foundation.

Table 4.11 Summary b

Model	R	R Square	Adjusted R Square	F	p-value
1	0.970a	0.940	0.931	66.83	0.000
a. Predictors: (Constant), building height, bearing capacity					
b. Dependent Variable: cost of foundation					

4.10. Regression coefficient

As it is shown on table 4.12 the variable with the highest beta coefficients contributes the most to explaining the dependent variable's variance, which is controlled by all other variable in the model, as shown in the beta column beneath standardized coefficients. Building height with value 0.953 is the most significant factor and bearing Capacity with value 0.05 is the lowest contributing factor in standardize beta coefficient column. Furthermore, the beta value is positive, which indicates the positive relationship between dependent and independent variables.

Table 4.12 Coefficients a						
Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
1		B	Std. Error	Beta		
	(Constant)	961,569.89	429,739.99		2.24	0.04
	Bearing Capacity	8.57	12.37	0.05	0.69	0.05
	Building height	864,588.84	64,833.53	0.953	13.34	0
a Dependent Variable: cost of foundation						

4.11. Regression equation

This study was conduct to determine if various forms of investigation can influence cost of foundation. It was hypothesized that bearing capacity and building height will positively predict the cost of building foundation. To test this hypothesis multiple regression analysis is used. An unstandardized coefficient is used in the equation as coefficient of different independent variables along with the constant term to predict the value of dependent variables. Therefore, as per the unit rate of Adama municipality in 2015, for foundations constructed on sandy silt and silt clay soils the predicted cost of foundation can be determined as follows,

Y (cost of foundation) = 961,569.9 + 8.6 (bearing capacity) + 864,588.9 (building height),

4.12. Limitations of the methodological framework

The equations for selection of cost-effective foundation type were familiarized with sandy silt and silt clay soil type. This study also limited to 5 bays by 4 bays of buildings dimension 25.5 by18 meters and four foundation types that is, isolated, combined, mat and pile foundations. The effect of seismic forces on the foundation systems of the case study buildings considered only seismic zone four of Ethiopia. Additionally, this investigate is limited to buildings of low and medium height. The unit rates used for estimation of the cost of foundation is done as per the estimation of unit rates done by Adama construction Authority for year 2015 of month March. The fluctuation of labor, construction material and equipment cost are not considered.

4.13. Future work

Further research with a geotechnical program can be extended this study for varies types of buildings with area coverage constructed on different soil type and foundations system not included in this thesis. The effect of seismic forces on the foundation systems other than seismic zone four also need detail investigation. Researches towards prediction technique for high rise building is also significant. A more complete check of buildings with different size of foundation type with medium and high bearing capacity is also necessary in future study. What is more, a better study considering fluctuation of labor, construction material and equipment cost can also improve the accuracy of calculation's feasibility.

Conclusions and Recommendations

Conclusion

Developing suitable cost estimation method plays a very important role by speed up the design period and reduces economy conflicts in foundation construction. The available selection methods of foundation type measure safeness of the design than the cost effectiveness. Actually, choosing an economical foundation should not compromise the safeness and strength of the foundation. But it is possible to attain both the safeness and economic effectiveness of foundation. The current research aimed to develop the suitable cost-effective foundation type selection method during early stages of foundation design for medium rise buildings on typical sandy clay and sandy silt soils stratum in Adama city. The simplified methods recommended in this dissertation has created a better understanding about suitable cost-effective foundation type selection during early stages of foundations design and concludes that under the defined situations, this method is feasible.

Therefore, as per the investigation of this dissertation the equations used to determine suitable cost-effective foundation type selection under the described conditions is as follow

$$\mathbf{Y} = 961{,}569.9 + 8.6\mathbf{X_1} + 864{,}588.9\mathbf{X_2} \qquad (05.01)$$

Where

Y- cost of foundation

X_1-bearing capacity

X_2-building height

Recommendations

The current study can be act as a bench mark in the research on cost-effective foundation type selection method. However, the result of this study should be treated with caution due to the small sample size, inadequate number of foundation and soil type.

Future studies could further examine and develop a more precise prediction method by considering the factor influence cost of foundation such as seismic zone, different soil type, varies stratum and fluctuation of construction cost.

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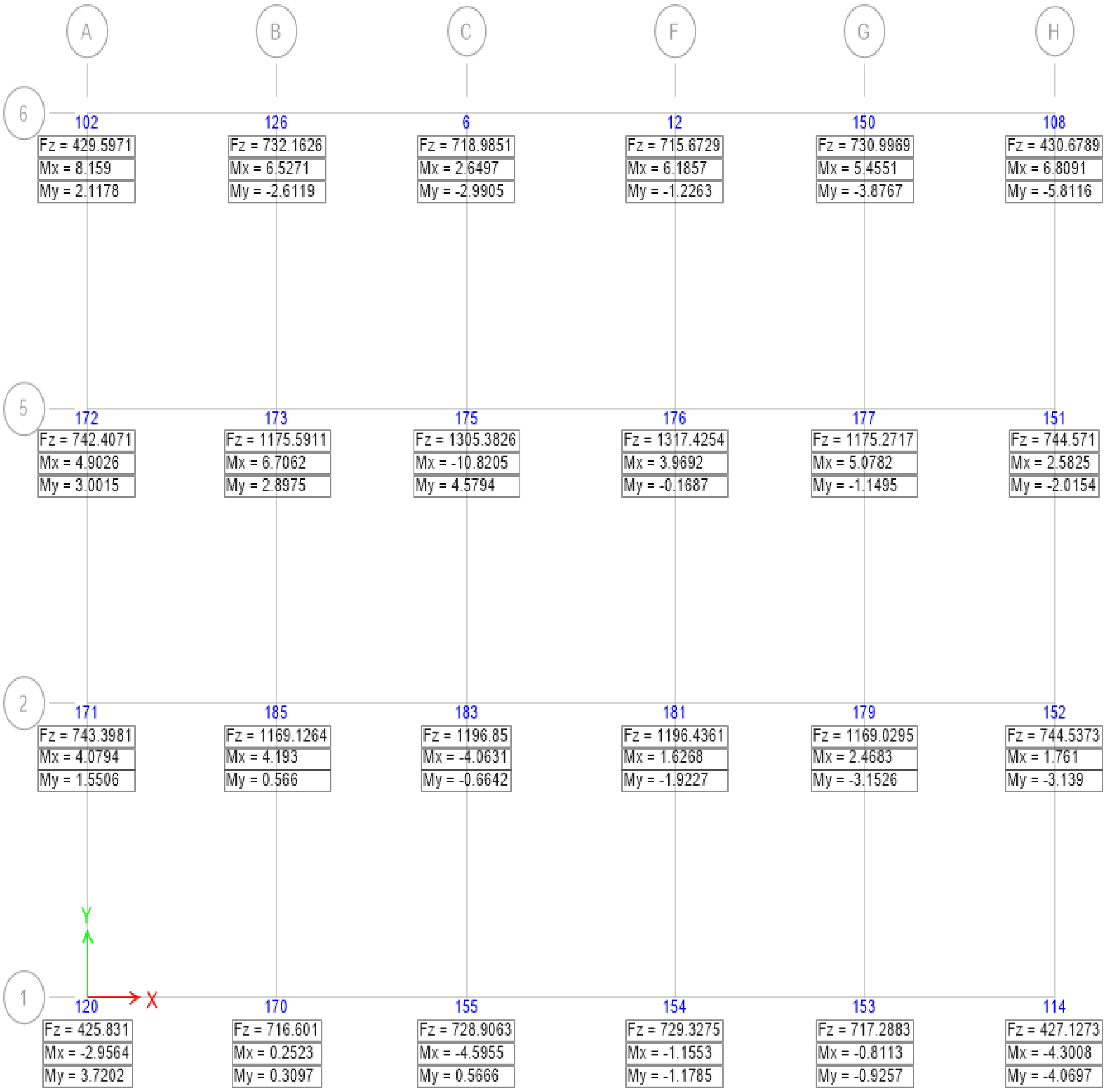
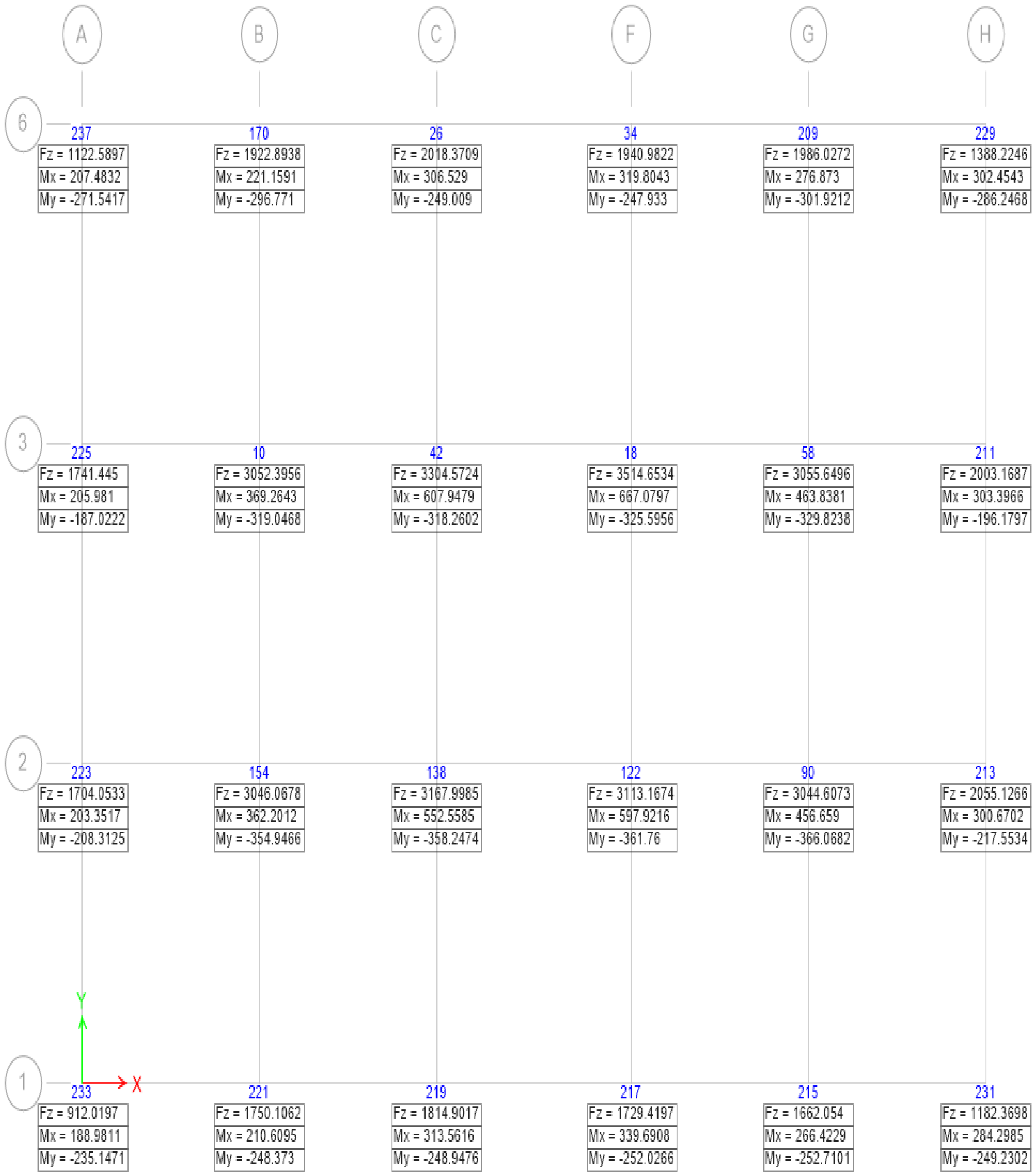
Annex- A

Values of forces and moments generated from super structure

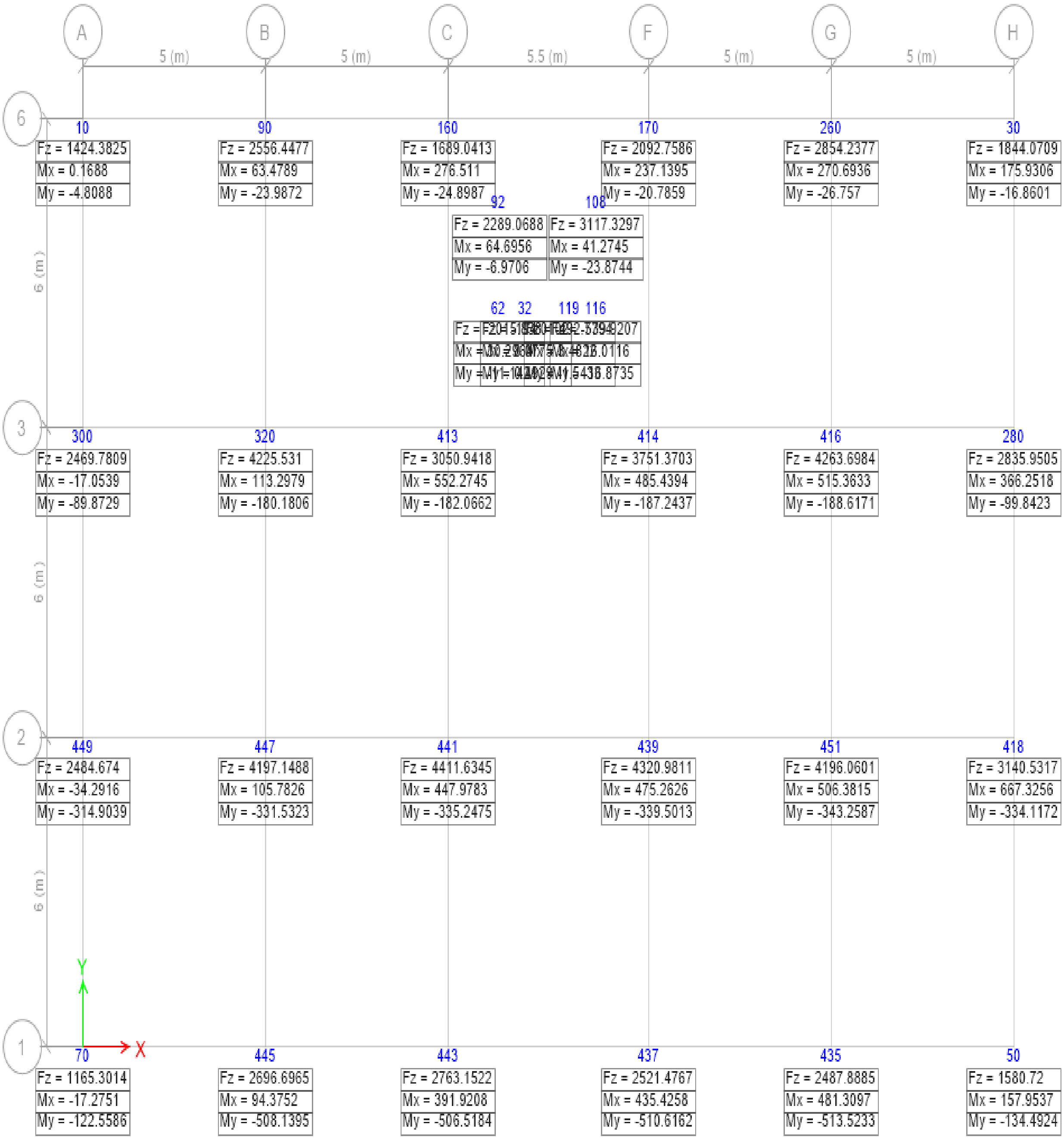
Values of forces and moments generated from super structure of G+2 building



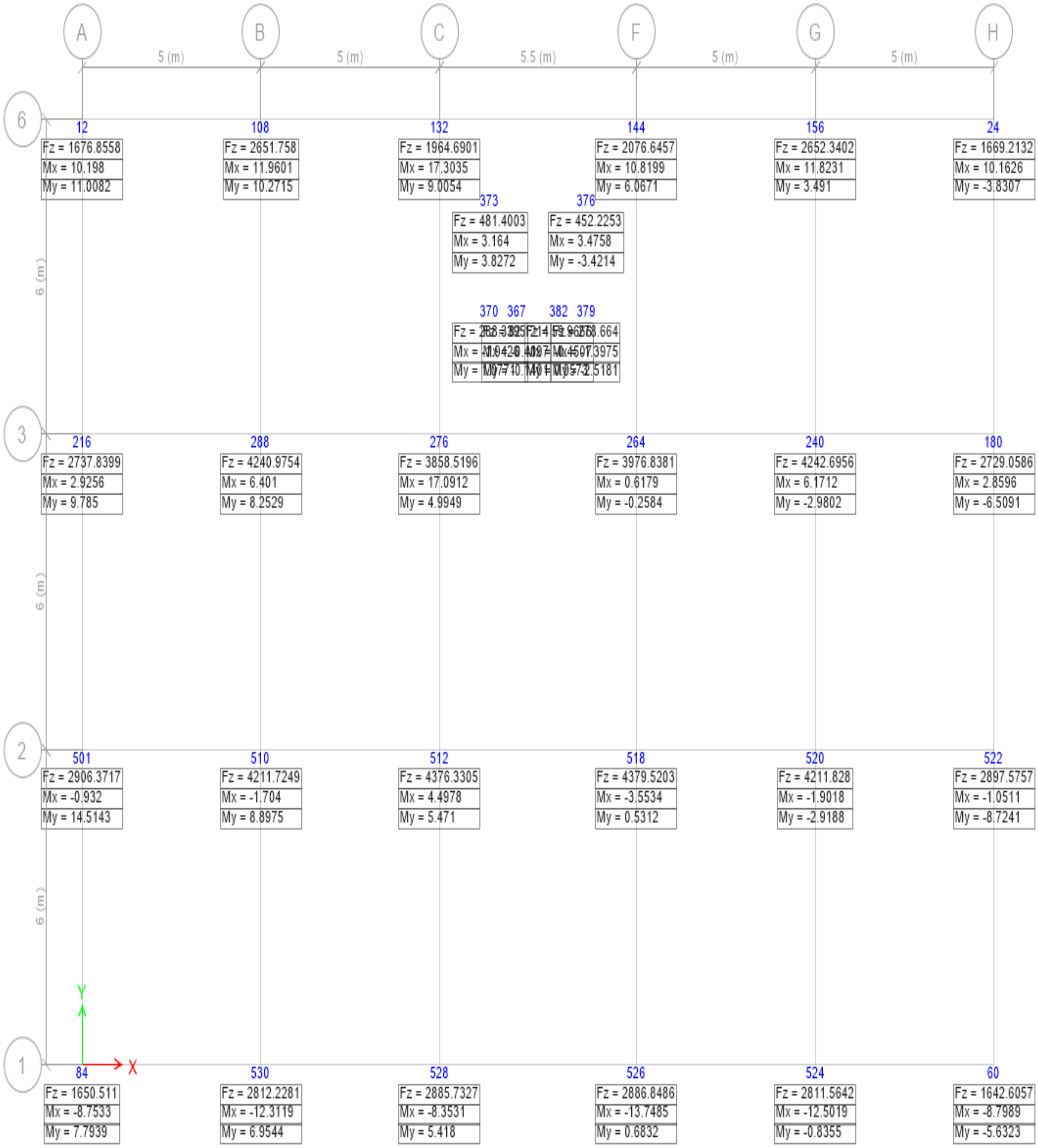
Values of forces and moments from super structure for G+4 building



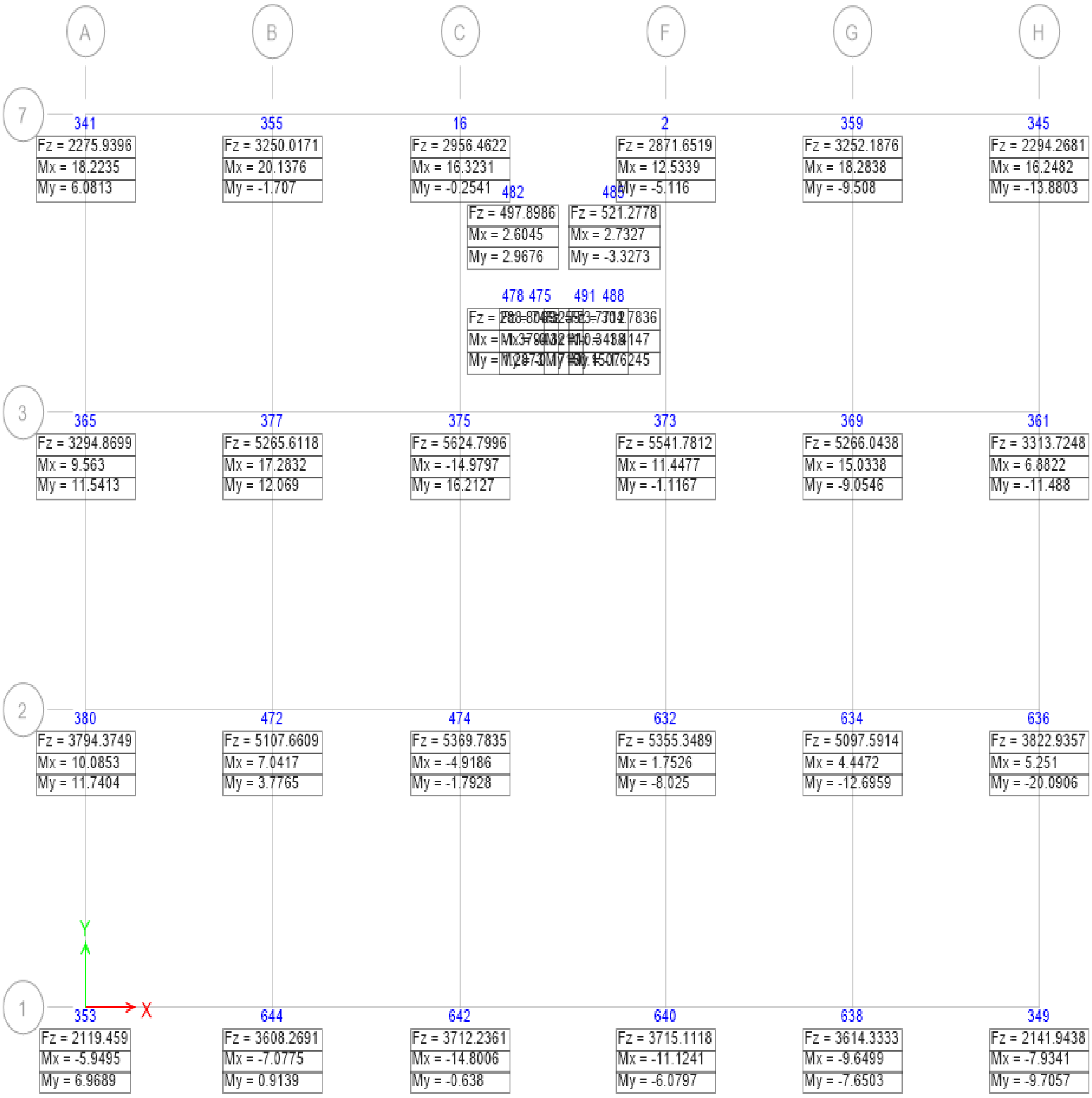
Values of forces and moments from super structure for G+6 building



Values of forces and moments from super structure for G+8 building



Values of forces and moments from super structure for G+10 building



Annex-B

Hand calculation excel sheet template for bearing capacity

The standard blow count (N_{60}) correced for overburden pressure											
η_H=	Donut,	0.8			η_B=	60-120	1		η_S=	Standar	1.2
	Blows count				Sum of the	η_R=	Correction to over burden pressure			C_N=0.77	$(N_1)_{60}=C_N*N$
Depth (z)	1st	2nd	3rd	source	N	Rod lengt	N_{60}	Y	$P'_o=Yz$	C_N	$_{60}\leq 2*N_{60}$
2.0	3.0	5.0	6.0	from BH_2	11.0	0.8	7.9	14.4	28.9	1.4	11.2
3.0	4.0	6.0	6.0	from BH_1	12.0	0.8	8.6	14.4	43.3	1.3	11.1
4.0	4.0	6.0	7.0	from BH_2	13.0	0.8	9.4	14.4	57.7	1.2	11.1
5.0	5.0	6.0	8.0	from BH_1	14.0	0.9	11.4	14.4	72.2	1.1	12.7
6.0	5.0	7.0	8.0	from BH_2	15.0	0.9	12.2	15.0	89.7	1.0	12.7
7.0	4.0	6.0	8.0	from BH_1	14.0	1.0	12.8	15.0	104.7	1.0	12.6
8.0	4.0	6.0	9.0	from BH_2	15.0	1.0	13.7	15.0	119.6	0.9	12.9
9.0	4.0	7.0	9.0	from BH_1	16.0	1.0	14.6	15.0	134.6	0.9	13.2
10.0	4.0	7.0	12.0	from BH_2	19.0	1.0	17.3	15.0	149.5	0.9	15.0
11.0	6.0	9.0	13.0	from BH_1	22.0	1.0	21.1	15.0	164.5	0.8	17.6
12.0	5.0	10.0	13.0	from BH_2	23.0	1.0	22.1	15.0	179.4	0.8	17.8
13.0	7.0	14.0	14.0	from BH_1	28.0	1.0	26.9	15.0	194.6	0.8	20.9
14.0	7.0	14.0	15.0	from BH_2	29.0	1.0	27.8	15.0	209.6	0.8	21.0

Bearing capacity for Isolated and Combined footing														
B		2	2.25	2.5	2.75	3	3.25	3.5	3.75	4	4.25	4.5	4.75	5
Z=0.5*B		1	1	1	1	2	2	2	2	2	2	2	2	3
Z=2*B		4	5	5	6	6	7	7	8	8	9	9	10	10
$Z_{avg}(D_f)$		3	3	3	3	3	3	3	3	3	3	3	3	3
D_f+2B		7	8	8	9	9	10	10	11	11	12	12	13	13
$N_{design}=\sum(N/i^2)/\sum(1/i^2)$		11	11	11	11	11	11	11	11	11	12	12	12	12
F_2	0.08			F_3	0									
$K_d=(1+(0.33*D_f/B))\leq 1.33$		1	1	1	1	1	1	1	1	1	1	1	1	1
$q_{all}=N/F_2*((B+F_3)/B)$^2*K		249.3	188.8	188.8	189.3	189.3	186.3	183.2	181.1	178.7	177.3	175.4	174.3	172.8

Bearing capacity for Mat foundation														
B		6	8	10	12	14	16	18	20	22	24	26	28	30
Z=0.5*B		3	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00	15.00
Z=2*B		12	16	20	24	28	32	36	40	44	48	52	56	60
$Z_{avg}(D_f)$		3	3	3	3	3	3	3	3	2	3	3	3	3
D_f+2B		15	19	23	27	31	35	39	43	46	51	55	59	63
$N_{design}=\sum(N/i^2)/\sum(1/i^2)$		11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.46	11.54
F_2	0.08			F_3	0.3		ΔH_a	50						
$K_d=(1+(0.33*D_f/B))\leq 1.33$		1.17	1.12	1.10	1.08	1.07	1.06	1.06	1.05	1.03	1.04	1.04	1.04	1.03
Allowable	(N/F_2)*(	335.0	323.2	316.0	311.3	307.9	305.4	303.4	301.8	297.1	299.4	298.5	296.6	298.0

Bearing capacity for Pile foundation (6m)																						
Width of Pile	0.65		No	6.00		No of	8.0		Width in x-	16.12			Width in	11.7		End	188.60		pile group	0.94	Tanδ (for	0.45
Length of Pile	6.0		FS	3.0		C/C	2.2		Total no of	48			Skin	333.84		Θ=tan-	3.3		single area of	0.3	perimeter(	2.0
K_s(for silt	0.45																					
Depth (z)	N_{60}	N_{55}	N_1 N-(L-2.5D	Soil type	φ	Y	n²(45 +φ/2)	(Nq+1)tan	q'_o=Yz	Kp=(1+sinφ)/(1-sinφ)	Ks=1-sinφ	Ka=1/K_p	B*K_p/K_o	situation	Unit Skin Friction (f_s)	end bearing (f_b)	(q_s)=(As)$_{Tg}$* Ks*q'o*tanδ	(q_b)=A_{bg}*q'_o* N_q	fs=∑(Kq'_otanδ*PΔL)	f_b=Ap* q'_o*N_q	A_b($q'_o$$N_q$)	0.5YBNγ)
2	8	9	8	sandy CL	40	14	67.1	116	29	2.8	0.4	0.4	5.2	group gov	10	198	1530	365,393	56	643	643	542
3	9	9	8	sandy CL	38	14	49.4	79	43	2.6	0.4	0.4	4.4	group gov	10	216	2494	402,994	92	709	709	370
4	9	10	8	sandy CL	37	14	41.7	64	58	2.5	0.4	0.4	4.0	group gov	11	234	3480	453,937	128	799	799	299
5	11	12	9	sandy CL	37	14	41.0	62	72	2.5	0.4	0.4	4.0	group gov	14	285.6	4371	557,296	160	981	981	293
6	12	13	9	sandy CL	36	15	36.8	55	90	2.4	0.4	0.4	3.8	group gov	15	306	5592	622,953	205	1096	1,096	265
7	13	14	9	sandy	35	15	34.2	50	105	2.4	0.4	0.4	3.6	group gov	11	1276.8	6656	675,032	244	1188	1,188	241
8	14	15	9	sandy	35	15	32.8	47	120	2.3	0.4	0.4	3.5	group gov	12	1368	7690	740,940	282	1304	1,304	229
9	15	16	9	sandy	35	15	31.8	45	135	2.3	0.4	0.4	3.5	group gov	13	1459.2	8726	807,185	320	1420	1,420	220
10	17	19	11	sandy	35	15	33.1	48	150	2.3	0.4	0.4	3.6	group gov	16	1732.8	9591	933,762	352	1643	1,643	232
11	21	23	14	sandy	35	15	35.4	52	164	2.4	0.4	0.4	3.7	group gov	19	2112	10366	1,096,604	380	1929	1,929	252
12	22	24	14	sandy CL	35	15	34.4	50	179	2.4	0.4	0.4	3.6	group gov	20	2208	11393	1,163,679	418	2047	2,047	243
13	27	29	18	sandy	36	15	37.1	55	195	2.4	0.4	0.4	3.8	group gov	24	2688	12106	1,362,834	444	2398	2,398	268
14	28	30	18	sandy	36	15	36.2	53	210	2.4	0.4	0.4	3.7	group gov	25	2784	13130	1,429,257	482	2515	2,515	259
$f_{s(avg)}$															12		17467		641		845	
$Q_{u(group)}$															57,820		16,372		78,154		49,965	
Q_{all}(group)															**19,273**		**5,457**		**26,051**		**16,655**	
Q_{all}(single)															**402**		**114**		**543**		**347**	

Bearing capacity for Pile foundation (8m)																						
Width of Pile	0.75		No of	7.00		No of	9.00		Width in x-	21.15			Width in y-	16.05		End bearing	339.5		pile group	0.94	Tanδ (for	0.45
Length of Pile	6.0		FS	3.0		C/C	2.6		Total no of	63.0			Skin	446.4		Θ=tan-	3.3		single area of	0.44	perimeter(P)	2.36
K_s(for silt clay)	0.45																					
Depth (z)	N_{60}	N_{55}	N_1=N-(L-	Soil type	φ	Y	Nq=tan²(45+	Nγ=2(Nq+1)	q'_o=Yz	sinφ)/(1-sinφ)	Ks=1-sinφ	Ka=1/K_p	B*K_p/K_o	situation	Unit Skin Friction (f_s)	end bearing (f_b)	(q_s)=(As)$_{Tg}$*K	(q_b)=A_{bg}*q'_o*N	fs=∑(Kq'_otan δ*PΔL)	f_b=Ab* q'_o*N_q	A_b($q'_o$$N_q$)	0.5YBNγ)
2	8	9	8	sandy CL	40	14	67.1	116	29	2.8	0.4	0.4	5.21808	group gov	9.504	198	2,046	657,650	65	856	856	626
3	9	9	8	sandy CL	38	14	49.4	79	43	2.6	0.4	0.4	4.42092	group gov	10.368	216	3,334	725,325	106	944	944	427
4	9	10	8	sandy CL	37	14	41.7	64	58	2.5	0.4	0.4	4.03621	group gov	11.232	234	4,653	817,015	147	1,063	1,063	345
5	11	12	9	sandy CL	37	14	41.0	62	72	2.5	0.4	0.4	3.99716	group gov	13.7088	286	5,845	1,003,046	185	1,305	1,305	338
6	12	13	9	sandy CL	36	15	36.8	55	90	2.4	0.4	0.4	3.77415	group gov	14.688	306	7,478	1,121,217	237	1,459	1,459	306
7	13	14	9	sandy ML	35	15	34.2	50	105	2.4	0.4	0.4	3.62669	group gov	11.4912	1277	8,900	1,214,951	282	1,581	1,581	279
8	14	15	9	sandy ML	35	15	32.8	47	120	2.3	0.4	0.4	3.54857	group gov	12.312	1368	10,282	1,333,575	326	1,736	1,736	265
9	15	16	9	sandy ML	35	15	31.8	45	135	2.3	0.4	0.4	3.48759	group gov	13.1328	1459	11,668	1,452,805	370	1,891	1,891	254
10	17	19	11	sandy ML	35	15	33.1	48	150	2.3	0.4	0.4	3.56422	group gov	15.5952	1733	12,825	1,680,624	406	2,187	2,187	267
11	21	23	14	sandy ML	35	15	35.4	52	164	2.4	0.4	0.4	3.69229	group gov	19.008	2112	13,861	1,973,714	439	2,569	2,569	291
12	22	24	14	sandy CL	35	15	34.4	50	179	2.4	0.4	0.4	3.63763	group gov	19.872	2208	15,234	2,094,438	482	2,726	2,726	281
13	27	29	18	sandy ML	36	15	37.1	55	195	2.4	0.4	0.4	3.79114	group gov	24.192	2688	16,187	2,452,886	513	3,192	3,192	310
14	28	30	18	sandy ML	36	15	36.2	53	210	2.4	0.4	0.4	3.73727	group gov	25.056	2784	17,558	2,572,438	556	3,348	3,348	299
$f_{s(avg)}$															12		42538		1347		1278	
$Q_{u(group)}$															439,607		1,287,976		181,773		90,951	
Q_{all}(group)															146,536		429,325		60,591		30,317	
Q_{all}(single)															3053		8944		1262		632	

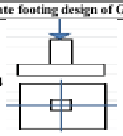
Annex-C

Hand calculation excel sheet template foundation design

Hand calculations excel sheet templet for Isolated footing

Project

Subject Isolate footing design of G+2 Building



0.4

Design value of the vertical action (V_{Ed})								Bearing resistance	
Imposed load	1025	Weight of foundation		Weight of backfill		N_{Ed}	Unfactored load	Allowable Bearing capacity	
M_x=	95	use B=	2.4	D_f=	2.40		P_{usf}	Q_{all}=	280
M_y=	25	use L=	2.4	γ=	18		The structural loads used in foundation design is actual loads not factored.	N_{60}=	25
H=	40	γ=	24	Overburden	2.00			N_{cor}=	20
Column size		G_k=	55.30					gravel sand=2,clay sand=3,silt sand=4	2
a=	0.5	G_k=	8.5		207.36	1379.59	719.30	footing type	1
		Q_k=	4					square=1,rectangle =2	0.82

Material's property

Reference	Concrete		
ES EN 1992-1-1. 3.1.3 Table 3.1	F_{ck}		30
	$F_{ck,cube}$		37
	f_{ctm}		2.9
	$f_{ctk}^{0.05}$		2
	$f_{ctk}^{0.95}$		3.8
	E_{cm}	modulus of elasticity	32
	$\varepsilon_{c1\%}$		2.2
	$\varepsilon_{cu1\%}$		3.5
	$\varepsilon_{c2\%}$		2
	$\varepsilon_{cu2\%}$		3.5
	n		2
	$\varepsilon_{c3\%}$		1.75
	$\varepsilon_{cu3\%}$		3.5
	γ_c	per=1.5,ac1.2	1.5
	f_{cd}		20.00
	f_{ctd}	$f_{ctk}^{0.05}/\gamma_c$	1.33
	E_{cd}	$E_{cm} / \gamma c_E$	26.7

Reference	Reinforcement		
ES EN 1992-1-1. 3.2.2	Class	A=1,B=2,C=3	3
	f_{yk} (cold)		460
	k	$(f_t/f_y)_k$	1.2
	$\varepsilon_{UK\%}$		7.5
	$\varepsilon_{UD\%}$	tensile strength	6.75
	γ_s		1.15
	f_{yd}		400
	E_s		200
	ρ_m	mean desnsity	7850
	$Ø_L$	longitudinal	14
	$Ø_s$	stirup	8
	soil property		
	μ=		0.15
	K=		2
	v=		0.35
	β_z=		1.1
	E=		31,200
	I_f=		0.82

Reference	Design cover		
ES EN 1992-1-1. 4.4.1	$c_{nom} = c_{min} + \Delta c_{dev}$		
	c_{min} =	$\max[c_{min,b}; c_{min,dur}]$	
	$c_{min,b}$	$Ø_L$- $Ø_s$	6
	$c_{min,dur}$	XC2	45
	c_{min}		45
	Δc_{dev}		10
	c_{nom}		55

Reference	Check Resistance to horizontal load			
	horizontal resistance due to base friction (H_{fb})	μP_{usf}	107.89	
	hor. Res. due to passive pressure (H_{fs})	Kγ*(overburden	72	
		Kγ*(overburden+h)	86.40	76.032
	H_f=	H_{fb}+H_{fs}	183.93	
	FS=	H_f/H	4.60	
	Max. allow.hor. load (with FS=2)	H_a=H_f/FS	91.96	
	T	T/BL	124.9	
	T_a		280	
	check	T/T_a	0.45	
		H/H_a	0.43	
		$T/T_a+H/H_a$<1		ok

Flexural Design

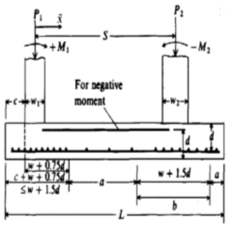
Reference					Output
	Plan size of footing	A_{req}=	P_{unf}/Q_{all}	2.57	
		b_{min}=	(P_{unf}/δ_{all})^0.5	1.60	1.7
	Bearing pressure	e_y=	$(-)M_{Ed,x}/N_{Ed}$	- 0.07	
		e_x=	$M_{Ed,y}/N_{Ed}$	0.02	
		L'=	L-2e_y	1.84	1.9
		B'=	L-2ex	1.66	1.8
		q_{max}=	N_{Ed}/B'L'	403.39	
	Check immediate settlement				
	S_v=	$PB(1-\mu^2)I_f/E_v$	24.87	25	ok
	Effective depth	Assume h=		0.40	
		d=	h-C_{nom}-φ/2	338.00	350
	Shear stress at column face			1.97	
		check	υc<0.8(f_{ck})^0.5	4.38	1.97
	Check thickness for punching	P_c=	C_p+8(1.5d)	6,200.00	
		A_p=	(a+3d)^2	2.40	
		V=	$q_{max}(b_{min}$^2-$A_p)$	337.84	
		υ=	V/C_pd	0.16	
	Ultimate shear stress	pt=	100A_{st}/(hd)	0.15	
		β=	(0.85f_{ck})/(6.89p_t) or 1 w	25.24	25
		υ_c=	0.85(0.8f_{ck})^0.5((1+5β)	0.31	
			υ<τ_c		ok
	Bending reinforcement	M=	q_{max}b(b/2-a/2)((b/2-a/2)	153.39	
	Bending for concrete	M_u=	0.156f_{ck} bd^2	1,031.94	ok
		$A_{s,min}$=	0.0013 bd	819.00	
		A_{sv}=	M/0.95f_{yk}z	1,055.65	6.86
		provide	use # 7Ø 14	1,077.53	ok
	Final check		100A_s/bh	0.15	
			check	υc >100As/bh	ok
	Critical section for shear			0.30	
		V=	q_{max}B'L_C	217.83	
		υ=	V/bd	0.35	
		check	υ<υ_c		FALSE
	spacing	$s_{v,max}$=	0.75d or 300 which less	262.50	250
		s_v<	2.175$f_{yk}A_{sv}$/b	586.76	ok
		provide	use # 8Ø 14 c/c 250		

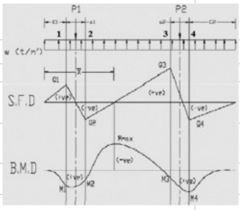
Reference	Cracking controling			Output
	Minimum area of reinforcement required in tension zone to control cracking			
ES EN 1992-1-1:2015	$A_{s,min}$ =	0.26(f_{ctm}/f_{yk})bd	1.0327	
	k=		1	
	$f_{ct,eff}$	0.3$f_{ck}^{(2/3)}$	2.90	
	σ_c=	N_{Ed}/bh	1916091.11	
	k_c=		0.40	
	A_{ct}=		0.885	
	$A_{s,min}$ =	$k_c k f_{ct,eff} A_{ct}/\sigma_s$	0.0022	
	check			ok!
7.3.3	$Ø_{max}$(max bar size)	from(32,25,16,12,10,8)	16	
	σ_s=	adjested from table	240	
	S_{max}=	from(300,250,200,150,100,5	250	
	σ_s=	adjested from table	200	
	G_k/Q_k=		2.13	
	Ψ_2	office	0.3	
	σ_{su}=	from chart	250	
	Flexural capacity	Redistribution from 15,20,25,30%)	15	
	δ value		0.85	
	K_{bal}=	0.363*(δ-0.44)-0.116*(δ-0.44)^2)	0.129	
	σ_s=	$\sigma_{su}(A_{s,req}/A_{s,prov})(1/\delta)$	288.14	
	check			ok
			Ø14 is sufficient	

Hand calculations excel sheet templet for combined footing

Project

Subject Combined footing





Design value of the vertical action (V_{Ed})						
Columns load				Bearing resistance		
	Unfactored load		Factored load	Allowable Bearing capacity		
Column 1 (P_1)	540	P_{d1}	810	q_{all}=		250
Column 2 (P_2)	890	P_{d2}	1335.0	$q_{ult}=P_{ult}/P*q_{all}$		375
P_{self} + back fill (P_3 (15%))	214.5	P_{d3}	321.8	Maximum and Minimum contact pressure		
P	1644.5	P_{ult}	2466.75	$q_{Max}=(p/(L*B))+(6*M_x/B*L^2)+(6*M_y/(L*B^2))$	229.56	OK
M_{x1}	85	M_{y1}	75	$q_{Min}=(p/(L*B))-(6*M_x/B*L^2)-$	155.87	OK
M_{x2}	50	M_{y2}	40	Net upward pressure on footing		
M_x	135	M_y	115	Intensity of Upward pressur	$P_u=P_{ult}/A$ (KN/m^2)	192.71
C/C Column Spacing	S(mm)	5		Soil pressure per m width (w)	$w=P_u*B$ (KN/m)	385.4
				Shear Force & Bending Moment Diagram		
Footing Dimension				Shear Force Diagram		
Location of resultant (x)	$x=(((P_{d2}*S)+M_x+My)/(P_{ult})$	3		Shear Force		Point of zero shear from left end
Column size 1 (a_1)	0.5	a_3	0.5	$Q_1= w*C_1$	154.17	$X_1=P_{d1}/w$ (from left end)
Column size 2 (a_2)	0.5	a_4	0.5	$Q_2 = w*(C_1+a_1)-P_{d1}$	-463.11	2.10
Requier Length of footing (L=2*((C1/2)+x))	6.1	use L	6.4	$Q_3= -w*(C2+a2)+P_{d2}$	949.57	$X_2=L-X_1$ (from right end)
Requier Width of footing (b= Pult/(L*qult))	1.08	use b	2	$Q_4 =-w*C_2$	-192.71	4.30
Requier area (A)	A= B*L	12.8		Maximum Bending Moment		
cantilever projection beyond a_1(C_1)		0.4		$M_1=w*C_1$^2/2	30.83	
cantilever projection beyond a_2(C_2)	$C_2=L-(C_1+(a_1/2)+S+a_2/$	0.5		$M_2=w*((C_1+a_1)$^2)/2+-$P_{d1}*a_1/2$	-46.40	
cantilever projection beyond P_1 (C_3)	$C_3=C_1+(a_1/2)$	0.65		$M_3=w*((C_2+a_2)$^2/)-$P_2*a_2/2$	-141.04	
cantilever projection beyond P_1 (C_4)	$C_4=C_2+(a_2/2)$	0.75		M4=$w*C_2$^2/2	48.18	
				M_{max}= w*Y^2/2-(P_{d1*}(Y-C_1-a_1/2)	81.42	

Material's property								
Reference	Concrete			Reference	Reinforcement			
	F_{ck}		25		Class	A=1,B=2,C=3	3	
	$F_{ck,cube}$		30		f_{yk} (cold)		460	
	f_{ctm}		2.6		k	$(f_t/f_y)_k$	1.2	
	$f_{ctk}^{0.05}$		1.8	ES EN	$\varepsilon_{UK\%}$		7.5	
	$f_{ctk}^{0.95}$		3.3	1992-1-1.	$\varepsilon_{Ud\%}$	tensile strength	6.75	
	E_{cm}	modulus of elasticity	31	3.2.2	γ_s		1.15	
ES EN	$\varepsilon_{c1\%}$		2.1		f_{yd}		400	
1992-1-1.	$\varepsilon_{cu1\%}$		3.5		E_s		200	
3.1.3	$\varepsilon_{c2\%}$		2		$Ø_L$	longitudinal	16	
Table 3.1	$\varepsilon_{cu2\%}$		3.5		$Ø_T$	Transverse	20	
	γ_c	per=1.5,ac1.2	1.5		$Ø_s$	stirup	8	
	f_{cd}		16.67		Effective Depth			
Reference	Design cover				h= Assumed		0.45	
	$c_{nom} = c_{min} + \Delta c_{dev}$				Top Reinforcement			
ES EN	c_{min} =	max[$c_{min,b}$; $c_{min,dur}$]			% of redistribution (from 10,15,20,25,30)	15	K_{bal}	0.168
1992-1-1.	$c_{min,b}$	$Ø_L$- $Ø_s$	8		K= $M_{Ed}/(bd_x^{2}f_{ck})$	0.011	If K<K_{bal} "no com. Rein" If K>K' "com. Rein.	no compression reinforcement
4.4.1	$c_{min,dur}$	XC2	45		$z=d_x*(0.25-(K/1.134)) \le 0.95*d_x$		93.04	ok
	c_{min}		45		$Areq=M_{Ed}/(0.87*f_{yk}*z)$ > A	1,293.93	$A_{s,min}=0.26(f_{ctm}/f_{yk})*bd_x$	ok
	Δc_{dev}		10		provide		1,809.56	ok
	c_{nom}		55		Number of bars required (	A_{req}/A of Ø	6.44	7
					Spacing (S)	$(b-(c_{nom}*2)-Ø_L/2)/N_b$	268.86	250
					provide	use Ø 16C/C 250	1,407.43	ok

Bottom Longitudinal Reinforcement			
% of redistribution (from 10,15,20,25,30)	15	K_{bal}	0.168
Effective depth (d_x)	d_x= h-c-(0.5*$Ø_L$)		387
K= M_{Ed}/(b*$d_x^{2*}f_{ck}$)	0.01	If K<K_{bal} "no com reinforcement" If K>K'	no com reinforcement'
z =d_x*(0.25-(K/1.134)) ≤ 0.95*d_x		94.55	ok
A_{req}=M_{Ed}/(0.87*f_{yk}*z)	814.85	$A_{s,min}$=0.26(f_{ctm}/f_{yk})*bd_x	1,137.44
provide		1,407.43	ok
Number of bars required (N_b)	A_{req}/A of Ø	5.66	8
Spacing (S)	(b-(c_{nom}*2)-$Ø_L$/2)/N	235.25	200
Bottom Transverse Reinforcement at C_1			
Consider (b_1)		1000	
Soil persure (P_s)	P_s=w*b_1	385.43	
(C_w)	C_w=(b-a_1)/2	0.75	
M_{Ed}	M_{Ed}=P_s*C_w*C_w/2	108.40	
% of redistribution (from 10,15,20,25,30)	20	K_{bal}	0.153
Effective depth (d_y)	d_y= h-c-(1.5*$Ø_T$)		365
K= M_{Ed}/(b_1*d_y^2*fck)	0.03	If K<K_{bal} "no com reinforcement" If K>K_{bal} "com reinforcement req"	no com reinforcement'
z=d_y*(0.25-(K/1.134)) ≤ 0.95*d_y		80.77	ok
A_{req}=M_{Ed}/(0.87*f_{yk}*z)	3,353.42	$A_{s,min}$=0.26(f_{ctm}/f_{yk})*$b_1$$d_y$	536.39
Number of bars required (N_b)	A_{req}/A of Ø	10.67	11
Spacing (S)	(b-(c_{nom}*2)-$Ø_T$/2)/N	80.00	80
provide	use # 11Ø 20 C/C	3,455.75	ok
Critical shear at 1d			
y_1=X_1-(C_1+a_1)		1.20	
y_2=X_2-(C_2+a_2)		3.30	
Design shear (P_{Ud})	V_{Ed}=Q_3(y_2-d_x)/y_2	838.16	
Critical shear stress (V_{Ed})	V_{Ed}=P_{ud}/(b*d_x) (MI	1.08	
k=1+(200/d_x)^0.5	k=1+(200/d_x)^0.5	1.72	ok
$V_{Rd,c}$ =V_{min}=(0.035*k^(3/2)*(f_{ck})^0.5	V_{Ed} < $V_{Rd,c}$	0.39	Shear rein required
Punching shear at 2d			
d_{avg}	(d_x+d_y)/2	376	
2d (d')	d_{avg}*2	752	
Control perimeter (U)	U=2((a_2+ a_2)+(2π*	4726.96	
Area within perimeter (A)	A=(a_2+a_2)+(2*a_2*d')+(2*a_2*d')+(π*d'^2)	1.78	
Design shear (P_{Ud})	P_{Ud}=P_{d2}-w*A (kN)	649.67	
Punching shear stress (V_{Ed})	V_{Ed}=P_{ud}/(U*d') (MI	0.18	
Punching shear resistance			
k=1+(200/d_x)^0.5	k=1+(200/d_x)^0.5	1.72	ok
$V_{Rd,c}$=V_{min}=0.035k^(3/2)f_{ck}^(1/2)	$V_{Rd,c}$> V_{Ed}	0.39	ok

	Bottom Transverse Reinforcement at C_2			
	Consider (b_1)		1000	
	Soil persure (P_s)	P_s=w*b_1	385.43	
	(C_w)	C_w=(b-a_2)/2	0.75	
	M_{Ed}	M_{Ed}=P_s*C_w*C_w/2	108.40	
	% of redistribution (from 10,15,20,25,30)	20	K_{bal}	0.153
	Effective depth (d_y)	d_y= h-c-(1.5*$Ø_T$)		365
	K= M_{Ed}/(b_1*d_y^2*fck)	0.03	If K<K_{bal} "no com reinforcement" If K>K_{bal} "com reinforcement req"	no com reinforcement'
	z=d_y*(0.25-(K/1.134)) ≤ 0.95*d_y		80.77	ok
	A_{req}=M_{Ed}/(0.87*f_{yk}*z)	3,353.42	$A_{s,min}$=0.26(f_{ctm}/f_{yk})*$b_1$$d_y$	412.61
	Number of bars required (	A_{req}/A of Ø	10.67	11
	Spacing (S)	(b-(c_{nom}*2)-$Ø_T$/2)/N_b	80.00	80
	provide	use # 11Ø 20 C/C 80	3,455.75	ok
	Maximum Punching Shear at Column Perimeter			
	Column Perimeter (U_o)	Uo=4*a_2	2	
	Maximum shear resistance ($V_{Rd,max}$)	$V_{Rd,max}$=0.5*U_o*d(0.6*(1-(fck/250)))*(fck/1.5)	3483000.0	ok
	Cracking controling			Output
	Minimum area of reinforcement required in tension zone to control cracking			
ES EN 1992-1-1:2015	A,min	0.0013bd	0.001	
	$A_{s,min}$ =	0.26(f_{ctm}/f_{yk})bd > 0.0013bd	0.001	0.0011
	k=		0.895	
	$f_{ct,eff}$	0.3$f_{ck}$$^{(2/3)}$	2.56	
	$σ_c$=	N_{Ed}/bh	2.74	
	k_c=		1.00	
	A_{ct}=	x*b/10^6	0.00	
	$A_{s1,min}$ =	k_ck$f_{ct,eff}$$A_{ct}$/$σ_s$	0.00	
	check	$A_{s1,min}$ <$A_{s,min}$		ok!
7.3.3	$Ø_{max}$(max bar size)	from(32,25,16,12,10,8)	16	
	$σ_s$=	adjested from table	240	
	S_{max}=	from(300,250,200,150,10	250	
	$σ_s$=	adjested from table	200	
	G_k/Q_k=		2.33	
	$Ψ_2$	office	0.3	
	$σ_{su}$=	from chart	250	
	Flexural capacity	Redistribution from 15,20,25,30%)	15	
	δ value		0.85	
	K_{bal}=	0.44)^2)	0.129	
	$σ_s$=	$σ_{su}$($A_{s,req}$/$A_{s,prov}$)(1/δ)	285.41	
	check			ok
			Ø20 is sufficient	

Hand calculations excel sheet templet for mat foundation

Project																			
Subject Mat foundation																			
Date																			
										21									
						10.5							10.5						
		O_1	C/C	0.5	5		5		5		5		5			0.5	O_1		
			0.5	A		B		C		D		E				F			
				2276	1	3250	2	2957	3	2872	4	5	3252	6	2376		3		
			5	P												Q		8	
				3295	7	5266	8	5625	9	5542	10	11	5266	12	3395		5		
			5	O												R			
		16		3795	13	5108	14	5370	15	5356	16	17	5098	18	3875		5		
			5	N												S			
				2120	19	3609	20	3712	21	3715	22	23	3615	24	2210		0.25	8	
			0.25	M												T			
				1	25	1	26	1	27	1	28	29	1	30	1		0.25		
			0.25	L		K		J		I		H				G			
					3		5		5		5		3						

a) Material's property

Reference					Output
	Concrete				
	F_{ck}		25		
ES EN	$F_{ck,cube}$		30		
1992-1-1,	γ_c	per=1.5,acc1.2	1.5		
3.1.3	f_{cd}		16.67		
Table 3.1	γ		25		
	Partial factors for action				
	γ_G (DL+LL)		1.5		
	Column size (C_s)				
		L= 0.5		B= 0.5	

b) Reinforcement

Reference				
	Class	A=1,B=2,C=3	3	
	f_{yk} (solid)		500	
	γ_s		1.15	
ES EN	f_{yd}		434.78	
1992-1-1,	E_s		200	
3.2.2	$Ø_L$	longitudinal	20	
	$Ø_s$	stirup	8	

c) Design cover

Reference				Output
	cnom = cmin + Δcdev			
ES EN	c_{min} =	max($c_{min,b}$; $c_{min,dur}$)		
1992-1-1,	$c_{min,b}$	$Ø_L$ - $Ø_s$	12	
4.4.1	$c_{min,dur}$	XC2	45	
	c_{min}		45	
	$Δc_{dev}$		10	
	c_{nom}			55

1. Loading

P_T=	Q_1+Q_2+Q_3+...		81,104
P_{sd}=	10%		8,110.40
R=			89,214.40
A=			336
q_{all}=			380

2. Location of resultant and Eccentricity in X and Y direction

x=	ΣQ	10.20	e_x=x-B/2	(0.30)	<B/6	ok
y=	ΣQ	7.10	e_y=y-L/2	(0.90)	<B/6	ok

3. Comparing Maximum Contact Pressure and Bearing Capacity

q_{max}=	R/BL(1+(6e_x/B)+(6e_y/L))	377.84	q_{all}>q_{max}	ok

4.2. Soil Reaction and Column Loads Adjustement Calculations

X-direction strips

Strip	Width (B_1)	point	q_{point}	q_{av}	q_{av}*B_1*B	F_{av}=(q_{av}+ΣF)/2	F=F_{av}/ΣF	$q_{av,mod}$=q_{av}(F_{av}/q_{av})
AFPQ	3	A,F	175.7	192.54	12129.71	13368.36	0.92	212.20
		P,Q	209.4					
PQOR	5	P,Q	209.4	237.45	24932.07	24963.04	1.00	237.74
		O,R	265.5					
ORNS	5	O,R	265.5	293.59	30826.93	27776.96	1.12	264.54
		N,S	321.7					
NSMT	0.25	N,S	321.7	336.40	1766.09	885.54	177.11	168.68
		M,T	351.1					
MTLG	0.25	M,T	351.1	353.24	1854.51	929.76	185.95	177.10
		L,G	355.3					

Y-direction strips

Strip	Width (B_1)	point	q_{point}	q_{av}	q_{av}*B_1*B	F_{av}=(q_{av}+ΣF)/2	F=F_{av}/ΣF	$q_{av,mod}$=q_{av}(F_{av}/q_{av})
ABKL	3	A,B	195.0	284.80	15679.25	13978.63	1.10	262.05
		K,L	374.6					
BCJK	5	B,C	186.4	276.23	22098.32	19666.16	1.14	245.83
		J,K	366.1					
CDIJ	5	C,D	175.7	265.52	21241.52	19453.26	1.10	243.17
		I,J	355.3					
DEHI	5	D,E	165.0	254.81	20384.73	18935.37	1.08	236.69
		H,I	344.6					
EFGH	3	E,F	156.4	246.24	11819.58	14525.79	0.84	302.62
		G,H	336.1					

4.3. Modified Column Loads / 5.1. Shear Force / 5.2. Bending Moment

X-direction Strips

Strip	Column	Q_n	F	Q'_n=Q*F	load per unit length ($Bq_{av,mod}$)	Distance	Shear left	Shear right	Distance (x) @ Max.BM	Moment left	Moment right
AFPQ	1	2276	0.92	2,083.00	636.59	0.5	318.3	(1,764.71)		79.57	
	2	3250		2,974.41		5	1,418.24	(1,556.17)	2.77	-2446.00	1579.82
	3	2957		2,706.25		5	1,626.77	(1,079.48)	2.44	-1902.07	2078.57
	4	2872		2,628.46		5	2,103.46	(525.00)	1.76	-915.25	3475.20
	5	3252		2,976.24		5	2657.94	(318.29)	0.82	-216.49	5548.84
PQOR	6	3295	1.00	3,290.92	1188.72	0.5	594.4	(2,696.56)		148.59	
	7	5266		5,259.48		5	3,247.02	(2,012.46)	2.37	-3058.53	4434.68
	8	5625		5,618.03		5	3,931.12	(1,686.91)	1.69	-1703.51	6500.18
	9	5542		5,535.13		5	4,256.67	(1,278.46)	1.42	-1196.95	7621.56
	10	5266		5,259.48		5	4665.12	(594.36)	1.08	-687.49	9154.13

4.1. Pressure Distribution

I_x=		$BL^3/12$		7,168.00			
I_y=		$LB^3/12$		12,348.00			
M_x=		$R*e_y$	-	80,484.65	M_x/I_x=	(11.23)	
M_y=		$R*e_x$	-	26,649.20	M_y/I_y=	(2.14)	
q=	$(P_{total}/A)\pm(M_y/I_y)x \pm(M_x/I_x)y$						
point	R/A	x	$(M_y/I_y)x$	y	$(M_x/I_x)y$	q'	check
			-2.1		-11.23		$q \le q_{all}$
A	265.5	-10.5	22.5	5.0	-89.83	198.2	ok
B	265.5	-7.5	16.1	5.0	-89.83	191.8	ok
C	265.5	-2.5	5.4	5.0	-89.8	181.0	ok
D	265.5	2.5	-5.4	5.0	-89.8	170.3	ok
E	265.5	7.5	-16.1	5.0	-89.8	159.6	ok
F	265.5	10.5	-22.5	5.0	-89.8	153.2	ok
P	265.5	-10.5	22.5	5.0	-56.1	231.9	ok
Q	265.5	10.5	-22.5	5.0	-56.1	186.9	ok
O	265.5	-10.5	22.5	0.0	0.0	288.0	ok
R	265.5	10.5	-22.5	0.0	0.0	243.0	ok
N	265.5	-10.5	22.5	-5.0	56.14	344.2	ok
S	265.5	10.5	-22.5	-5.0	56.14	299.2	ok
M	265.5	-10.5	22.5	-7.6	85.62	373.6	ok
T	265.5	10.5	-22.5	-7.6	85.62	328.6	ok
L	265.5	-10.5	22.5	-8.0	89.83	377.8	ok
K	265.5	-7.5	16.1	-8.0	89.83	371.4	ok
J	265.5	-2.5	5.4	-8.0	89.83	360.7	ok
I	265.5	2.5	-5.4	-8.0	89.83	350.0	ok
H	265.5	7.5	-16.1	-8.0	89.83	339.3	ok
G	265.5	10.5	-22.5	-8.0	89.8	332.9	ok

Strip	Column	Q_u	F	Q'_u	load per unit length	Distance	Shear left	Shear right	Distance (x) @ Max.BM	Moment left	Moment right
ORNS	11	3795	1.12	4,263.10	1322.71	0.5	661.4	(3,601.74)		165.36	
	12	5108		5,738.05		5	3,011.82	(2,726.23)	2.72	-4903.76	3428.97
	13	5370		6,032.37		5	3,887.34	(2,145.03)	2.86	-2809.49	5712.27
	14	5356		6,016.64		5	4,468.53	(1,548.10)	1.62	-1739.28	7548.05
	15	5098		5,726.82		5	5065.46	(661.36)	1.17	-905.95	9699.34
NSST	16	1	177.11	177.11	42.17	0.5	21.1	(156.02)		5.27	
	17	1		177.11		5	54.82	(122.29)	3.78	-288.65	35.63
	18	1		177.11		5	88.55	(88.55)	2.98	-177.32	92.98
	19	1		177.11		5	122.29	(54.82)	2.18	-92.98	177.32
	20	1		177.11		5	156.02	(21.08)	1.38	-35.63	288.65
MRLG	21	1	185.95	185.95	44.27	0.5	22.1	(163.81)		5.53	
	22	1		185.95		5	57.56	(128.39)	3.78	-303.06	37.41
	23	1		185.95		5	92.98	(92.98)	2.98	-186.17	97.62
	24	1		185.95		5	128.39	(57.56)	2.18	-97.62	186.17
	25	1		185.95		5	163.81	(22.14)	1.38	-37.41	303.06

Y-direction Strips

Strip	Column	Q_u	F	Q'_u	load per unit length ($B_{q_{Average}}$)	Distance	Shear left	Shear right	Distance (x) @ Max.BM	Moment left	Moment right
ABKL	1	2276	1.10	2,492.29	786.16	0.5	393.1	(2,099.21)		98.27	
	6	3295		3,608.13		5	1,831.61	(1,776.52)	2.67	-2002.65	2133.65
	11	3795		4,155.64		5	2,154.30	(2,001.34)	2.26	-2007.22	2951.69
	16	2120		2,321.47		5	1,929.48	(391.99)	2.58	-2547.41	2367.76
	21	1		1.10		0.25	-195.45	(196.54)	0.50	-97.72	24.29
BCJK	2	3250	1.14	3,708.66	1229.13	0.5	614.6	(3,094.09)		153.64	
	7	5266		6,009.17		5	3,051.58	(2,957.58)	2.52	-3894.36	3788.10
	12	5108		5,828.87		5	3,188.09	(2,640.78)	2.41	-3558.32	4134.58
	17	3609		4,118.32		5	3,504.90	(613.43)	2.15	-2856.84	4997.13
	22	1		1.14		0.25	-306.14	(307.28)	0.50	-153.07	38.13
CDIJ	3	2957	1.10	3,256.34	1215.83	0.5	607.9	(2,648.43)		151.98	
	8	5625		6,194.45		5	3,430.72	(2,763.71)	2.18	-2886.52	4840.24
	13	5370		5,913.62		5	3,315.43	(2,598.18)	2.27	-3141.11	4520.41
	18	3712		4,087.77		5	3,400.96	(686.81)	2.14	-2776.12	4983.05
	23	1		1.10		0.25	-302.86	(303.96)	0.50	-151.43	37.72
DEHI	4	2872	1.08	3,110.05	1183.46	0.5	591.7	(2,518.32)		147.93	
	9	5542		6,001.36		5	3,398.98	(2,602.38)	2.13	-2679.41	4881.05
	14	5356		5,799.94		5	3,314.92	(2,485.02)	2.20	-2861.26	4642.61
	19	3715		4,022.93		5	3,432.28	(590.65)	2.10	-2609.02	4977.16
	24	1		1.08		0.25	-294.78	(295.87)	0.50	-147.39	36.71
EFGH	5	3252	0.84	2,741.29	907.86	0.5	453.9	(2,287.36)		113.48	
	10	5266		4,439.00		5	2,251.95	(2,187.04)	2.52	-2881.50	2792.99
	15	5098		4,297.38		5	2,352.26	(1,945.12)	2.41	-2634.30	3647.35
	20	3615		3,047.28		5	2,594.19	(453.09)	2.14	-2083.73	3786.42
	25	1		0.84		0.25	-226.12	(226.97)	0.50	-113.06	28.16

6. Effective Depth (d) of the Mat

$U = b_o d \varphi(0.34(f_{ck})^{0.5})$

Column	Factored max. col. Load	Perimeter (b_o)			$\varphi*0.34*(f_{ck})^{0.5}$		
Corner Column	2.98	$2C_1+C_2+d$	1.50	d	1.18		
Edge Column	5.73	$2C_1+2C_2+2d$	2	2d	1.18		
Center Column	6.03	$4(d+C_1)$	2	4d	1.18		
	c	b	a	d	sat mat		
Corner Column	2.98	1.77	1.18	0.84			
Edge Column	5.73	2.36	1.18	1.79			
Center Column	6.03	2.36	1.18	1.85	1849.1		
	h=	$d+cover+\Phi_b/2$	1914.07	take h=	1900.00	take d=	1727

7.1.1. Maximum Positive Moments per Unit Width / 7.1.1. Maximum Negative Moments per Unit Width

X-direction Strips				M_u	X-direction Strips				M_u
Strip		Maximum Moment	Max. Mom. per unit width (M')	M' *(load factor) (s)	Strip		Maximum Moment	Max. Mom. per unit width	M' *(load factor) (s)
ORNS	Bottom (M+)	9699.34	1939.87	2,179.14	ORNS	Top (M-)	-4903.76	-980.75	-1101.72

7.2.1. Maximum Positive Moments per Unit Width / 7.2.2. Maximum Negative Moments per Unit Width

Y-direction Strips				M_u	Y-direction Strips				M_u
Strip		Maximum Moment	Max. Mom. per unit width (M')	M' *(load factor)	Strip		Maximum Moment	Max. Mom. per unit width (M')	M' *(load factor)
BCJK	Bottom (M+)	4997.13	999.43	1,140.47	BCJK	Top (M-)	-3894.36	-778.87	-888.79

8.1.1. Bottom Reinforcement / 8.1.2. Top Reinforcement

Clause 6.1	Redistribution from (0,5,10,15,20,25,30%)	15	δ	0.85		Redistribution from (0,5,10,15,20,25,30%)	15	δ	0.85	
ES EN	K'	0.167				K'	0.167			
1992-1-1:2004	$K_{bal}=M_{ed}/f_{ck}bd^2$	0.029	K_{bal}=	0.363*(δ-0.44)-0.116*(δ-0.44)^2)	0.129	$K_{bal}=M_{ed}/f_{ck}bd^2$	-0.015	K_{bal}=	0.363*(δ-0.44)-(	0.129
	$K_{bal} < K_{bal}$	no comp.				$K_{bal} < K_{bal}$	no comp.			
				a	s				a	s
	z=	d(0.5+(0.25-(K/1.134))^0.5)	1681.28	9.5	105.64	z=	d(0.5+(0.25-(K/1.134))^0.5)	1749.22	-4.6	-216.98
	A_s=	$M_{ed}/(0.87f_{yk}z)$	2979.50	use φ20 c/c	300	A_s=	$M_{ed}/(0.87f_{yk}z)$	-1447.90	use φ20 c/c	300
	Check Mc					Check Mc				
	a=	$A_sf_{yk}/(0.85f_{ck}b)$	78.11			a=	$A_sf_{yk}/(0.85f_{ck}b)$	-34.07		
	c=	a/B_1	82.48			c=	a/B_1	-40.08		
	ε_t=	(d-c)/c*0.003	0.060	εt>0.005	tension control	ε_t=	(d-c)/c*0.003	-0.132	εt>0.005	com control
	Mc=	$\varphi A_sf_{yk}(d-a/2)$	2260583799	(Mc>Mu) "ok"	ok	Mc=	$\varphi A_sf_{yk}(d-a/2)$	-1136337228	(Mc>Mu) "ok"	ok

8.2.1. Bottom Reinforcement / 8.2.2. Top Reinforcement

Redistribution from (0,5,10,15,20,25,30%)	15	δ	0.85		Redistribution from (0,5,10,15,20,25,30%)	15	δ	0.85	
K'	0.167				K'	0.167			
$K_{bal}=M_{ed}/f_{ck}bd^2$	0.015	K_{bal}=	0.363*(δ-0.44)-0.116*(δ-0.44)^2)	0.129	$K_{bal}=M_{ed}/f_{ck}bd^2$	-0.012	K_{bal}=	0.363*(δ-0.44)-0.116*(δ-0.44)^2)	0.129
$K_{bal} < K_{bal}$	no comp.				$K_{bal} < K_{bal}$	no comp.			
			a	s			a	s	
z=	d(0.5+(0.25-(K/1.134))^0.5)	1703.38	4.9	204.11	z=	d(0.5+(0.25-(K/1.134))^0.5)	1744.97	-3.7	-268.30
A_s=	$M_{ed}/(0.87f_{yk}z)$	1539.16	use φ20 c/c	300	A_s=	$M_{ed}/(0.87f_{yk}z)$	-1170.91	use φ20 c/c	300
Check Mc					Check Mc				
a=	$A_sf_{yk}/(0.85f_{ck}b)$	36.22			a=	$A_sf_{yk}/(0.85f_{ck}b)$	-27.55		
c=	a/B_1	42.61			c=	a/B_1	-32.41		
ε_t=	(d-c)/c*0.003	0.119	εt>0.005	control	ε_t=	(d-c)/c*0.003	-0.163	εt>0.005	com control
Mc=	$\varphi A_sf_{yk}(d-a/2)$	1183612933	(Mc>Mu) "ok"	ok	Mc=	$\varphi A_sf_{yk}(d-a/2)$	-917231422.6	(Mc>Mu) "ok"	ok

Check for Shear

X-direction Strips										Y-direction Strips									
Strip	$V_{ED}=V_{max}$	ρ_l=	$\rho_l<0.02$	$C_{Rd,c}$=	K=	K≤2	$V_{Rd,c}$=	$V_{Rd,c}>V_{ED}$		Strip	$V_{ED}=V_{max}$	ρ_l=	$\rho_l<0.02$	$C_{Rd,c}$=	K=	K≤2	$V_{Rd,c}$=	$V_{Rd,c}>V_{ED}$	
		$A_s/(b_wd)$		$0.18/\gamma_c$	$1+(200/d)^{0.5}$		$(C_{Rd,c}k(100\rho_lf_{ck})^{(1/3)})b_wd$		adequate shear capacity			$A_s/(b_wd)$		$0.18/\gamma_c$	$1+(200/d)^{0.5}$		$(C_{Rd,c}k(100\rho_lf_{ck})^{(1/3)})b_wd$		adequate shear capacity
ORNS	5,065.5	0.0017	ok	0.12	1.34	ok	452,145.80	ok		ORNS	3,504.9	0.0009	ok	0.12	1.34	ok	362,788.97	ok	

Hand calculations excel sheet templet for pile foundation

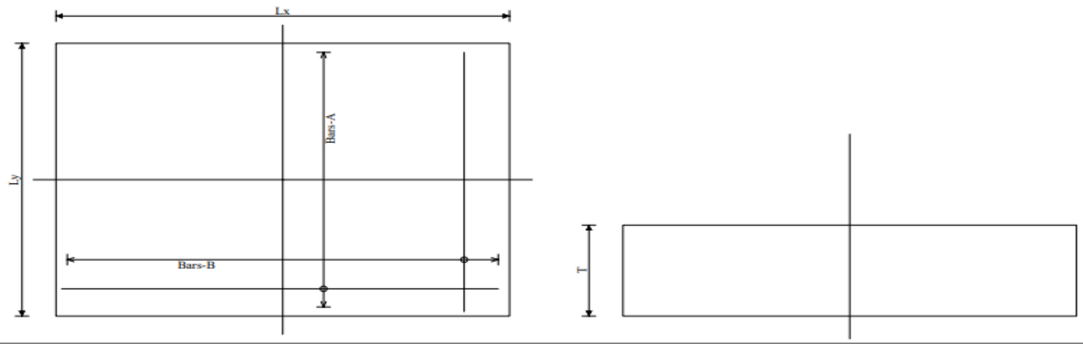
Pile design in put for G+6					
In put data					
Material property					
	Concrete	Rebar			
f_{ck}	30				
f_{yk}		500			
E	32,000.00	200,000.00	N/mm^2		
	32,000,000.00		KN/m^2		
Geotechnical report (Meyerhof's method)					
Pile data					
For homogeneous granular soil					
Pile dimension / width (D)				0.75	
Area crossection of pile (A)				0.4	m^2
Length of pile (L)				10	m
FS (point bearing)				3	
FS (skine friction)				1.5	
Point Bearing Capacity					
N_{60} (Above 10D and below 4D pile point		above 10D	2.5	23.5	
		Below 4D	13		
use				24	
q_a (Ultimate point resistance, using SPT meyerhof)		$0.4*P_a*N_{60}*L/D <4P_a*N_{60}$		9600.00	
Q_p(pile point bearing capacity)		A_pq_a		4,241.15	
Frictional Resistance					
displacement					
displacement pile		use	non displacement (cast in	use	
N_{60}	14.7	15	14.7	15	
f_{av}	$0.02p_aN_{60}$	30	$0.01p_aN_{60}$	15	
Qs	pLf_{av}	900		450	
safe load of pile (**Q_{all}**)	$(Q_p+Q_s)/FS$			**1,713.72**	KN
Loading data			**Spring constant or point spring**		
Axial Load (P_x)			**Spring constant (K)**	E_cA/L	**1,413,716.69**
Axial Load (P_y)					
Axial Load (P_z)	**55393**		**Piles Arrangement**		
M_x	**0**		e_x=	M_y/p	0.00
M_y	**0**		e_y=	M_x/p	0.00
M_z			Required number of piles (n)	$1.2*P/Q_{all}*(1+e_x)*(1$	38.79
			use number of total pile		**48**
X-direction			**Y-direction**		
Number of raw		6	Number of column		8
Number of pile on each raw (n)		8	Number of pile on each		6
Spacing between pile (S_{min})	2.5D (for end beari	1.875	Spacing between pile (S_{min})	2.5D (for end bearing	1.875
	3D (for friction)	2.25		3D (for friction)	2.25
S		**2.06**	**S**		**2.06**
Spacing between pile (**S_{max}**)	6D	**4.5**	Spacing between pile (**S_{max}**)	6D	**4.5**
Spacing at edge of footing (e)	D up to 1.5D	**1.125**	**Spacing at edge of footing**	1.5D	**1.125**
Total length (L_g)	(n-1)S+2(D/2)	**16.31**	**Total Width (B_g)**	(m-1)S+2(D/2)	**12.94**

Annex D

Structural design of foundation by Safe soft ware

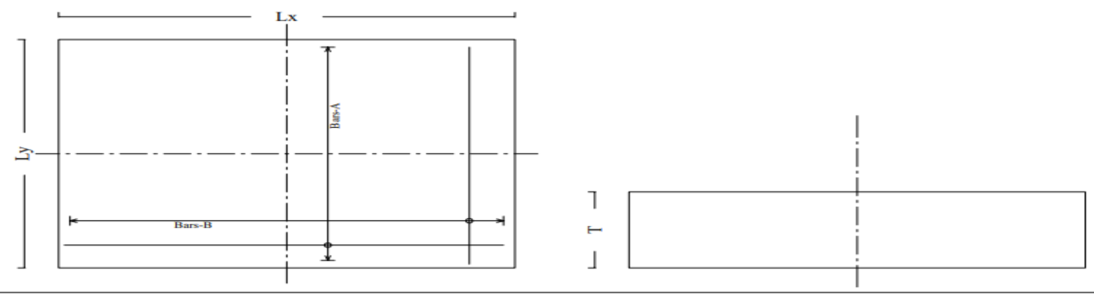
G+2 Isolated footing table

Sr. No.	Type	Nos	Lx	Ly	T	Rebars-A	Rebars-B
1	F1	12	2.20 m	2.20 m	0.45 m	12-16	12-16
2	F2	4	2.00 m	2.00 m	0.42 m	7-16	6-16
3	F3	8	2.4 m	2.4 m	0.55 m	18-16	19-16



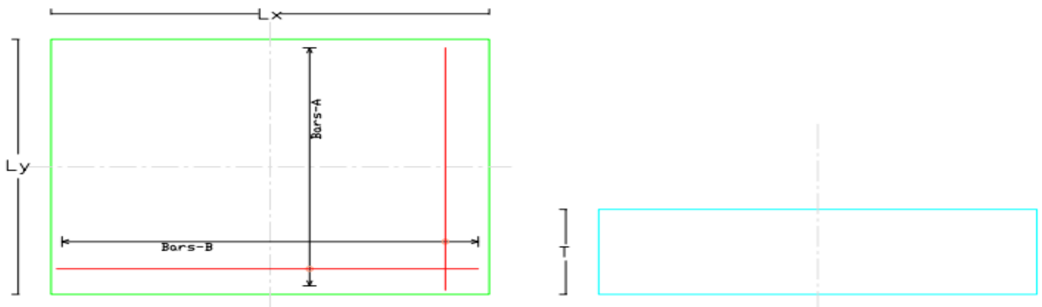
G+4 Isolated footing table

Sr. No.	Type	Nos	Lx	Ly	T	Rebars-A	Rebars-B
1	F1	4	3.00 m	3.00 m	0.48 m	12-16	13-16
2	F2	12	3.50 m	3.50 m	0.58 m	17-16	17-16
3	F3	8	4.20 m	4.20 m	0.68 m	27-16	28-16



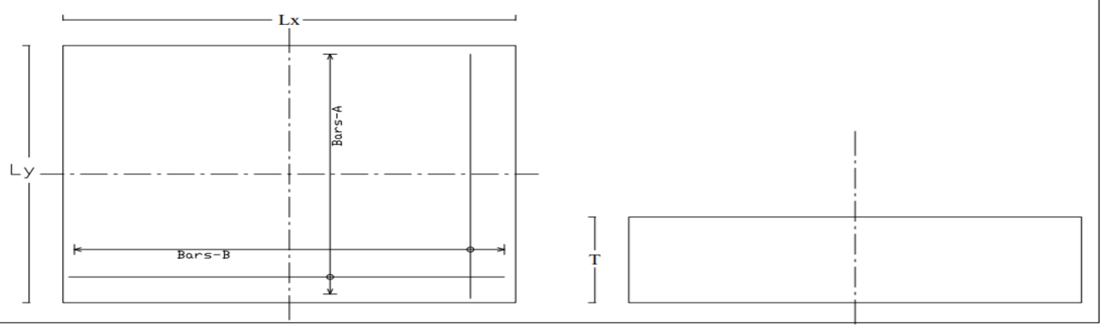
G+6 Isolated footing table

Sr. No.	Type	Nos	Lx	Ly	T	Rebars-A	Rebars-B
1	F1	4	3.00 m	3.00 m	0.50 m	15-16	14-16
2	F2	6	3.20 m	3.20 m	0.75 m	31-16	30-16
3	F3	10	3.6 m	3.6 m	0.65 m	24-16	27-16
4	F4	1	10.50 m	11.00 m	1.00 m	47-16	78-16



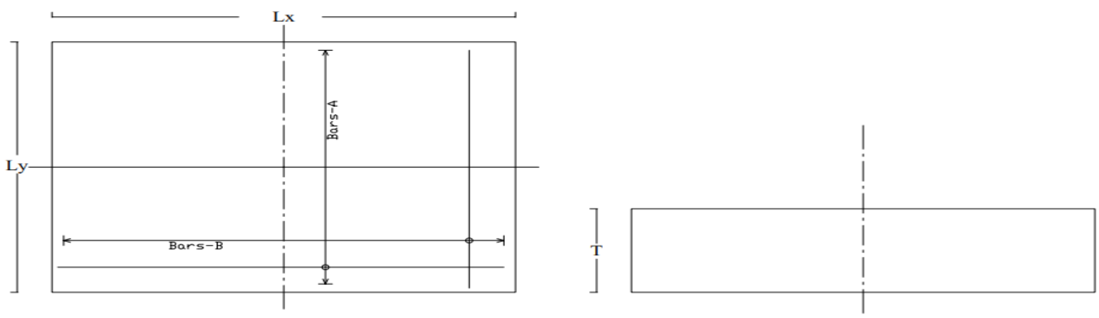
G+6 Combined footing table

Sr. No.	Type	Nos	Lx	Ly	T	Rebars-A	Rebars-B
1	F1	4	9.000 m	4.000 m	0.600 m	30-16	28-16
2	F2	1	9.000 m	4.000 m	0.700 m	33-16	32-16
3	F3	3	9.000 m	4.000 m	0.750 m	31-16	29-16
4	F4	2	9.500 m	4.000 m	0.750 m	30-16	33-16



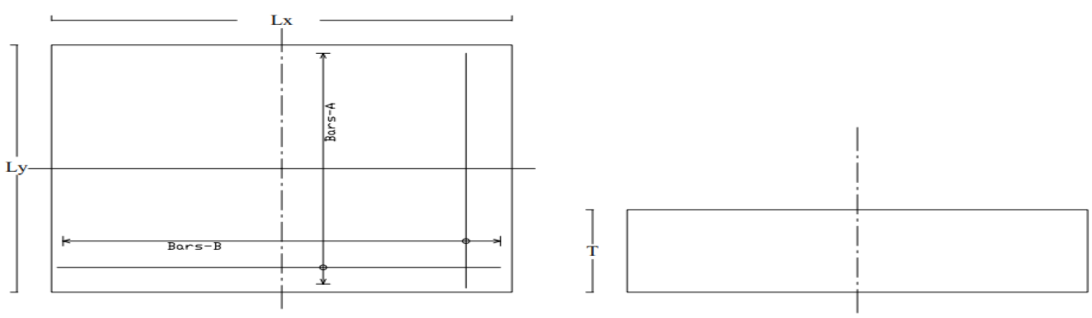
G+8 Combined footing table

Sr. No.	Type	Nos	Lx	Ly	T	Rebars-A	Rebars-B
1	F1	1	**8.50m**	**3.60m**	0.850 m	54-16	34-16
2	F2	1	**9.00m**	**4.00m**	0.850 m	54-16	44-16
3	F3	2	**9.00m**	**4.00m**	0.850 m	52-16	46-16
4	F4	2	**8.50m**	**3.60m**	0.650 m	39-16	37-16
5	F5	2	**9.00m**	**4.00m**	0.700 m	56-16	52-16
6	F6	2	**8.50m**	**3.60m**	0.700 m	61-16	43-16

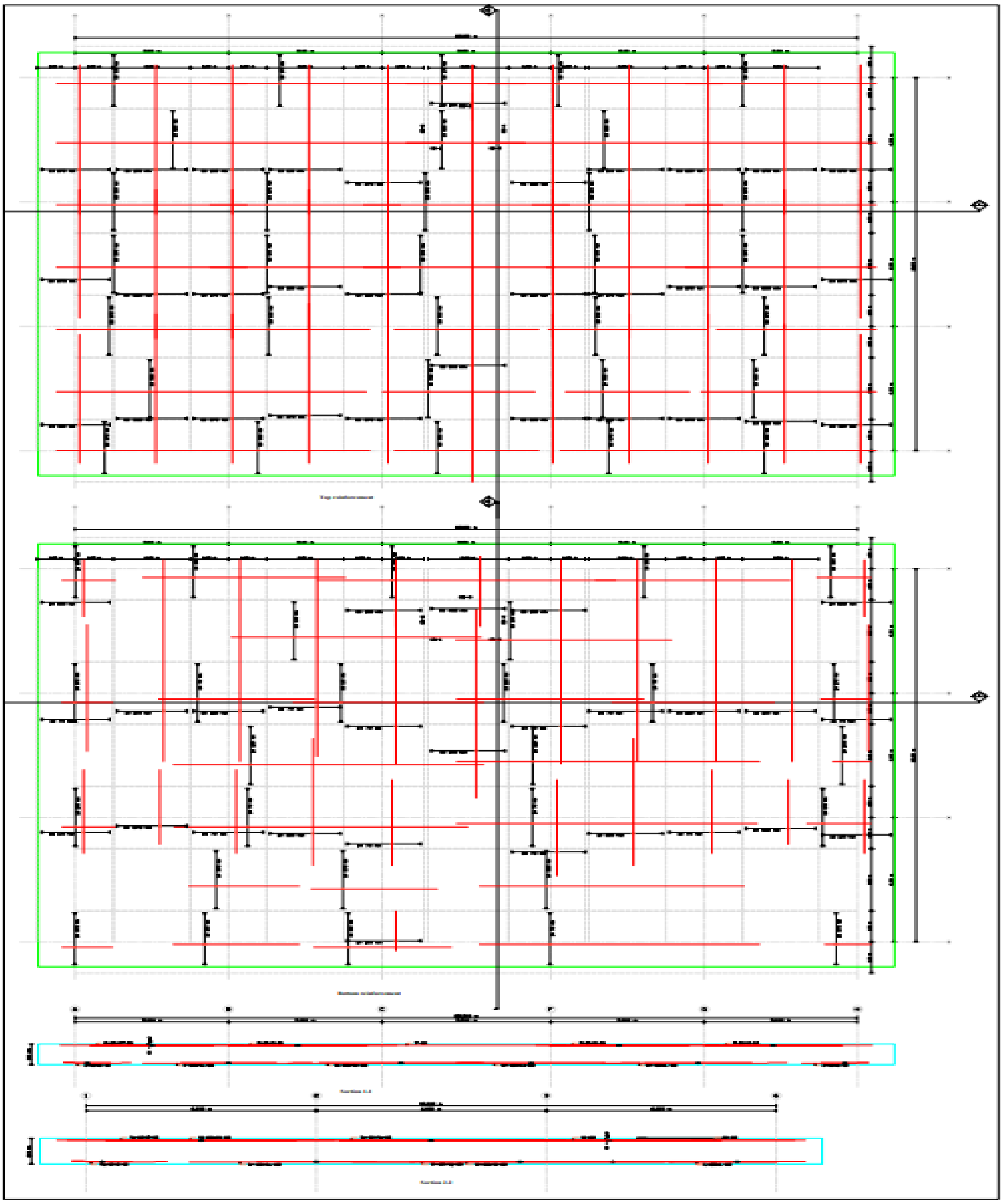


G+10 Combined footing table

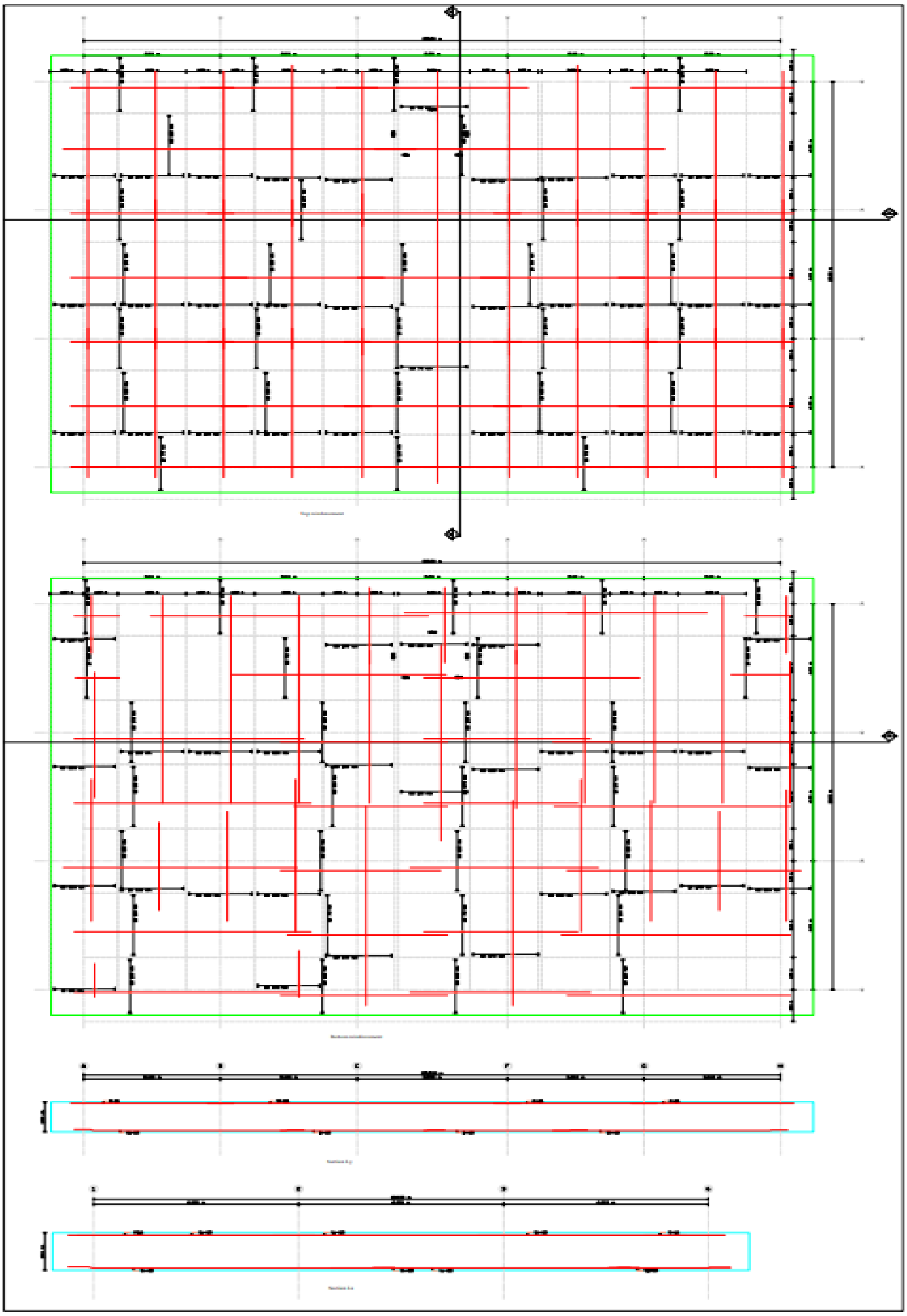
Sr. No.	Type	Nos	Lx	Ly	T	Rebars-A	Rebars-B
1	F1	1	**9.00m**	**4.00m**	0.950 m	54-16	34-16
2	F2	1	**9.00m**	**4.00m**	0.950 m	54-16	44-16
3	F3	2	**9.00m**	**4.00m**	0.950 m	52-16	46-16
4	F4	2	**8.50m**	**3.60m**	0.700 m	39-16	37-16
5	F5	2	**9.00m**	**4.00m**	0.800 m	56-16	52-16
6	F6	2	**8.50m**	**3.60m**	0.800 m	61-16	43-16



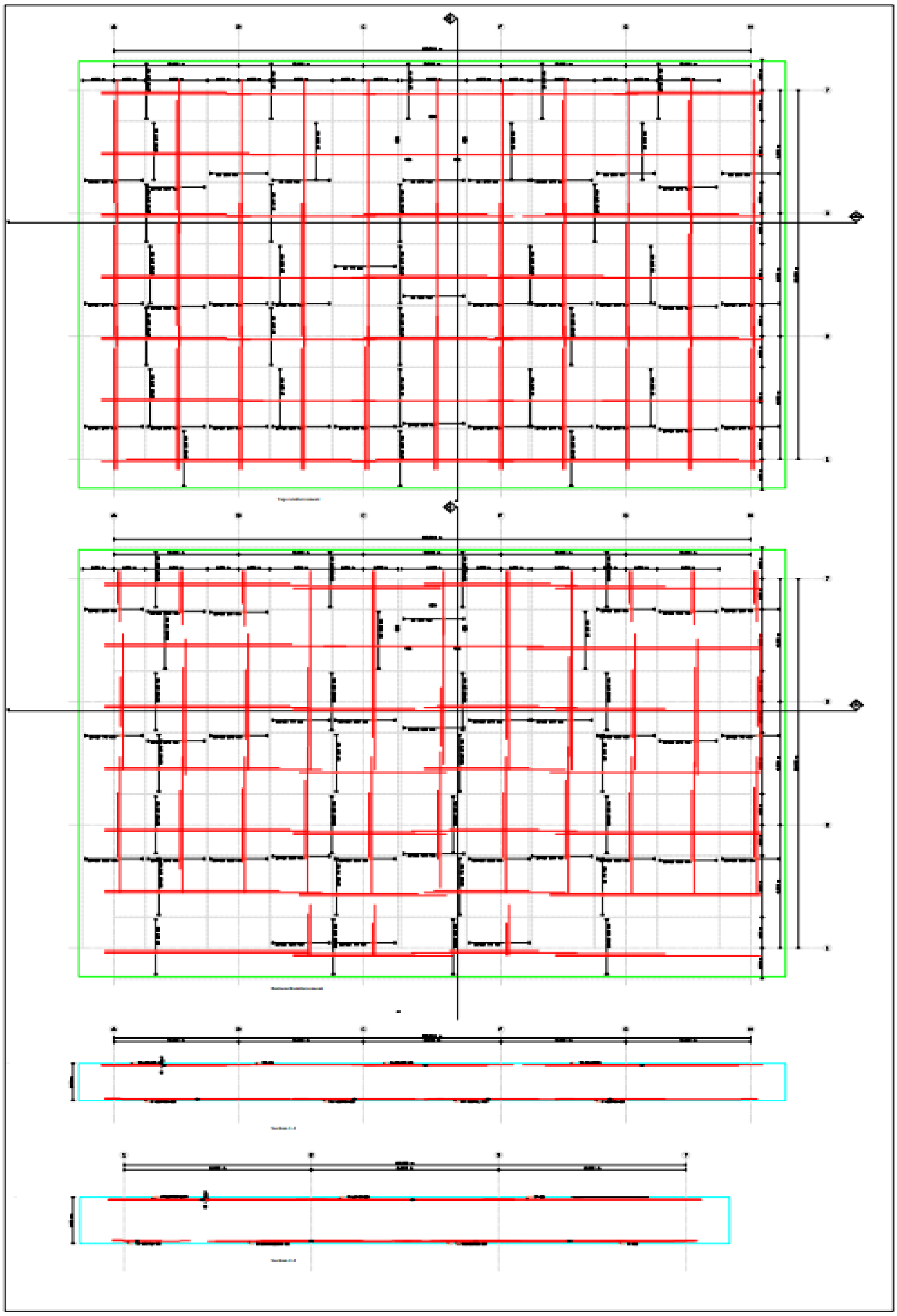
G+6 Mat foundation reinforcement



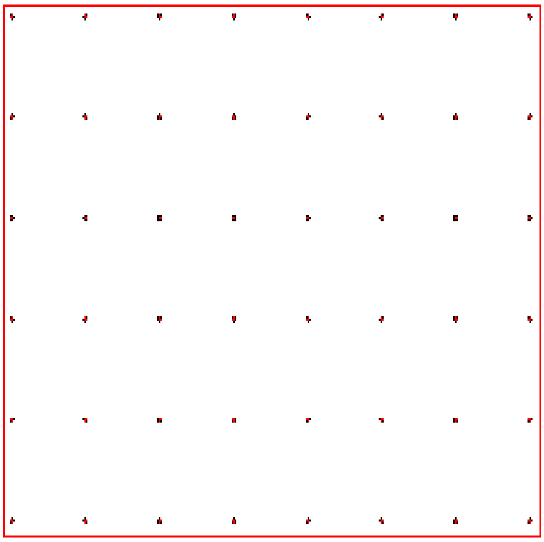
G+8 Mat foundation reinforcement



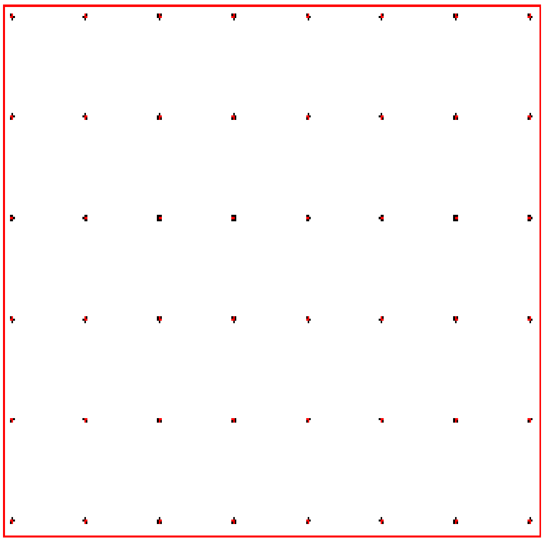
G+10 Mat foundation reinforcement



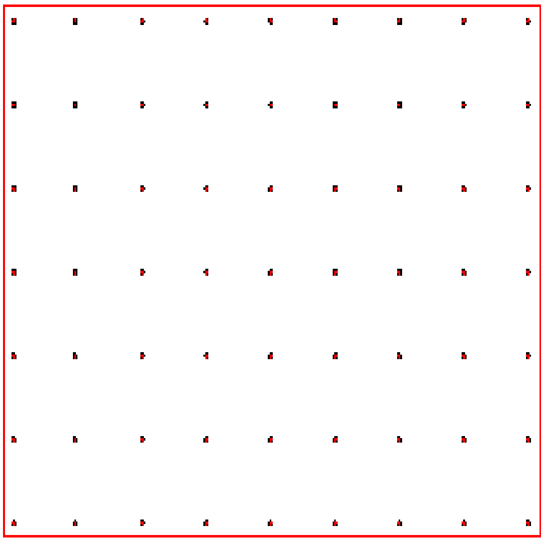
Pile foundations layout



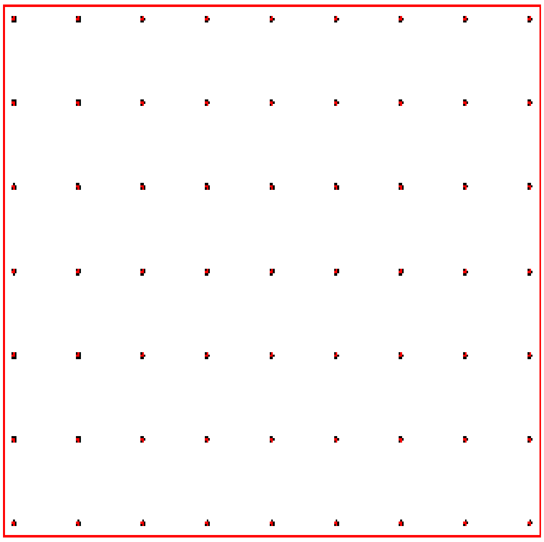
G+4 pile layout



G+6 pile layout

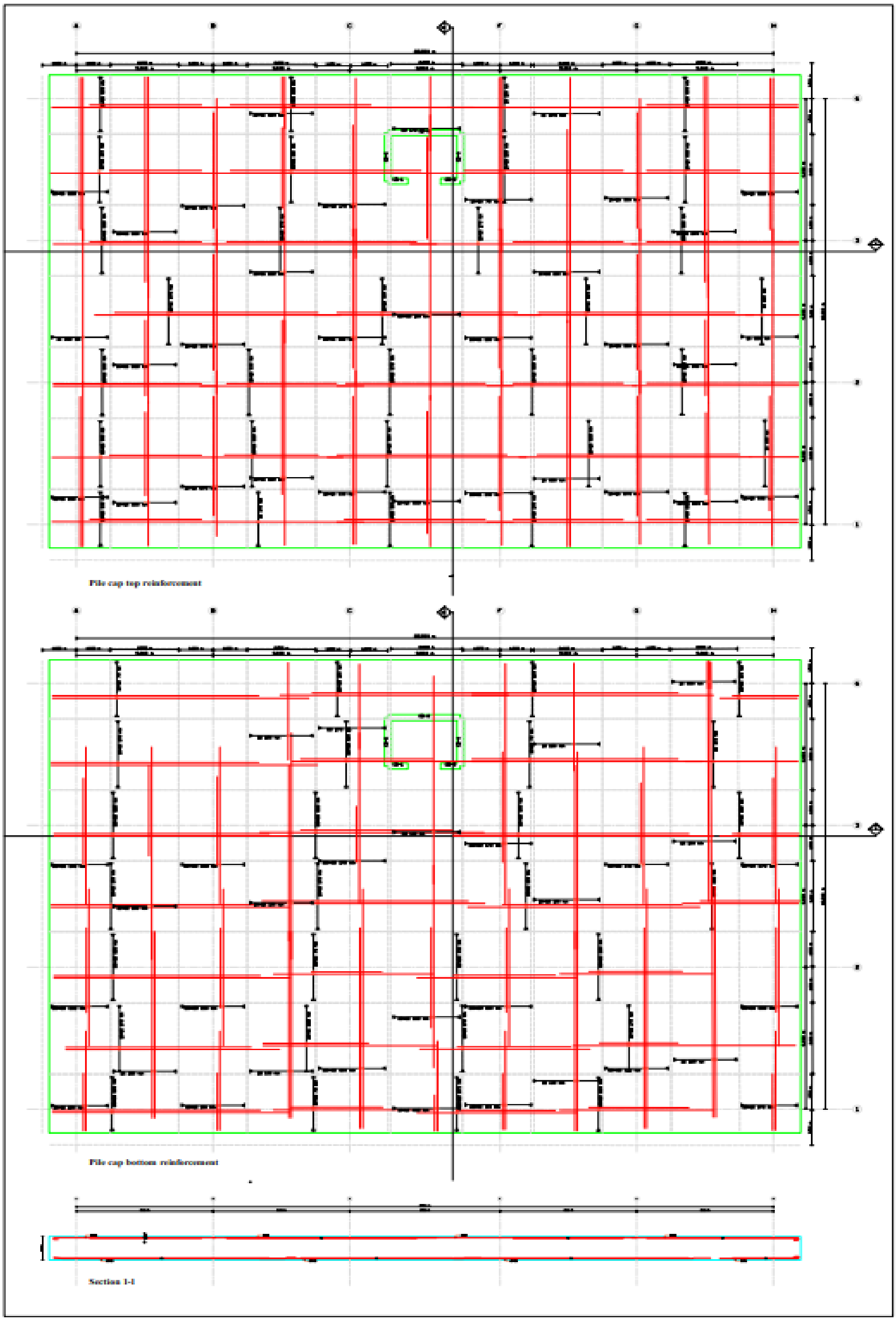


G+8 pile layout

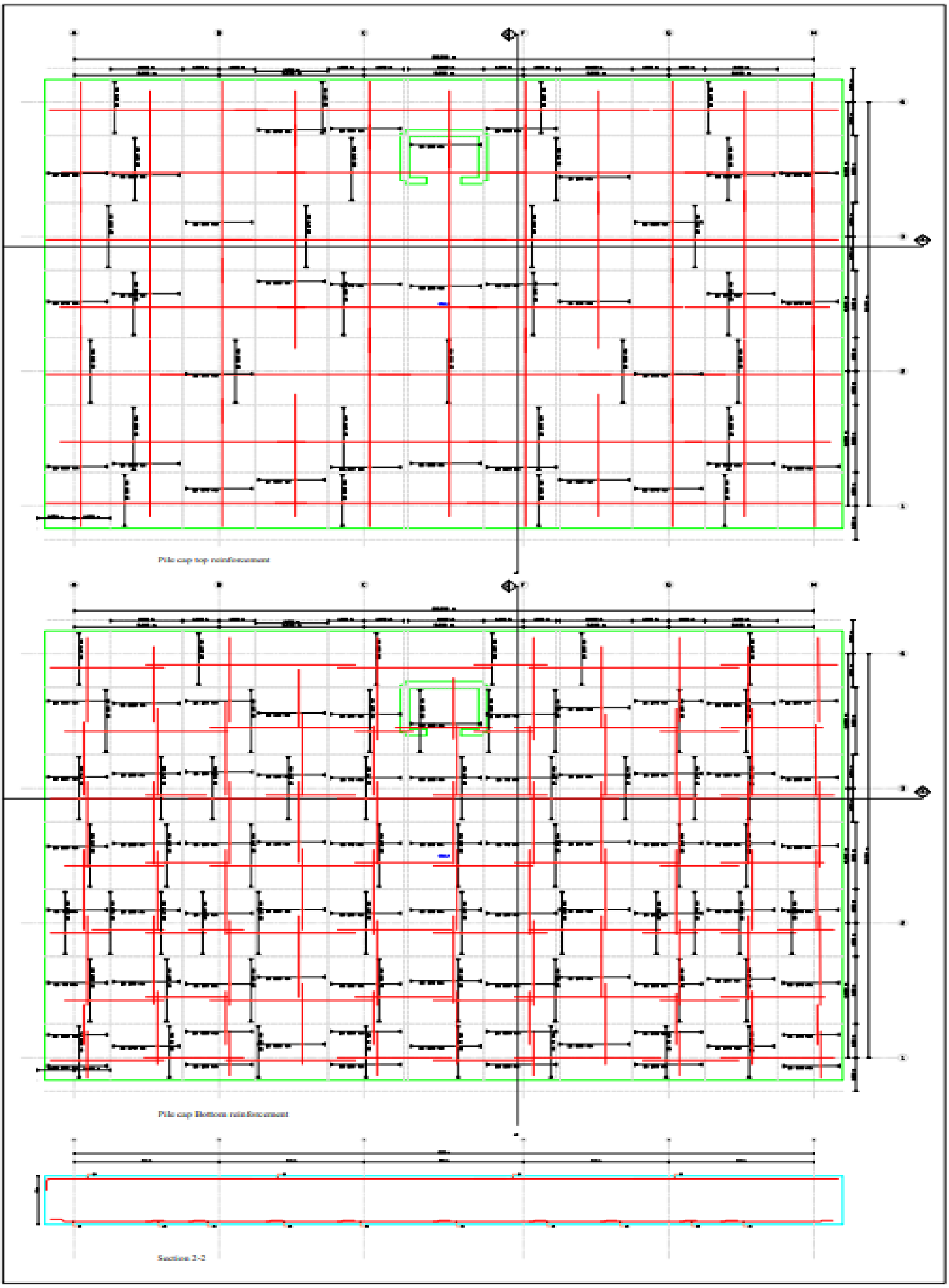


G+10 pile layout

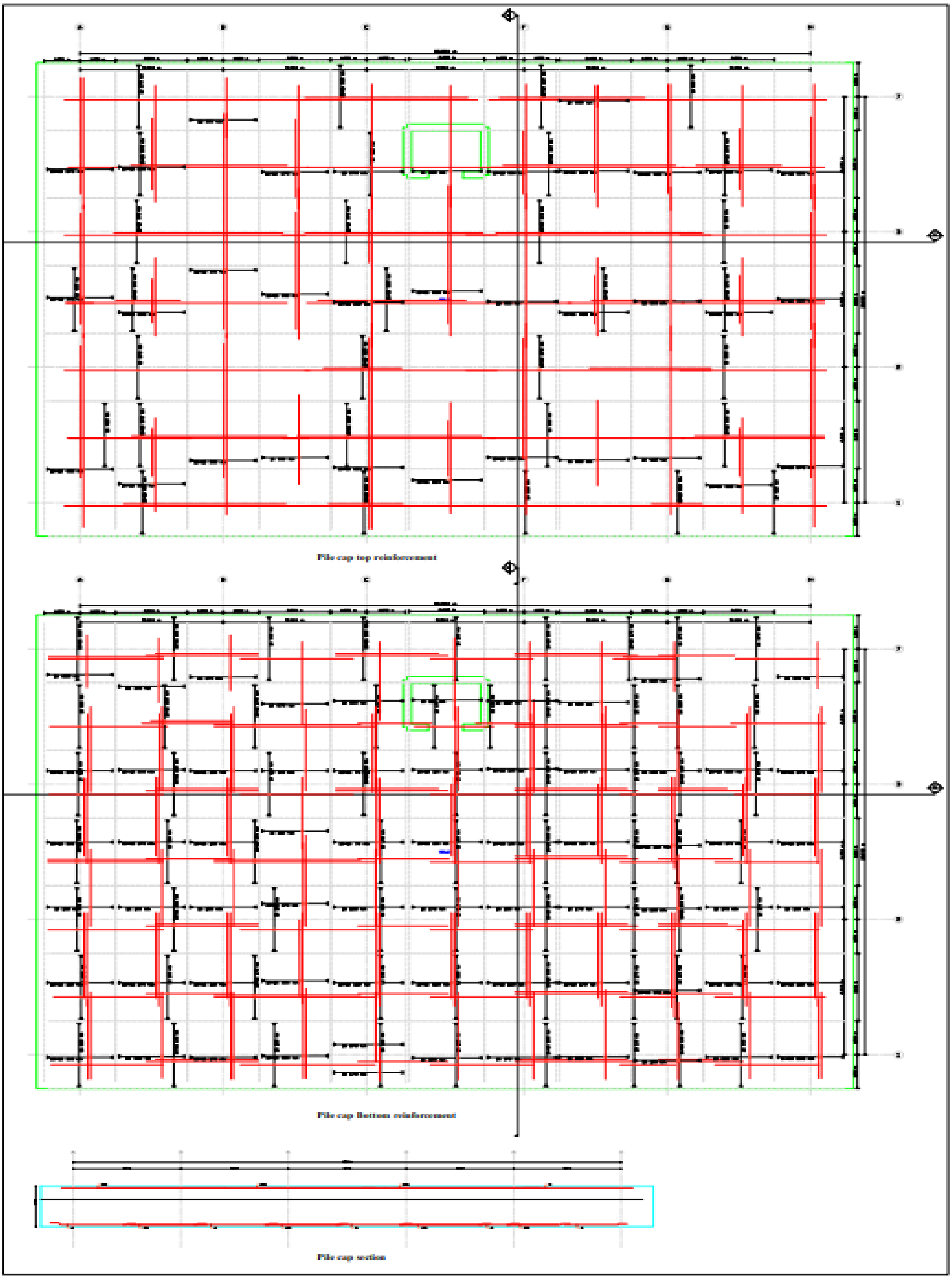
G+6 Pile cap reinforcement



G+8 Pile cap reinforcement



G+10 Pile cap reinforcement



Annex E

Bill off quantities and specification of different foundation

Cost of G+2 Isolated footing

	ISO-G+2 BILL OFF QUANTITY AND SPECIFICATION				
No	**Description**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	550.00	34.97	19231.93
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	459.00	219.17	100601.00
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	0.00	262.52	0.00
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	0.00	327.53	0.00
1.06	Excavation for pit to a depth not exceeding 1500mm starting from reduced level.	m^3	338.94	219.17	74286.93
1.07	Ditto but over 1500mm not exceeding 3000mm.	m^3	321.84	262.52	84488.52
1.08	Excavation for continuous Trench to a depth not exceeding 1500mm starting from reduced ground level.	m^3	109.44	262.52	28729.88
1.09	Back fill under mat slab, around isolated footing and masonry with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	778.24	858.77	668331.64
1.1	Back fill under hard core with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	592.67	858.77	508969.76
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	1017.38	240.26	244435.72
1.12	25cm thick basaltic or equivalent stone hardcore well rolled, consolidated and blinded with crushed stone.	m^2	375	519.35	194757.32
	Sub total				**1,923,832.69**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	84	128.86	10824.06
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	32.032	3136.54	100469.78
2.2.2	For foundation column	m^3	10.881	4216.36	45878.25
2.2.3	For footing	m^3	96.96	2891.42	280348.86
2.3	C-20 mass concrete poured on ground floor slab.	m^2	454.97	274.69	124975.60
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.1	For grade beams.	m^2	172.17	494.318	85107.65
2.4.2	For foundation column	m^2	46.70	520.224	24294.05
2.4.3	For footing	m^2	97.95	494.318	48419.77
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	0	111.852	0
2.5.2	Φ10 Reinforced bar	kg	0	112.852	0
2.5.3	Φ12 Reinforced bar	kg	0	113.008	0
2.5.4	Φ14 Reinforced bar	kg	3024	113.838	344245.5694
2.5.5	Φ16 Reinforced bar	kg	0	114.769	0
2.5.6	Φ20 Reinforced bar	kg	0	115.174	0
2.5.7	Φ24 Reinforced bar	kg	0	116.282	0
	Sub total				**1,064,563.59**
	3. MASONRY WORK				
3.1	50cm thick Masonary below natural ground level	m^3	80.08		1720.88
3.2	50cm thick Masonary above natural ground level	m^3	60.06		2029.43
	Sub total				**3750.31**
	Total				**2,992,146.58**

Cost of G+4 Isolated footing

No	ISO-G+4 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	550.00	34.97	19231.93
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	459.00	219.17	100601.00
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	0.00	262.52	0.00
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	0.00	327.53	0.00
1.06	Excavation for pit to a depth not exceeding 1500mm starting from reduced level.	m^3	536.72	219.17	117636.10
1.07	Ditto but over 1500mm not exceeding 3000mm.	m^3	694.91	262.52	182425.47
1.08	Excavation for continuous Trench to a depth not exceeding 1500mm starting from reduced ground level.	m^3	94.08	262.52	24697.61
1.09	Back fill under mat slab, around isolated footing and masonry with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	1613.25	858.77	1385415.73
1.1	Back fill under hard core with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	590.29	858.77	506926.56
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	1199.80	240.26	288264.91
1.12	25cm thick basaltic or equivalent stone hardcore well rolled, consolidated and blinded with crushed stone.	m^2	375	519.35	194757.32
	Sub total				**2,819,956.63**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	84	128.86	10824.06
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	31.6224	3136.54	99185.05
2.2.2	For foundation column	m^3	22.4322	4216.36	94582.30
2.2.3	For footing	m^3	198.50	2891.42	573951.99
2.3	C-20 mass concrete poured on ground floor slab.	m^2	452.59	274.69	124322.05
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.1	For grade beams.	m^2	169.97	494.318	84019.36
2.4.2	For foundation column	m^2	73.22	520.224	38092.90
2.4.3	For footing	m^2	200.09	494.318	98908.33
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	0	111.852	0
2.5.2	Φ10 Reinforced bar	kg	0	112.852	0
2.5.3	Φ12 Reinforced bar	kg	0	113.008	0
2.5.4	Φ14 Reinforced bar	kg	0	113.838	0
2.5.5	Φ16 Reinforced bar	kg	5645	114.769	647871.9247
2.5.6	Φ20 Reinforced bar	kg	0	115.174	0
2.5.7	Φ24 Reinforced bar	kg	0	116.282	0
	Sub total				**1,771,757.97**
	3. MASONRY WORK				
3.1	50cm thick Masonary below natural ground level	m^3	79.056		1720.88
3.2	50cm thick Masonary above natural ground level	m^3	59.292		2029.43
	Sub total				**3750.31**
	Total				**4,595,464.90**

Cost of G+6 Isolated footing

	ISO-G+6 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	550.00	34.97	19231.93
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	459.00	219.17	100601.00
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	0.00	262.52	0.00
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	0.00	327.53	0.00
1.06	Excavation for pit to a depth not exceeding 1500mm starting from reduced level.	m^3	704.61	219.17	154432.39
1.07	Ditto but over 1500mm not exceeding 3000mm.	m^3	1080.40	262.52	283624.05
1.08	Excavation for continuous Trench to a depth not exceeding 1500mm starting from reduced ground level.	m^3	101.60	262.52	26671.74
1.09	Back fill under mat slab, around isolated footing and masonry with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	1803.80	858.77	1549056.20
1.1	Back fill under hard core with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	588.16	858.77	505096.68
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	1375.21	240.26	330407.95
1.12	25cm thick basaltic or equivalent stone hardcore well rolled, consolidated and blinded with crushed stone.	m^2	375	519.35	194757.32
	Sub total				**3,163,879.26**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	84	128.86	10824.06
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	31.328	3136.54	98261.65
2.2.2	For foundation column	m^3	26.841	4216.36	113171.40
2.2.3	For footing	m^3	328.63	2891.42	950193.72
2.3	C-20 mass concrete poured on ground floor slab.	m^2	450.46	274.69	123736.74
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.1	For grade beams.	m^2	168.39	494.318	83237.15
2.4.2	For foundation column	m^2	112.72	520.224	58641.96
2.4.3	For footing	m^2	330.52	494.318	163383.09
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	7	111.852	782.9653905
2.5.2	Φ10 Reinforced bar	kg	0	112.852	0
2.5.3	Φ12 Reinforced bar	kg	41	113.008	4633.316154
2.5.4	Φ14 Reinforced bar	kg	22	113.838	2504.432052
2.5.5	Φ16 Reinforced bar	kg	837	114.769	96061.78937
2.5.6	Φ20 Reinforced bar	kg	12797	115.174	1473885.477
2.5.7	Φ24 Reinforced bar	kg	1379	116.282	160352.4132
	Sub total				**3,339,670.18**
	3. MASONRY WORK				
3.1	50cm thick Masonary below natural ground level	m^3	78.32		1720.88
3.2	50cm thick Masonary above natural ground level	m^3	58.74		2029.43
	Sub total				**3750.31**
	Total				**6,507,299.75**

Cost of G+8 Isolated footing

	ISO-G+8 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit Price**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	550.00	34.97	19231.93
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	459.00	219.17	100601.00
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	0.00	262.52	0.00
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	0.00	327.53	0.00
1.06	Excavation for pit to a depth not exceeding 1500mm starting from reduced level.	m^3	704.61	219.17	154432.39
1.07	Ditto but over 1500mm not exceeding 3000mm.	m^3	1080.40	262.52	283624.05
1.08	Excavation for continuous Trench to a depth not exceeding 1500mm starting from reduced ground level.	m^3	101.60	262.52	26671.74
1.09	Back fill under mat slab, around isolated footing and masonry with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	1803.80	858.77	1549056.20
1.1	Back fill under hard core with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	588.16	858.77	505096.68
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	1375.21	240.26	330407.95
1.12	25cm thick basaltic or equivalent stone hardcore well rolled, consolidated and blinded with crushed stone.	m^2	375	519.35	194757.32
	Sub total				**3,163,879.26**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	130	128.86	16751.52
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	38	3136.54	119188.67
2.2.2	For foundation column	m^3	27	4216.36	113841.80
2.2.3	For footing	m^3	328.63	2891.42	950193.72
2.3	C-20 mass concrete poured on ground floor slab.	m^2	450.46	274.69	123736.74
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.1	For grade beams.	m^2	168.39	494.318	83237.15
2.4.2	For foundation column	m^2	214.00	520.224	111327.98
2.4.3	For footing	m^2	330.52	494.318	163383.09
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	120	111.852	13422.26384
2.5.2	Φ10 Reinforced bar	kg	0	112.852	0
2.5.3	Φ12 Reinforced bar	kg	420	113.008	47463.23865
2.5.4	Φ14 Reinforced bar	kg	22	113.838	2504.432052
2.5.5	Φ16 Reinforced bar	kg	837	114.769	96061.78937
2.5.6	Φ20 Reinforced bar	kg	12797	115.174	1473885.477
2.5.7	Φ24 Reinforced bar	kg	1379	116.282	160352.4132
	Sub total				**3,475,350.30**
	3. MASONRY WORK				
3.1	50cm thick Masonary below natural ground level	m^3	178		1720.88
3.2	50cm thick Masonary above natural ground level	m^3	58.74		2029.43
	Sub total				**3750.31**
	Total				**8,419,135.00**

Cost of G+4 Combined footing

	C0-G+4 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	550.00	34.97	19231.93
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	459.00	219.17	100601.00
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	0.00	262.52	0.00
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	0.00	327.53	0.00
1.06	Excavation for pit to a depth not exceeding 1500mm starting from reduced level.	m^3	539.46	219.17	118235.76
1.07	Ditto but over 1500mm not exceeding 3000mm.	m^3	866.10	262.52	227365.05
1.08	Excavation for continuous Trench to a depth not exceeding 1500mm starting from reduced ground level.	m^3	97.20	262.52	25516.67
1.09	Back fill under mat slab, around isolated footing and masonry with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	1943.23	858.77	1668797.33
1.1	Back fill under hard core with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	589.99	858.77	506669.27
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	1205.66	240.26	289671.87
1.12	25cm thick basaltic or equivalent stone hardcore well rolled, consolidated and blinded with crushed stone.	m^2	375	519.35	194757.32
	Sub total				**3,150,846.19**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	84	128.86	10824.06
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	31.5776	3136.54	99044.53
2.2.2	For foundation column	m^3	20.2884	4216.36	85543.26
2.2.3	For footing	m^3	221.81	2891.42	641353.65
2.3	C-20 mass concrete poured on ground floor slab.	m^2	452.29	274.69	124239.76
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.1	For grade beams.	m^2	169.73	494.318	83900.33
2.4.2	For foundation column	m^2	85.75	520.224	44607.77
2.4.3	For footing	m^2	222.58	494.318	110024.72
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	0	111.852	0
2.5.2	Φ10 Reinforced bar	kg	0	112.852	0
2.5.3	Φ12 Reinforced bar	kg	0	113.008	0
2.5.4	Φ14 Reinforced bar	kg	311	113.838	35403.5622
2.5.5	Φ16 Reinforced bar	kg	4590	114.769	526790.4579
2.5.6	Φ20 Reinforced bar	kg	0	115.174	0
2.5.7	Φ24 Reinforced bar	kg	0	116.282	0
	Sub total				**1,761,732.10**
	3. MASONRY WORK				
3.1	50cm thick Masonary below natural ground level	m^3	78.944		1720.88
3.2	50cm thick Masonary above natural ground level	m^3	59.208		2029.43
	Sub total				**3750.31**
	Total				**4,916,328.60**

Cost of G+6 Combined footing

	C0-G+6 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	550.00	34.97	19231.93
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	459.00	219.17	100601.00
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	0.00	262.52	0.00
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	0.00	327.53	0.00
1.06	Excavation for pit to a depth not exceeding 1500mm starting from reduced level.	m^3	585.90	219.17	128414.21
1.07	Ditto but over 1500mm not exceeding 3000mm.	m^3	1915.40	262.52	502824.81
1.08	Excavation for continuous Trench to a depth not exceeding 1500mm starting from reduced ground level.	m^3	109.12	262.52	28645.87
1.09	Back fill under mat slab, around isolated footing and masonry with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	2365.43	858.77	2031368.74
1.1	Back fill under hard core with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	588.40	858.77	505302.79
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	1264.02	240.26	303693.45
1.12	25cm thick basaltic or equivalent stone hardcore well rolled, consolidated and blinded with crushed stone.	m^2	375	519.35	194757.32
	Sub total				**3,814,840.12**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	84	128.86	10824.06
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	31.36	3136.54	98362.02
2.2.2	For foundation column	m^3	29.212	4216.36	123168.40
2.2.3	For footing	m^3	266.85	2891.42	771576.09
2.3	C-20 mass concrete poured on ground floor slab.	m^2	450.70	274.69	123802.67
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.1	For grade beams.	m^2	168.56	494.318	83322.18
2.4.2	For foundation column	m^2	122.77	520.224	63868.55
2.4.3	For footing	m^2	383.84	494.318	189739.86
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	13	111.852	1454.078582
2.5.2	Φ10 Reinforced bar	kg	0	112.852	0
2.5.3	Φ12 Reinforced bar	kg	6	113.008	678.0462664
2.5.4	Φ14 Reinforced bar	kg	77.3	113.838	8799.66353
2.5.5	Φ16 Reinforced bar	kg	196	114.769	22494.75593
2.5.6	Φ20 Reinforced bar	kg	3474	115.174	400115.5073
2.5.7	Φ24 Reinforced bar	kg	0	116.282	0
	Sub total				**1,898,205.88**
	3. MASONRY WORK				
3.1	50cm thick Masonary below natural ground level	m^3	78.4		1720.88
3.2	50cm thick Masonary above natural ground level	m^3	58.8		2029.43
	Sub total				**3750.31**
	Total				**5,716,796.31**

Cost of G+8 Combined footing

	C0-G+8 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	550.00	34.97	19231.93
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	459.00	219.17	100601.00
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	0.00	262.52	0.00
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	0.00	327.53	0.00
1.06	Excavation for pit to a depth not exceeding 1500mm starting from reduced level.	m^3	800.85	219.17	175525.73
1.07	Ditto but over 1500mm not exceeding 3000mm.	m^3	2316.17	262.52	608034.33
1.08	Excavation for continuous Trench to a depth not exceeding 1500mm starting from reduced ground level.	m^3	136.80	262.52	35912.35
1.09	Back fill under mat slab, around isolated footing and masonry with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	1027.54	858.77	882422.35
1.1	Back fill under hard core with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	581.84	858.77	499669.23
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	1506.65	240.26	361987.73
1.12	25cm thick basaltic or equivalent stone hardcore well rolled, consolidated and blinded with crushed stone.	m^2	375	519.35	194757.32
	Sub total				**2,878,141.96**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	84	128.86	10824.06
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	30.592	3136.54	95953.15
2.2.2	For foundation column	m^3	53.563	4216.36	225841.06
2.2.3	For footing	m^3	485.98	2891.42	1405167.72
2.3	C-20 mass concrete poured on ground floor slab.	m^2	444.14	274.69	122000.71
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.1	For grade beams.	m^2	164.43	494.318	81281.63
2.4.2	For foundation column	m^2	222.36	520.224	115677.89
2.4.3	For footing	m^2	487.49	494.318	240976.38
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	12	111.852	1342.226384
2.5.2	Φ10 Reinforced bar	kg	0	112.852	0
2.5.3	Φ12 Reinforced bar	kg	45	113.008	5085.346998
2.5.4	Φ14 Reinforced bar	kg	0	113.838	0
2.5.5	Φ16 Reinforced bar	kg	12679	114.769	1455158.217
2.5.6	Φ20 Reinforced bar	kg	85	115.174	9789.815233
2.5.7	Φ24 Reinforced bar	kg	6338	116.282	736993.1797
	Sub total				**4,506,091.39**
	3. MASONRY WORK				
3.1	50cm thick Masonary below natural ground level	m^3	76.48		1720.88
3.2	50cm thick Masonary above natural ground level	m^3	57.36		2029.43
	Sub total				**3750.31**
	Total				**7,387,983.66**

Cost of G+4 Mat foundation

No	DESCRIPTION	Unit	QTY	Unit Price	Amount
	Ma-G+4 BILL OFF QUANTITY AND SPECIFICATION				
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	649.00	34.97	22693.68
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	524.18	219.17	114886.23
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	55.00	262.52	14438.44
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	55.00	327.53	18014.23
1.09	Back fill under mat slab with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	205.98	858.77	176886.03
1.1	Back fill around masonry foundation with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	55.20	858.77	47404.34
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	502.80	240.26	120803.33
	Sub total				**515,126.28**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	514.9375	128.86	66353.75
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	140	3136.543929	439116.15
2.2.2	For foundation column	m^3	2.49	4216.363125	10498.74
2.2.3	For footing	m^3	356.00	2891.4225	1029346.41
2.3	C-20 mass concrete poured on ground floor slab.	m^2	405.00	274.68975	111249.35
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.2	For foundation column	m^2	10.46	520.2242093	5443.63
2.4.3	For footing	m^2	323.96	494.3176159	160140.86
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	180	111.8521986	20133.39576
2.5.2	Φ10 Reinforced bar	kg	76.96	112.8524003	8685.120724
2.5.3	Φ12 Reinforced bar	kg	340.45	113.0077111	38473.47523
2.5.4	Φ14 Reinforced bar	kg	689.62	113.8378206	78504.83782
2.5.5	Φ16 Reinforced bar	kg	870	114.7691629	99849.17175
2.5.6	Φ20 Reinforced bar	kg	1100	115.1742969	126691.7265
2.5.7	Φ24 Reinforced bar	kg	4100	116.2816629	476754.818
	Sub total				**2,671,241.44**
	Total				**3,152,446**

Cost of G+6 Mat foundation

	Ma-G+6 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary	m^2	649.00	34.97	22693.68
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	524.18	219.17	114886.23
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	55.00	262.52	14438.44
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	55.00	327.53	18014.23
1.09	Back fill under mat slab with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	205.98	858.77	176886.03
1.1	Back fill around masonry foundation with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	55.20	858.77	47404.34
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	502.80	240.26	120803.33
	Sub total				**515,126.28**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	514.9375	128.86	66353.75
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	170.505	3136.54	534796.42
2.2.2	For foundation column	m^3	2.49	4216.36	10498.74
2.2.3	For footing	m^3	463.44	2891.42	1340011.69
2.3	C-20 mass concrete poured on ground floor slab.	m^2	506.64	274.69	139168.13
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.2	For foundation column	m^2	10.46	520.224	5443.63
2.4.3	For footing	m^2	293.46	494.318	145061.71
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	0	111.852	0
2.5.2	Φ10 Reinforced bar	kg	26.96	112.852	3042.500711
2.5.3	Φ12 Reinforced bar	kg	340.45	113.008	38473.47523
2.5.4	Φ14 Reinforced bar	kg	689.62	113.838	78504.83782
2.5.5	Φ16 Reinforced bar	kg	1087.73	114.769	124837.8616
2.5.6	Φ20 Reinforced bar	kg	19841.34	115.174	2285212.383
2.5.7	Φ24 Reinforced bar	kg	6100	116.282	709318.1439
	Sub total				**5,480,723.27**
	Total				**5,995,849.55**

Cost of G+8 Mat foundation

	Ma-G+8 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	649.00	34.97	22693.68
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	530.92	219.17	116362.92
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	55.00	262.52	14438.44
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	55.00	327.53	18014.23
1.09	Back fill under mat slab with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	208.65	858.77	179183.25
1.1	Back fill around masonry foundation with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	55.50	858.77	47661.97
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	506.57	240.26	121707.31
	Sub total				**520,061.80**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	521.625	128.86	67215.49
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	298.368	3136.54	935844.34
2.2.2	For foundation column	m^3	0	4216.36	0.00
2.2.3	For footing	m^3	625.95	2891.42	1809885.91
2.3	C-20 mass concrete poured on ground floor slab.	m^2	506.77	274.69	139203.15
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.2	For foundation column	m^2	0.00	520.224	0.00
2.4.3	For footing	m^2	328.28	494.318	162276.56
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	27.18	111.852	3040.142759
2.5.2	Φ10 Reinforced bar	kg	13.27	112.852	1497.551351
2.5.3	Φ12 Reinforced bar	kg	37.99	113.008	4293.162943
2.5.4	Φ14 Reinforced bar	kg	143.16	113.838	16297.02239
2.5.5	Φ16 Reinforced bar	kg	511.72	114.769	58729.67606
2.5.6	Φ20 Reinforced bar	kg	32207.99	115.174	3709532.602
2.5.7	Φ24 Reinforced bar	kg	12137.56	116.282	1411375.661
	Sub total				**8,319,191.27**
	Total				**8,839,253.07**

Cost of G+10 Mat foundation

	Ma-G+10 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit Price**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	649.00	34.97	22693.68
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	796.37	219.17	174544.37
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	110.00	262.52	28876.89
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	110.00	327.53	36028.46
1.09	Back fill under mat slab with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	312.98	858.77	268774.88
1.1	Back fill around masonry foundation with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	55.50	858.77	47661.97
1.11	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	777.70	240.26	186849.60
	Sub total				**765,429.85**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	521.625	128.86	67215.49
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.1	For grade beams.	m^3	663.552	3136.543929	2081260.00
2.2.2	For foundation column	m^3	1.886	4216.363125	7952.06
2.2.3	For footing	m^3	938.93	2891.4225	2714828.87
2.3	C-20 mass concrete poured on ground floor slab.	m^2	502.77	274.68975	138104.39
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.2	For foundation column	m^2	7.80	520.2242093	4056.92
2.4.3	For footing	m^2	148.10	494.3176159	73206.46
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	10.34	111.8521986	1156.551734
2.5.2	Φ10 Reinforced bar	kg	91.75	112.8524003	10354.20772
2.5.3	Φ12 Reinforced bar	kg	70.15	113.0077111	7927.490931
2.5.4	Φ14 Reinforced bar	kg	322.52	113.8378206	36714.97389
2.5.5	Φ16 Reinforced bar	kg	1219.6	114.7691629	139972.4711
2.5.6	Φ20 Reinforced bar	kg	2096.22	115.1742969	241430.6646
2.5.7	Φ24 Reinforced bar	kg	28530.86	116.2816629	3317615.846
	Sub total				**8,841,796.39**
	Total				**9,607,226.24**

Cost of G+6 Pile foundation

	PI-G+6 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	649.00	34.97	22693.68
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	796.37	219.17	174544.37
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	55.00	262.52	14438.44
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	55.00	327.53	18014.23
1.05	Excavation for pile pit in ordinary soil to a depth not exceeding 15m from reduced level.	m^3	296.88	427.00	126767.98
1.06	Back fill under mat slab with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	312.98	858.77	268774.88
1.07	Back fill around masonry foundation with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	55.50	858.77	47661.97
1.08	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	667.70	240.26	160421.00
	Sub total				**833,316.55**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	521.625	128.86	67215.49
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.2	For foundation column	m^3	13.202	4216.36	55664.43
2.2.3	For piles foundation	m^3	296.88	4216.36	1251756.02
2.2.4	For pile cap	m^3	521.63	2891.42	1508238.26
2.3	C-20 mass concrete poured on ground floor slab.	m^2	502.77	274.69	138104.39
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.2	For foundation column	m^2	54.59	520.224	28398.42
2.4.3	For pile cap	m^2	148.10	494.318	73206.46
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	18	111.852	2013.34
2.5.2	Φ10 Reinforced bar	kg	241.63	112.852	27268.53
2.5.3	Φ12 Reinforced bar	kg	69.3	113.008	7831.43
2.5.4	Φ14 Reinforced bar	kg	525	113.838	59764.86
2.5.5	Φ16 Reinforced bar	kg	16881.9	114.769	1937521.53
2.5.6	Φ20 Reinforced bar	kg	28	115.174	3224.88
2.5.7	Φ24 Reinforced bar	kg	4030	116.282	468615.10
	Sub total				**5628823.13**
	Total				**6,465,889.98**

Cost of G+8 Pile foundation

	PI-G+8 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	649.00	34.97	22693.68
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	796.37	219.17	174544.37
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	110.00	262.52	28876.89
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	110.00	327.53	36028.46
1.05	Excavation for pile pit in ordinary soil to a depth not exceeding 15m from reduced level.	m^3	389.66	427.00	166382.97
1.06	Back fill under mat slab with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	312.98	858.77	268774.88
1.07	Back fill around masonry foundation with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	55.50	858.77	47661.97
1.08	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	777.70	240.26	186849.60
	Sub total				**931,812.82**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	521.625	128.86	67215.49
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.2	For foundation column	m^3	1.886	4216.36	7952.06
2.2.3	For piles foundation	m^3	389.66	4216.36	1642929.77
2.2.4	For pile cap	m^3	938.93	2891.42	2714828.87
2.3	C-20 mass concrete poured on ground floor slab.	m^2	502.77	274.69	138104.39
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.2	For foundation column	m^2	7.80	520.224	4056.92
2.4.3	For pile cap	m^2	148.10	494.318	73206.46
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	30	111.852	3355.57
2.5.2	Φ10 Reinforced bar	kg	166	112.852	18733.50
2.5.3	Φ12 Reinforced bar	kg	119	113.008	13447.92
2.5.4	Φ14 Reinforced bar	kg	551	113.838	62724.64
2.5.5	Φ16 Reinforced bar	kg	6999	114.769	803269.37
2.5.6	Φ20 Reinforced bar	kg	18783.2	115.174	2163341.85
2.5.7	Φ24 Reinforced bar	kg	0	116.282	0.00
	Sub total				**7713166.81**
	Total				**8,648,729.93**

Cost of G+10 Pile foundation

	PI-G+10 BILL OFF QUANTITY AND SPECIFICATION				
No	**DESCRIPTION**	**Unit**	**QTY**	**Unit**	**Amount**
	1. EXCAVATION & EARTH WORK				
1.01	Site clearing to remove the top soil to an average depth of 20cm for ordinary soil.	m^2	649.00	34.97	22693.68
1.02	Bulk excavation in ordinary soil to a depth not exceeding 1500mm from reduced level.	m^3	796.37	219.17	174544.37
1.03	Ditto but over 1500mm not exceeding 3000mm.	m^3	220.00	262.52	57753.77
1.04	Ditto but over 3000mm not exceeding 4500mm.	m^3	220.00	327.53	72056.91
1.05	Excavation for pile pit in ordinary soil to a depth not exceeding 15m from reduced level.	m^3	692.72	427.00	295791.94
1.06	Back fill under mat slab with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	312.98	858.77	268774.88
1.07	Back fill around masonry foundation with non-expansive selected material from quarry and compact in layers not exceeding 200mm thick by sprinkling water to insure the required dry density .	m^3	55.50	858.77	47661.97
1.08	Cart away surplus excavated material and Dispose it at a distance not less than 5km from site as per the Engineers Direction.	m^3	997.70	240.26	239706.80
	Sub total				**1,178,984.34**
	2. CONCRETE WORK				
2.1	c-5 lean concrete under grade beam	m^2	521.625	128.86	67215.49
2.2	Reinforced concrete class C-30 with 325kg of concrete filled in to formworkand vibrated around reinforcement bars				
2.2.2	For foundation column	m^3	1.886	4216.36	7952.06
2.2.3	For piles foundation	m^3	692.72	4216.36	2920764.04
2.2.4	For pile cap	m^3	1147.58	2891.42	3318124.18
2.3	C-20 mass concrete poured on ground floor slab.	m^2	502.77	274.69	138104.39
2.4	Provide, cut and fix in position timber, plywood, Acrow spans, Acrow pipe (or) Doka type formwork. For fairfaced concrete and exposed concrete. The form work shall consist of shores, bracings, sides of beams and columns, bottom of slabs etc, including ties, anchors, hangers, inserts etc.				
2.4.2	For foundation column	m^2	7.80	520.224	4056.92
2.4.3	For pile cap	m^2	148.10	494.318	73206.46
2.5	C1 The following are held to be included: (a) tying wire (b) cutting and bending (c) provision of test certificates and reinforcement for testing				
2.5.1	Φ8 Reinforced bar	kg	53	111.852	5928.17
2.5.2	Φ10 Reinforced bar	kg	309.2	112.852	34893.96
2.5.3	Φ12 Reinforced bar	kg	313	113.008	35371.41
2.5.4	Φ14 Reinforced bar	kg	1404.2	113.838	159851.07
2.5.5	Φ16 Reinforced bar	kg	6085.4	114.769	698416.26
2.5.6	Φ20 Reinforced bar	kg	2608	115.174	300374.57
2.5.7	Φ24 Reinforced bar	kg	5918	116.282	688154.88
	Sub total				**8452413.86**
	Total				**9,635,148.49**