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Soil-Structure Interaction of a Predominant Raft Foundation with Different Allowable Bearing Capacities in Nigeria

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Abstract

This research investigates Nigeria's predominant raft foundation (beam and slab). This kind of raft foundation is commonly used in Nigeria due to cost and Nigerian terrain. Nigeria generally has a flatter terrain; hence, the raft foundation beam is assumed to be used primarily in raising the ground floor above the road level. The research reviewed that the beam's primary function is not limited to adjusting building level, however, the beam carried a substantial quantity of loads. The raft beam is usually found at the deeper strata whilst the raft slab is located at the topmost strata. Since the bearing capacity increases with depth because of overburden then one may advocate there is a difference between the bearing capacity of the raft beam and raft slab because both the raft beam and slab are founded at different strata. Hence, this research investigates which bearing capacity should govern the design of the foundation, and the loading carrying capacity (load sharing) of the raft beam and raft slab.

Given the above, a Plaxis model was used to simulate a typical real-life constructed project to understand the soil-structure interaction between the beam and slab because of the varying founding depth. A consolidation laboratory test was performed on the sample of the filling material. From the consolidated laboratory parameters and soil investigation report, other parameters required to use certain models in the Plaxis software were determined. Two models from Plaxis were used in this project work, modified cam clay was used for the filling material whilst the hardening soil was used beyond the filling material.

The overall study shows that the maximum stress is located at the tip and along the raft beam. The beam which is often considered as a stiffener because of the limited contact surface (in this case, it is just 250mm) in Nigeria is subjected to a significant amount of stress/pressure from the soil. In this work, the beam carried about 20% less than the raft slab. When the beam stiffness was reduced in Plaxis, all the foundation stresses are now transferred to the slab, implying that the beam is not redundant but has a load-carrying capacity and should not be completely ignored.

Sammendrag

Dette forskningsarbeidet undersøker det dominerende flåtefundamentet (bjelke og plate) i Nigeria. Dette flåtefundamentet brukes ofte i Nigeria på grunn av kostnader sammenlignet med andre typer, og Nigeria-terrenget. Nigeria ser generelt ut til å ha et flatere terreng, og derfor antas bjelken i flåtefundamentet primært å brukes til å heve første etasje over vegnivå. Forskningen vurderte at strålens primære funksjon ikke er begrenset ved å justere bygningen, men strålen bar en betydelig mengde belastninger. Bjelken til flåten finnes vanligvis i de dypere strataene mens platen er plassert i de øverste strataene, forskjellen i grunnlag og andre faktorer resulterer i forskjellen i bæreevnen til både bjelken og platen på grunn av kumulativ effekt av overdekning derfor er det viktig å undersøke hvilken bæreevne som styrer designet.

For å forstå jord-struktur-samspillet mellom bjelken og platen på grunn av den varierende fundamentdybden, ble en Plaxis-modell brukt for å simulere en nærliggende situasjon av et typisk real-life konstruert prosjekt. En konsolideringstest ble utført på prøven av fyllmaterialet. Fra de konsoliderte laboratorieparametrene og jordundersøkelsesrapporten ble andre parametere som kreves for å kunne bruke en bestemt modell i Plaxis-programvaren bestemt. To modeller ble brukt i dette prosjektarbeidet, modifisert kamleiremodell ble brukt for fyllmaterialet, mens herdende jordmodell ble brukt utover fyllmaterialet. Kapittel tre diskuterte utførlig parameterevalueringstrinnene, og det var også en laboratoriekurvetilpasningsseksjon av Plaxis.

Den overordnede studien viser at maksimal spenning er plassert i spissen og langs bjelken. Bjelken som ofte regnes som avstivere på grunn av den begrensede kontaktflaten (i dette tilfellet er den bare 250 mm) i Nigeria er utsatt for betydelige påkjenninger/trykk fra jorda. I dette arbeidet bar

bjelken ca 20 % mindre av det flåteplaten bar. Når bjelkestivheten ble redusert i Plaxis, overføres nå alle fundamentspenningene til platen som derfor innebærer at bjelken ikke er overflødig, men har bæreevne og ikke bør ignoreres fullstendig.

Preface

This dissertation is a self-effort research work where the laboratory data were obtained from the laboratory technician (Salami and Samuel) from the University of Lagos, Nigeria. The architectural drawings were drawn by FEA architects (lead architect: Bankole Andrew Bosun), the structural engineer is MIBBS Consults (a company founded by my humble self), and the soil investigation report was conducted by Apex Geoservices & Geotechnics (Lead engineer: Taiwo Tifase).

The consolidation laboratory test as presented in Table 4.0 was carried out primarily by Salami and Samuel, figure 1.1.3 was extracted from Apex Geoservices & Geotechnics report, and the floor plans and sections through the floor plans used in sections 1.1 and 3.1 were from the architectural Revit drawings obtained from FEA whilst the foundations loads presented in figure 3.1.9 is from MIBBS consults.

The correlation graph between standard penetration number and wave velocity was primarily obtained from the research work of Muge et. al (2011) in Figure 3.3.2.1.1 and the empirical relationship/equation derived by Anbazhagen et. al 2012 was used to determine the shear modulus in section 3.3.2.2 of this work.

This dissertation was entirely written by me whilst the analysis & the interpretation of the Plaxis Model, the interpretation of the laboratory tests, and the soil investigation report were made comprehensible with the tutelage of my supervisor-Professor Gudmund Reidar Eiksund.

Acknowledgments

I would like to start by giving thanks and praises to Almighty Allah for granting me the wisdom, knowledge, strength, and perseverance to complete this master's thesis even when there are no required data, by extension praises are extended to his beloved messenger Seyidina Muhammad Ibn Abdullah (S.A.W).

Also, I would like to express my gratitude to my family for their firm support, encouragement, love, endurance, and prayers, especially from my mum throughout my academic journey.

In addition, I would like to thank my supervisor, Professor Gudmund Reider for his unrelenting tutelage, feedback, and mentorship. Prof.'s expertise and invaluable insights have been beneficial in shaping the content of this thesis.

Similarly, I would like to acknowledge Engr. Chinedu Oko-Director at Arup Nigeria for his support and contributions to the success of this research work. I honestly thank you for providing me with a platform in the form of reference materials to explore this topic extensively and for sharing your wealth of experience with me.

Finally, I would like to appreciate all those who have contributed towards the successful completion of the research project ranging from friends Architect Bosun Andre, Engr. Adeleke Adeyemi, Mr. Salami, and Samuel of the University of Lagos laboratory, I said Thank you and God bless you all!

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List of Abbreviations or Symbols

Q = Soil Pressure

V = Column Loads

My = Moment in y direction

Mx = Moment in x direction

x = Eccentricity in x direction

y = Eccentricity in the y direction

I.y= Second moment of area

A = Area of the raft/mat

P = Total loads

B = Length of the footing

K_s = Modulus of subgrade reaction

q_a = Allowable bearing capacity (in Kip/square foot)

Nc & Nq = Bearing Capacity

Sc / Sq = Shape factor for soil

SSI = Soil structure interaction

FEM = Finite element method

Gui = Graphical user interface

E_{50} = Average stiffness

σy = Vertical stress

σx = Horizontal stress

ε_v = Volumetric strain

ε_y = Vertical strain (axial strain

σx’ = Effective confining stress

Pref = Atmospheric pressure

E_{50}^{ref} = Average stiffness reference

Eoed = Oedometer stiffness

υ = Poison ratio

τ = Shear stress

Cu = Undrained shear strength

E_u^{50} = Average undrained stiffness

σc' = Effective preconsolidated pressure

OCR = Over consolidated ratio

NC = Normally consolidated clay

OC = Over consolidated clay

m = Power index

Cc = Compression index

e_0 = Initial void ratio

e_1 = Final void ratio

DH = Change in height

Log DP = Logarithm of change in stress

h_f = Final height

Cr = Recompression index

λ = Slope of normal compression line in υ: ln p’ plane

κ = Slope for unloading & reloading line in υ: ln p’ plane

λ^* = Normal compression index

κ^* = unloading index

Mcsl = Moving critical state line

Vs = Shear wave velocity

SPTn = Standard penetration test number

ɣb = Soil bulk density

Mc = Moisture content

φu = undrained friction angle

Gref = Shear modulus

Gmax = Maximum shear modulus

E_{ur} = Unloading & reloading stiffness

Ws = Width of strip

BC = Bearing capacity

UB = Universal Beam

UC = Universal Column

M = Oedometer Modulus

ULS = Ultimate Limit State

SLS = Serviceability Limit State

BIM = Building Information Modelling

CHAPTER ONE

1.1 Introduction

Generally, sub-structural elements are founded on soil strata and sometimes deeper into the rock top surface. Foundations are sub-structural elements that help in transferring super-structural loads deep into soil strata with acceptable settlement limits. During this transferring process, there are stresses/strains interaction between soil and the foundation structural elements according to many scholars. For both structural and geotechnical engineers, the stress-strain response of the soil mass under the loaded foundation is of paramount importance in ensuring the safety of the buildings. Geotechnical response on the substructural element could be raft slab bulging, pile snapping, and differential settlement amongst others.

The focus of this project work is to examine the soil-structure interactions of a predominant foundation type in Nigeria. According to global classification, foundations are either deep or shallow. The choice of foundation depends on the magnitude of imposed loads, the nature and behavior of soil, tolerable settlements, buildability, and cost.

Deep foundations include pile, diaphragm, and pier foundations whilst shallow foundations are strip footings, strap footings, isolated footings, and raft foundations. Raft foundation is quite common in Nigeria and is adopted in a situation where there is a need to spread the load over a larger area, a situation where the foundation bases of an isolated footing clash with each other, and when pile foundation can be justifiably (justifiable because the choice of raft over pile has to be proved by designs) avoided due to cost. Raft foundations are of the following; Slab raft, slab and beam raft, and Cellular raft.

Based on the types of raft foundations listed above, beam and slab raft foundations are predominant in Nigeria due to cost/value engineering however, the slab and beam raft type has its challenges that seem not to have been investigated technically and numerically (the used of finite element program that has the capacities to simulate soil structure interaction). It is therefore essential to scientifically justified and make public the findings of the challenges associated with the predominate raft foundation in Nigeria. Hence, the focus of this research work is to simulate the foundation using FEM where the soil stress-strain behaviour is included and the load distribution on the soil mass under the loaded raft foundation (beam and slab) is calculated.

During the research work, a finite element software that has the capabilities to simulate the soil response/behaviour in fairly all states (initial, loaded, or consolidated, etc.) was adopted. Plaxis software has the capability to simulate the stress /strain behaviour of soil as long as the input parameters that were inputted represent the soil's true behaviour in both rest and disturbed states. However, the input parameters are considered very important and should be determined with utmost attention because wrong parameters tend give wrong output. On this note, this thesis therefore devoted close attention to all input parameters used for the simulations processes.

In view of determining the right input to use for the purpose of simulation, soil exploration becomes a priority to obtain the right input data. Due to spatial variability of soil properties (that is, the tendency of soil parameters to change with location) and the ambiguity of determining the soil parameters, this work considered soil data from the in-situ test (an example Standard Penetration Test) and Laboratory test result (an example is Oedometer). Plaxis software was also used to curve-fit the laboratory result and comparisons between the two curves were discussed.

1.1 Justification for Study

The focus of the research is to investigate the popular raft foundation used in Nigeria. The justification for investigating this predominant raft foundation is enshrouded in one of its (raft foundation) advantages. The major advantage of the raft foundation that necessitated this investigation is that the beam component of the raft foundation is used for raising the ground floor above the founding depth specified in the geotechnical soil report. To avoid constructing the ground floor deeper into the ground (that is creating an undesirable basement) because of the soil report recommendation, the beam component of the beam and slab raft foundation is placed at the recommended founding depth (as specified in the soil report) whilst the slab is usually placed at the soil makeup level on top of the raft beam as shown in Figure 1.1.1 and 1.1.2 below.

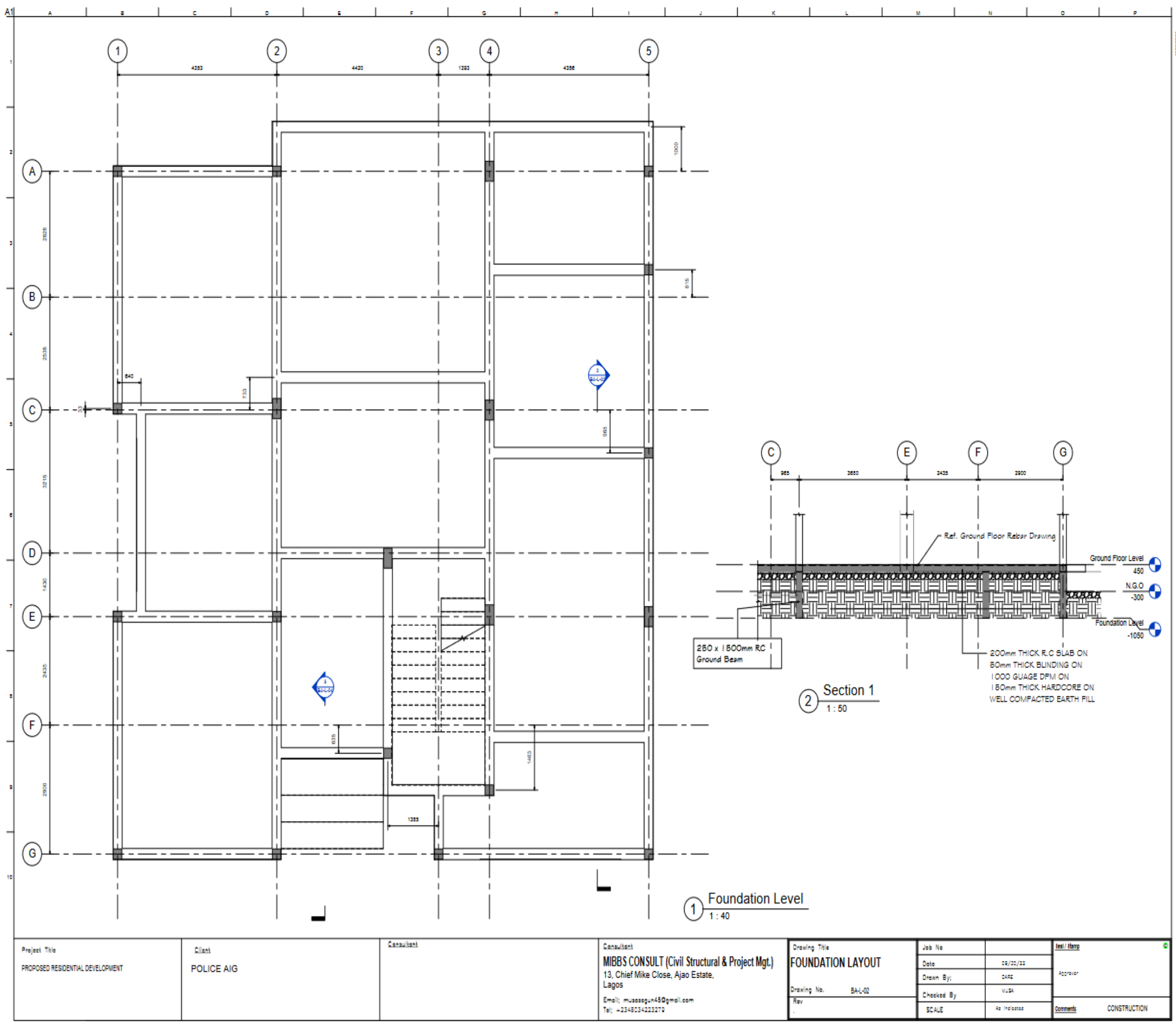

FIGURE 1.1.1: STRUCTURAL GROUND FLOOR/PLAN AND SECTION OF BEAM AND SLAB FOUNDATION IN NIGERIA

A closer look at the enlarged section 1 from Figure 1.1.2 below shows that the beam bottom tip (also referred to as beam soffit) is at negative 1050mm (-1050mm) whilst the natural ground level (NGL) is at the road level (site level). The ground floor level is at positive 450mm (+450mm) above the road level to prevent ingress of water from the road into the proposed development.

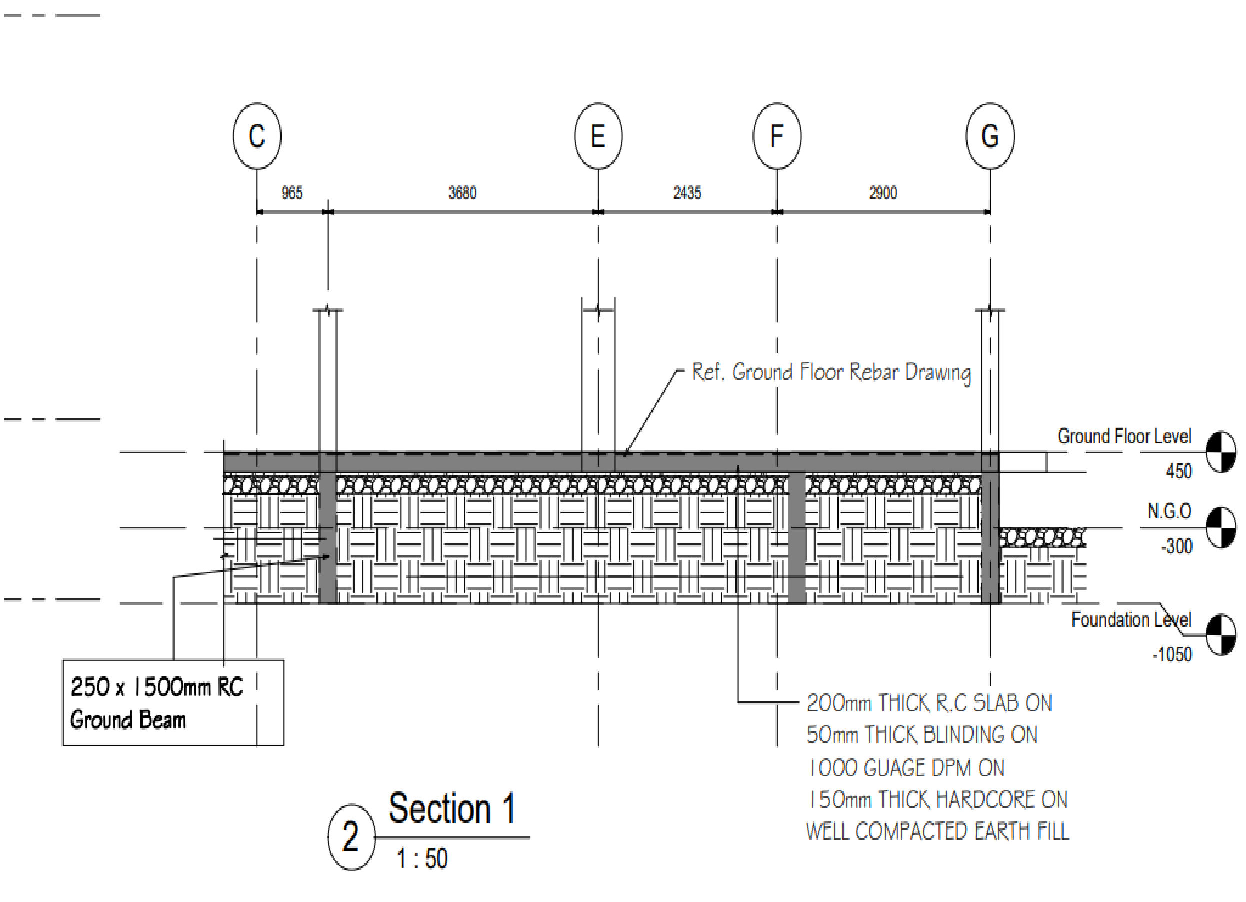

FIGURE 1.1.2: AN ENLARGED SECTION THROUGH THE GROUND FLOOR

Meanwhile, after soil exploration, the allowing bearing capacities for different depths into the ground are usually expressed by the geotechnician in the soil report. The typical allowable bearing capacities recommended for this case study (the site) by the geotechnical engineer are shown in Figure 1.1.3 below. Figure 1.1.3 (an image) is an extract from the soil investigation report for the same site. The full report is attached as Appendix 1 which is confidentially limited to this research work only.

Sub-Soil Investigation Report
For a proposed Development At Police College Ikeja, Lagos State.

APEX BN 2450347

See table below for allowable bearing capacities for the top **5.00m.**

TABLE 5-1: ALLOWABLE BEARING CAPACITIES TO-5.00M

Depth(m)	Bearing Capacity(KN/m^2)
1.00	54.0
2.00	81.0
3.00	94.5
4.00	121.5
5.00	135.0

FIGURE 1.1.3: ALLOWABLE BEARING CAPACITIES FOR DIFFERENT DEPTHS

From Figure 1.1.3, the allowable bearing capacity at negative 1m (1m into the ground) is 54kPa. By this, it means that the raft slab should have been placed at negative 1m to mobilize the allowable bearing capacity of 54kPa as stated by the geotechnical engineer. Placing the raft slab at negative 1m would mean that the ground floor would be at negative 1m below the road level. To mitigate the flooding issue, structural engineers in Nigeria improvised on the use of raft beams to elevate the raft slab above this founding depth specified during soil investigation since there is no consideration for building a basement.

Since the allowable bearing capacity would be of different value at the makeup level (+450mm level), that is the allowable bearing capacity of compacted infill soil and at the beam bottom tip (beam soffit) hence the need to study and established the active bearing capacity that would govern the structural design of this type of foundation.

1.2 Aim and Objectives

The objectives and aim of the research are:

- To investigate how the loads from the superstructure are mobilizing contact stresses at slab level and beam level. The calculated contact stresses are then to be compared with the allowable pressure (bearing capacity) recommended in the geotechnical design report;
- To investigate how allowable bearing capacity vary with depth;
- To investigate how larger allowable bearing capacity at beam foundation depth than at slab level depth may influence the foundation design;
- To investigate the effect of settlement beneath the beam and how it transfers load to the infill materials under the slab.

1.3 Methodology

To achieve the above objectives, finite element software program-Plaxis was employed in the design of only the enlarged section shown in Figure 1.1.2 above for simplicity purposes. The total loads from the column at the level of the foundation would be picked from the structural analysis software. The foundation loads on the foundation can be found in Appendix 2. The model would also be kept simple using a 2D Plaxis instead of the 3D Plaxis, and just a single bay of the building (Figure 1.1.2) was investigated.

Since the adequacy of the result depends on the soil input parameters that would be used in Plaxis, the input parameters were given proper attention. The parameters were extracted from the soil investigation report, correlation graphs, and empirical formula.

1.4 Limitation of the Research

The first limitation of the study is the input parameters to the finite element software (Plaxis) since the site under investigation is not compelled to conduct all the laboratory and field tests required to obtain the necessary parameters. The project also relies on external correlations and empirical formular done by other researchers to obtain some parameters that cannot be determined directly.

The second limitation is that the study only focuses on the load shearing principle (that is the load carrying capacity between the substructural elements-beams and slabs) between the slab and beam and displacement is not used as criteria in this project work. This is because the soil investigation report presented an allowable bearing pressure and not bearing capacity/deflection table or curve to show the deflection that are associated with each foundation loading. Although, the allowable bearing pressure presented in the soil investigation report has taken settlement into consideration (by further reducing the safe bearing capacity, for example if the ultimate bearing capacity is $450kN/m^2$, factor of safety of 3, the safe bearing capacity becomes $150kN/m^2$, then the allowable bearing capacity can be taken to be say $110kN/m^2$ depending on the sensitivity of the structure to the effect of settlement) but presenting the load-settlement graph in the soil report as well would have been helpful for comparison (i.e., comparing the soil investigation report's load-settlement graph with Plaxis' load-settlement graph).

Lastly, simplified geometry was modelled for the project because Plaxis 2D was used in the research work hence the decision to simplify the problem. In this work, a single frame geometry was modelled for the research, and the result of the single frame was studied in Plaxis. As a result of this simplicity in the model, the findings in this project work may be affected because the stress contribution from the adjoining substructures and soil had been neglected due to the simplified geometry.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Raft Foundations

Raft foundation is the predominant foundation in Nigeria commonly used in situations where the bases of isolated footings clash with one another hence it is therefore intuitive to combine the bases into a single mat or base (This is otherwise called combined footing). This single mat is also used in situations where the underlying soils have varying compressibility properties. The partial rigidity given by the raft acts as a spread over varying soil compressibility properties under the mat hence reducing the differential settlement of the raft foundation [1].

According to Oyenuga [2] raft foundation is a continuous monolithic concrete slab under buildings that has the capability of receiving all the structural loads of the superstructure and transferring the same loads over an area sufficient to avoid overstressing the soil beyond its bearing capacity.

Oyenuga [2] expressed his thought and experience over the years in Nigeria and listed some Sites in Lagos State, Nigeria with either swamp or sand-filled deposits thus having an exceptionally low bearing capacity of about approximately 20kPa. The cities expressed by [2] in Lagos include but are not limited to Lekki, Victoria Island, Ikoyi, Okota, Isolo, Festac and Satellite Town. Raft foundation tends to overturn (Uplift) due to hydrostatic pressure and it should be studied especially in areas of high-water tables. This hydraulic static effect can be rectified by either a dried construction (the process of continuous pumping) or the use of anchor piles [2].

2.2 Importance of the Raft Foundation

As already mentioned in section 2.1 above, a raft is a continuous slab that covers all the entire footprint of a building in the form of a foundation where the columns and walls of the superstructure rest on. This slab (raft slab) rests directly on the soil strata (layer) or sometimes rests on the pile in the case of piled raft.

According to [3], a raft foundation becomes a better option where some or all the itemized conditions below are encountered while choosing a preferred foundation that will meet the test of time:

(a) Raft foundation becomes imperative when superstructure loads are so heavy, and the allowable bearing capacity of the soil is so small that individual footings would cover a substantial area;
(b) Raft foundation becomes the preferred option when the soils contain compressible strata, or the soil tends to exhibit an erratic trait and it is almost impossible to define and measure the extent of each of the weak pockets which could result in differential settlement along the footprint;
(c) Raft foundation becomes ideal when structures (for example Silos, Chimneys, Water towers, etc.) and equipment to be supported are extremely sensitive to differential settlement;
(d) Raft foundation becomes a preferred option in the case of a floating foundation in which the supporting soil bearing capacity is reasonably extremely poor and the weight of the super-structure is proposed to be balanced by the weight of the soil excavated;
(e) Adequate where the water table is high above the founding base of the sub-structure, the raft slab helps to prevent the buoyancy effect;
(f) Single slab is considered economical in a situation where the individual foundation, if provided, will be subjected to large fluctuating bending moments which may result in differential rotation and differential settlement of individual base thereby affecting the entity of the superstructure.

2.3 Classification & Types of Raft Foundation

According to [3], raft foundations can be classified based on the following:

1. According to their method of support;
2. According to the structural elements used for constructing the raft foundation;

3. According to structural superstructure loads where stiffness plays a key role in the excessive distortion of the superstructure.

2.3.1 According to their Method of Support

Generally, the raft foundation can be supported on any of the following platforms:

(a) Raft foundation supported directly on the soil surface;
(b) Raft foundation supported on piles;
(c) Raft foundation supported based on Buoyancy effect.

2.3.2 According to the Structural Elements Used for Construction

The classification in accordance with the structural elements used for constructing the raft foundation can be as follows:

(a) The slab thickness can have a uniform thickness throughout the section, or the slab may have a pedestal along its section however, a Raft of uniform depth is most popular due to its simplicity in terms of design and construction;
(b) The sub-structure can either be beam and slab where the beam is either upturn or downturn;
(c) Where the substructure floor can function as part of the raft foundation to give additional rigidity to the raft foundation, this is often called the cellular raft.

2.3.3 According to the Structural Superstructure Loads & Stiffness

The slab and beam raft foundation falls within this classification and is therefore used for large, loaded buildings to avoid excessive distortion of the superstructure which may be linked to variation in the loads spread over the raft or compressibility of the supporting soil. The introduction of beams to the slab helps to increase the stiffness of the foundation thus increasing the overall behaviour of this raft system.

[2] expressed his opinion concerning the slab and beam raft has some pitfalls which likely should be considered when making a choice. One of the pitfalls is during excavation of the beams which have the tendency of creating disturbance to the soil therefore affecting the bearing capacity of the soil. The second pitfall is in the case of upturn (i.e., upstand) beams in the basement, the floor usage of the basement becomes impaired due to the upstand beams within the floor areas. The third disadvantage as expressed by [2] is in the case of placing water proofing membrane (also called Damp proof membrane) around the beams, which was considered to be almost an impossible task to achieve by [2].

Figure 2.3.3.1 shows a typical upstand beam raft foundation option. DPC in Figure 2.3.3.1 is referring to Damp Proof Course which was considered by [2] has been the third disadvantage while Figure 2.3.3.2 shows the typical down-stand beam and slab raft foundation.

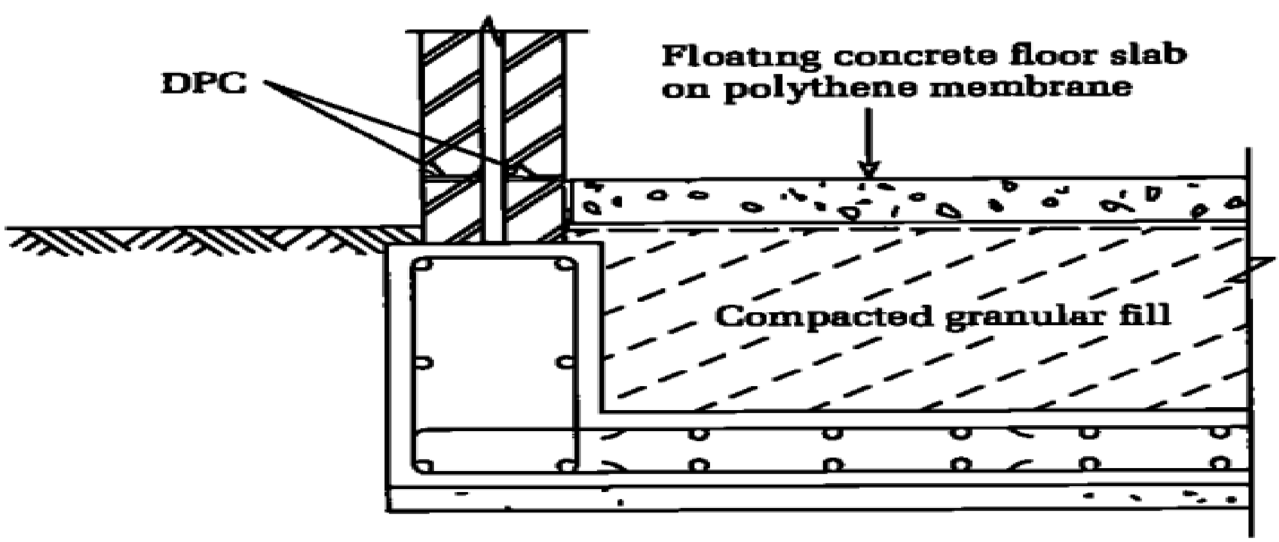

FIGURE 2.3.3.1: SLAB AND BEAM (UPSTAND BEAM) RAFT FOUNDATION [1]

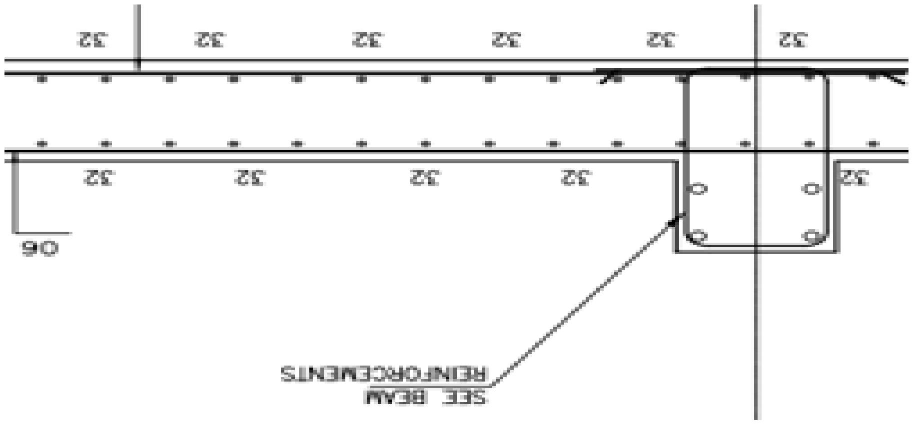

FIGURE 2.3.3.2: SLAB AND BEAM (DOWN STAND BEAM) RAFT FOUNDATION

2.3.4 According to the Structural Superstructure Loads & Stiffness

A buoyancy raft is the kind of raft in which the total weight of the soil removed during the excavation of a basement balances the imposed load that the soil will be subjected to.

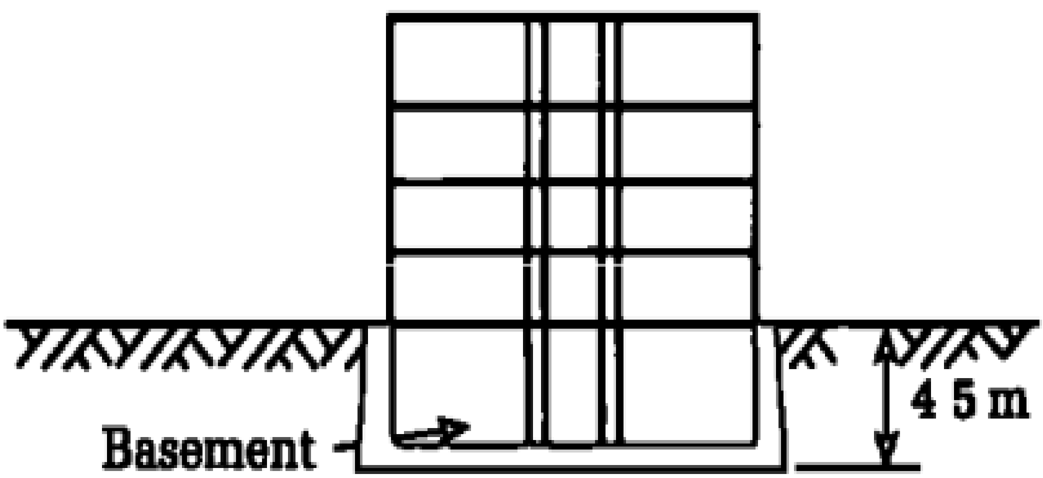

FIGURE 2.3.4.1: TYPICAL BUOYANCY RAFT AT 4.5M DEPTH [1]

According to the figure above and as expressed by [1], the excavation to the depth of about 4.5m could generate about 80kPa pressure (assuming the soil density is between 17 to 20kN/m^3), if the self-weight of the sub-structure is determined and subtracted from the generated pressure, the deficit or the weight difference between the substructure self-weight and pressure generated due to excavated hence becomes the weight that can be placed on the excavated pit.

2.3.5 Cellular Raft Foundation

According to [2], cellular raft foundation becomes a preferred option when very weak soil is encountered however, raft foundation is still considered adequate after thoughtful consideration, hence cellular raft can be considered for such a situation whereas Figure 2.3.5.1 below, illustrate the concept of a cellular raft. [2] believes that additional load carrying of the cellular raft is possible because the additional load carrying capacity is proportional to the excavated earth materials.[2] further stated that the dead space within the foundation can be used for storage/services and concluded that this kind of raft foundation is expensive and rarely used.

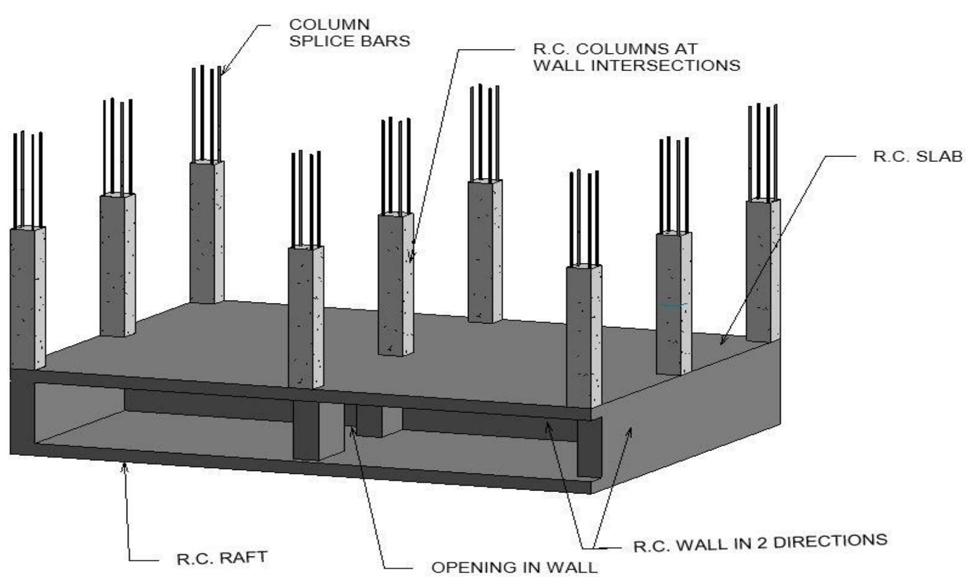

FIGURE 2.3.5.1: CELLULAR RAFT FOUNDATION

2.4 Raft Foundation Synopsis by Different Authors

Several authors have distinct approaches in designing and analyzing the raft foundation, some of the approaches are discussed below. Likewise, the bending moment and shear forces are estimated using independent approaches by individual authors. Although the design approach remains the same, the only difference is in the method of estimating the bending and shear force on the slab. Different author’s bending moments and shear forces estimation approaches are itemized below:

2.4.1 Raft Foundation Design Approach by Peck, Hansen and Thornburn

Peck et al [4] regards raft slabs as continuous inverted flab slab plate supported within individual panel by columns or walls and there are no expected upward deflections. Figure 2.4.1 below shows a typical inverted flat slab (the load from the soil causes the top of the slab to be in tension therefore requires tension reinforcement also referred to as a Top reinforcement and the bottom of the slab which is in contact with the soil is in compression and requires compression/bottom reinforcement). The surface load on the raft is the soil pressure that acts at the soffit (i.e., bottom) of the raft slab and is usually taken to be uniformly distributed over the entire raft slab. Based on Peck, the soil pressure or the imposed load on the raft slab can be estimated by the algebraic sum of all the column’s loads times by necessary factor safety and divided by the area of the footprint of the raft.

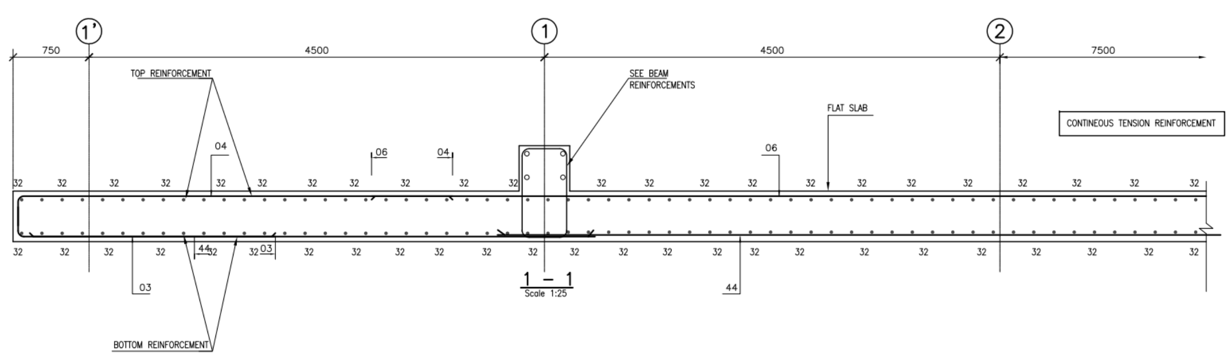

FIGURE 2.4.1: TYPICAL INVERTED FLAT SLAB

Since the assumption adopted by Peck is similar to a flat slab (but inverted for raft), thus the bending moments and shear force in individual panels are estimated using the appropriate moment and shear coefficients specified in different design codes for the design of flat slab floors.

The above approach is believed to have disadvantages as the differential settlement of the structure is not considered and the erratic nature of the soil is not taken into consideration. Also, it is believed that the differential settlement is likely to cause a redistribution of the design moments and could possibly have a significant effect on the estimated moment and shear. Peck concluded that the raft should be designed as though the slab is seated on a platform of close and equal-spaced elastic springs of approximately the same stiffness and the contact pressure on the raft slab varies directly to the deflection of the spring where the constant of the proportionality is considered to be the modulus of subgrade reaction. Peck opined that the estimation of this constant (modulus of subgrade) is never constant as it is shrouded with lots of uncertainties.

However [3] concluded that the above method did not exhaustively address the issue of deflection in the raft foundation and the issue of differential settlement is sometimes solved by engineers-by providing high strength and adequate stiffness in the raft slab. He however considered this approach as not been sufficient to solve the deflection problem whereas noted that the stiff raft is likely to increase the bending moment of the raft and the value calculated is too high than the value calculated through flat coefficients stipulated in the design codes.

2.4.2 Raft Foundation Design Approach by Elwyn. E.S. Seelye

In this method, the same approach as proposed by Peck is used to determine the soil pressure. However, in this method, the pressure is determined at different points of the raft foundation. After soil pressure had been determined, the footprint of the building is divided into strips (bands) and designs are as per each band [5].

2.4.3 Raft Foundation Design Approach by Teng

According to [6], the raft plate is assumed to be seated on an infinitely rigid surface, and the bearing pressure under the raft plate follows the planner distribution. He proposed that the raft plate should be analyzed as an entity in the two perpendicular directions and the total shear force or bending moment becomes the algebraic sum of forces or moments in each direction. The principle of summing the moments and forces is based on assumptions because the problem is complex, and the structure is indeterminate in nature which cannot be solved by static principle (by three moment equations).

[6] suggested that the middle column strip of the raft plate can be analyzed as independent continuous or combined footings as this result gives a high upper limit and seems to be conservative for the design.

2.4.4 Raft Foundation Design Approach by Dunham

Duncan [7] studies the behavior of the soil under each column and stated that the grouping of the columns according to many researchers could be misleading or unjustified due to the erratic properties of the soil. Duncan opined that; it is conservative to assume that the raft plate is rigid, the load is constant and the soil which is plastic in nature will compress and spread under individual column loads (the load of the columns should be treated as point loads).

The assumption here is that the analysis should be by strips and the strips should be designed as though the mat is a beam supporting the columns (see figure 2.4.4.1 below). The width of the strip is denoted as Ws, and the design of the strip is based on this demarcation.

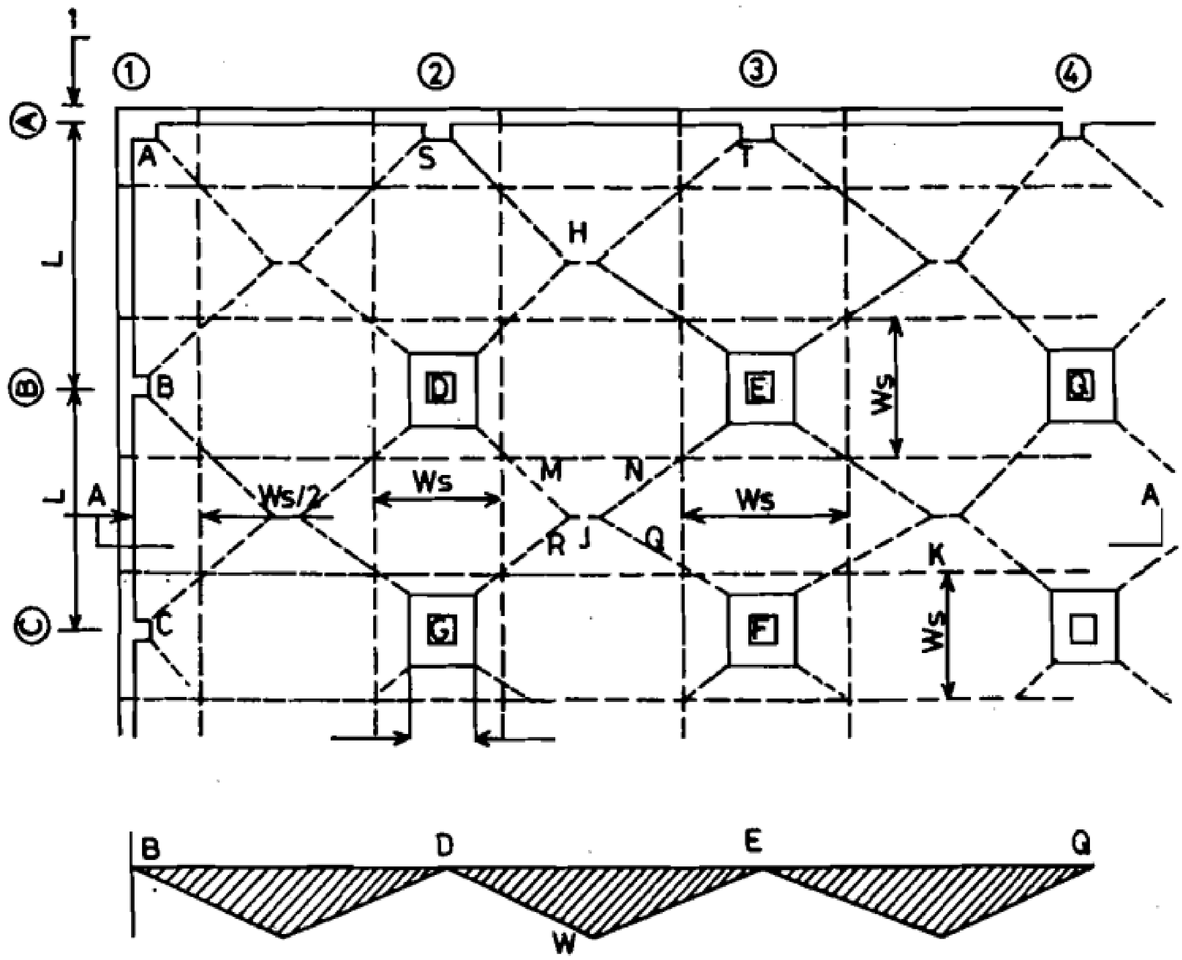

FIGURE 2.4.4.1: PLAN OF ASSUMED COLUMNS STRIPS AND LOADS DISTRIBUTION BY DUNCAN [3]

2.4.5 Raft Foundation Design Approach by Indian Standard Code of Practice IS 2950-1965

The Indian code of practice grouped raft design into a convectional and elastic approach. The formal approach assumed that the raft foundation is an infinitely rigid structure, and the pressure distribution is not a function of the deflection of the raft. In this case, the soil pressure distribution is assumed to be acted in plain whereby the centroid of the soil pressure coincides with the resultant force of all the loads acting on the foundation.

Whilst the latter approach (Elastic Method) is based on either the assumption that the soil has numerous numbers of isolated springs or the assumption that the soil behaves as though it is an elastic medium and obeys Hooke's law. Though, these assumptions have its limitation and one of the limitations expressed by the Indian code is that the area of the mat should be relatively small, and the loads should be concentrated around the location [8].

2.4.6 Raft Foundation Design Approach by A.L.L Baker

According to Baker [9], designing a raft foundation as a reverse floor is not ideal (i.e., the surface of the slab in contact with the soil is reversed during design by structural engineer for ease of analysis but the reinforcement are placed at the top of the slab because that is the sagging location for a raft slab), However, this approach has always been the consensus of structural engineers in my opinion where the load on this mat is taken as a uniform earth pressure from the soil on the plate.

Baker's concern is that using the soil uniform pressure (in this case refers to as loads on the plate, which would be used in sizing the substructural elements) to estimate the section moment (moment used in sizing the raft slab) is too conservative. [9] proposed straight-line law to effectively determine the variation in the soil pressures and its corresponding deflection.

2.4.7 Raft Foundation Design Approach by Joseph E. Bowles

According to Bowles [10], the raft plate should be designed as a rigid structure and proposed that the soil pressure should be calculated based on equation 1.0 given below and in the case where the resultant force coincides with the centre of the raft plate, equation 2.0 should be adopted:

$$Q = \frac{V}{A} \qquad \textbf{Eqn. 1.0}$$

Whereas if the resultant force does not coincide with the centre of the plate that is there is eccentricity between the centre of the mat and resultant force, the equation below becomes useful:

$$Q = \frac{V}{A} \pm \frac{M_Y.X}{I_Y} \pm \frac{M_X.Y}{I_Y}$$

Eqn. 2.0

Where Q = Soil pressure

V = Column loads

A = Area of the raft

My & Mx= Moment in the Y and X direction respectively

X & Y = Eccentricity in X and Y direction respectively

I y = second moment area

Bowles stated that if the eccentricity is somewhat high, there will be errors in the bending moment and shear forces within the plate calculated would contain errors. He then concluded that it will then be possible to calculate the soil pressure at various locations under the raft mat once the structure dimensions are known.

2.5 Raft Foundation Design Approach and Consideration

By the foregoing (Section 2.4), it was concluded that raft could either be designed as a rigid or flexible foundation. Both the rigid and flexible foundation as regards raft foundation would then be discussed in detail below:

2.5.1 Rigid Raft Foundation Approach

As mentioned earlier by several Authors above, that raft being rigid connotes that the raft slab provides an adequate bridge over the soil irregularities. According to [3], he believes that soil pressure under the raft foundation for a rigid plate is either uniformly distributed or varying linearly, and [3] also mentioned that the differential settlement is rather low in rigid foundation whereas the bending moment/ shear force in the plate is relatively high in a rigid raft. The high moment and shear force values are believed to be related to the higher stiffness of the structural element used.

The design of a rigid foundation according to [3] is based on these two approaches:

1. Design based on an Inverted Floor system (same as Figure 2.4.1 above);
2. Combining footing approach system.

2.5.2 Flexible Raft Foundation Approach

In a flexible raft system, many Authors (a few mentioned above in section 2.3), stated that the flexible plate does not have the capacity to serve as a bridge over the non-uniformities of the soil structure instead the column points should be idealized locally.

In contrast to the rigid plate, the differential settlement for the flexible plate is relatively higher than the rigid plate and the bending moment/shear force in the raft plate due to the soil pressure is relatively low compared to the rigid foundation system.

[3] proposes that the analysis and design of a flexible raft foundation can be based on two theories which are:

1. Flexible foundation plate assumed to be supported on elastic foundation:

2. Flexible foundation plates are assumed to be supported on the bed of uniformly distributed elastic springs where the spring constant is determined from the coefficient of subgrade reaction.

In addition to the statement credited to [3] above, [3] further stated that the two methods above can be evaluated by manual computation but with so some limitations and hence can be best solved by computer programs such as finite element and finite differences methods.

The finite element method is presumed by [3] to be used for solving approach 1 (1-Flexible foundation plate assumed to be supported on an elastic foundation) above, where the plate on the elastic foundation is converted into a computer program of matrix structural analysis. This converted plate in the matrix structural form is transformed into a mesh of several tiny plates (finite elements) that are interconnected at each end of these tiny plates (otherwise known as the nodes). In this instance, the soil is modeled as a set of isolated springs.

Meanwhile, the finite difference method used for solving approach 2 (2-Flexible foundation plate assumed to be supported on a bed of uniformly distributed elastic springs where the spring constant is determined from a coefficient of subgrade reaction) where in this method a single value of sub-grade modulus can be adopted for the whole foundation footprint.

2.5.3 Raft Foundation Design Parameters

The crucial parameters needed in the design of a raft foundation are grouped into the following according to [3];

1. The rigidity of the raft plate;
2. Pressure distribution under the raft plate;
3. Subgrade modulus value;
4. Other information obtained from soil/geotechnical investigation report.

2.5.3.1 The Rigidity of the Raft Plate

This has been one of the major discussions of section 2.1 and is extensively discussed in section 2.5 above.

2.5.3.2 Pressure Distribution Under the Raft Plate

This topic is the major concern of some research works. However, [3] mentioned that the major problem of designing a raft is the determination of the actual contact pressure of the soil under the raft foundation against the plate. Up until now, there are project works-to theoretically find ways to estimate the exact contact pressure under the raft but an exact solution to this problem seems not to be known yet. Likewise, there are experimented works (although limited) conducted in quest of the exact contact pressure with no pinpoint recommendation/solution [3].

Based on the [3], soil pressure determination is an extraordinarily complex task that has so many variabilities. However, this variability compounded the problem. A few of the variabilities considered by [3] are mentioned below:

I. Time of inspecting/inquiring the pressure;
II. The size of the plate;
III. Influence from surrounding foundations;
IV. The influence of different magnitude loads;
V. Rigidity of the superstructure;
VI. The nature of the foundation (rigid or flexible);
VII. Spatial Variabilities (properties) of the soil;
VIII. The kind of soil beneath the raft foundation (single homogeneous soil mass, layered formation, strata of various layers, among others);
IX. Built up groundwater table.

Gupta further argues that the concept of assuming a uniform pressure distribution under a rigid plate is not true. He believes that; to truly have a uniform distribution of soil pressure, the footing has to be a very flexible, and if the design concept of soil being elastic (which led to using modulus of elasticity otherwise called coefficient of sub-grade modulus) is appropriate to go with, then the settlement under the rigid foundation should have been uniform and that of flexible foundation will be non-uniform. This completely contradicts the foregoing argument in section 2.5.1. Based on this forgoing argument, [3] finds it difficult to comprehend why then would the contact pressure under a rigid footing be uniform. [3] finally concluded that this is a pure case of soil structure interaction issue and that the soil pressure under the footing is non-uniform in nature and the settlement within the footing would be a differential settlement in the real sense. He believed that the soil pressure distribution for footing on sand soils is reasonably uniform.

[3] however proposed that for a square footing, the average pressure distribution should be calculated as stated in equation 3.0 below:

$$\text{Square footing soil pressure} = \frac{0.6\ x\ P}{A} \qquad \textbf{Eqn. 3.0}$$

He advised that the above pressure calculated by equation 3 should be added with additional pressures calculated by equation 4.0 to give allowance to the influence from the edges (that is the pressure obtained from equation 4.0 give account to the pressure contribution from the edges of the footprint which is different from the pressure within the footprint).

$$\text{Square footing soil pressure due to contribution along edges} = \frac{\mathbf{0.1\ x\ P}}{\mathbf{A}} \qquad \textbf{Eqn. 4.0}$$

Whilst for a rectangular footing with a large length, the average soil pressure suggested by [3] is given by equation 5.0 below;

$$\text{Rectangular footing soil pressure} = 0.8 * P\ aveage + \ 0.1\ P/B \ \ (for\ the\ edeges) \quad \textbf{Eqn. 5.0}$$

where P = total loads

A = area of the mat

B = length of the footing

To demonstrate the behaviour of contact pressure under footing further, [3] presented the contact pressure of raft foundation based on different founding strata as shown below:

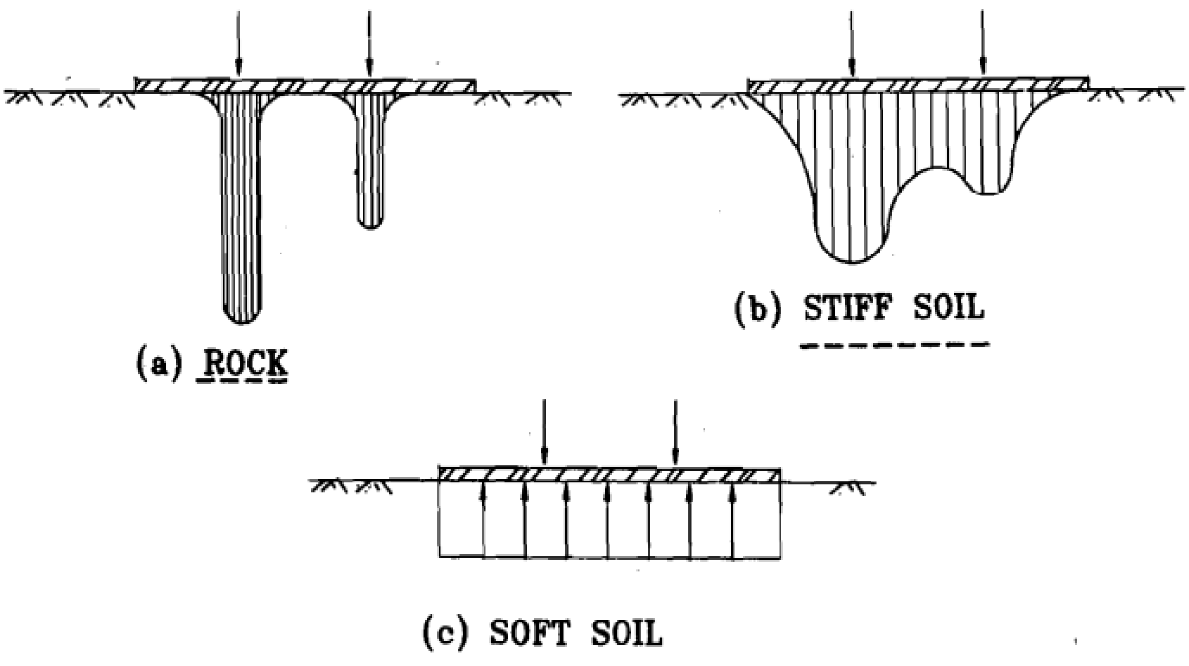

FIGURE 2.5.3.2: DISTRIBUTION PRESSURE PATTERN BASED ON DIFFERENT STRATA [3]

2.5.3 Modulus of Sub-Grade Reaction

According to [3], he regarded the modulus of subgrade reaction as one of the crucial parameters in estimating soil pressure and this parameter can sometimes be referred to as the coefficient of subgrade reaction. He numerically finds the relationship between the modulus of subgrade and deflection and stated that the modulus of subgrade reaction as being the magnitude of soil pressure needed to result in a unit deflection of the system. To prove his theory, he recommended conducting a plate load test and plotting a curve of soil pressure against the deflection. He however mentioned that the real result on site can be influenced by some, or all the parameters listed below (not limited) as against the result that would be obtained in a control environment (Laboratory investigation/test);

I. The magnitude of the soil pressure and deflection are altered because the soil is not perfectly elastic (Soil is elastoplastic);
II. Size of the footing affects the value;
III. The shape of the footing;
IV. The founding of the footing also has a significant effect;
V. Soil strata which may not come to play when testing with a small plate also affect;

Whilst the laboratory and field estimation of subgrade modulus may seem overly complex, several authors have recommended simple empirical formulation of estimating the modulus of subgrade from bearing capacity factors in Terzaghi bearing capacity equation. Some of the recommendations are discussed below.

2.5.4.1. Recommendation On Modulus of Sub-Grade Reaction by Bowles

According to [11], the empirical relationship between the modulus of subgrade and safe bearing capacity is related by equation 6.0 below;

$$Ks = 36 * qa \qquad \textbf{Eqn. 6.0}$$

Where Ks = modulus of subgrade reaction

qa = allowable bearing capacity in kips per sq. ft.

In conclusion. [11] suggested that the ranges of values for sub-grade modulus for several types of soil are tabulated in Table 1.0 given below;

TABLE 1.00: RANGES OF VALUES FOR KS FOR DIFFERENT SOILS

Soil	Ranges of Ks. Kef
Loose sand	30 - 100
Medium sand	60 -500
Dense sand	100- 800
Clayed sand (Medium)	200 - 500
Silty sand (Medium)	150 - 300
Clayed soil:	
qu ≤ 4 * Ksf	75 - 150
4 < qu ≤ 8 ksf	150 - 300
8 < qu	> 300

2.5.4.2 Recommendation on Modulus of Sub-Grade Reaction by IS:2950-1981

According to Indian standard [3], the procedure for determining the modulus of the subgrade is stated in Appendix B of the code IS 2950-1981 and the tables are shown below:

TABLE 2.00: MODULUS OF SUBGRADE REACTIONS (K) FOR COHESIVE SOIL BASED IS: 2950-1981 [12]

Soil Characteristic		*Modulus Of Subgrade Reactions (K) in kg/cm^{3}*	
Relative Density (1)	*Standard Penetration test value (N)* (2)	*For dry or moist state* (3)	*For submerged state* (4)
Loose	*< 10*	*1.5 (95)*	*0.9 (57)*
Medium	*10 to 30*	*1.5 to 4.7 (95 to 300)*	*0.9 to 2.9 (57 to 185)*
Dense	*30 and over*	*4.7 to 18.0 (300 to 1146)*	*2.9 to 10.8 (185 to 687)*

** The above values apply to a square plate 30 X 30 cm or beams 30 cm wide*

TABLE 3.00: MODULUS OF SUBGRADE REACTIONS (K) FOR COHESIVE SOIL BASED IS: 2950-1981 [12]

Soil Characteristic		*Modulus of Subgrade Reaction (K) in Kg/cm³*
Consistency	*Unconfined compressive strength, kg/cm²*	
(1)	*(2)*	*(3)*
Stiff	*1 to 2*	*2.7 (172)*
Very stiff	*2 to 4*	*2.7 to 5.4 (172 to 344)*
Hard	*4 and over*	*5.4 to 10.8 (344 to 688)*

** The values apply to a square plate 30 × 30 cm. The above values are based on the assumption that the average loading intensity does not exceed half the ultimate bearing capacity.*

Based on my observation, the difference between the two tables is in the footnote which states that the average loading intensity of the second table does not exceed half the ultimate bearing capacity whilst the figures in the bracket in the tables are in Kips per cubic foot units.

2.5.4.3 Recommendation on Modulus of Sub-Grade Reaction by Alpan and Prof. Alam Singh

Alpan [13] conducted a similar test suggested in section 2.5.4 that is measuring a deflection from a loaded plate as prescribed by Terzaghi et al. and during Alpan test, he finds the relationship between the inverse of the modulus of subgrade reaction and Standard Penetration Test (SPT) blows. According to the work of Singh [14], these two parameters-the inverse values of subgrade modulus and SPT plotted the correlation between these two parameters in SI units. Part of the work of Singh also involves charts for overburden and SPT.

Part of [14] conclusions is that the value of the modulus of subgrade should be not greater than three times the original value of N (number of blows), and when N is greater than 15 it should be further corrected.

2.6 Soil Structure-Interaction of a Raft Foundation

Soil structure interaction (SSI) is believed to be exigent by most researchers especially in the study dynamic behaviour of soil and structures during earth movement. According to [15], the research works begins in the 1930s and suggested that the structure and soil beneath should be considered as a single system during analysis. He further advised that SSI analysis can be simplified into direct and substructure methods.

The soil and structure are said to be treated as a whole system in the direct method, in this method the soil in the surroundings is advised to be explicitly modeled with adequate boundaries conditions to resemble real-life situations whereas, in the substructures methods, both are treated as an independent entity with each modeled and analysis independently.

According to [15], if one can establish the force–displacement relationship/equation at the connecting phase of the soil under the substructure and superstructure hence this established equation can simply be taken as the combining motion equation of the entire system.

Based on the research made so far on this topic, there has been no citation of soil interaction of raft foundation and the soil underneath rather all research works focus on the dynamic behaviour of SSI during the earthquake hence an attempt would be made at the end of this work to illustrate the behaviour of the soil and structure under loading but this time under non-dynamic loading.

2.6.1 Finite Element Modelling and Designing Software (Plaxis)

According to [16], the finite element method/approach (FEM) is an approximate numerical method which is deployed in solving a variety of problems involving engineering and sciences. The finite element is believed to be originally developed by engineers in the 1920s and advanced further by mathematicians.

According to [16], FEM in solving geotechnical problems is very much similar in the way it is applied to solving structural problems. In geotechnical approach, force is applied to the soil strata, and the corresponding response due to this applied force on the soil body is studied with respect to deformation/displacement. However, [17] itemized the basic steps that go within the finite element software program below:

I. Discretization of the problem;
II. Calculation of the element stiffness matrices;
III. Assembling of the element stiffness matrices;
IV. Determination of the element load vectors;
V. Compiling the element load vectors;
VI. Introducing the relevant boundary condition;
VII. Introducing external forces;
VIII. Determination of the displacement vectors;
IX. Finally, calculate the strain and stress field.

According to [16], there are several FEM applications, but the common ones mentioned by [16] are listed below:

a) Used for static structural analysis, for example, the plates, shells soil, and force-displacement rock, etc.;
b) Used in dynamic problems (ground movement);
c) Used in heat flux and heat flow;
d) Used in coupled problems for example when you consolidate there is seepage and deformation at the same time hence this is coupled problem.

2.6.2 Synopsis of Plaxis 2D Finite Element Program

According to [18] ‘’*PLAXIS 2D is a special-purpose two-dimensional finite element program used to perform deformation, stability, and flow analysis for various types of geotechnical applications*’’ Plaxis 2D provides an avenue to either model the problem at hand as a plain strain or axisymmetric. It is currently a product of Bentley and is believed to have a friendly graphical user interface (GUI).

According to [17], Plaxis 2D uses 6 and 15 nodal point triangular elements in the two-dimensional problems whilst in the 3- dimensional tetrahedral element [17] believe 10 nodal points may be adequate. The accuracy of the problem can be improved by using fine-meshed elements. However, this fine mesh requires a longer processing time.

[17] concluded that the 15-node triangular elements should be used in the event that the designing computer has the capacity to process the problem rather than using the 6-node triangular approach however, the 6-node triangle may as well be used in case of computer constraint and the problem is far from global collapse with reasonable plastic strain.

2.6.3 Soil Models and Evaluation of Stiffness of Soils

Plaxis has several soil models that can be used for simulations and solving different problems among which are Mohr-Coulomb, Modified Cam Clay, Soft soil creep, and Hardening soil, among others. To adequately predict the behaviour of soil, one needs to first understand the theory behind each soil model which is tied to the available soil parameters because each model requires slightly different parameters to be able to simulate the soil behaviour [16]. According to [16], the simple soil model requires fewer input parameters to be able to simulate the soil behaviour whilst the advanced model requires more input to simulate. The more soil parameters require the better the accuracy of the prediction of the soil behaviour by the model of choice. The complex model requires a good understanding of soil behaviour to be able to follow through with why the simulation is behaving in any particular way

According to [16], Mohr-Coulomb is believed to be a simple easy-to-follow model where strength is based on the Mohr criterion and stiffness is based on the stress-strain relationship (Hooke's Law). This model is of the opinion that soil is linearly elastic and isotropic material however, soil exhibits a completely different behaviour and complex material which require careful attention and good engineering judgment in the selection of models and parameters.

However, [16] concluded that the hardening soil model is a complex model that has numerous advantages over the simple model and thus recommended over the simple model in cases where the deformation behaviour of soil is required in a problem. This model requires more input parameters than the simple soil model.

2.6.3.1 Evaluation of Stiffness Parameters for Sand

The average stiffness (E_{50}) can be determined from triaxial test. According to [16], the vertical stresses of sand in a triaxial compression test increase with horizontal stresses being kept constant (this form of triaxial could be referred to as a uniaxial compression test). The stress–strain relationship of this test can be represented by the curve below.

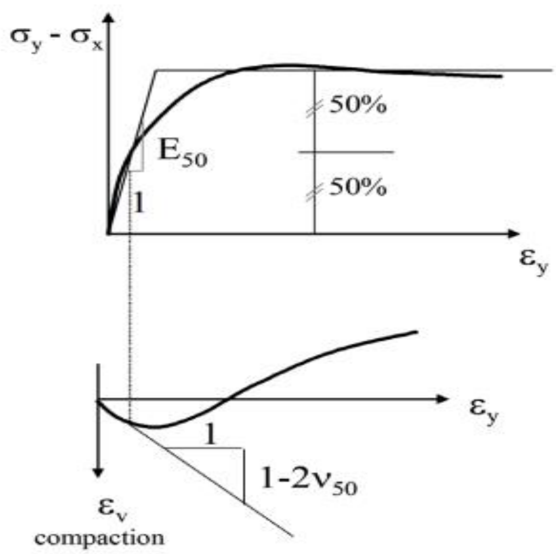

FIGURE 2.6.3.1.1: EVALUATION OF E50 AND V50 OF SAND DRAINED TRIAXIAL TEST [16]

In the stress–strain curve, the tangential line to the topmost curve gives the failure line while the average of the failure line is indicated as E_{50}. A secant line drawn from the origin intersects the curve thus, the slope of the gives the average stiffness. Similarly, the poison ratio can be evaluated from the axial strain ε_y versus the volumetric strain ε_v in the second curve.

2.6.3.2 Evaluation of Reference Stiffness Parameters for Sand

The mathematical relationship for the reference stiffness parameter is according to the below expression:

$$E_{50} = E_{50}^{ref} \sqrt{\frac{\sigma_x'}{p_{ref}}}$$

Eqn. 7.0

where σ_x' = effective confining stress

P_{ref} = atmospheric pressure (usually 100kPa) and

E_{50} = average stiffness as determined above.

The major unknown is the reference average stiffness.

According to [16], the values of the confining stress selected for the test will affect the value of the average stiffness. [16] summarizes the output of considering three different values of cell pressure in Figure 2.6.3.2.1 below and the conclusion is that loose sands have E_{50}^{ref} of about 15MPa and dense sands have $E_{50}^{ref\ of}$ about 50MPa.

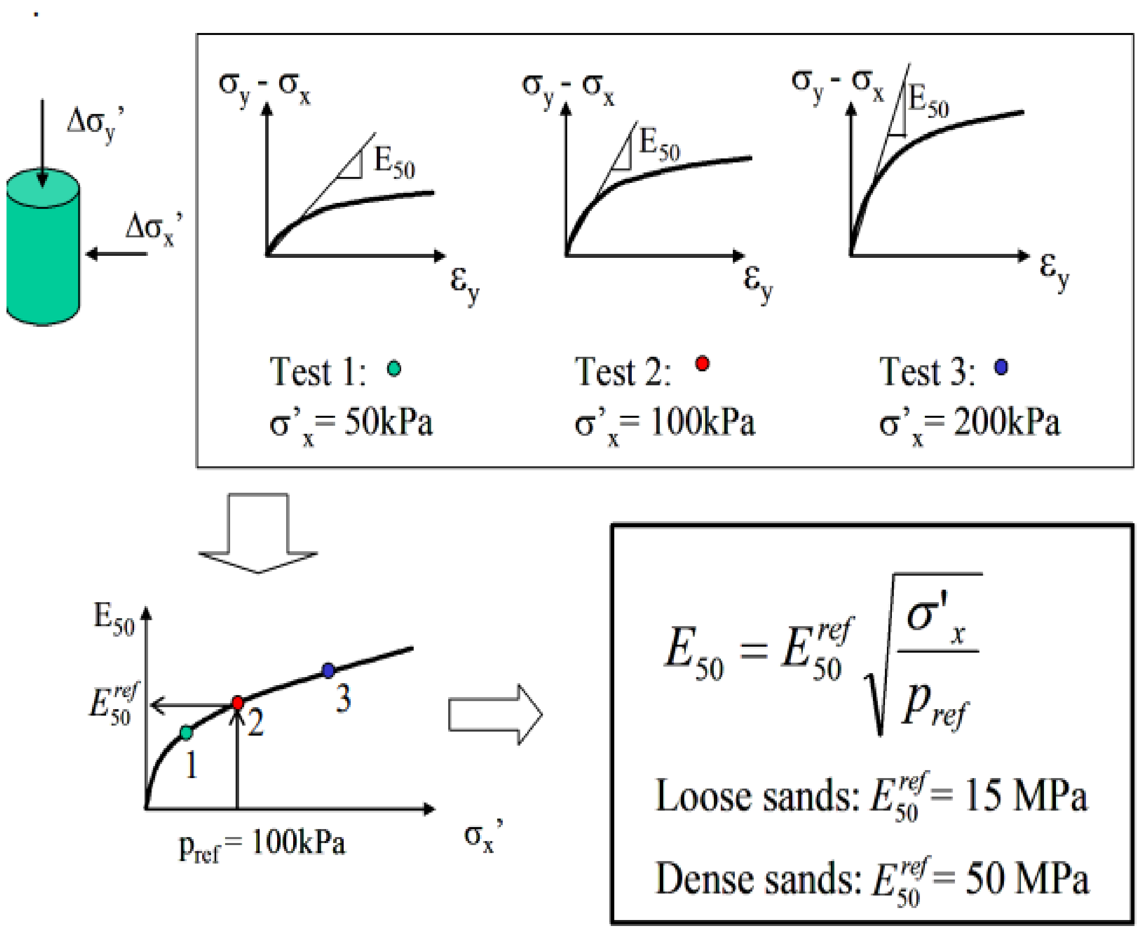

FIGURE 2.6.3.2.1: AVERAGE REFERENCE STIFFNESS FOR SANDS EVALUATION [16]

2.6.3.3. Evaluation of Oedometer Stiffness Parameters of Sand

The relationship between Oedometer stiffness and young modulus is based on the equation below and on the assumption that there is a linear elasticity relationship. Adopting a poison ratio of 1/3 reduces the equation to 1.5 times the young modulus as seen in the equation below:

$$Eoed = \frac{E\,(1-\upsilon)}{(1-2\upsilon)(1+\upsilon)} = \frac{3}{2}E \qquad \textbf{Eqn. 8.0}$$

The conclusion here is that the oedometer stiffness is 1.5 times stiffer for soil placed in a cylindrical steel ring because the sample with a constant cell pressure is supported sideways.

Meanwhile, the oedometer stiffness is related as expressed in the equation below and here the major principal stress is used in the equation:

$$E_{oed} = E_{oed}^{ref} \sqrt{\frac{\sigma_y'}{p_{ref}}}$$

Eqn. 9.0

2.6.3.4. Evaluation of Stiffness Parameters for Clay

According to [16], clay may be considered as undrained during short-term loading stated which could be because of the water retention time of clay material and cannot be all expunged under short-term loading, this however, leads to no volume change in the system due to the fact the water or undrained clay is incompressible due to presence of porewater therefore one can conclude that the deformation for undrained clay would only likely have shear deformation or distortion.

In contrast to the above paragraph, long-term loading of clay would therefore result in volume change as it is likely that all porewater retain would be dissipated over time. [16] expressed concern over a soft clay loaded under this condition that the deformation would be excessive and not behave as though it was elastic but rather plastic and permanent and this plastic behaviour of the soft clay cannot be simulated using the simple linear elastic – perfectly plastic soil model in Plaxis.

However, [16] believe that the solution to the selecting right model is by selecting the correct parameters for several types of loading and unloading conditions hence it will be difficult to ascertain the true stiffness or clay properties for clay rather than the stiffness and other clay properties that would be evaluated will be for specific loading conditions (either loading or unloading condition).

2.6.3.5. Evaluation of Undrained Average Stiffness $G_{50}{}^{U}$ from Triaxial Test

This follows the same approach (secant line through the origin, two lines drawn with one of the lines tangential to the failure point, and the second line through the midpoint) as explained in section 2.6.3.1 above - Evaluation of stiffness parameters for sand. This concept is also well explained in Figure 2.6.3.5.1 below:

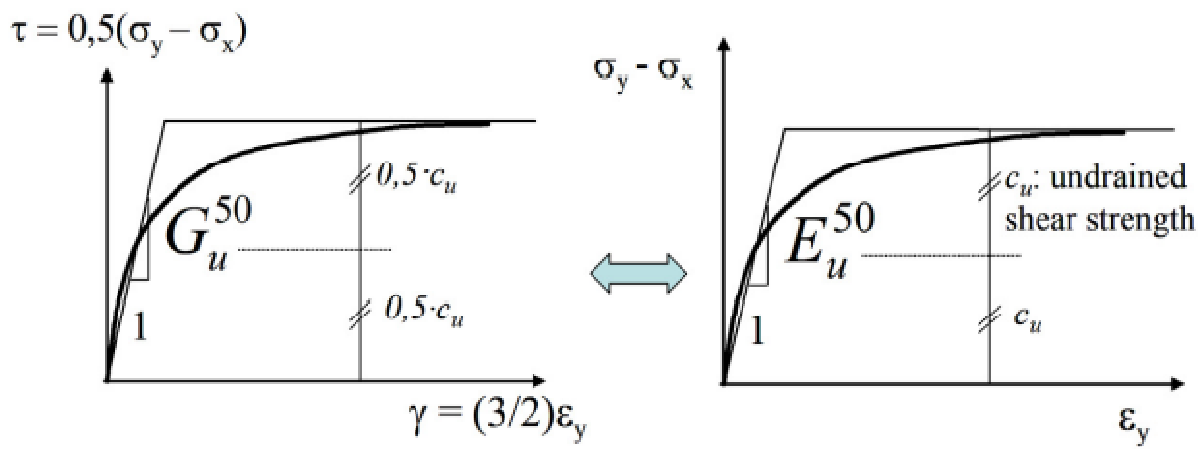

FIGURE 2.6.3.5.1: EVALUATION OF UNDRAINED STIFFNESS FROM AN UNDRAINED TRIAXIAL TEST [16]

[16] expressed the undrained stiffness with respect to some parameters and some of them will only be quoted below:

With undrained poison ratio υ_u taken as 0.5 due to the incompressibility nature of undrained sample because porewater is not allowed to dissipate, the undrained shear strength can be expressed as given in the equation below:

$$G_u = \frac{E_u}{2(1+\nu_u)} = \frac{E_u}{3}$$

Eqn. 10.0

Undrained stiffness is also expressed in terms of the percentage of strain at which it reaches failure, for instance, if the failure is reached when the percentage strain is about one percentage, the undrained stiffness becomes:

$$G_u^{50} = \frac{c_u}{0,01} = 100 \cdot c_u$$

Eqn. 11.0

In terms of plastic index, undrained stiffness can be expressed as follows:

$$E_u^{50} = \frac{15000 c_u}{I_p}$$

Eqn. 12.0

2.6.3.6. Evaluation of Undrained Shear Strength

From the above equations, we can see that undrained stiffness is tied to undrained strength hence it is imperative to first determine the undrained strength. This section is aimed at finding the expression for undrained shear strength.

[16] expressed the undrained shear strength for a normally consolidated soil as 25% of the principal effective stresses which is expressed in the equation below, an expression that was linked to Janbu's research works:

$$c_u = 0,25 \cdot \sigma_y'$$

Eqn. 13.0

For over-consolidated soil, the undrained shear strength can be determined by the expression below:

$$c_u = 0,2 \cdot \sigma_c'$$

Eqn. 14.0

Where σ'_c is the effective pre-consolidated pressure.

[16] also expressed the undrained strength for an over-consolidated clay in terms of OCR as expressed in the equation below:

$$c_u = 0,22 \cdot \sigma_y' \cdot OCR^{0,8}$$

Eqn. 15.0

Finally, [16] concludes that the above expressions should not be used as a complete replacement for the geotechnical site investigation test but rather as a guide for expected results.

2.6.3.7. Evaluation of Drained Stiffness of Clays

The drained stiffness of clay is normally expected to be lower than the undrained stiffness because of the dissipated pore pressure in the drained set up which results in a change in volume while the undrained gives rise to a change in shape. The drained stiffness is practically investigated in an Oedometer setup where a gradual increase in stress produces vertical deformation of the clay in a ring. The slope of the applied stresses and the measured vertical deformation or strain gives the oedometer stiffness.

The figure below summarizes the experimental procedure of evaluating the drained stiffness of clay;

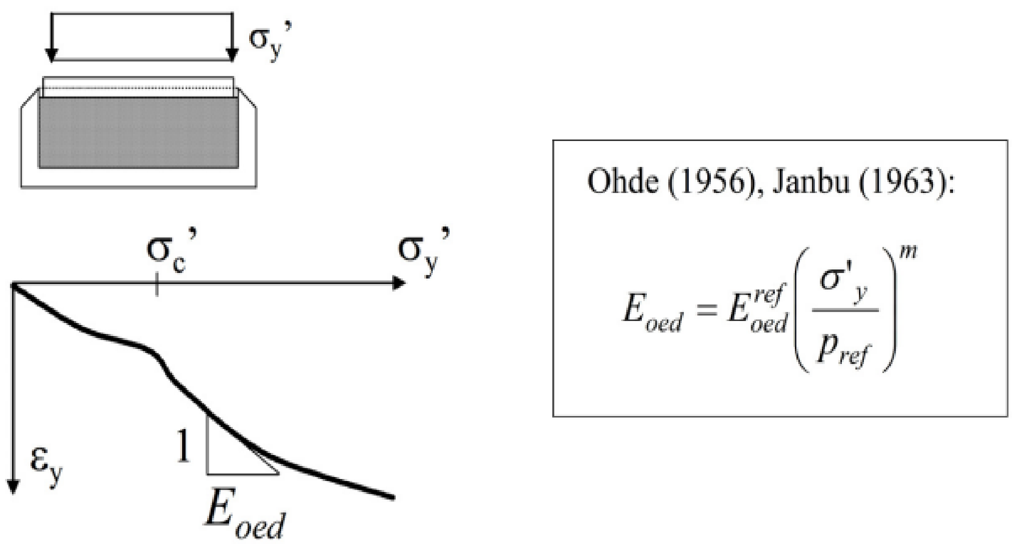

FIGURE 2.6.3.7.1: DETERMINATION OF DRAINED STIFFNESS FROM AN OEDOMETER SETUP [16]

The oedometer stiffness is expressed by the equation below:

$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma_y'}{p_{ref}}\right)$$

Eqn. 16.0

In the above equation, m = 1 for a normally consolidated (NC) clay, P_{ref} =100kPa, and the slope of the stress-strain gives the E_{oed} hence $E_{oed}{}^{ref}$ can be determined from the equation above.

For over-consolidated clay (OC), the oedometer stiffness can be written as given in the equation below:

$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma_c'}{p_{ref}}\right) \cdot n$$

Eqn. 17.0

where n is between 2 to 5

2.6.3.8. Evaluation Of Undrained Stiffness of Clays

As mentioned earlier, the undrained stiffness is generally higher than the drained stiffness. [16] intuitively suggest that the same soil skeleton can be used for both drained and undrained conditions in Plaxis just by adding a constraint of no volume change when the soil skeleton is considered under an undrained model however the option of adding no volume change may sometimes not be available especially when the soil models used is a simple model and govern by elasticity.

The relationship between the undrained stiffness and drained stiffness is proved by [16] to be as given below when using 0.5 for the undrained poison ratio and 0.3 for the drained poison ratio:

$$E_u = \frac{1,5}{(1+\upsilon')}E' \approx 1,1 \cdot E'$$

Eqn. 18.0

2.6.3.9 Soft Soil Model

According to [18] , Cam-Clay model is a soft soil model that is used to simulate the behavior of soft soil (example of soft soils are normally consolidated soil and peat). [18] believes that Cam clay model behaves well for primary compression situation. The figure below shows the compression test graph for both sand and clay.

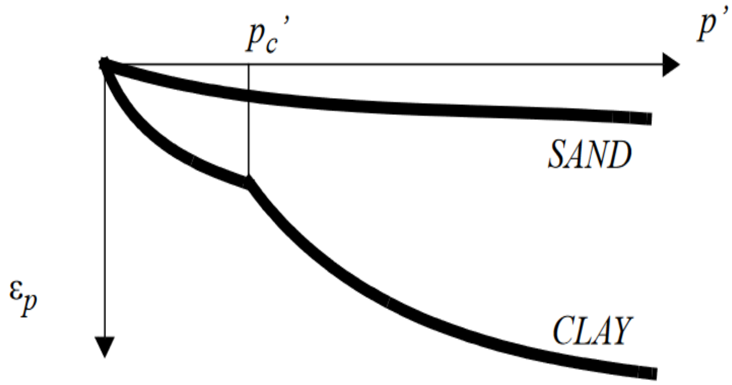

FIGURE 2.6.3.9.1: OEDOMETER/COMPRESSION TEST PLOT FOR SAND AND CLAY [16]

There are several parameters that can be obtained from an Oedometer test. These parameters, including Oedometer Modulus according to [16] are shown in the figure below:

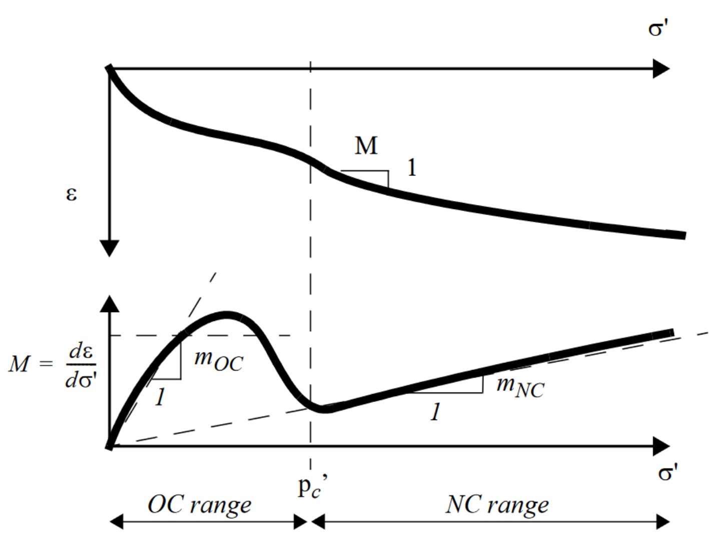

FIGURE 2.6.3.9.1: THE OEDOMETER PARAMETERS AND OEDOMETER MODULUS [16]

The figure below shows the flexibility parameters for Kappa and Lambda for over-consolidated and normal consolidated ranges according to [16].

$$\text{OC range: } \Delta\upsilon = -\kappa\Delta\ln\sigma' = -\kappa\ln\frac{\sigma'}{\sigma'_0}$$

$$\text{NC range: } \Delta\upsilon = -\lambda\Delta\ln\sigma' = -\lambda\ln\frac{\sigma'}{\sigma'_0}$$

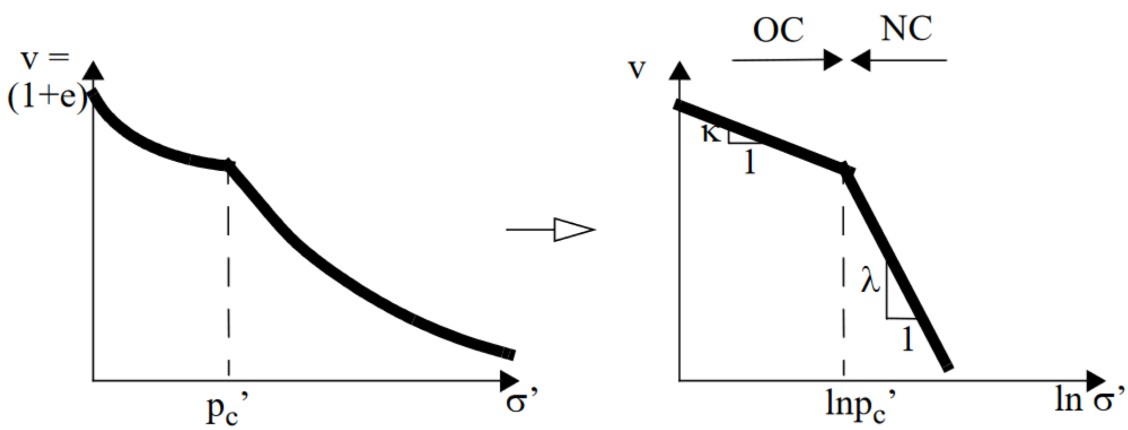

FIGURE 2.6.3.9.2: FLEXIBILITY PARAMETERS FOR KAPPA AND LAMBDA [16]

Where: κ = Kappa

λ = Lambda

σ' = Vertical effective stress

P’ = Mean Effective stress

D υ = Change in specific volume

e = void ratio

OC = Over consolidation

NC = Normally consolidation

Pc’ = Vertical Pre-consolidation pressure

Po= Pre-consolidation pressure

σo ‘ = Initial effective stress

υ = Specific volume

CHAPTER THREE

3.0 RESEARCH METHODOLOGY

3.1 Introduction

The approach adopted in this research work involves the investigation of a section of an ongoing construction project with a beam and slab raft foundation. The building is structurally designed with Prota Structural software and the structural drawings are modeled in Revit Structural software. Therefore, the general arrangement drawings of the buildings would be extracted from the Revit BIM, and a section through the structural floor plan would be investigated.

The building under consideration is a 4-storey structure comprises of a ground floor, first floor, second floor, and a pent floor and to be used for domestic purposes. The superstructure is composed entirely of reinforced concrete structural elements, the suspended floors are schemed in beam and slab option and the columns are all reinforced concrete. The structural floor plans of the building that are extracted from the Revit drawings suite are shown in the figures below:

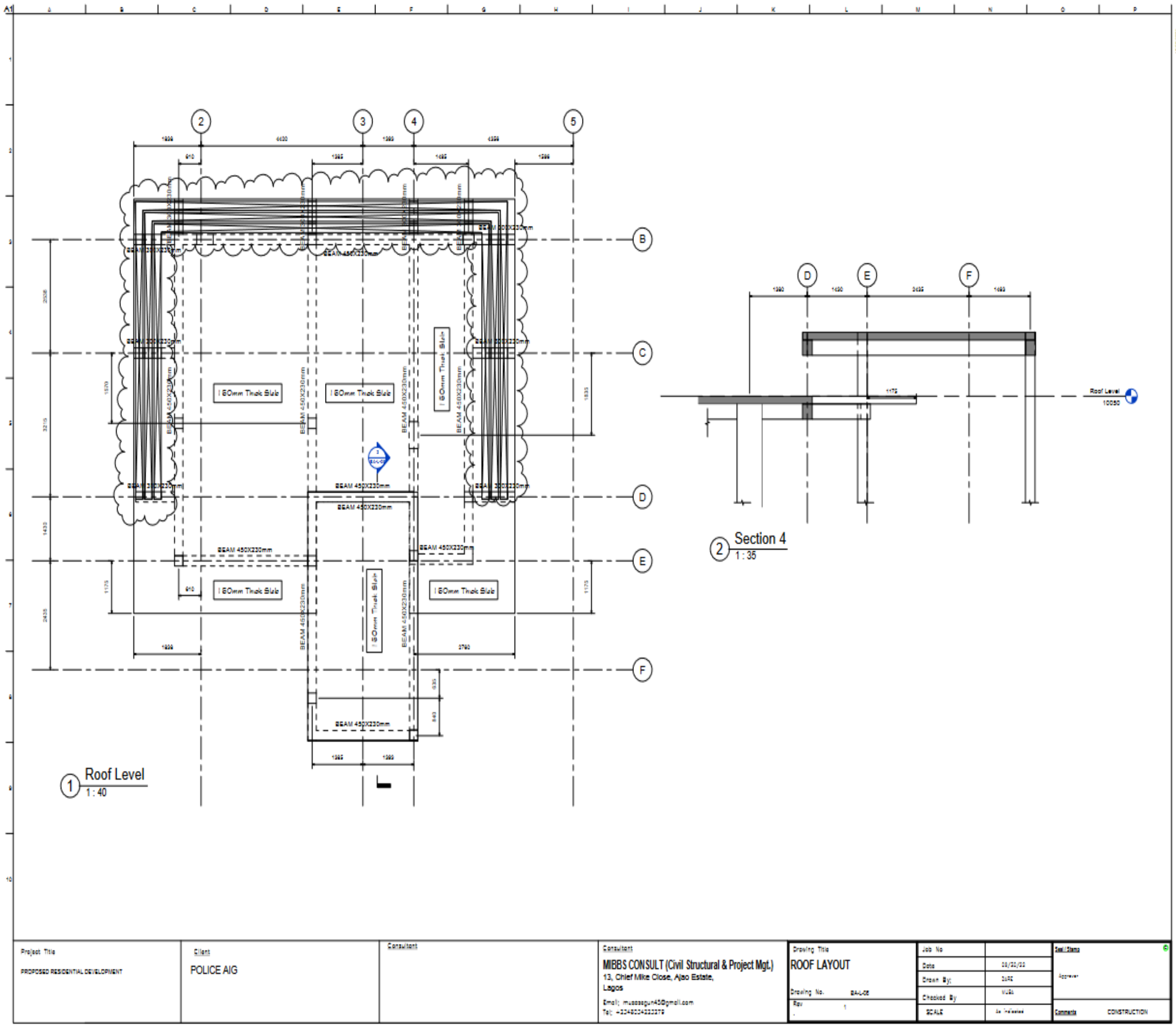

FIGURE 3.1.1: STRUCTURAL PENT FLOOR OF THE INVESTIGATED BUILDING

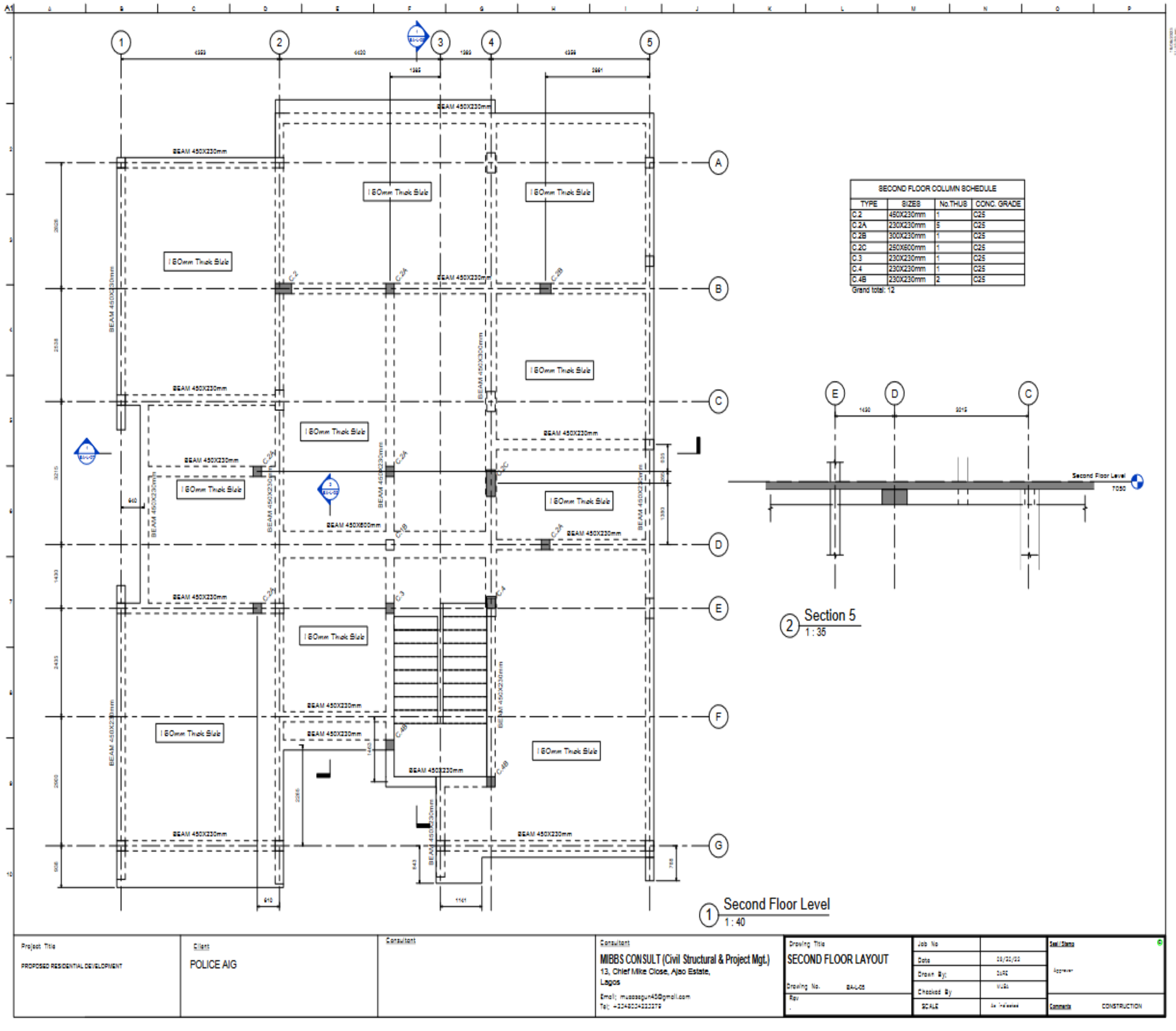

FIGURE 3.1.2: STRUCTURAL SECOND FLOOR OF THE INVESTIGATED BUILDING

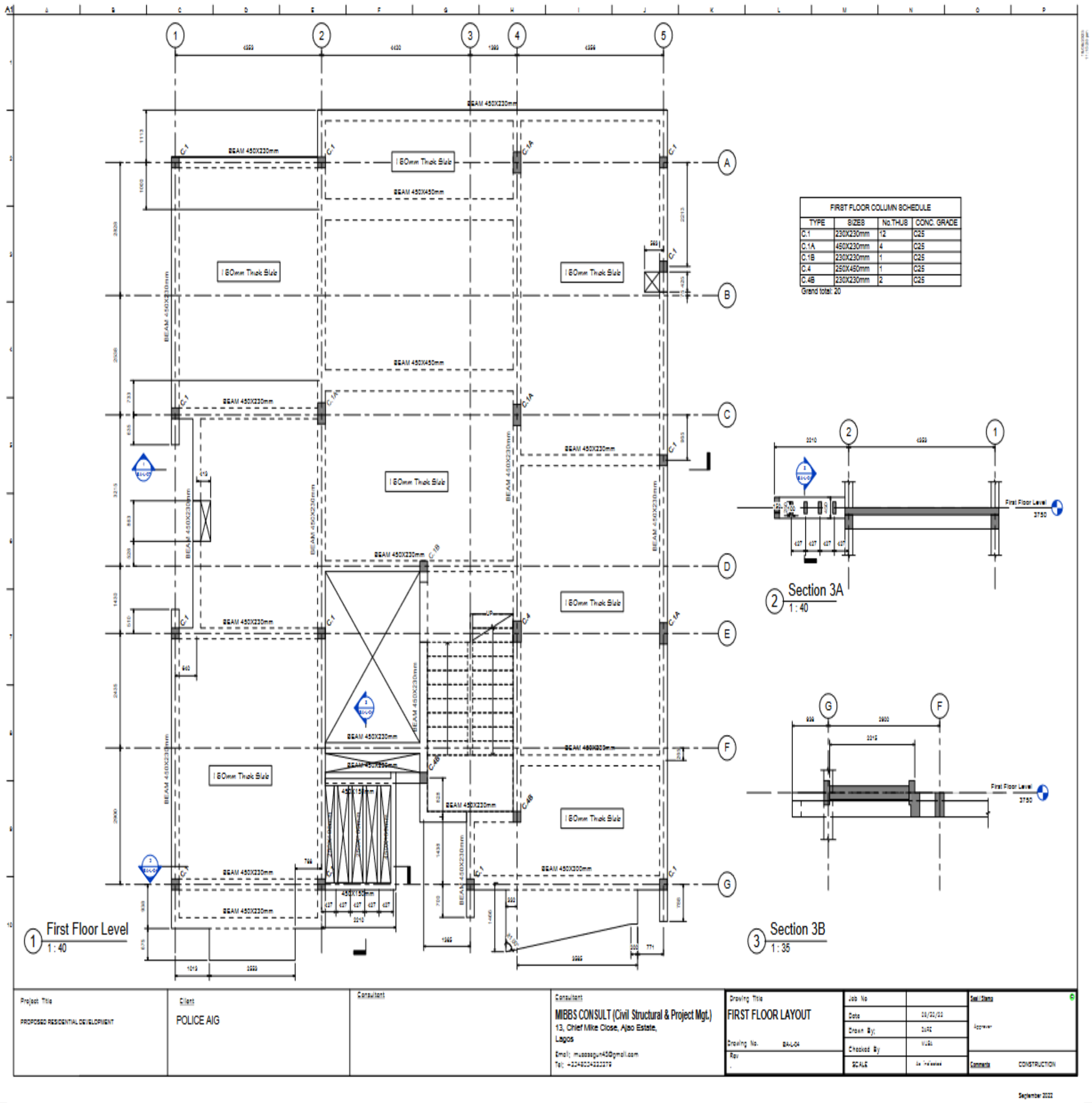

FIGURE 3.1.3: STRUCTURAL FIRST FLOOR OF THE INVESTIGATED BUILDING

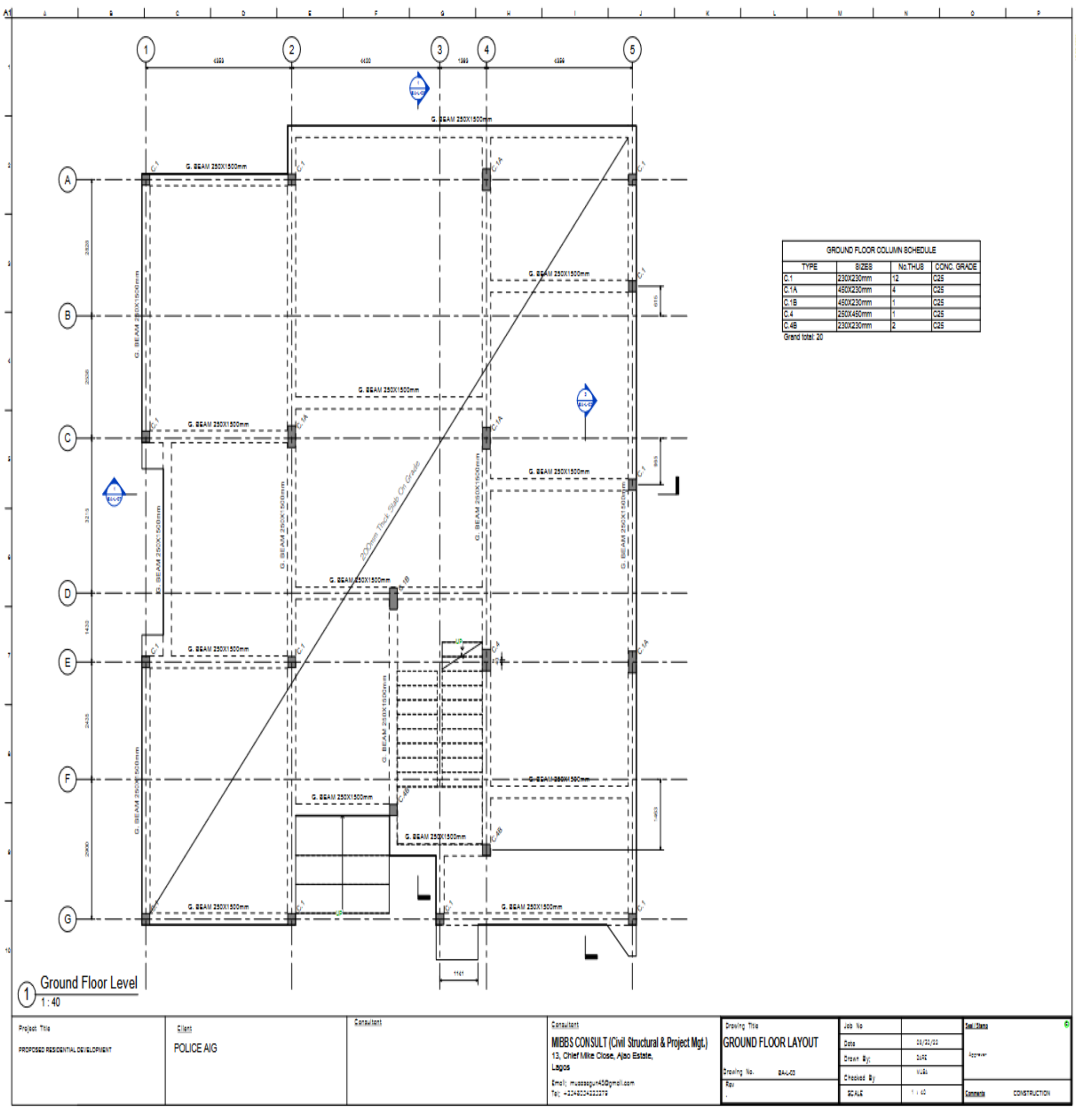

FIGURE 3.1.4: STRUCTURAL GROUND FLOOR OF THE INVESTIGATED BUILDING

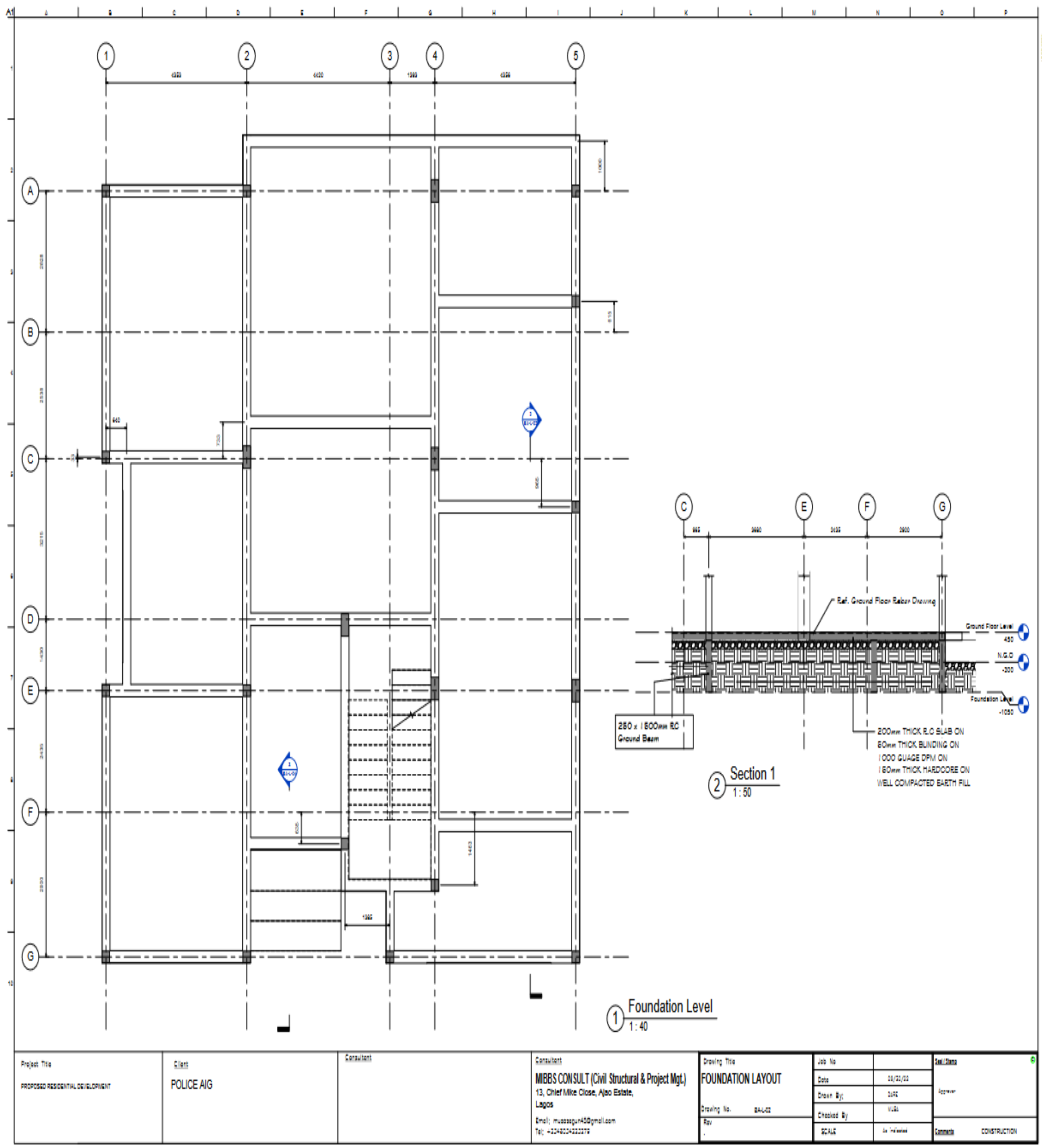

FIGURE 3.1.5: FOUNDATION LAYOUT OF THE INVESTIGATED BUILDING

FIGURE 3.1.6: 3D FRAME OF THE INVESTIGATED BUILDING

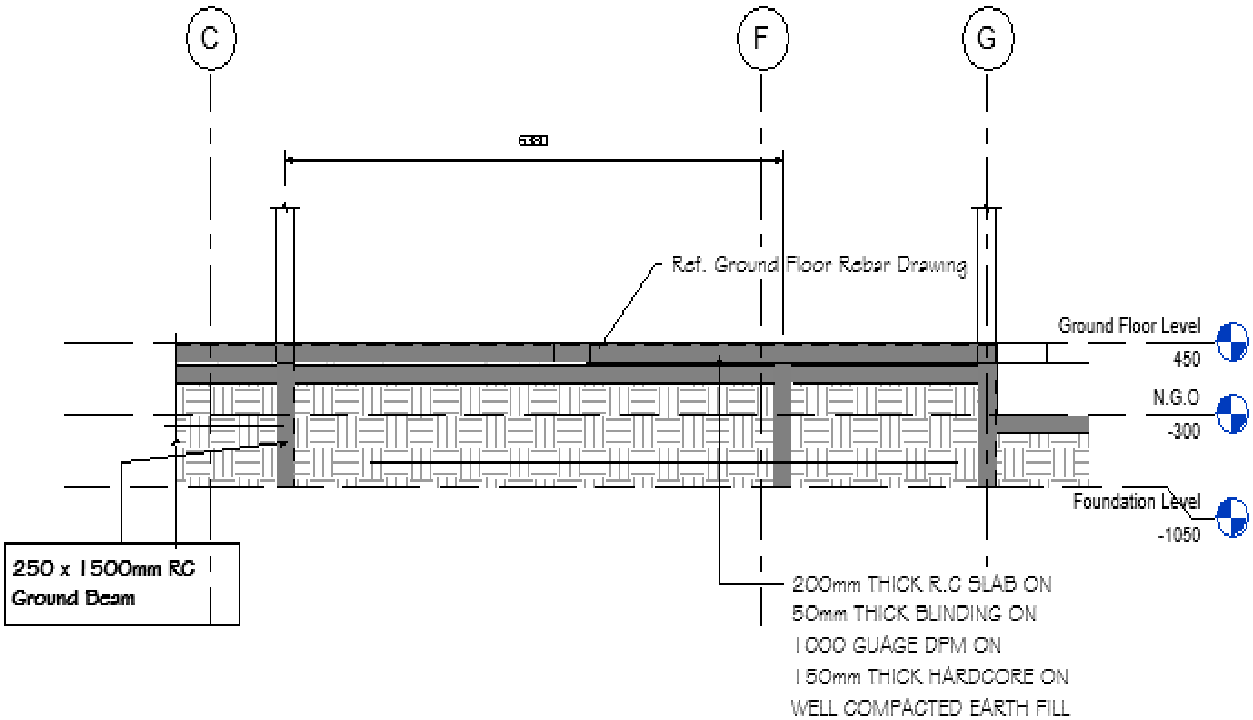

FIGURE 3.1.7: TYPICAL SECTION OF THE FOUNDATION OF THE BUILDING

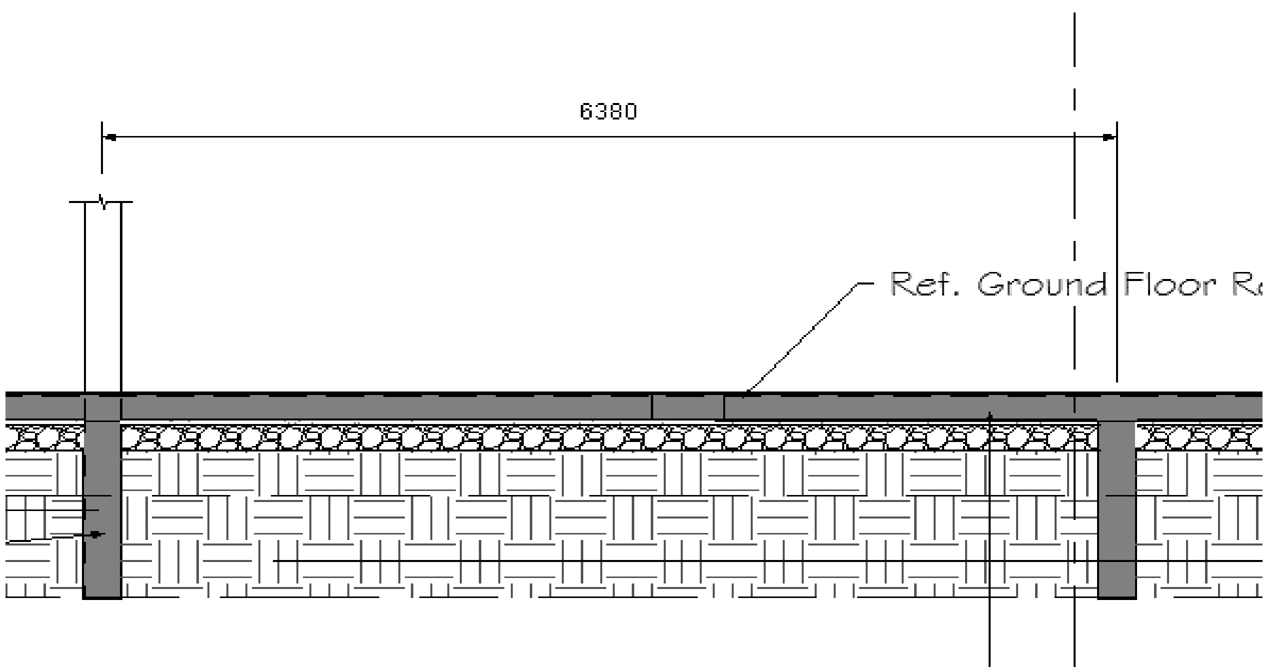

FIGURE 3.1.8: MODEL USED FOR INVESTIGATION (MODEL USED IN PLAXIS FOR INVESTIGATION)

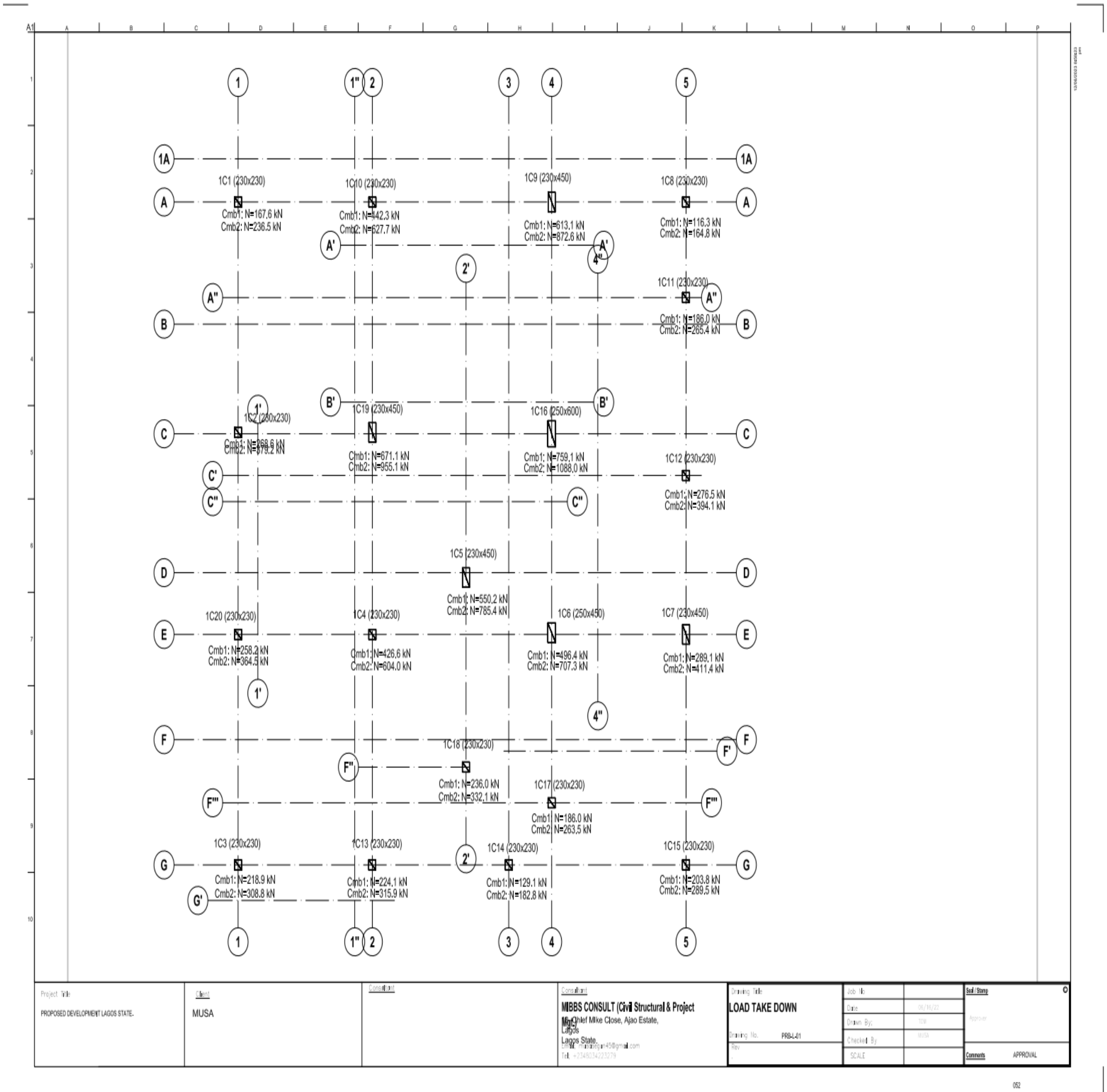

FIGURE 3.1.9: FOUNDATION LOADS (C1_SLS & C2_ULS) FROM SUPERSTRUCTURE (SEE APPENDIX 2 FOR A3 SIZE)

Note: Due to the large scale of the building, the loads in Figure 3.1.9 are not so legible. Therefore, A3 paper size is attached as an appendix, and AutoCAD version is uploaded in Insperra.

3.2 The Geometry of the Raft Foundation

The typical section that would be used for the investigation of this research is shown in Figure 3.2.1 below and it is a section through the largest span of the building.

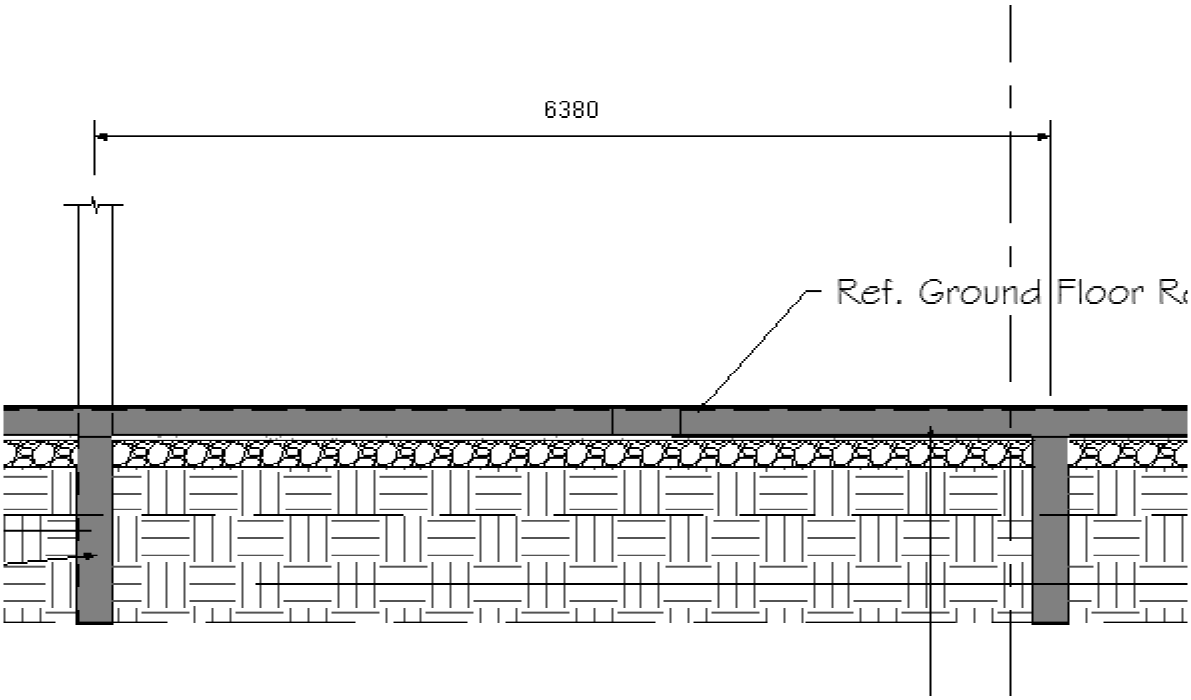

FIGURE 3.2.1: TYPICAL SECTION USED FOR THE INVESTIGATION.

In this instance, the beam is 6380mm apart and of size 1500mm deep and 250mm wide. The slab is an integral part of the depth of the beam; the slab thickness is 200mm thick. The block of a soil sample to be considered during the investigation would be 5000mm in depth since it is infinite beneath the substructure. The compacted earth fill to raise the raft slab above the natural ground level is 250mm, hence making the ground floor to be at +450mm (200mm slab + 250mm filling) above the natural road level.

A fair description of the above illustration can be shown in Figure 3.2.2 below and the same would be modeled in Plaxis 2D for investigation.

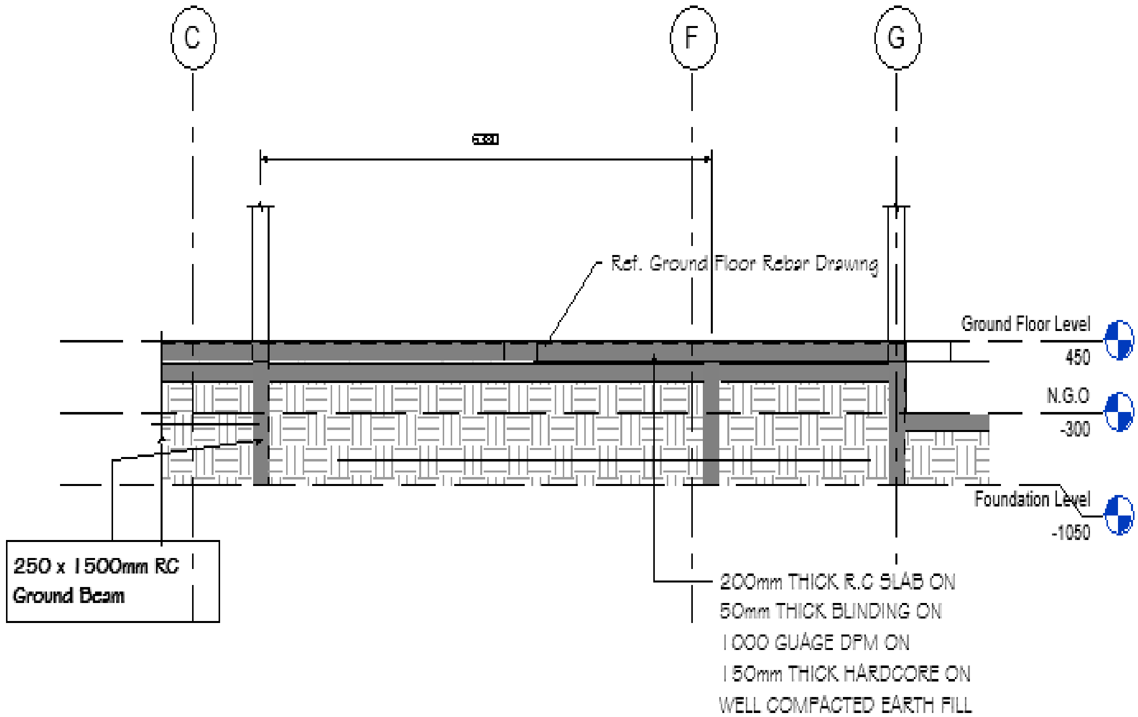

FIGURE 3.2.2: TYPICAL SECTION USED FOR THE INVESTIGATION WITH SEVERAL LEVELS.

3.2.1 Structural Loads on Foundation and Estimation of Surface Pressure

The building was analyzed and designed with Prota design software and the loads take-down (that is the load on the foundation) were directly extracted from the software. A quick alternative to using the loads from the software would be the influence area approach, this approach is commonly called the tributary area of a column by a structural engineer. The tributary area of a column method involves estimating an area over each column (for each floor) and multiplying by the design loads of each floor. The tributary method was not used in this work since there is a structural analysis model from the structural engineer.

Conservatively, the columns load (SLS and ULS) around the pent floor were used for this research work because these columns have higher loads compared to others. Meanwhile, the column loads obtained from the software (or influence area approach) is a point load whilst the load input required by Plaxis 2D is a line load hence there is a need to divide the point load by either the length or width. The blown-up plan of the area with the highest column loads is shown in the figure below:

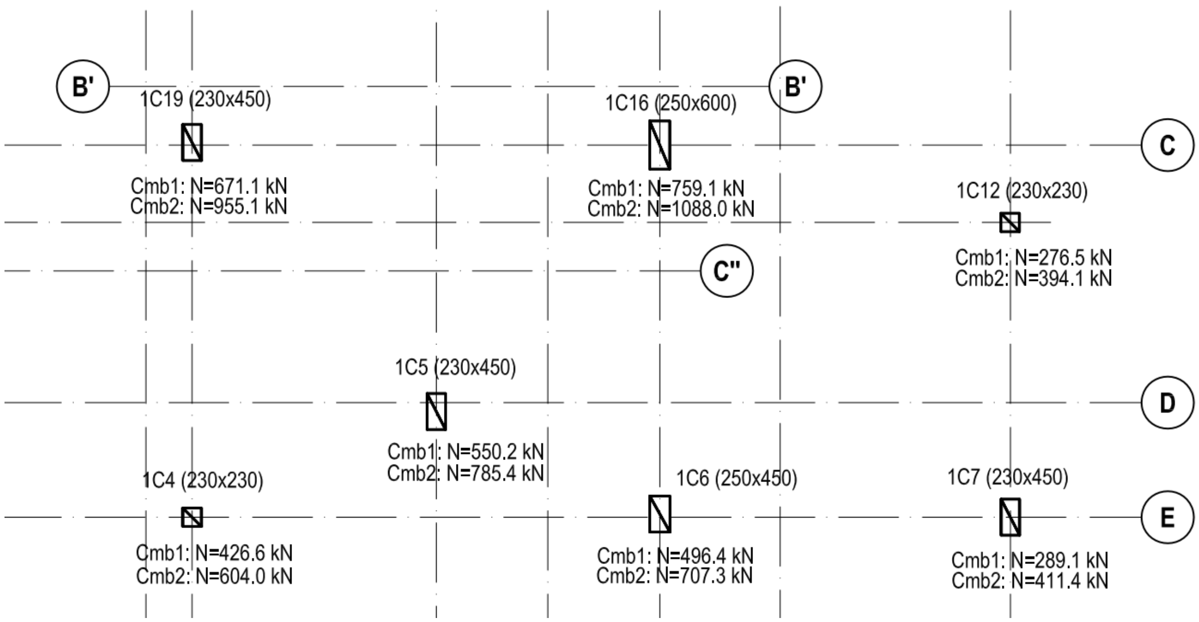

FIGURE 3.2.2A: BLOWN-UP OF FOUNDATION LOADS AROUND THE PENT FLOOR

The terms Cmb1 and Cmb2 in Figure 3.2.2A refer to combination 1 and 2. Combination 1 means serviceability limit state (SLS) loads while combination 2 refers to ultimate limit state (ULS) loads. Since the study involves investigating the foundation, the serviceability limits state load was used because the soil parameters are factored already.

The line load that was used in Plaxis is obtained by dividing each column load by the length/width. The length is uniquely calculated by summing half of the length on both sides of the column.

Estimation of length = 0.5 (Bi +Bii);   Bi / Bii = first length/ second length (on plan)

After consideration of the loads and lengths (measured from AutoCAD) of Figure 3.2.2A, a conservative line load of 170kN/m was obtained and used as the foundation load for this research work.

Using the calculated line load above (170kN/m) and the beam distance apart of (6380mm) as shown in Figure 3.2.1, one could roughly calculate (as a quick check) the surface pressure as follows:

Surface pressure due to foundation load = 170 * 2 (number of loads) / 6.38 = 53.3kN/m2

Slab self-weight = 0.2 * 24 = 4.8kN/m2 (slab is 200mm; unit weight of concrete is 24kN/m3)

Hence, the total surface loads = 4.8 + 53.3 = 58.13kN/m2

Comparing the surface pressure obtained above with the soil investigation extract in Figure 1.1.3, the comparison shows that the surface load (58.13kN/m2) exceeds the allowable bearing capacity of 54kN/m^2 at 1m depth (obtained from Figure 1.13). The implication of this is that an ordinary raft slab of 200mm thick is not sufficient even when the allowable bearing capacity of the soil at 1m depth is used. A thicker raft slab at a depth (depth that is sufficient to mobilize the counter pressure to withstand the exacted pressure of 58.13kN/m^2 by the structure) into the ground would be recommended based on these preliminary checks. The option of having a thicker slab into the ground would result in having a basement, the basement proposal is a concept that is not required by the client hence the reason for the beam and slab raft option.

Meanwhile, a close study of soil reports (referring to Figure 1.1.3) shows that the effect of overburden is not considered. This consideration would increase the allowable bearing capacity presented in Figure 1.1.3. The significance of this increment is not studied.

3.3 The Parameters Input into Plaxis 2D

Plaxis' input parameters are vital to the success of the project as we are aware that for computer program simulations the statement garbage in is garbage out holds. Hence, the input parameters were extensively discussed, and efforts were channeled into obtaining reasonable and fairly accurate values for the chosen parameters. In cases where the parameters were not available, mathematical relationships and parameter correlations were used to establish the unknown parameters.

However, since the input into Plaxis is the function of the soil models used, it will be logical to first discuss the soil model used for the two soils in this investigation. Among the various soil models in Plaxis, two of which were considered are Modified Cam Clay (MCC) and Hardening Soil (HS). Although, these two models are advantageous over the simple models as discussed in Chapter 2. Apart from being advantageous, MCC and HS were also considered based on the available parameters and the mathematical relationships that are available to determine the unknown parameters. The decision to use the two soil models was established after all the required input parameters to Plaxis were completely sorted.

3.3.1 Modified Cam Clay Input Parameters into Plaxis 2D

The modified cam clay was the soil model used to simulate the behaviour of the filling soil (the soil is popularly called Laterite in Nigeria) in the foundation. The filling material-lateritic soil was used to fill the pockets within the beams, from the natural ground level to the ground floor which is 450mm (top of the structural concrete slab) above the natural ground level. For this soil material, an Oedometer test (consolidation test) was carried out on the sample of Lateritic soil used, and the result of the test result was attached to this research work as Appendix 3.

A desk study was carried out in Appendix 3 to understand the behaviour of the soil and to conduct a curve fitting. The curve fitting to the laboratory result was carried out by the Plaxis test simulator, and the curve obtained in the laboratory was compared with the Plaxis' curve. Table 4.0 below is the consolidation test reading obtained from the laboratory.

TABLE 4.00: CONSOLIDATION TEST FOR FILLING MATERIAL (LATERITIC CLAY)

	LOADING					**UNLOADING**		
TIME	**25**	**50**	**100**	**200**	**400**	**50**	**0**	**DP (kN/m²)**
0 sec	0	25	77	160	281	447	375	
16 sec	20	70	151	267	415			
36 sec	21	71	152	269	418			
1mins	21	71	152	269	421			
2. 1/4 mins	22	72	153	270	424			
4mins	22	73	154	271	427			
6. 1/4mins	23	74	155	272	430			
9 mins	23	74	156	273	433			
12. 1/4 mins	24	75	156	274	436			
16 mins	24	75	157	275	438			
25 mins	24	76	157	276	440			
36 mins	24	76	158	277	442			
49 mins	25	76	158	278	444			
64 mins	25	77	159	279	445			
81 mins	25	77	160	280	446			
100 mins	25	77	160	281	447	375	290	

Table 4.0 is the raw data that was obtained from the laboratory whilst some parameters were calculated from the laboratory data and tabulated in Table 5.0. The formulae required in developing Table 5.0 were highlighted in the right column of the Table and sample calculations were also displayed beneath each value.

TABLE 5.00: OUTPUT OF MATHEMATICAL COMPUTATION CONDUCTED ON TABLE 4.0

Ring height =	18.6	mm	e_o =	0.894		$(1+e_o)/H_o)$=	0.102
ring correction factor	0.002		Ho =	18.6			
Dh (change in height) in mm =Change final * 0.002	0.05 (25*0.002)	0.154 (77*0.002)	0.32 (160*0.002)	0.562 (281*0.002)	0.894 (447*0.002)	0.75 (375*0.002)	0.58 (290*0.002)
Final Height hf in mm =18.60- Dh	18.55 (18.6-0.05)	18.446 (18.6-0.154)	18.28 (18.6-0.32)	18.038 (18.6-0.562)	17.71 (18.6-0.894)	17.85 (18.0-0.75)	18 (18.0-0.58)
Initial Height in mm	18.6 (18.6-0.0)	18.55 (18.6-0.05)	18.446 (18.6-0.154)	18.28 (18.6-0.32)	18.04 (18.6-0.562)	17.706 (18.6-0.894)	17.9 (18.6-0.75)
DH in mm	0.05 (18.6-18.55)	0.104 (18.55-18.44)	0.166 (18.446-18.28)	0.242 (18.28-18.038)	0.332 (18.04-17.71)	-	-
M_v (m^2 /MN) = (DH * 1000/ (Hf * DP))	0.108	0.224	0.182	0.132	0.094		
De (change in void)= $((1+e_o)/Ho)$ * Dh)	0.005	0.016	0.033	0.057	0.091	0.076	0.059
e = e_o -De	0.889	0.878	0.861	0.837	0.803	0.818	0.835
C_v = 0.111* Haverage square)/ t	NA	NA	NA	NA	NA	NA	NA
log Dσ	1.398	1.699	2.000	2.301	2.602	1.699	

The first attempt was to plot the logarithm of the change in stresses versus the void ratio from Table 5.0 above and study the loading and unloading behaviour of the soil sample. Hence, the graph of the logarithm change in stress versus void ratio is then shown in Figure 3.3.1.2 below:

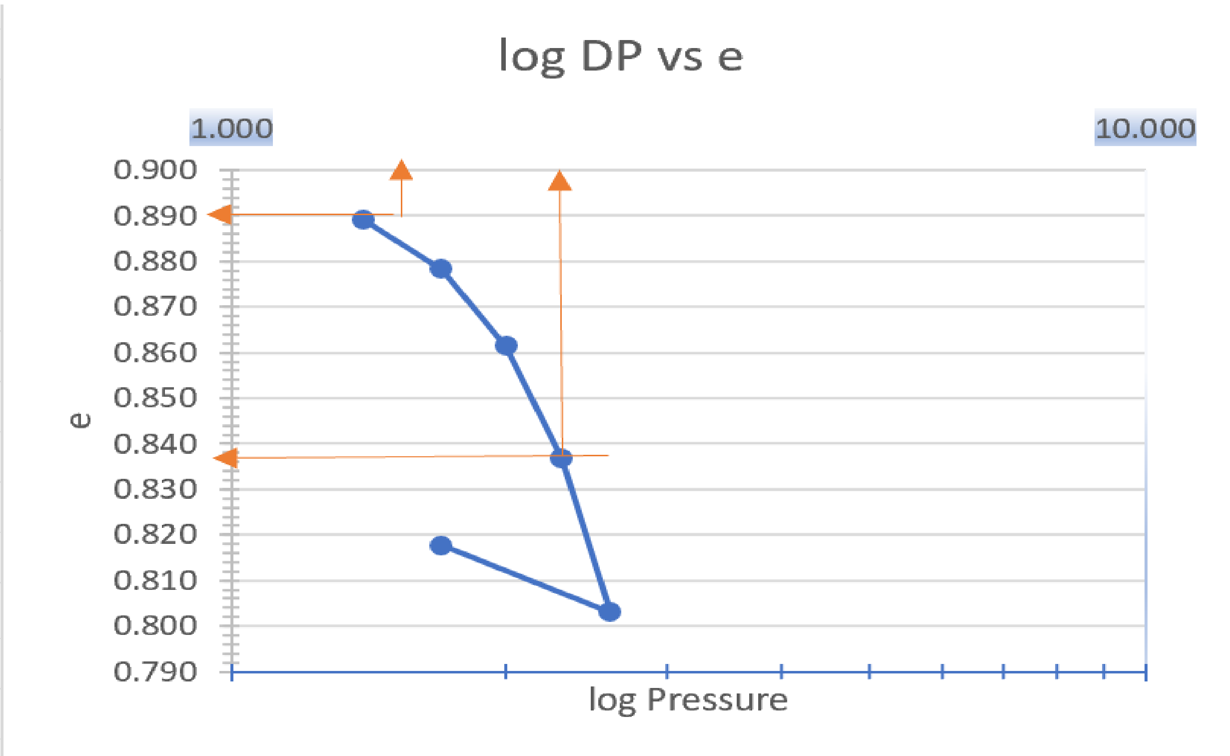

FIGURE 3.3.1.2: LOGARITHM OF CHANGE IN STRESS VERSUS VOID-RATIO

Mathematically, the compression index (Cc) in consolidation (loading curve) is expressed as given in the equation below:

$$C_c = e_0 - e_1 / \log (p_1 / p_0) \qquad \textbf{Eqn. 19.0}$$

The formular in the above is synonymous to the slope of the above graph. The slope of the graph is evaluated as:

$$\text{Slope} = C_c = (0.89 - 0.835)/ (4 - 1.8) = 0.025$$

Similarly, the recompression index (Cr) obtained along the unloading curve is evaluated by taking the slope along the same curve (unloading curve) and estimated to be Cr = 0.016.

Alternatively, the compression and recompression indices are mathematically investigated further by determining the oedometer or the modulus at each stress and strain point. The Oedometer modulus is denoted as M which is determined by dividing the change in stress by the change in strain at each loading step.

The change in stress is denoted as **Dσ** and is the average of the summed stress at loading steps 1 and 2 that is **Dσ** = ((0 + 25) * 0.5)) = 12.5kN/m^2, same with 37.5, 75 etcetera. Thus, the result of the change in stress and change in strain is tabulated in Table 6.0 below:

TABLE 6.0: EVALUATION OF CHANGE IN STRESS AND CHANGE IN STRAIN

σ	0.00	25.00	50.00	100.00	200.00	400.00	50.00	0.00
ε	0.00	0.0027	0.0083	0.0172	0.0302	0.0481	0.0403	0.0312
Dσ	12.50	37.50	75.00	150.00	300.00	225.00	25.00	0.00
M= dσ/dε	9300.0	4471.2	5602.4	7686.0	11204.8	45208.3	5470.6	
m=dM/dσ		30.17	27.78	23.46		198.69	218.82	
m = M/σ		119.23	74.70	51.24		200.93	218.82	

where M = change in stress / change in strain

Note that m was calculated as the change in M divided by the change in stress or direct value without change that is M divided by the stress. The calculated m was therefore used for calculating the slope of the normal compression line (λ) and slope the of unloading – reloading line (κ) by first evaluating the normal compression index (λ *) and unloading index (κ *).

λ * was determined by taking the inverse of the average of the values of loaded m calculated above. Sample calculations are shown below:

Loaded λ * = 1/ ((30.17+27.78+23.46)/3)) = 0.03685 for m=dM/dσ

And loaded λ * = 1/ ((119.23+74.70+51.24)/3)) = 0.01224 for **m = M/σ**

Whilst κ * was determined by taking the inverse of the average of the unloaded m. Sample calculations are also shown below:

κ * (unloaded m) = 1/((198.689+218.82)/2) = 0.0047903 for m=dM/dσ

κ * (unloaded m) = 1/ ((200.926+218.824)/2) = 0.0047647 for **m = M/σ**

The relationship between the κ *, κ and void ratio was then used to determine κ and as well as λ. The connecting relationship between κ *, κ and, void ratio is expressed in the equations below:

From $\lambda * = \frac{\lambda}{1+e}$ **Eqn. 20.0**

Then $\lambda = \lambda * (1 + e)$ **Eqn. 21.0**

Similarly, $\kappa = \kappa^* (1 + e)$ **Eqn. 22.0**

From equation 23, $\lambda = \lambda * (1 + e)$ = 0.01224 * (1 + 0.894)

where e_o = 0.894 as obtained for laboratory

Therefore, $\lambda = 0.023$

And $\kappa = \kappa^* (1 + e)$ = 0.0047647 (1 + 0.894) = 0.009

Finally, the Cc and Cr were computed based on the lambda * and Kappa * calculated above as follows:

NOTE: $\mathbf{\ln 10 = 2.3}$

$C_c = \lambda * \; * 2.3 * (1 + e_o)$ = 0.03685 * 2.3 * (1+0.894) **= 0.16053** for when $\lambda * = 0.03685$

Similarly, $C_c = \lambda * \; * 2.3 * (1 + e_o)$= **0.0533** for when $\lambda * = 0.01224$

Also, the Cr was determined equivalently but κ^* values were used instead.

Cr = κ^* * 2.3 * (1+ e_o) = 0.0047903 * 2.3* (1+0.894) = **0.0208675** for κ^* = 0.0047903

Similarly, **Cr = 0.027562** for κ^* = 0.0047647

To determine the loading and unloading stiffness from the same laboratory test, the graph of change in M (i.e., dσ/dε) against Dσ is plotted from Table 6.0 data and shown in Figure 3.31.3 below:

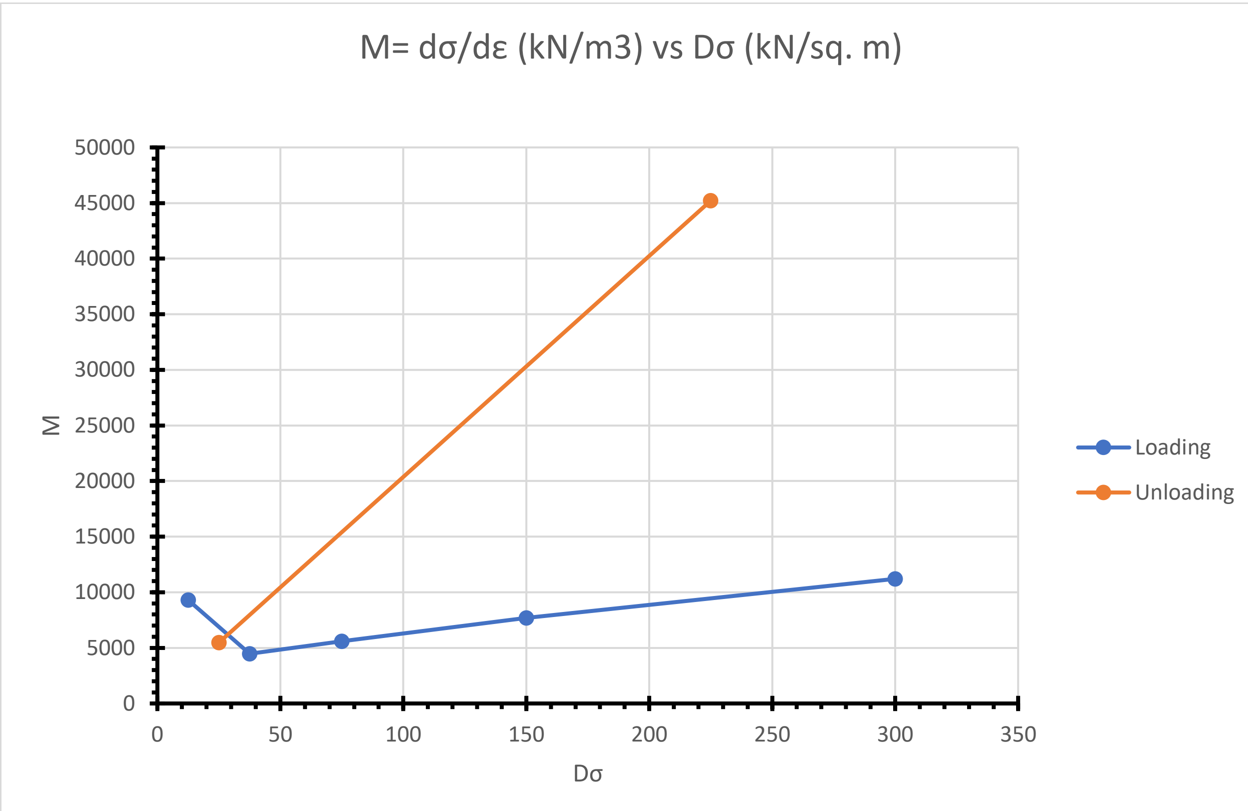

FIGURE 3.3.1.3: GRAPH OF M VERSUS CHANGE IN STRESS

From the loading and unloading graph above, the maximum load stiffness is approximately 11000kN/m^3 (This stiffness can be adopted for embarkment problems) while the unloading is 45000kN/m^3 (used for excavation problems). This increasing value of the unloading stiffness is because soil is not linearly elastic hence, the deformation during loading remains permanent and does not recover during unloading. This high unloading stiffness is synonymous with what is experienced during the excavation process (higher energy is expended during excavation) and it is generally believed in some literature that the unloading stiffness for sand is approximately three times higher than the loading stiffness whilst the unloading for clay is approximately five times higher than the loading.

Table 7.0 below is an extract from the above table 6.0, where the strain for each applied stress was calculated by a change in height divided by the original height. Hence, the stress versus strain table below is determined based on the applied stress and calculated strain.

TABLE 7.0: STRESS VERSUS STRAIN

σ	0.000	25.000	50.000	100.000	200.000	400.000	50.000	0.000
ε	0.000	0.003	0.008	0.017	0.030	0.048	0.040	0.031

From the above table, the graph of stress versus strain was plotted as indicated below and the slope of the loading curve is determined. The slope of the graph which gives the Oedometer stiffness is calculated as:

$$E_{oed} = (200 - 100)/(0.0285 - 0.0185) = 10{,}000\text{kN/m}^2$$

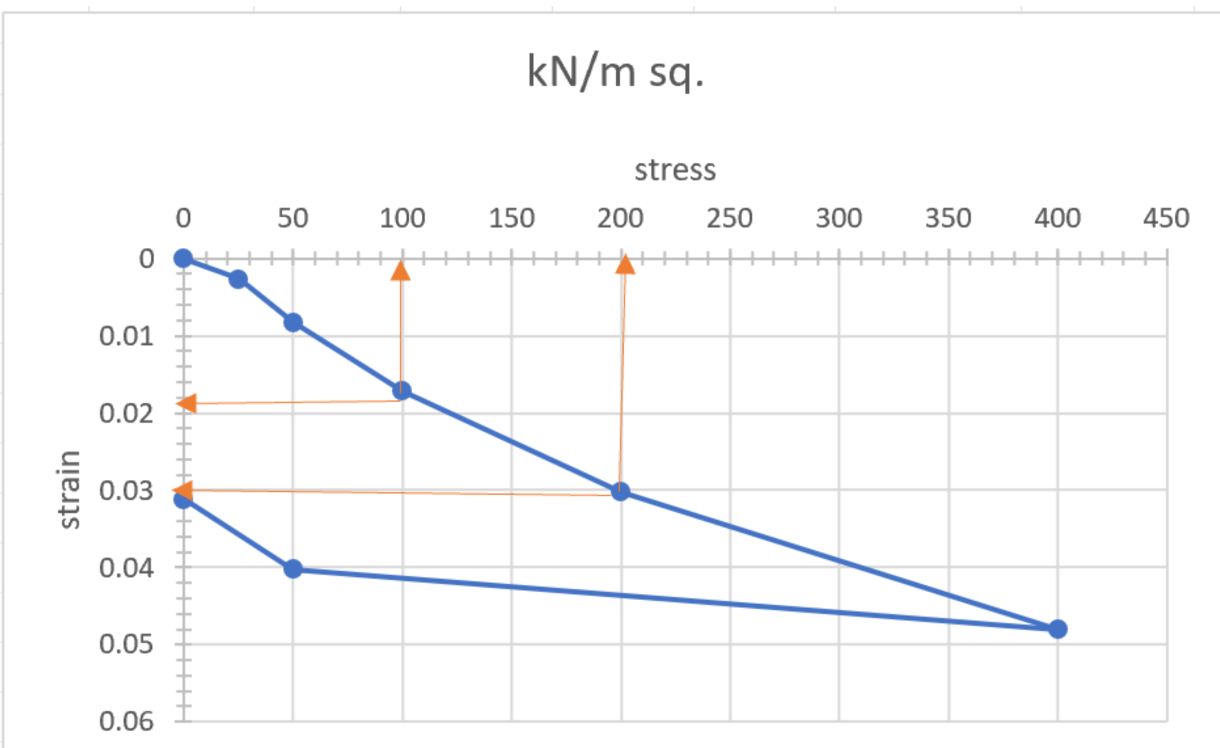

FIGURE 3.3.1.4: STRESS VERSUS STRAIN CURVE TO EVALUATE THE OEDOMETER STIFFNESS

For comparison purposes, there was the need to simulate the above test in Plaxis. The same stress increments used in the laboratory were adopted in Plaxis. The stress in the Plaxis is loaded incrementally as shown in Figure 3.3.1.5 below and the vertical preconsolidated stress was iterated until there was somewhat resemblance with the curve obtained in the laboratory result. The vertical preconsolidated stress of 40kN/m^2 gave a strain of about 5% as equally obtained in the laboratory plot of Figure 3.3.1.4.

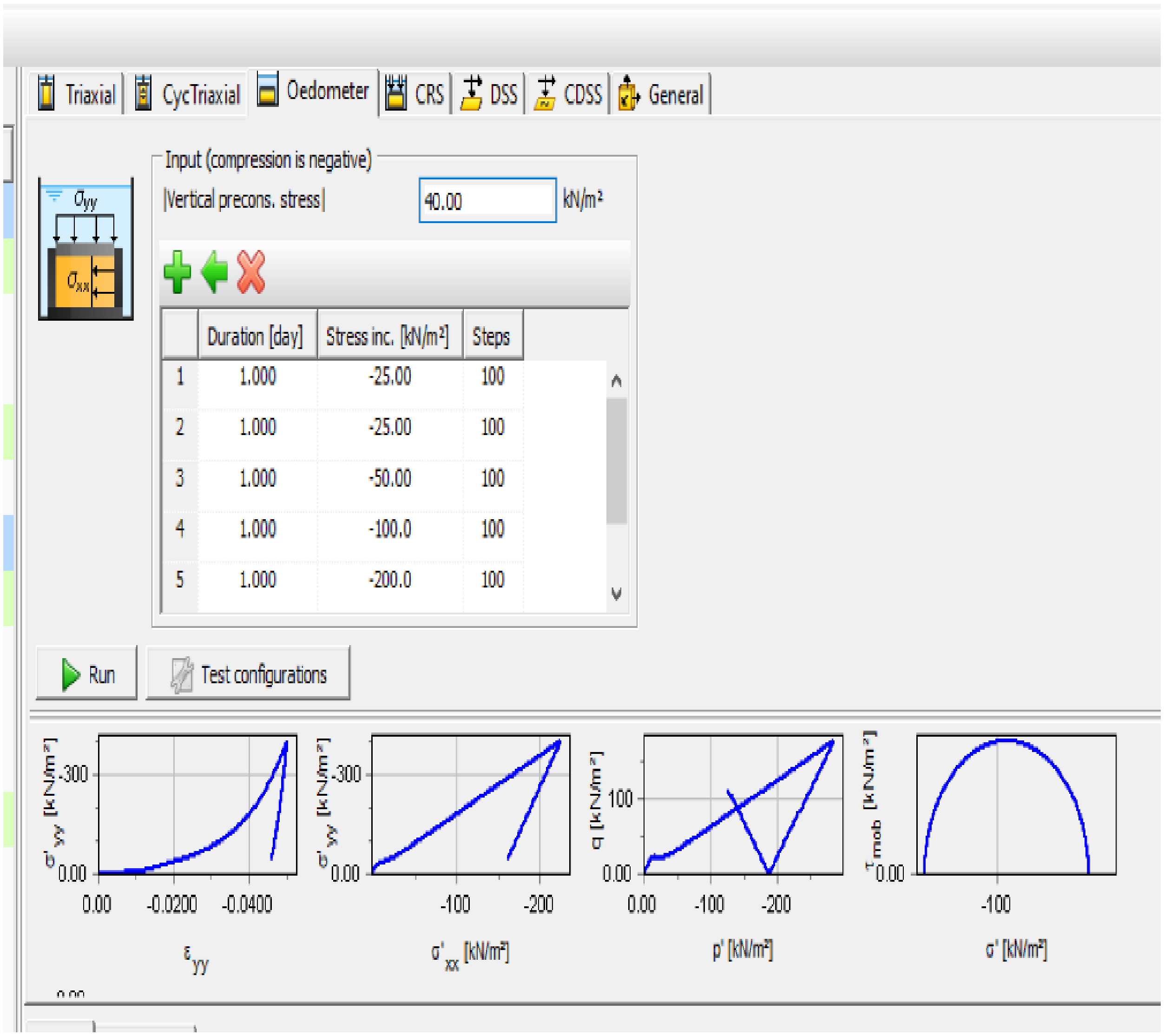

FIGURE 3.3.1.5: INPUT INTO PLAXIS TO SIMULATE THE OEDOMETER TEST

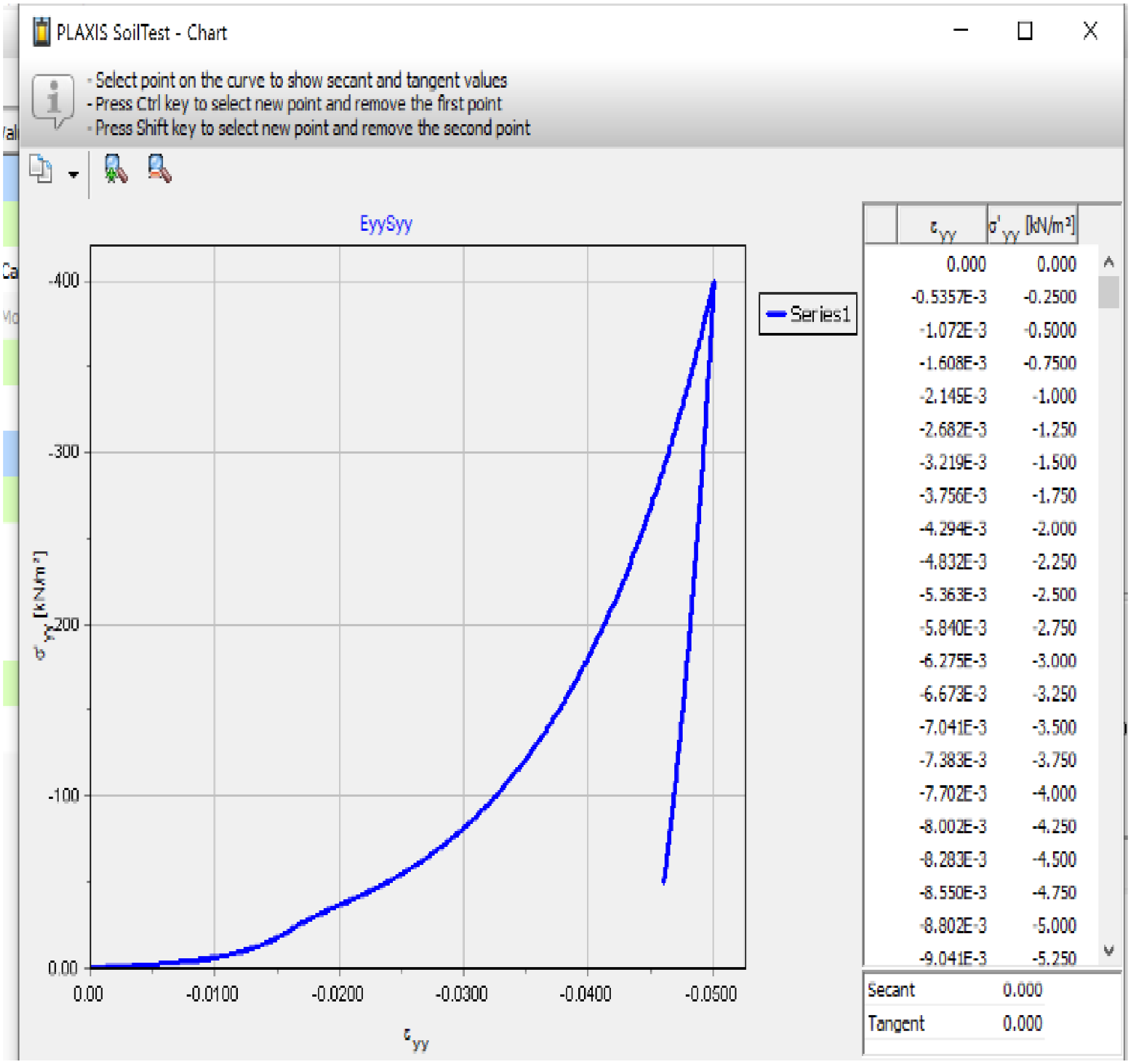

FIGURE 3.3.1.6: STRESS_ STRAIN CURVE OBTAINED FROM PLAXIS OEDOMETER TEST

By superimposing both the laboratory and Plaxis plots, there were noticeable differences thus, there was the need to adjust Kappa and Lambda so that the curve could fit with the laboratory curve. In Figure 3.3.1.6A below, the Kappa and Lambda used to generate the Plaxis curve were 0.0018 and 0.033 respectively. Therefore, Figure 3.3.1.6A below shows the plot when Kappa and Lambda were adjusted (to 0.0018 and 0.033 to match with the laboratory curve) and superimposed on the laboratory result. For this project work, the calculated Kappa (0.009) and

Lambda (0.023) were used as the adjusted values of Lambda and Kappa were determined at the end of the research work.

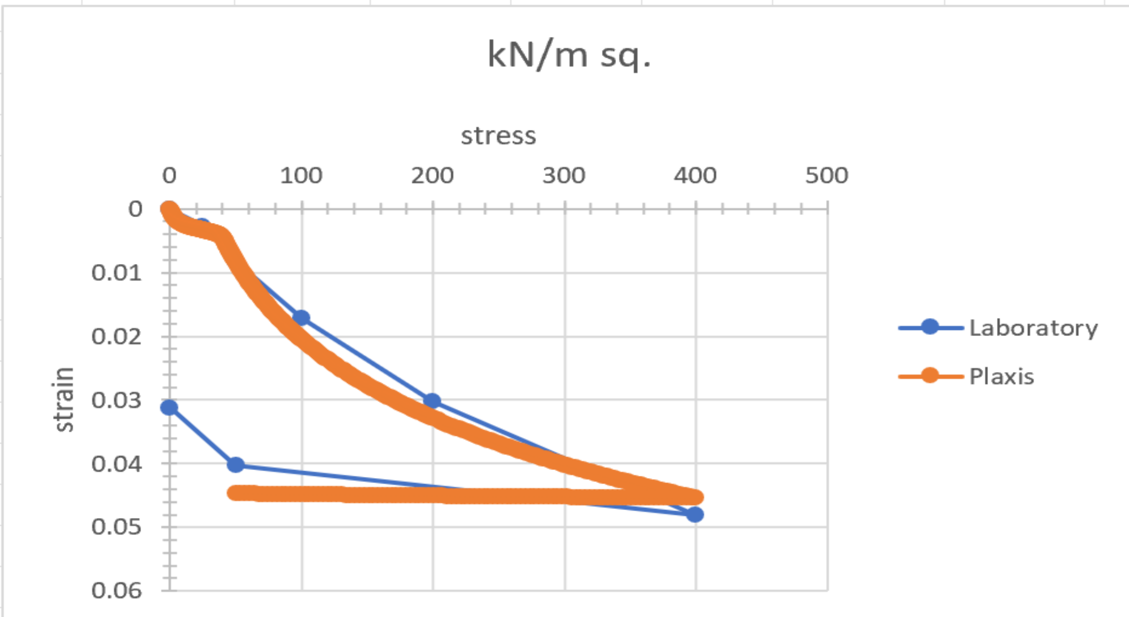

FIGURE 3.3.1.6A: STRESS_ STRAIN CURVE OBTAINED FROM PLAXIS AND OEDOMETER (LABORATORY) TEST

The moving critical state line Mcsl is required input in Plaxis when using the Modified cam clay model and this is calculated based on equation 26 below:

$$Mcsl \ = \frac{6sin\varphi}{3\pm sin\varphi} \qquad \textbf{Eqn. 23.0}$$

A positive sign in the above formula is adopted for expansion and a negative sign is adopted for compression. Since our experiment is consolidation which is compression hence the need to set the formular to as expressed below:

$$\text{Mcsl} = \frac{6sin\varphi}{3-sin\varphi} \qquad \textbf{Eqn. 24.0}$$

Setting φ=30 Mcsl becomes 1.2

Therefore table 8.0 presented all the parameters used for the modified cam clay model in Plaxis as follows:

TABLE 8.00: SUMMARY OF THE INPUT PARAMETER FOR MODIFIED CAM CLAY (MODEL TYPE IS DRAINAGE)

S/no	Parameters	Values
1	Λ (calculated)	0.023
2	K (calculated)	0.009
3	υ_{ur}	0.15
4	M_{csl}	1.2
5	Cohesion ref (C'_{ref})	10
6	φ_{inter}	30
7	Dilatancy ψ	0
8	γ_{sat}	20
9	γ_{unsat}	18.6
10	void ratio e_0	0.894

3.3.2 Hardening Soil Input Parameters into Plaxis 2D

In this section, the soil investigation report was referred to and the full report is attached and tagged as Appendix 1. The essential parameters extracted from the soil investigation report conducted on site were itemized in the table below:

TABLE 9.00: PARAMETERS EXTRACTED FROM THE GEOTECHNICAL SOIL REPORT

S/no	Parameters	Values
1	SPT N between 0.75 - 3.0m	6
2	Soil bulk density γ_b	$1.89Mg/m^3$
3	Moisture content	19%
4	Initial Void ratio e_o	0.709
5	Undrained shear strength C_u	$67kN/m^2$
6	Phil (φ $u_{ndrained)}$	12 degrees

The soil parameters in Table 9.00 are the parameters presented in the geotechnical soil investigation report and these parameters were used to obtain other parameters that were necessary to use the hardening soil model in Plaxis. Correlation relationships (in the form of empirical formula and graphs) between the parameters presented in Table 9.0 and other parameters required in Plaxis (for hardening soil model) were also determined using either empirical formulae or graphs. The correlation relationships used to obtain the required parameters are discussed below.

3.3.2.1 Determination of Shear Wave Velocities from SPT_N

For instance, the shear wave velocity (Vs) was obtained from the research work of [19] where SPT is related to the kind of soil deposit and shear wave velocities. The soil investigation report states that the soil deposit is Pliocene soil hence the graph below was used to obtain Vs.

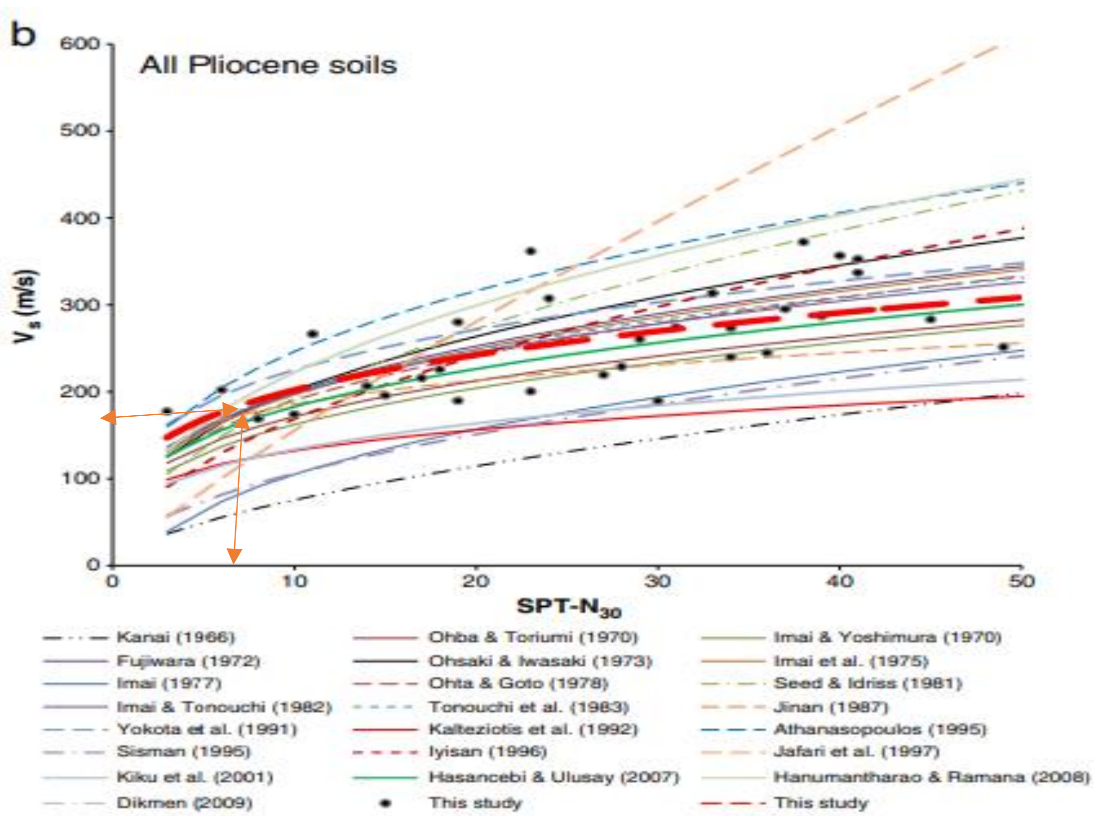

FIGURE 3.3.2.1.1: EMPIRICAL RELATIONS BETWEEN SPT $_N$ VERSUS VS [19]

From the graph above, the shear wave velocity Vs is fairly extrapolated to be about **180m/s** (as roughly indicated by the orange arrow in the above figure-that is SPTn is six (6) versus the study red line between investigated points in black in Figure 3.3.2.1.1).

3.3.2.2 Determination of Shear modulus from SPT_N

The relationship between the shear modulus G_{ref} and SPT_N was obtained from the research work of [20]. [20] formulated an empirical formula that relates the shear modulus and standard penetration values. This empirical formular is given in the equation below:

$$Gref \ = \ 19.43 * N^{0.51} \qquad \textbf{Eqn. 25.0}$$

Substituting SPT_N into equation 28.0 gives:

$$Gref \ = \ 19.43 \ * \ 6^{0.51} \ = \ 48.50\text{MPa}$$

Gmax can be calculated by multiplying the bulk density of the soil and the shear wave velocity calculated above. Hence Gmax becomes:

$$Gmax \ = \ Vs \ * \ \gamma_b \ = \ 180\text{m/s} * 18900\text{g/m3} \ = 3.4\text{MPa}$$

3.3.2.3 Determination of E_{ur}, $E_{50}{}^{ref}$ E_{oed} from G_{ref}

The relationship between unloading and reloading stiffness is related by the equation below:

$$Gref \ = \frac{Eur}{2(1+\upsilon)} \qquad \textbf{Eqn. 26.0}$$

υ posion ratio taken as 0.2 and G^{ref} calculated above as 48.50MPa.

Hence the unloading and reloading stiffness equals:

$Eur = Gref * 2(1 + \upsilon)$ = 48.50 * 2*(1+0.2) = 116.4MPa.

The E_{50}^{ref} is taken as one-third of E_{ur}, hence:

$E_{50}^{ref} = 1/3 * 116.4 = 38.8MPa$

The oedometer stiffness can be computed from the equation below:

$$Eoed = M = \frac{E\,(1-\upsilon)}{(1+\upsilon)\,(1-2\upsilon)} \qquad \textbf{Eqn. 27.0}$$

Substituting E_{50}^{ref} = 38.8MPa and υ= 0.2 E_{oed} =M = 42.680MPa

The summary of all the parameters needed for hardening soil model is itemized in Table 10 below:

TABLE 10.00: SUMMARY OF THE INPUT PARAMETER FOR HARDENING SOIL MODEL (MODEL TYPE IS DRAINAGE)

S/no	Parameters	Values
1	E_{50}^{ref}	38.80 *1000kN/m^2
2	E_{oed}^{ref}	42.68*1000 kN/m^2
3	υ_{ur}	0.2
4	P_{ref}	100 kN/m^2
5	Cohesion ^{ref} (C' ^{ref})	67kg/m^2
6	φ_{inter}	12 degrees
7	Dilatancy ψ	0
8	γ_{sat}	20kN/m^3
9	γ_{unsat}	18.9kN/m^3
10	void ratio e_0	0.709
11	E_{ur}^{ref}	116.4*1000
12	Power (m)	0.7

3.3.3 Sub-structure (Concrete) Parameters into Plaxis 2D

Though there are various options in Plaxis to model several structures which makes its interphase user-friendly. Plaxis is considered in my opinion to be a rigid and versatile software for safety design purposes. Just like every other software, there is a need to always consider soil-structure interactions and other governing parameters that may likely affect the output of the design.

At the beginning of the project, there was an attempt to use plates to model the concrete beams and slab because of the ease of assigning the stiffnesses (EA and EI) of the concrete to the adopted plate. If the plate option were used the EI for the 250mm wide beam would have been 40.364 * 10^9 kNm^2/m using this formula (E * 0.08333 * b*h^3), where E = Ecm as specified in Table 3.10 of EC2 [21] which is this case is 31GPa. Table 3.10 of Eurocode 2 expressed E as a function of the compressive strength of concrete and since the fck used for the substructural element is $25N/mm^2$ hence E becomes 31GPa. Whilst EA will be equal to 7.75 *10^6 kN/m (using E * b*h) with b =1m. A similar formular is used to estimate the slab stiffness parameters.

Although the plate approach was unable to complete the analysis (when analyzed in Plaxis), the analysis was forced to run when the punching option (this option is available in Plaxis) was checked. The image of the plate model option is shown in Figure 3.3.3.1 below:

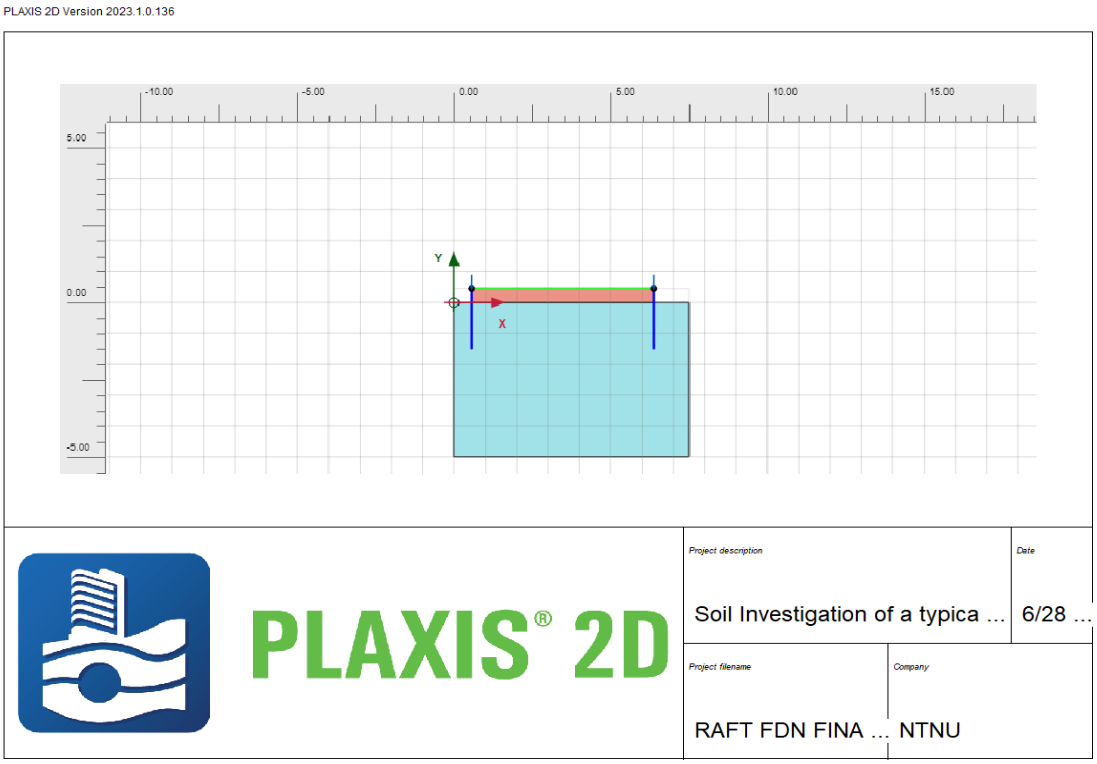

FIGURE 3.3.3.1: IMAGE OF DISCARDED PLATE OPTION OF THE PLAXIS MODEL

However, the approach of the plate option was jettisoned after consideration and tutelage as to what the effect of preventing punching option in Plaxis would be on the settlement of the structure. Another reason to discard the plate option is because the vertical plates for the beams are two-dimensional systems that were connected to a single node at the tip end of the plate. This therefore results in a singular point (a nodal point) making the contact stress between the plate and the soil to have high stresses. This higher stress is due to the nodal contact caused by the plate. Meanwhile, the concrete beam geometry is a three-dimensional element and would not possess a nodal property when in contact with the soil.

The nodal tip at the end of the plate is believed to further aggravates the tip stresses of the plate (shown by the Plaxis result before being discarded) which was the main objective of the research work (that is investigating the effect of the concrete beam which even has a wider contact area compared to the plate with a nodal contact behaviour).

To prevent the tip problem that may arise from using the plate option, a soil cluster was adopted. In this method, a soil volume of exact reinforced concrete beam dimensions was used to model the concrete for the beams and slab. This soil volume is not necessary to model the slab but was used for consistency and ease then, the concrete material properties were assigned to the soil volume.

This soil cluster option enables the Plaxis model to behave like concrete with three-dimensional in property and helps to avert the high tip stresses/pressure that was encountered when the plate option was adopted. Hence, the Plaxis model is remarkably close to what was constructed on-site, and the result can be somewhat close to reality. The summary of the parameters used for the soil volume is presented in Table 11 below:

TABLE 11.00: SUMMARY OF THE INPUT PARAMETER FOR THE CONCRETE STRUCTURAL MODEL

S/no	Parameters	Values
1	Soil model: Linear Elastic	
2	Drainage type: non-porous	
3	Unsaturated weight	24kN/m^2
4	E_{ref} (from EC2 based on fck=25N/mm^2)	31*10E6 kN/m^2
5	Poison ratio υ (nu)	0.25

CHAPTER FOUR

4.0 PLAXIS GEOMETRY MODEL AND DISPLAY OF RESULT

4.1 Display of the Plaxis' Geometry Models and Results

Based on the version of the Plaxis used [22], both the analysis and geometry for this research works would be described in steps below. The parameters summary tables for the modified cam clay model, hardening soil model and the concrete are presented under each model. The images which are extracted from the Plaxis model (i.e., from the geometry interphase and output interphase) are presented and discussed below:

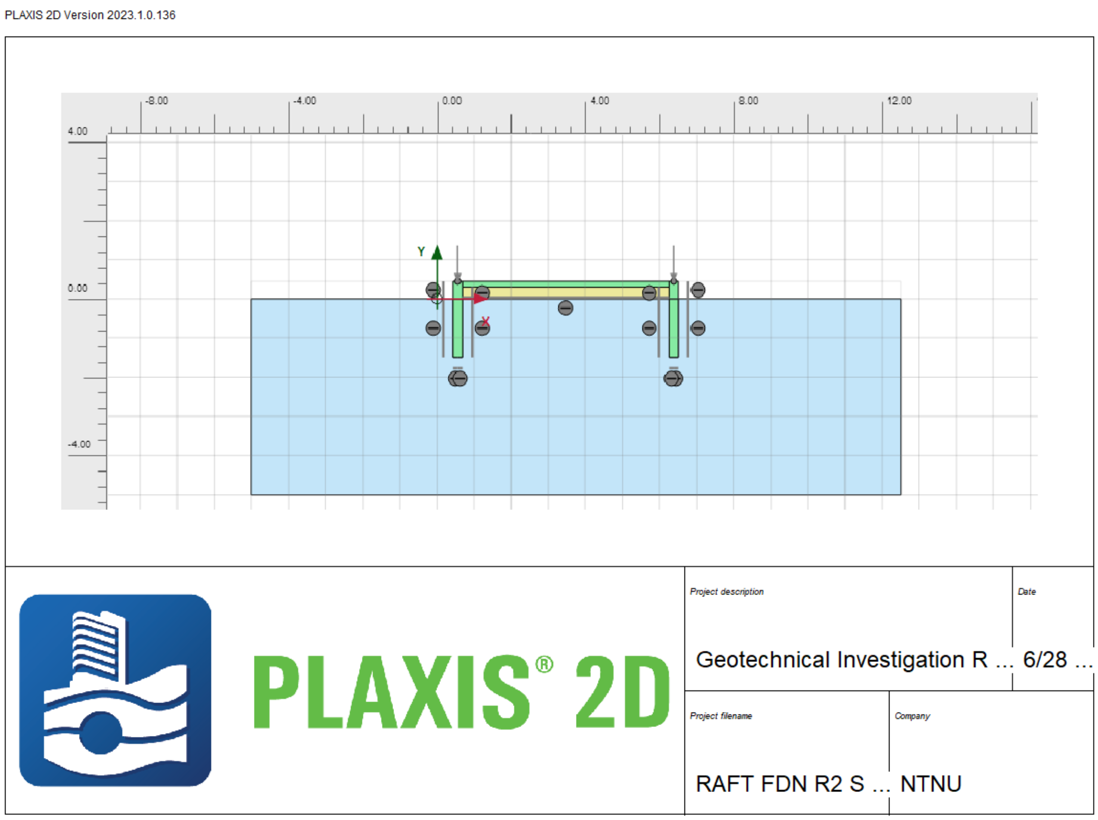

FIGURE 4.1.1: SOIL INTERPHASE AS MODEL IN PLAXIS [LIGHT BLUE IS HS MODEL & YELLOW IS CAM CLAY]

Figure 4.1.1 above shows that the soil was extended on both sides of the proposed typical building section, this is to ensure that there are contributions from both sides, which is what is expected on site.

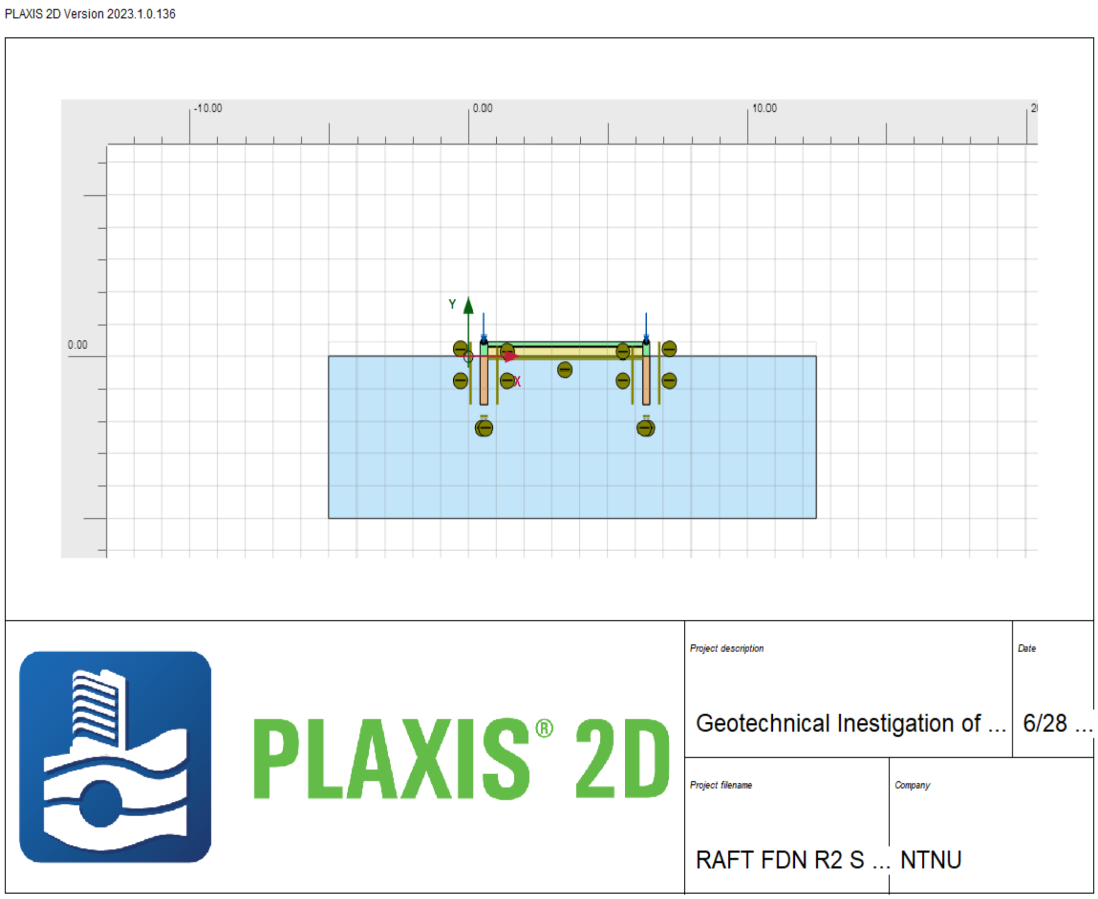

FIGURE 4.1.2: STRUCTURE INTERPHASE WITH CONCRETE ELEMENTS AND CONCRETE INTERPHASES

The interphase is necessary because of the differences in material bonding properties between the soil and concrete. Using the exact correct interphase value results in a good estimation of the shear stress value between the dissimilar materials but in this work, the interphase was assumed to be 0.5.

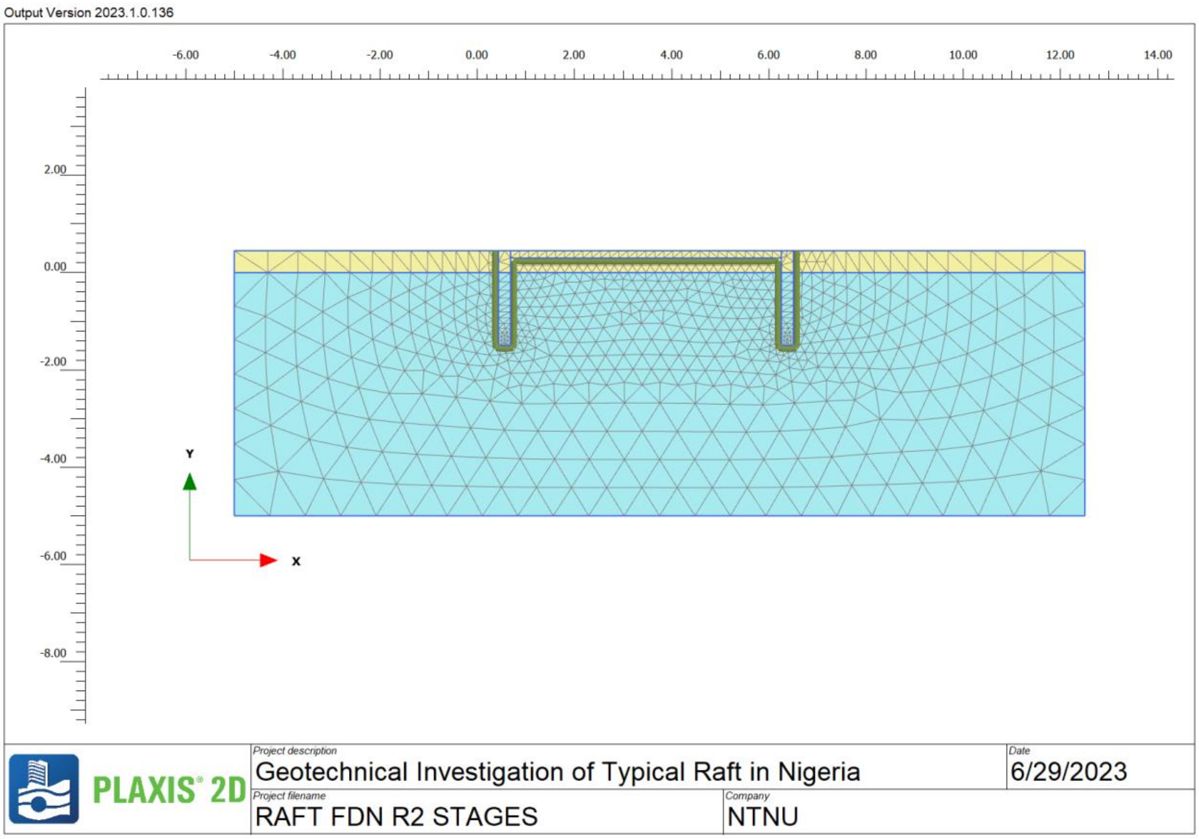

FIGURE 4.1.3: FINE MESH ADOPTED IN THE PLAXIS MODEL

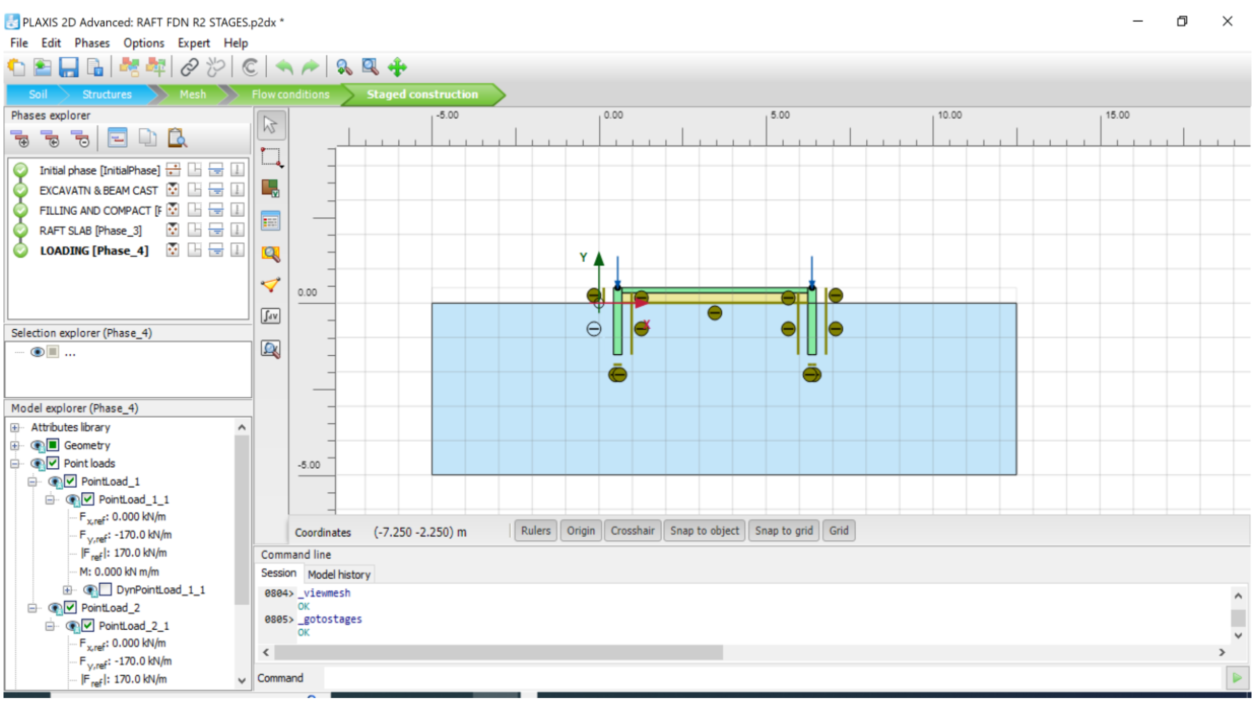

FIGURE 4.1.4: SEQUENCE OF MODEL STAGES ADOPTED FOR THE FULL MODEL

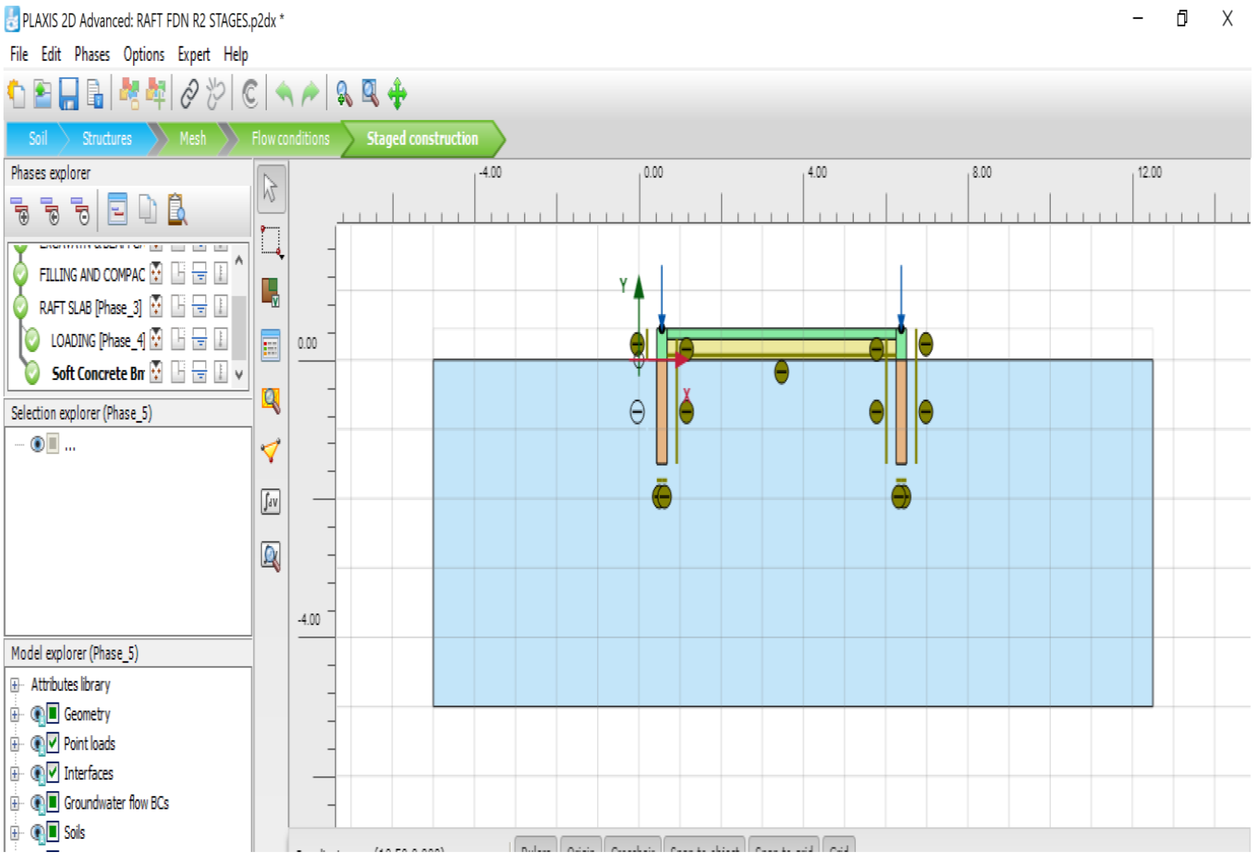

FIGURE 4.1.5: STAGES WITH CONCRETE BEAM SET TO SOFT STIFFNESS (IGNORING THE BEAM)

The brown material in Figure 4.1.5 indicates the concrete beam that the stiffness was reduced 3100kN/m^2. The aim was making the concrete beam behaves like soil so that the beam and slab raft foundation can be seen just like a regular slab foundation (this is discussed in detail below).

4.2 Display of the Input Parameters as Derived in Chapter Three.

As discussed, and derived in chapter three above, the input parameters into the Plaxis model are presented in the parameter summary table below:

4.2.1 Display of the Hardening Soil Parameters

The hardening soil parameters used for the research work is presented in Table 12 below:

TABLE 12: THE HARDENING SOIL PARAMETERS SUMMARY

S/no	Parameters	Values
1	$E_{50}{}^{ref}$	38.80 *1000kN/m^2
2	$E_{oed}{}^{ref}$	42.68*1000 kN/m^2
3	υ_{ur}	0.2
4	P_{ref}	100kPa
5	Cohesion ^{ref} (C' ^{ref})	67kN/m^2
6	φ_{inter}	12 degrees
7	Dilatancy ψ	0
8	γ_{sat}	20kg/m^3
9	γ_{unsat}	18.6kg/m^3
10	void ratio e_0	0.709
11	$E_{ur}{}^{ref}$	116.4*1000
12	Power (m)	0.7
13	R_{inter}	0.5
14	n_{init}	0.4149
15	$\upsilon_{u,equivalent}$ (nu)	0.4950
16	POP	10kN/m^2
17	OCR	1

The interphase of 0.5 used is based on the cohesion and friction values given in the soil report (the friction angle of twelve is given in the soil report). The friction angle of 12 degrees seems high and could result in sliding between the soil and concrete interphases hence the decision to use an interphase of 0.5.

At the initial stage, the preconsolidation value of say 10kN/m^2 is assumed (10 kN/m^2 is assumed to be conservative as against 40kN/m^2 used in Plaxis for curve fitting) to be the historical loaded values of the soil's current state. This value is just assumed for completeness as there is no information that can be used to estimate the historical loading state of the soil.

4.2.2 The Modified Cam Clay Parameters

The modified cam clay parameters used for the research work is presented in Table 13 below:

TABLE 13: THE MODIFIED CAM CLAY PARAMETERS SUMMARY

S/no	Parameters	Values
1	Λ (calculated)	0.023
2	K (calculated)	0.009
3	υ_{ur}	0.15
4	M_{csl}	1.2
5	Cohesion ref (C'_{ref})	10
6	φ_{inter}	30
7	Dilatancy ψ	0
8	γ_{sat}	20
9	γ_{unsat}	18.6
10	void ratio e_0	0.894
11	POP	25
12	OCR	1.0

For emphasis, the model used for the compacted filling material is a modified cam clay model, and the over-consolidated ratio (OCR) is assumed to be one whilst the pre-consolidation stress (POP) is taken to be 25kN/m^2. These assumed values are based on the procedural field exercise for the compaction process (for instance POP of 25kN/m^2 would likely be exacted by the dumper compactor used on-site during compaction) that is the infill materials are compacted in layers with the aid of the compactor machine and water.

4.2.3 Display of the Concrete Structural Parameters

The concrete structural parameters used for the research work is presented in Table 14 below:

TABLE 14: THE STRUCTURAL CONCRETE PARAMETER SUMMARY

S/no	Parameters	Values
1	Soil model: Linear Elastic	
2	Drainage type: non-porous	
3	Unsaturated weight	24kN/m^2
4	E_{ref} (from EC2 based on fck=25N/mm^2)	31*10E6 kN/m^2
5	Poison ratio υ (nu)	0.25
6	e_{init}	0.50
7	n_{init}	0.33
8	G_{ref}	12.40E6 kN/m^2
9	E_{oed}	37.20E6 kN/m^2
10	V_s	2251
11	V_p	3899

4.2.4 Display of the Soft Concrete Parameters

This material is adopted to ensure that the Plaxis model behaves as though the concrete beam is somewhat ignored because the stiffness parameter is reduced. The concrete stiffness was arbitrary and drastically reduced from 31.0* 10^6 to 3100 (0.01% reduction). The concrete beam stiffness reduction can therefore suggest that the beam component of the substructure can be ignored (the intention of this stiffness reduction is to make the concrete beam have the same strength properties as the soil) . Hence, it is logical to consider the concrete beam as a dummy element in the Plaxis model for this stage. The only parameter that was altered in this construction phase is the mechanical stiffness of the concrete and the reduction in stiffness is as shown below:

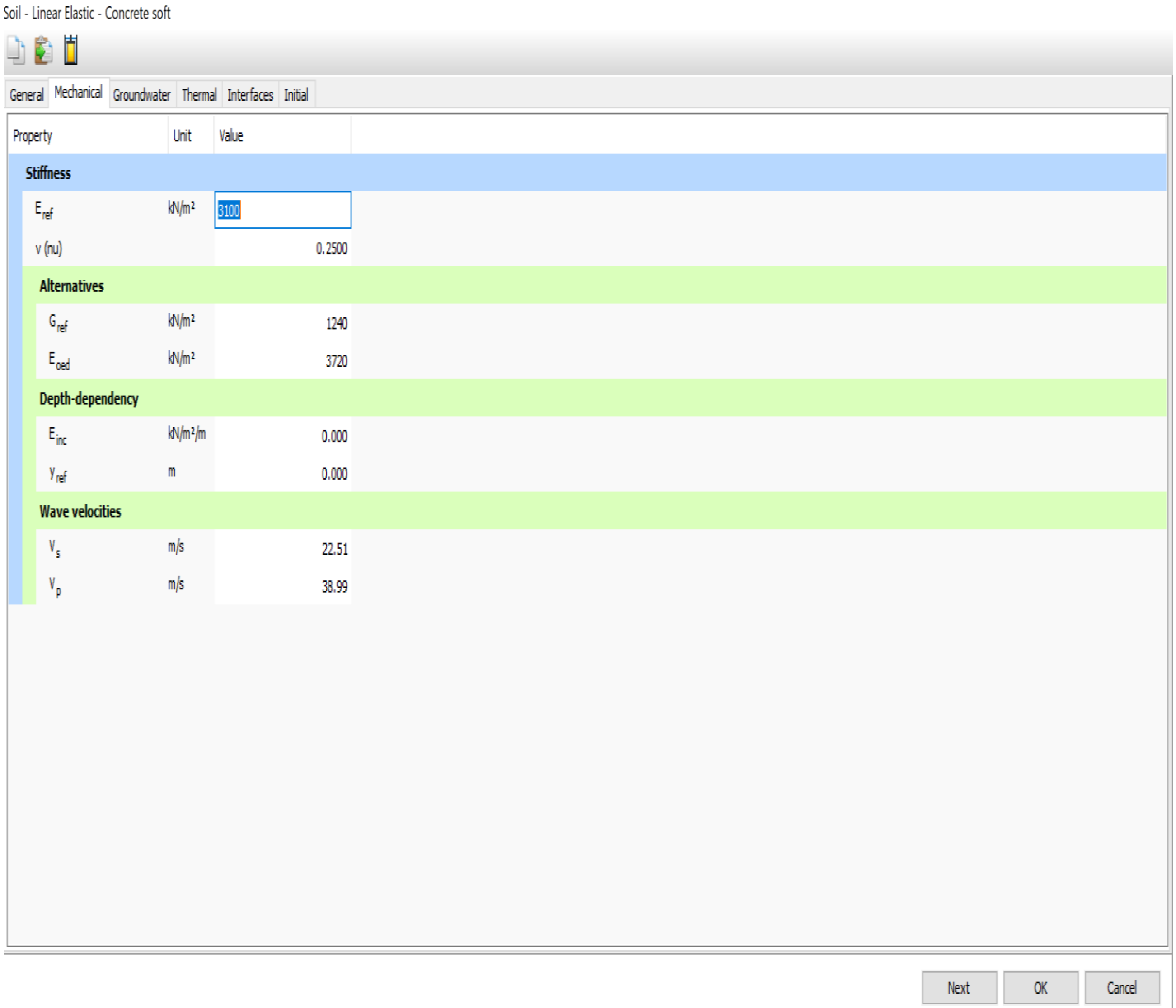

FIGURE 4.2.4.1: STRUCTURAL CONCRETE MECHANICAL PARAMETERS [WITH E_{REF} REDUCED TO 3100KN/M^2]

4.3: Discussion of Results from the Plaxis Model Based on the Input Parameters

The results based on the input parameters are displayed below, the interpretations of the displayed results were discussed briefly under each result/graph and the findings of all the results would be summarized in Chapter 5.

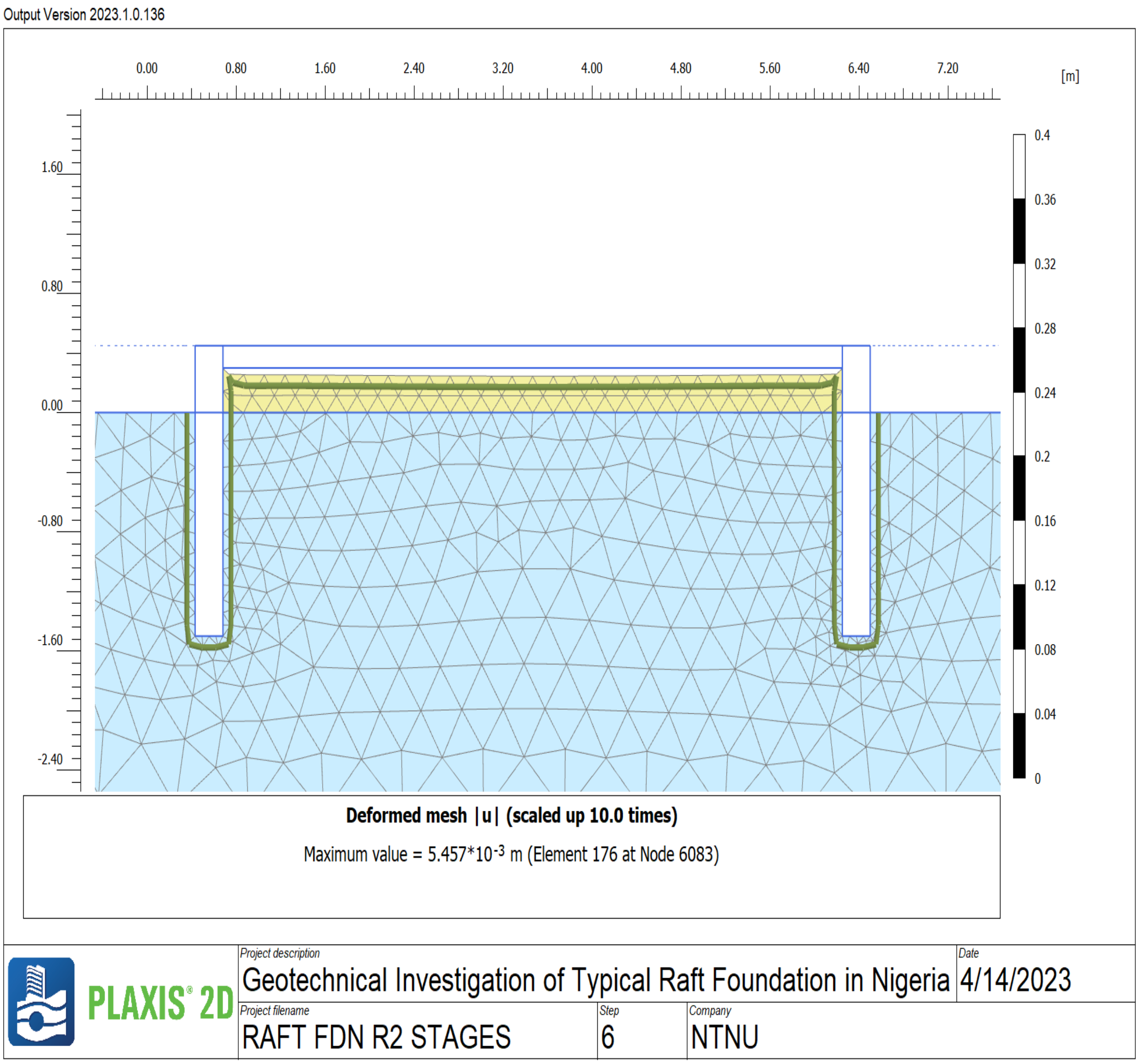

FIGURE 4.3.1: DEFORMED MESH FOR FULL STRUCTURAL MODEL WITHOUT LOAD [5.5MM SETTLEMENT]

Figure 4.3.1 shows that the settlement of the foundation under its self-weight is 5.5mm. The settlement of about 5.5mm may not be considered too extreme for a residential building, however, the tolerable limit under all load combinations (that is service, live loads, or ultimate is clearly stated in relevant standard codes).

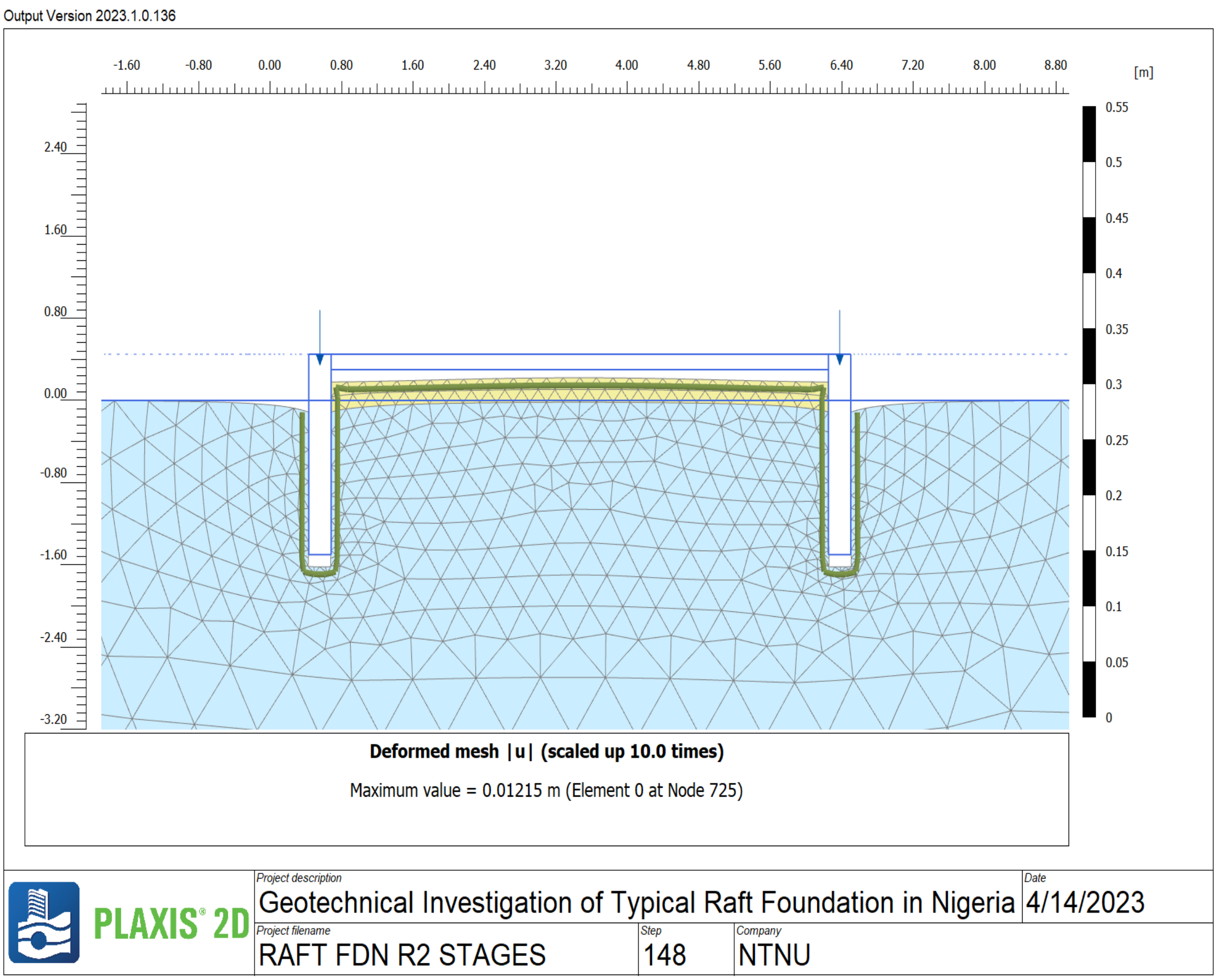

FIGURE 4.3.2: DEFORMED MESH FOR FULL LOADED STRUCTURAL MODEL [12.15MM SETTLEMENT]

Figure 4.3.2 shows the result of the total settlement under serviceability loading to be about 13mm and this settlement represents the long-term settlement because the analysis is designed under the drained analysis. The maximum settlement is located at the tip of the beam which could likely be linked to the fact that the applied structural load is directly on the beam (it is so because the structural columns emanated from the beam in real construction). The column starting from the top of the beam prevents punching through the slab. The effect of having a column (a point load) on the raft beam led to a pronounced settlement of the beam. Meanwhile, the deformed shape of the slab in Figure 4.3.2 (in an enlarged scale) shows an upward bulge which suggests the effect of pressure from the soil caused by the foundation loads.

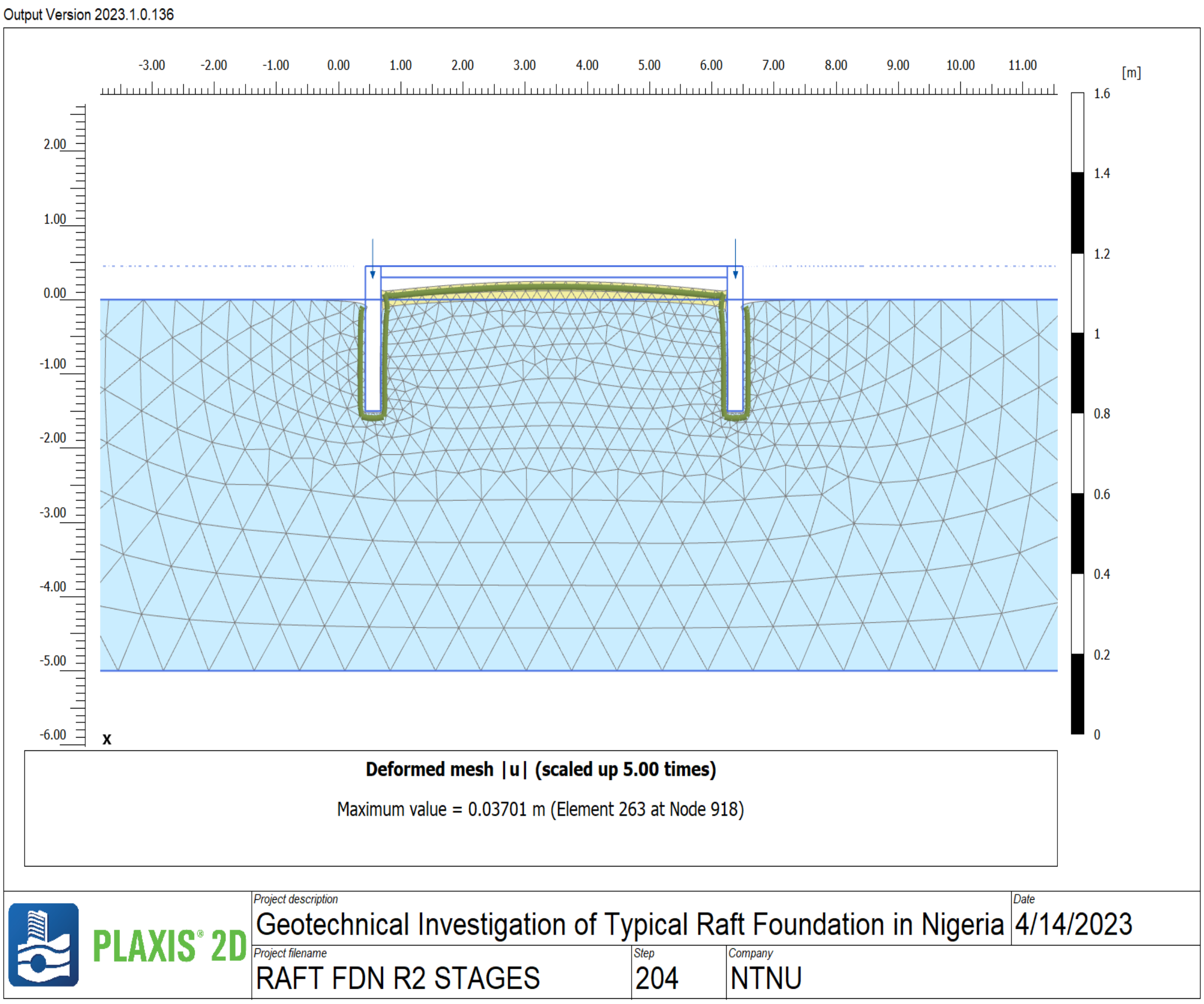

FIGURE 4.3.3: DEFORMED MESH FOR FULL LOADED STRUCTURAL MODEL AND SETTING TO CONCRETE BEAM TO SOFT

Close examination of the deformed image of Figure 4.3.3 (deformed shape where the beam stiffness is reduced to 3100kN/m^2. Due to this reduction, the beam is a dummy beam as structural engineers would say). The aim of creating this beam (i.e., dummy, or false beam) is to almost make the beam have equal stiffness as the soil. The load on the dummy beam would likely result in excessive deflection of the beam and since the substructure is monolithic, the raft slab would also be required to move downward but this movement would be prevented due to the compacted soil. This restriction in movement would cause the slab to bulge and the slab top is subjected to tensile force which is the primary reason to place the main reinforcement at the top of the raft slab. Also, placing the column load on the dummy beam could result in cracks at the beam-slab joint and punching as well.

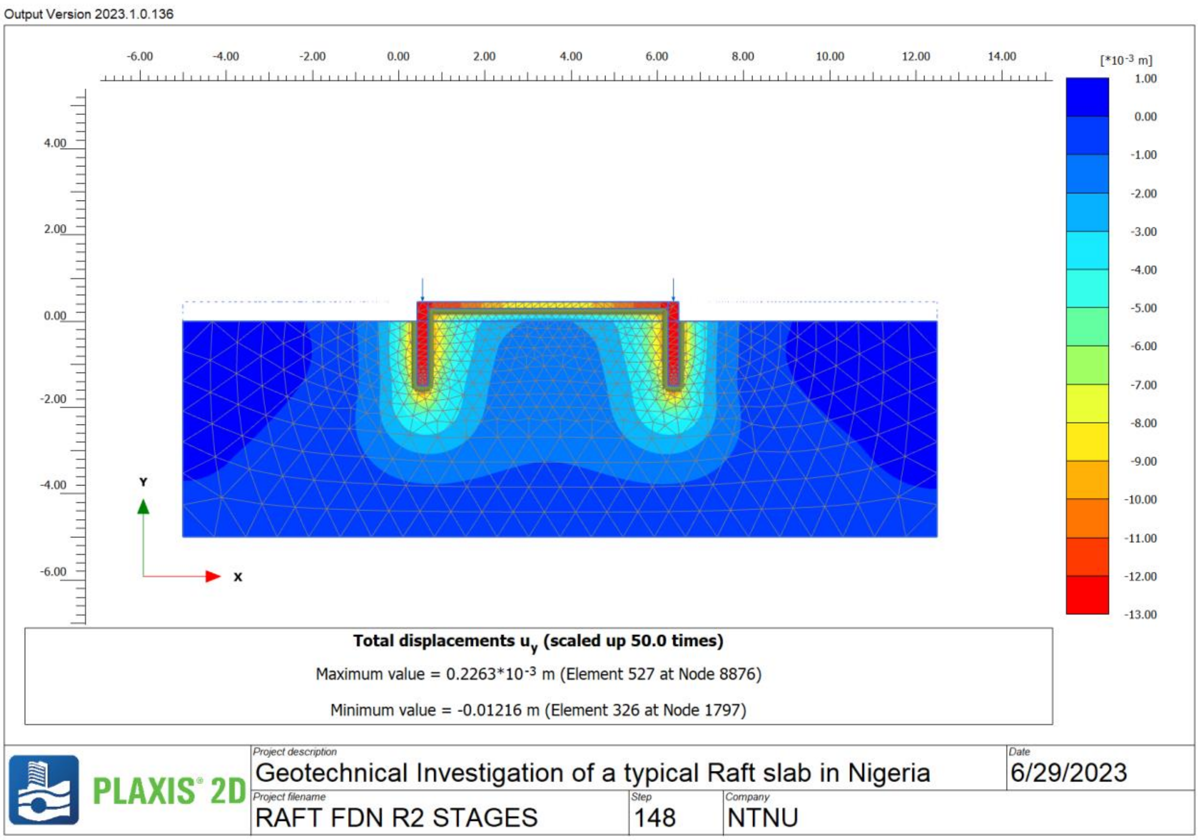

FIGURE 4.3.4: DEFORMED PLATE SHOWING PATTERN THROUGH WHICH THE STRUCTURE DEFORMED

The deformation plate shown above indicates clearly that the rigid body (in this is the concrete structure) deformed in a downward manner through the flexible body (in this case is the soil-the filling material and the soil block). The deformed pattern and direction are clearly shown in Figure 4.3.4 where the sliding is along the beam. There is less displacement at the centre of the slab and high displacement at the tip of the beam.

Based on the colour pattern, red means higher stress location. Then one would conclude that the concrete beams and the edge of the slab are highly stressed while the central part of the slab has less stress compared to the beam. This supports the argument that the slab is bulging at the centre.

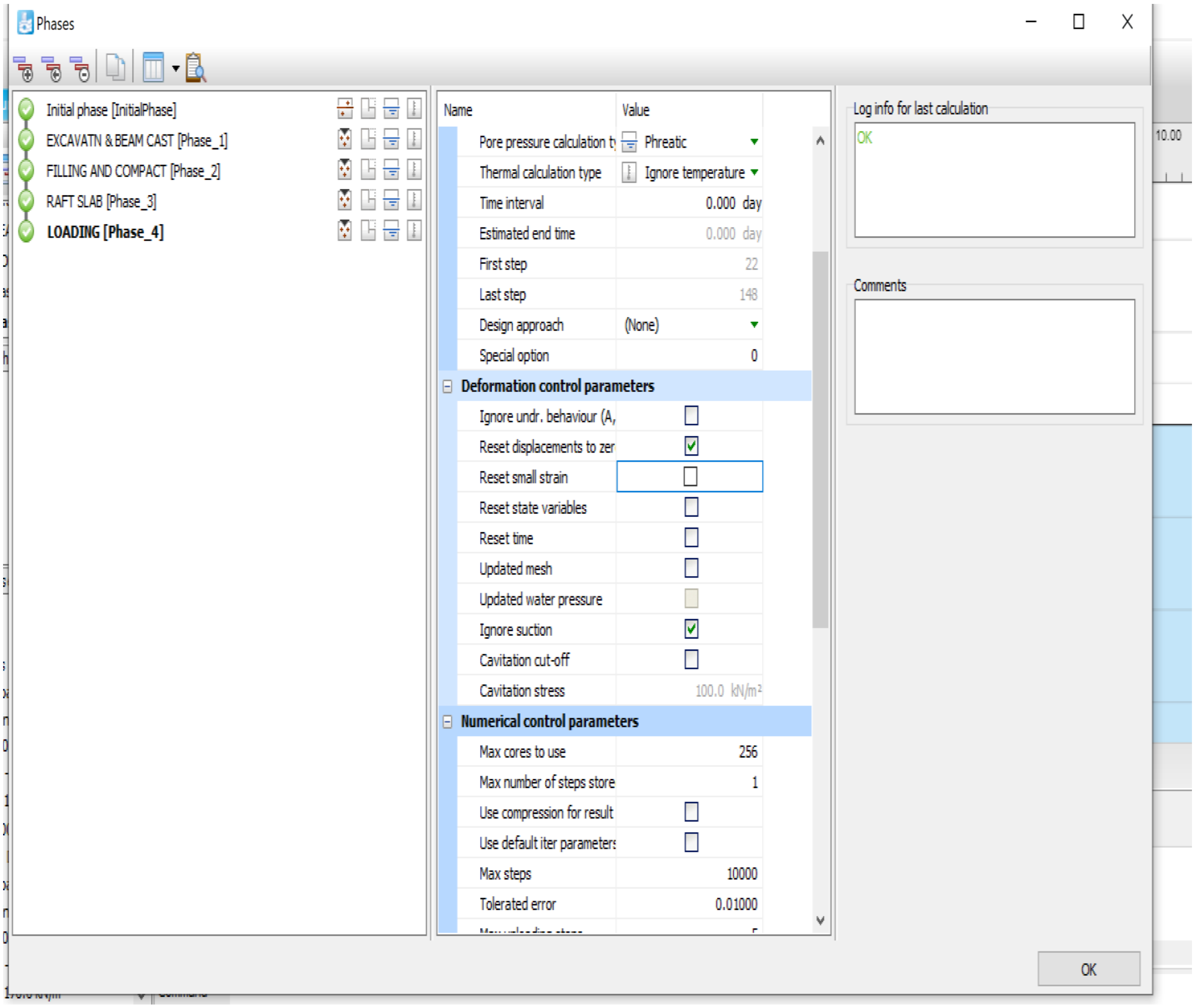

FIGURE 4.3.5: ACTIVATION OF ZERO DISPLACEMENT OPTION TO RESET THE GEOMETRY TO ZERO

It is however worth noting here that the displacement reset option as indicated in the image above was activated at the beginning of the final loading step, this was to ensure that the geometry of the compacted soil is set to zero as it would be obtained in the real construction works. This is because the concrete or compacted infill materials would eventually fill the ground floor to the required natural stipulated level (that is zero level position). This option is believed not to have any influence on the result in Plaxis as it is just a geometry correction tab.

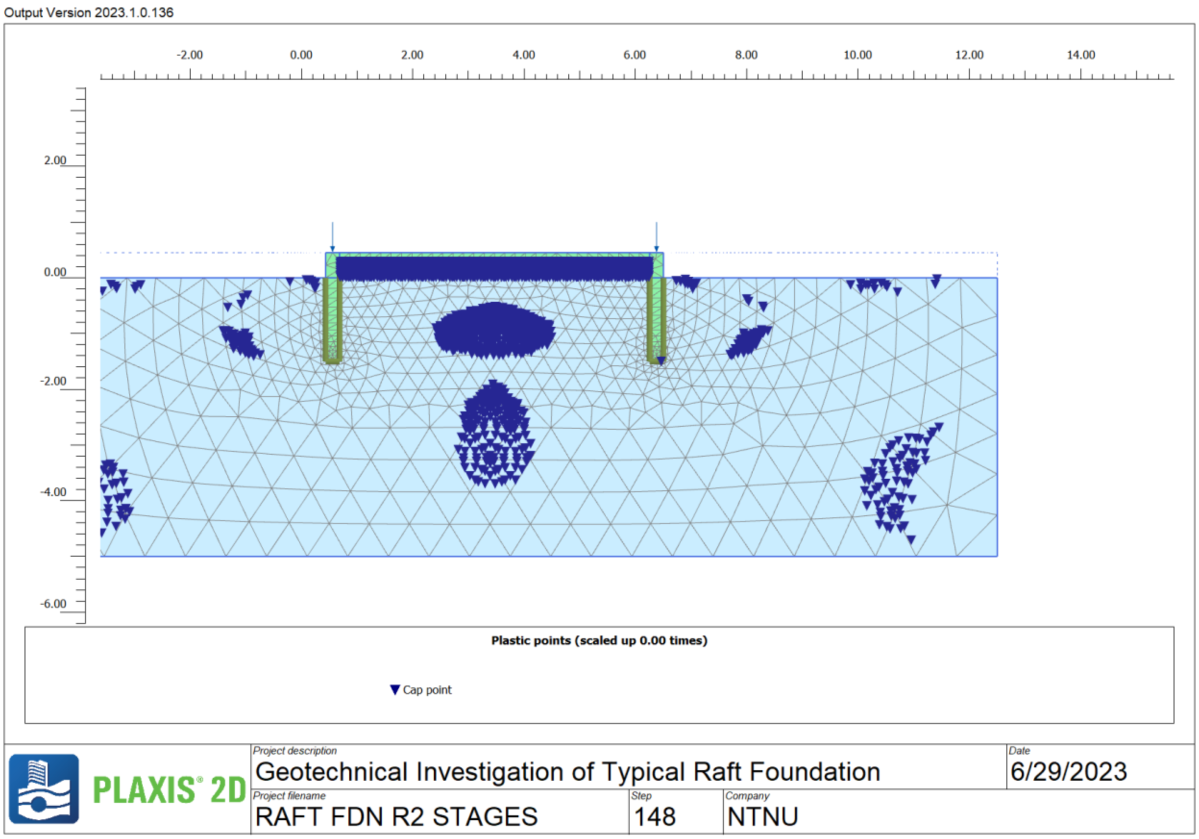

FIGURE 4.3.6: LOCATION OF CAP POINTS IN THE MODEL

Figure 4.3.6 shows the infill material reaches the cap point, although the cap point is not limited to the infill but is predominant in the infill material. The term cap point is pushed outwards means that the value of the pre-consolidation stress adopted/assumed (POP assumed is 25kN/m^2 for MCC and 10kN/m^2 for HS) in the input parameter is expanded (pushed outward). The outward expansion is possible because mobilized friction (tan ρ) is utilized in the model instead of the failure value (tan φ) whereas the cap in this case is controlled by the value of POP that is assumed (because it is hardening soil model). However, loading beyond the cap (POP) would likely result in plastic volumetric strains.

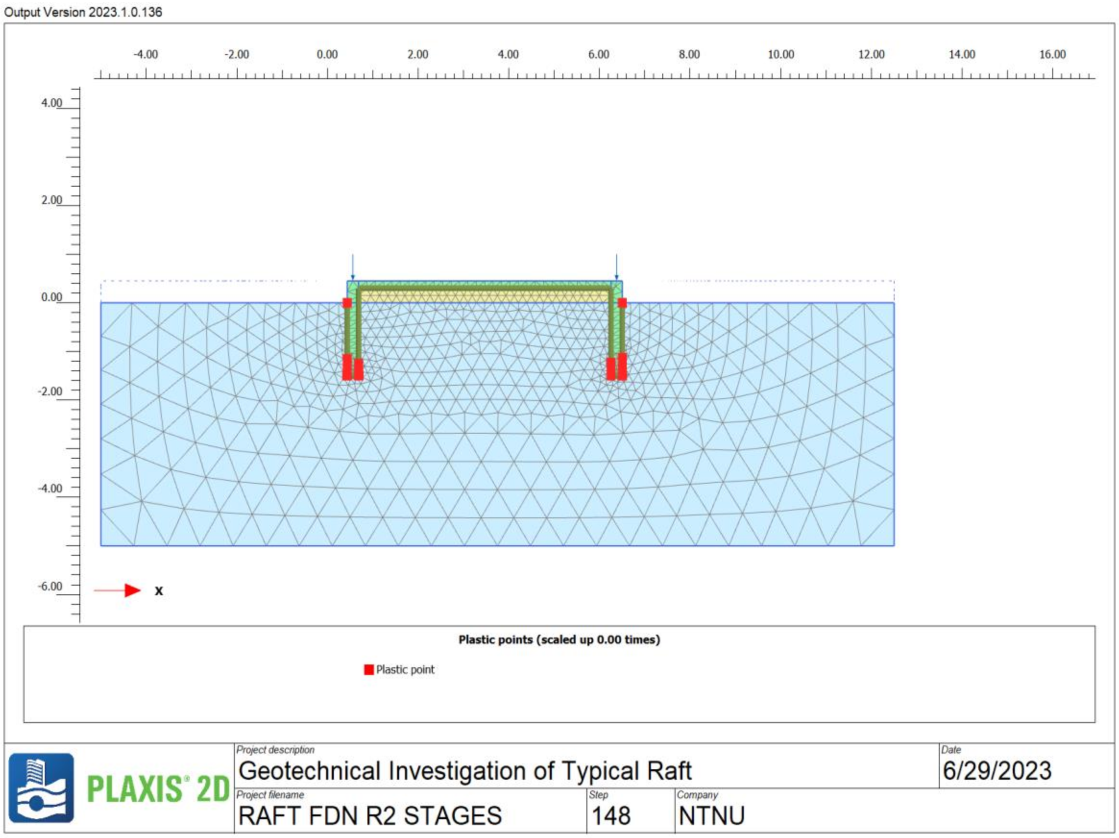

FIGURE 4.3.7 LOCATION OF FAILURE POINTS IN THE MODEL

Figure 4.3.7 shows the location of the failure point in the investigated model, the soil capacity (plastic point) is reached majorly along the beam interface and one can conclude that the failure point is localized along the raft beam, and the soil within the structural raft slab has no capacity problem. However, since the R_{inter} is set to a 0.5 limiting value hence there is a redundant value of about 50% saving (along the beam) in the failure point displayed in Figure 4.3.7.

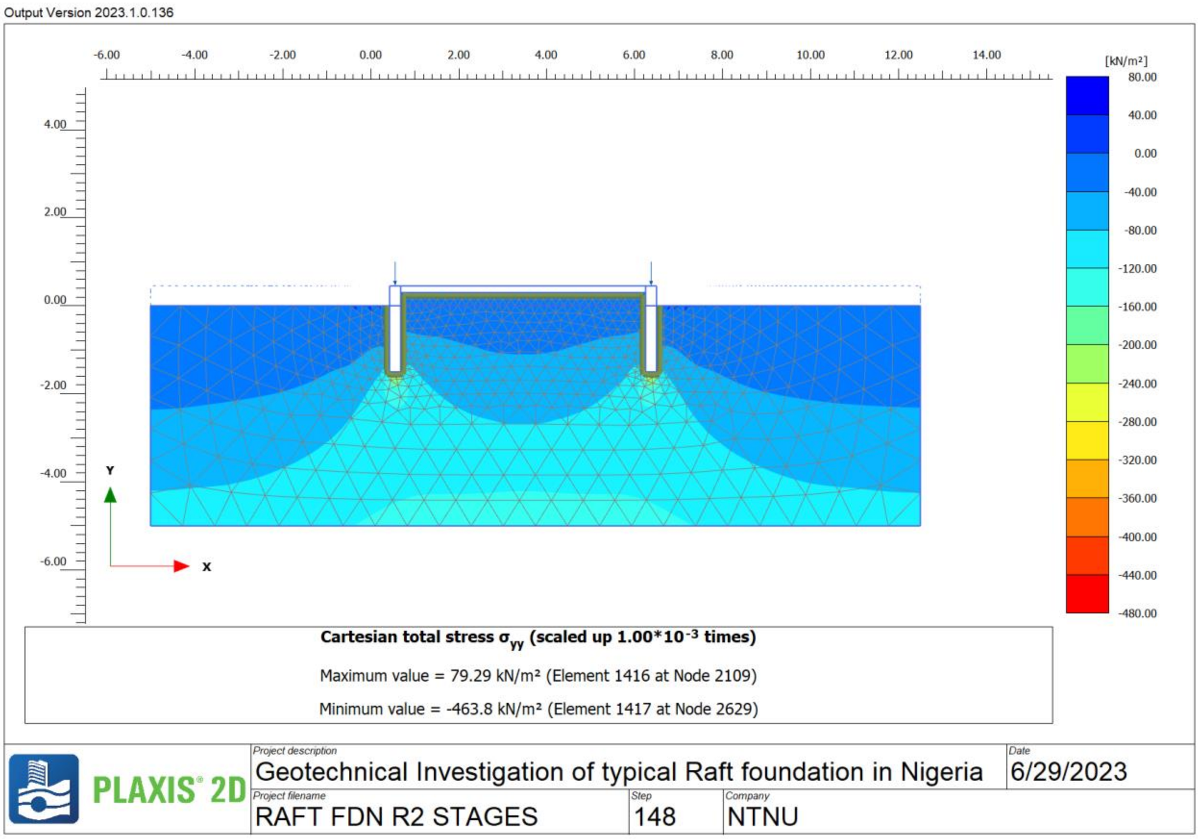

FIGURE 4.3.8: TOTAL VERTICAL STRESS PLATE (HIGHEST AT THE TIP OF THE BEAM)

The beam vertical stress has the highest stress of about 464kN/m^2 (although, the colour pattern in Figure 4.3.8, shows that the beam tip highest stress is about 275kN/m^2, hence one can conclude that the 464kN/m^2 displayed could be just a single point) whilst the middle of the slab has between 70 to 80kN/m^2 based on the colour pattern. Meanwhile, the geotechnical soil investigation report recommends that the of value allowable bearing pressure at a founding depth of 1.5 be about 75kN/m^2. Comparing the vertical stress in Figure 4.3.8 with the recommended allowable bearing pressure from the soil report seems not ideal because the allowable bearing pressure has a reduced value even after divided by factor of safety and the effect of overburden appears not considered during bearing capacity estimation (this is discussed in Chapter one as part of the limitations of the research). The effect of these limitations would be an increase to the recommended allowable bearing pressure of 75kN/m^2 as stated in the soil report.

Ignoring the limitation, one may however conclude that the pressure within the slab could be considered satisfactory while the contact pressure at the beam tip fails and could result in settlement from the tip. Since the whole structure is monolithic, there is the likelihood that the whole structure would settle uniformly and with no differential settlement thus compressing the filling material/soils and resulting in a further redistribution of soil stresses under the slab and perhaps making the slab carry extra loads. This discussion is further investigated by considering the deviatoric stress and localizing the stress pattern in the subsequent graphs below.

The figure 4.3.8A below investigated the point where the higher stress of 464kN/m^2 could possibly be located. Hence, Figure 4.3.8A is used to present localized stress of maximum of 270kN/m^2 (that is adjusting the legend setting in Plaxis to only stresses within 270kN/m^2 zone for clarification).

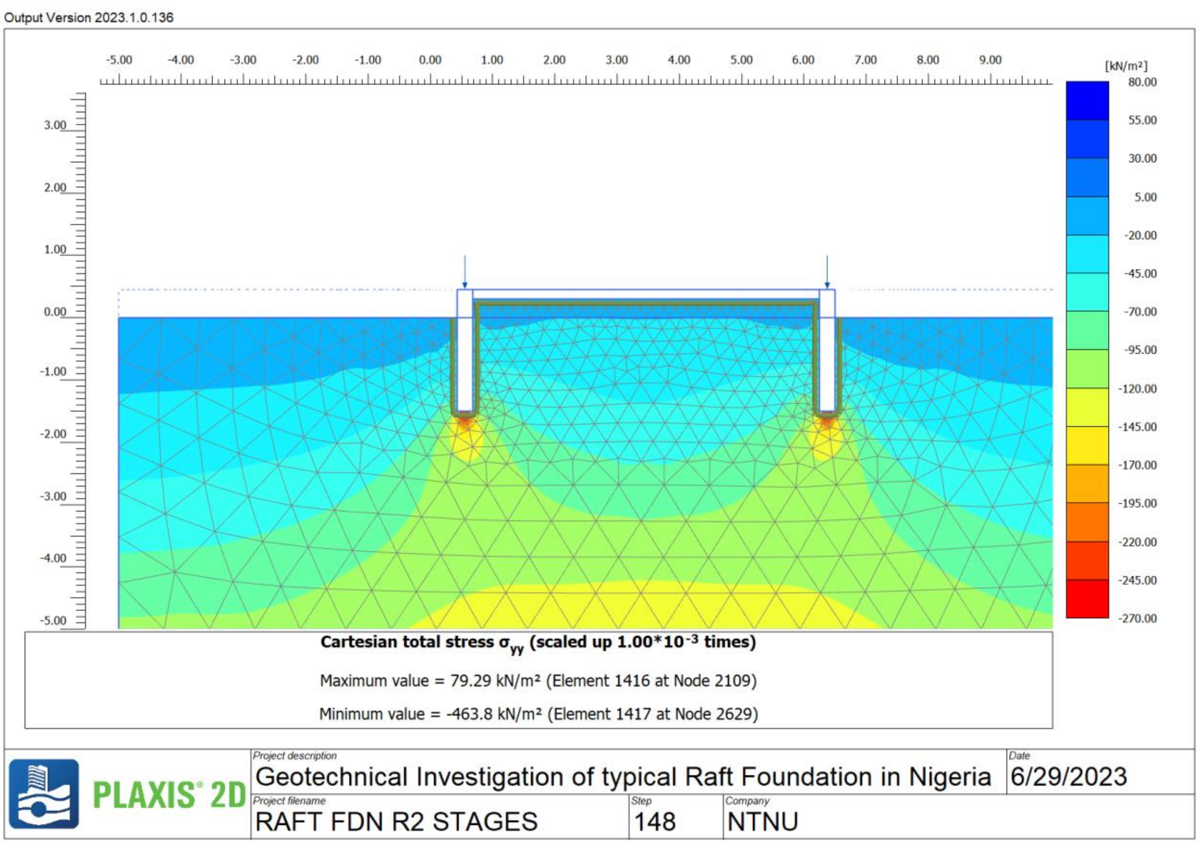

FIGURE 4.3.8A: TOTAL VERTICAL STRESS PLATE [WHEN THE LEGEND IS LIMITED TO 270KN/M^2]

Magnifying the tip of beam in Figure 4.3.8A to confirm where the highest stress of 464kN/m^2 could be located brings about the image (Figure 4.3.8C) below. Figure 4.3.8C shows the magnifying image of the stress under the beam and the grey colour in Figure 4.3.8C indicate the points of these higher stresses. These higher stresses are confined to the corners of the beam as called out below.

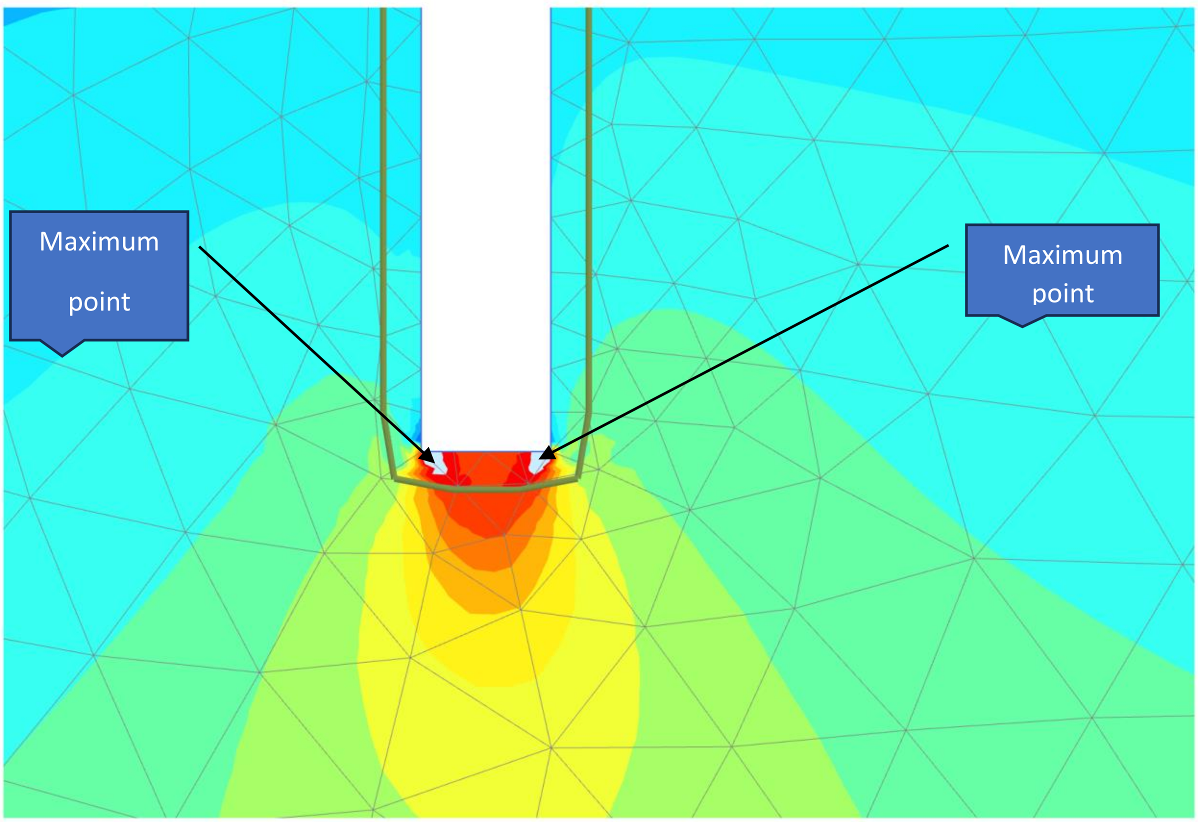

FIGURE 4.3.8C: MAGNIFYING THE LOCATION OF THE HIGHEST STRESS IN THE BEAM OF A RAFT FOUNDATION

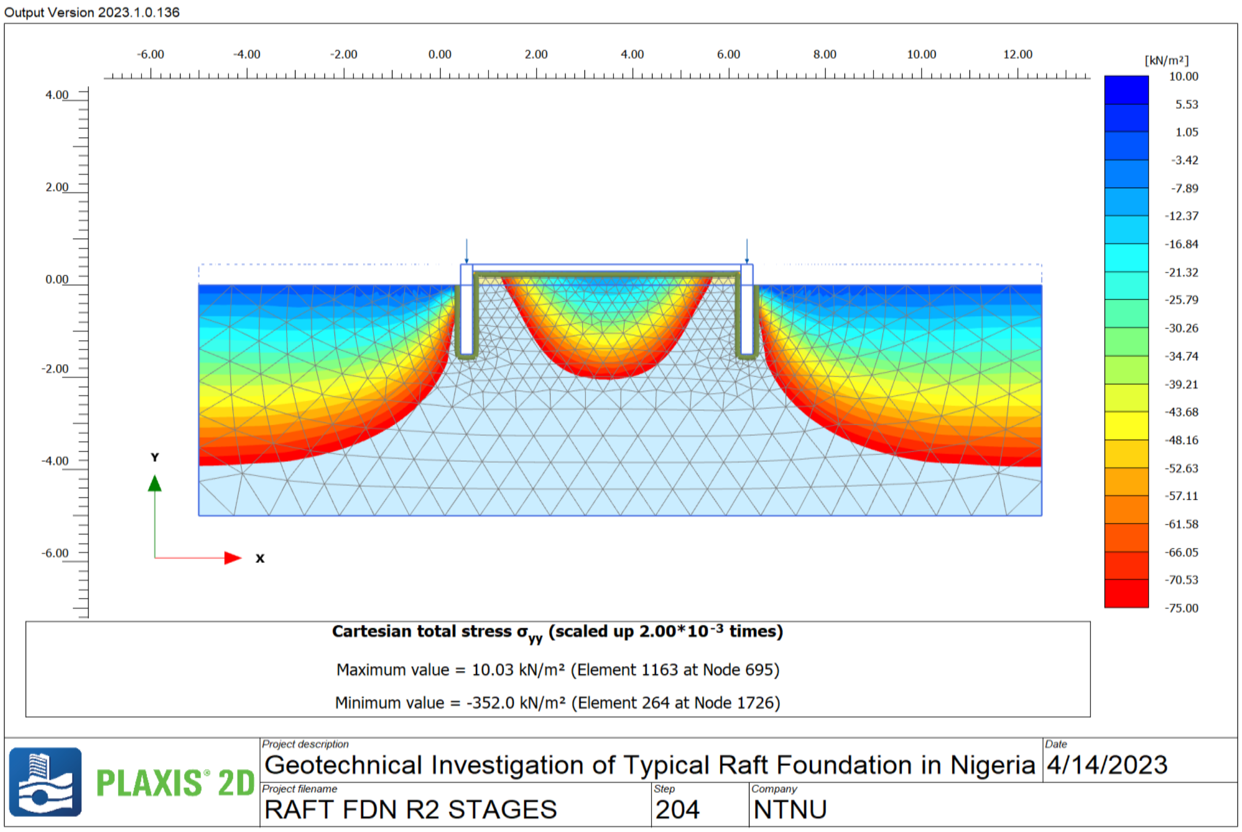

FIGURE 4.3.9: TOTAL VERTICAL STRESS PLATE BASED ON THE LIMITING MAXIMUM BC OF 75 AT THE 1.5M DEPTH

The figure above shows the result when the legend in the Plaxis model was set as per the soil investigation report that is specifying the maximum pressure as 75kN/m^2 by the soil report. The grey colour pattern in Figure 4.3.9 indicated areas that exceeded the recommended value of 75kN/m^2 according to the soil report. The figure clearly shows (though scaled up by 2.0*E-3) that the beams/interphases are highly stressed and exceed the allowable bearing capacity.

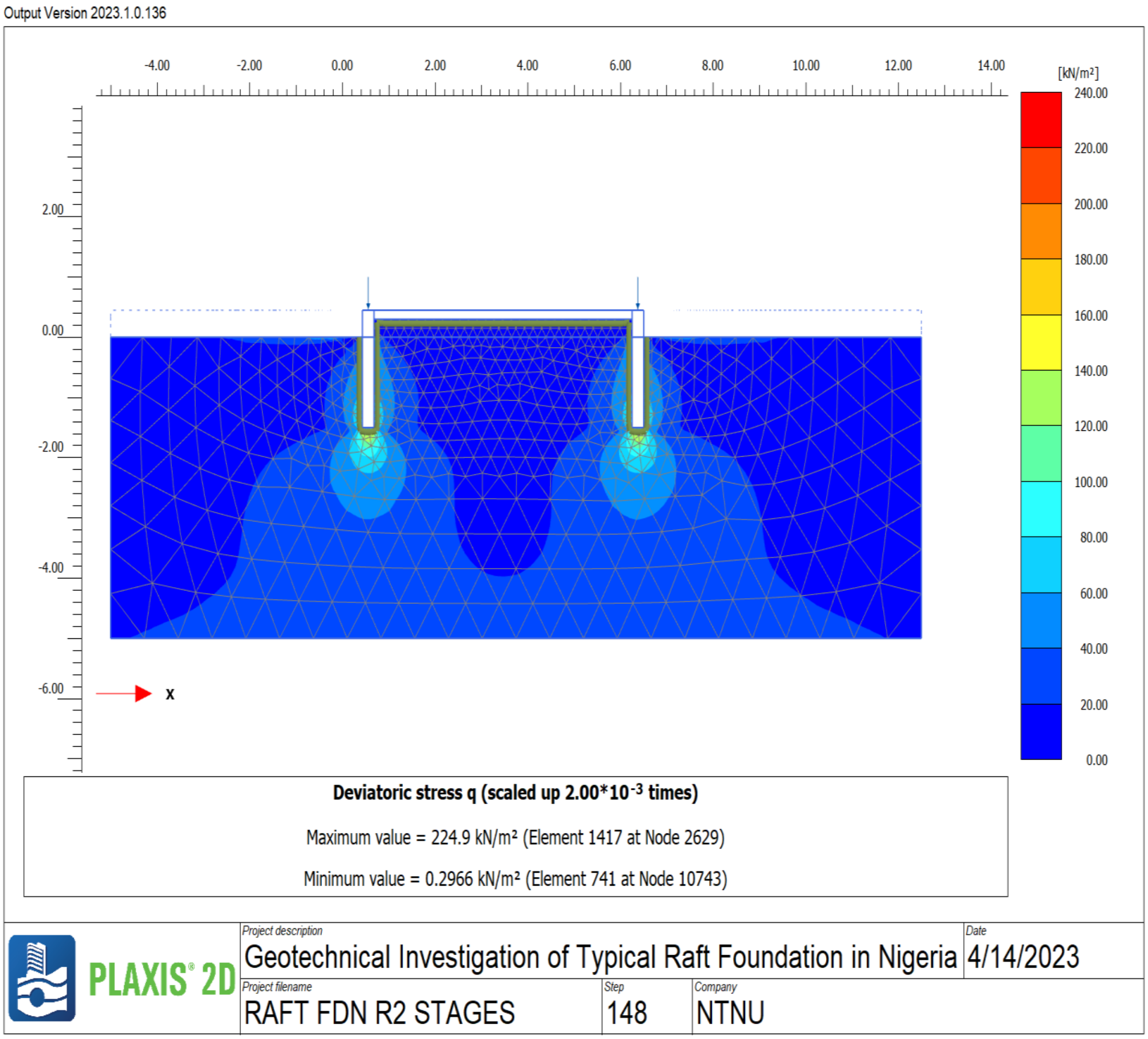

FIGURE 4.3.10: DEVIATORIC STRESS

The deviator stress can simply be defined as the difference between the major and minor principal stress. The deviatoric stress primarily is the soil/material property that shows the distortional shape of soil or material. The deviatoric stress also indicates the shear stress in the soil/structural elements. Therefore, the small bulb around the beam in the deviatoric stress graph above indicates the level of the stress at the tip of the beam and this collaborates with the foregoing argument that the stresses at the beam tip are higher compared to the slab and hence results in high settlement at the tip of the beam.

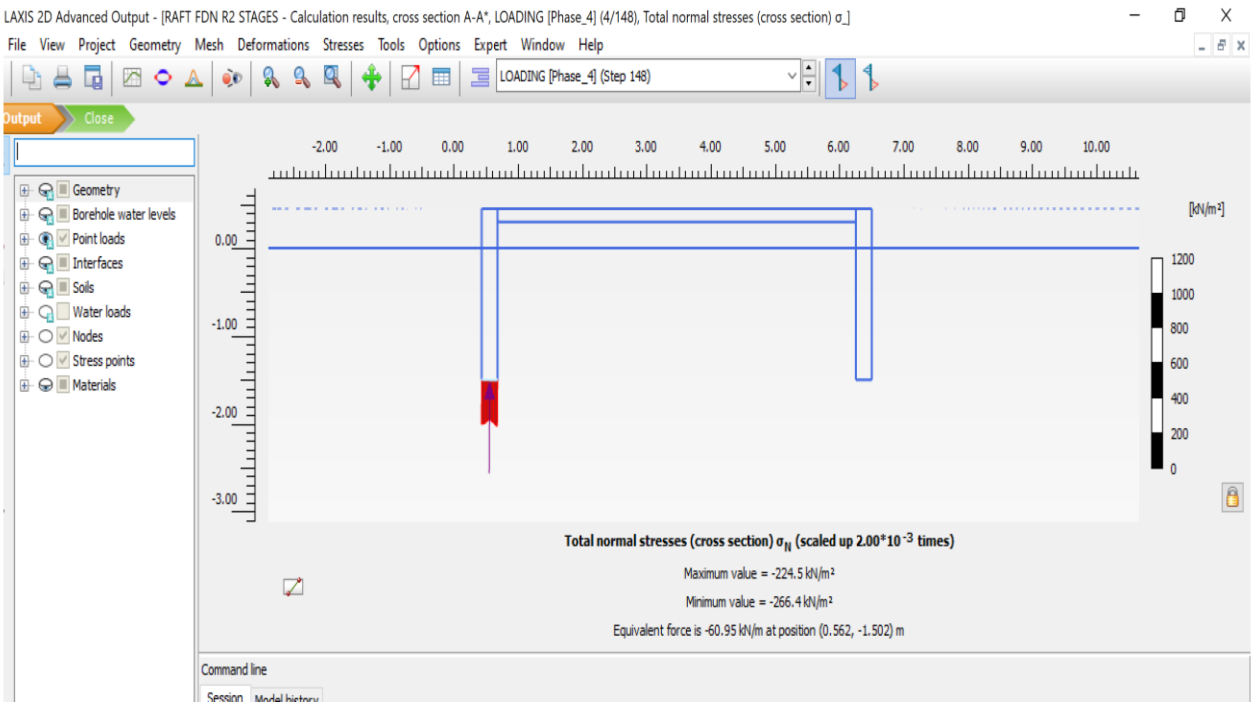

FIGURE 4.3.11: TOTAL NORMAL STRESS IN THE SOIL AT THE BOTTOM OF THE BEAM AS A RESULT OF THE LOAD

The total normal stress at the tip of the beam in the soil due to the foundation load is depicted by the image above and the reaction is 60.95kN/m as seen in Figure 4.3.11. The maximum and minimum stresses in the soil (at the beam tip) are also displayed in the same figure and have somewhat higher stress values as compared to the stresses in the slab (refer to figure 4.3.12 below for the slab stresses). This is possibly so because the beam is a rigid monolithic body (concrete), much stiffer, hence the beam attracts more stresses from the soil in comparison to the slab which is seated directly on soft infill material and compressible soil. This can be synonymous with two different springs which are loaded in a parallel loop (that is the load is at the connecting node of the beam and slab, the beam and the slab can be assumed to be parallelly connected in this case), the stiffer spring in this instance is the beam thus attract more loads. Although it is expected that the equilibrium of forces should be met, the forces would be idealized after discussing the force in the slab in Figure 4.3.12 below.

Based on the beam reaction of 60.95kN/m, it is therefore evident that this is the amount of load required to be supported by the beam and as such the beam is needed to carry some significant quantity of imposed load hence introducing the beam is not completely out of place. Some engineers advocated for the complete removal of the raft beam because of the limited contact width. Instead of complete removal, a toe beam/edge beam /tapered slab or chamfered slab should be considered or used. This toe beam/edge beam also helps to prevent punching, prevent ingress of water from the edges of the slab, and acts as a stiffener. Thus, the notion that the beam is acting as just a stiffener by most engineers in Nigeria could be wrong due to the reaction obtained above.

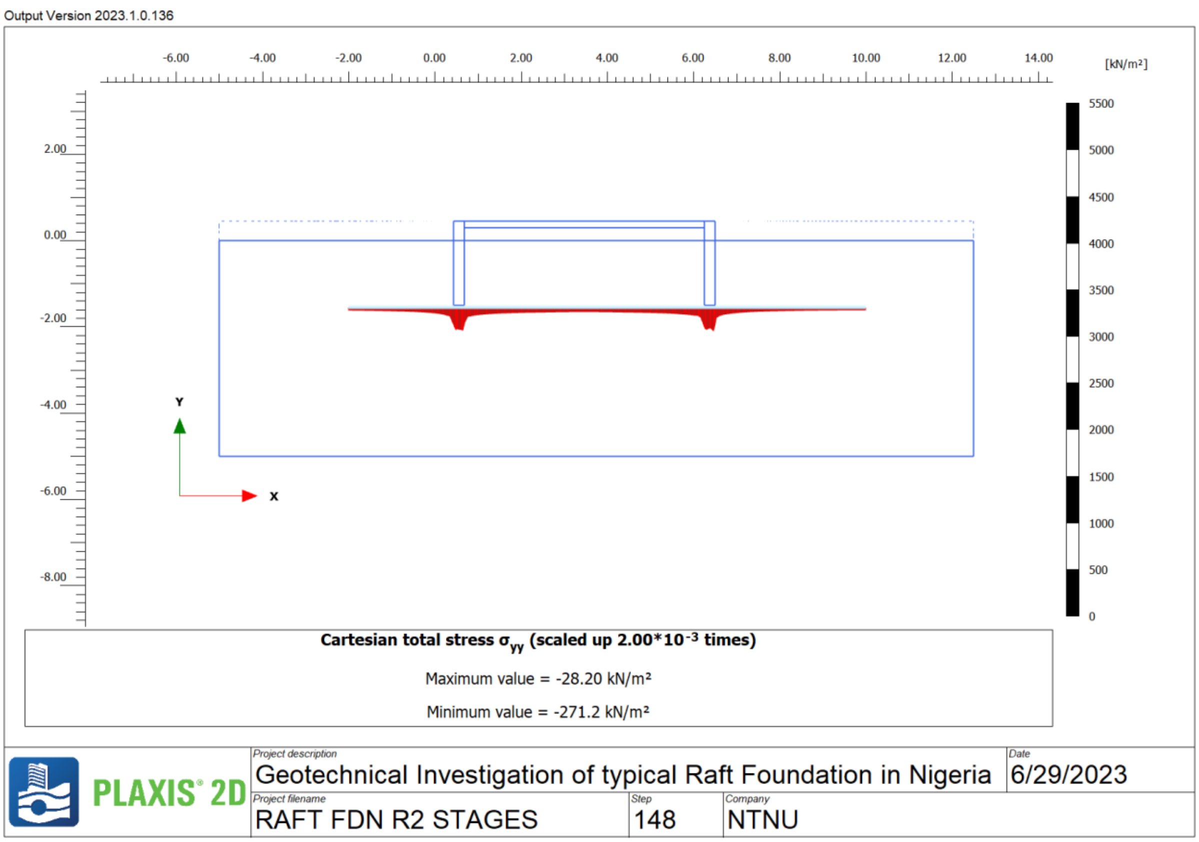

FIGURE 4.3.11A: STRESS IN THE SOIL AT 500MM BELOW THE DEPTH OF THE BEAM

Figure 4.3.11A shows the stress profile in the soil under the beam. The choice of 500mm under the beam is to make allowance for blinding (i.e., non-structural concrete material). Figure 4.3.11A shows that the stress in the soil under the beam has 271.2kN/m^2 while the stress in the soil under the slab has 28.02kN/m^2.

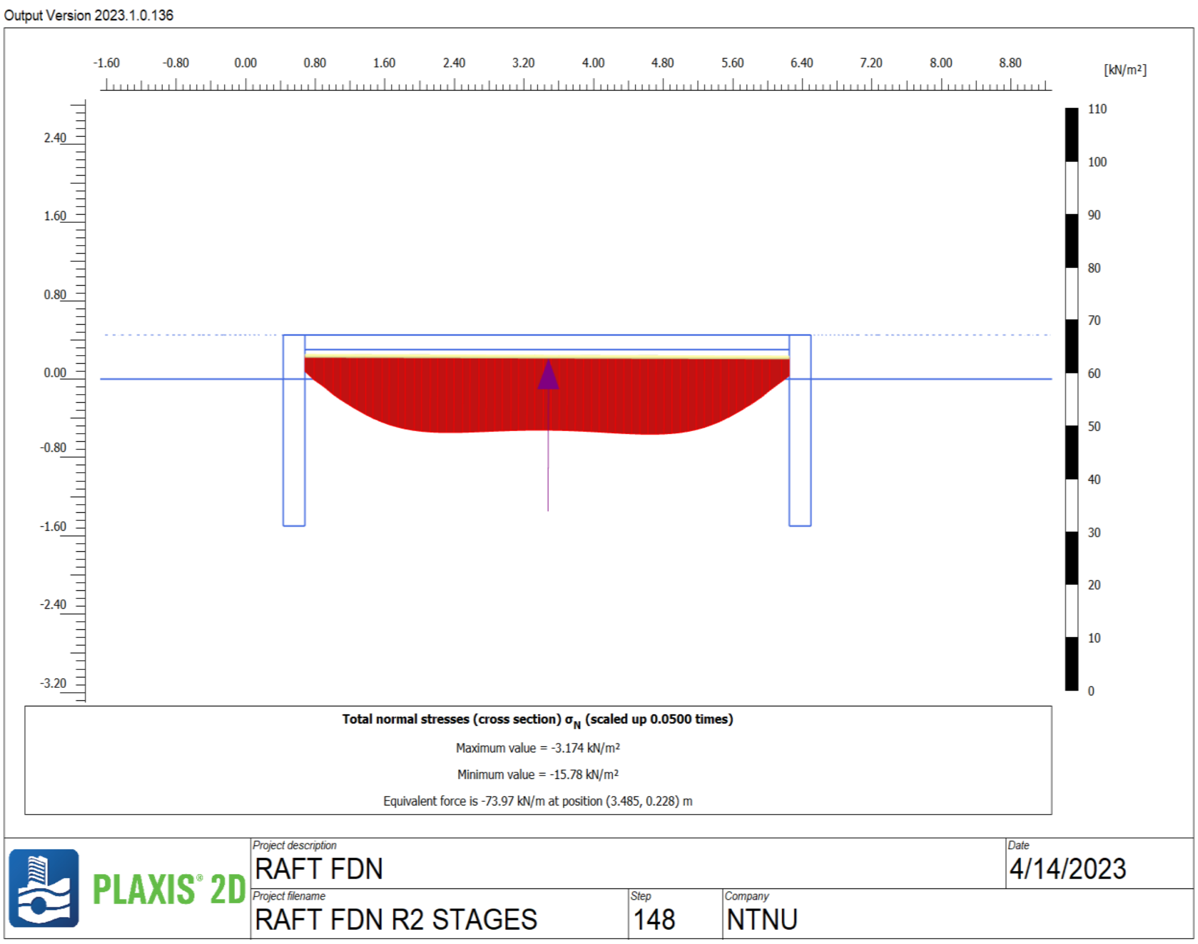

FIGURE 4.3.12: TOTAL NORMAL STRESS IN THE SOIL AT THE BOTTOM OF THE SLAB AS A RESULT OF THE LOAD

Similarly, the load from the slab is equally shown in Figure 4.3.12, where the reaction in the slab is 73.97kN/m due to the same foundation load from the structure. One can see that the load required to be supported by the raft slab is higher than in the beam despite the fact the foundation loads are directly on the beam and not on the slab. Although the stresses within the slab are considerably low in relation to the beam, this follows the same justification given above that the slab is seated on less rigid bodies (the soil and infill material) when compared to the beam that was found about 1.5m into the ground.

To demonstrate the above claim, there is a need to idealize the forces obtained in the foundation based on the applied loads, the resultants reactions obtained in the Plaxis model, the shear forces at both ends of the beams, and the self-weight of the beams/slab are listed below:

Imposed loads on the foundation is = 170kN/m (2 no);

Slab resultant reaction = 73.97kN/m (obtained in Plaxis model)

Beam resultant reaction = 60.95kN/m (obtained in Plaxis model)

Beam self-weight = 1.5*0.25* 24 *2 = 18kN/m (2 no calculated based on the size)

Slab self-weight = 6.3*0.2*24= 30.24kN/m (calculated based on size)

The shear stress on all sides of the beam (internal and external) = 4*τ_s

The algebraic sum of vertical forces is = +170 * 2 - 60.95*2 -73.97 + 18*2 + 30.24 – 4* τ_s = 0

τ_s = 48.1kN/m

The shear stress in the interphase is often transferred to the substructures (slabs and beam tips) after the effect of creep and relaxation in the soil.

Meanwhile, the shear stress obtained numerically above can also be determined in Plaxis, although the result obtained in Plaxis would require integrating over the entire length of the beam to determine the shear stress. Generally, the shear stress in the inner face of the beam is expected to be relatively smaller than the outer face of the beam.

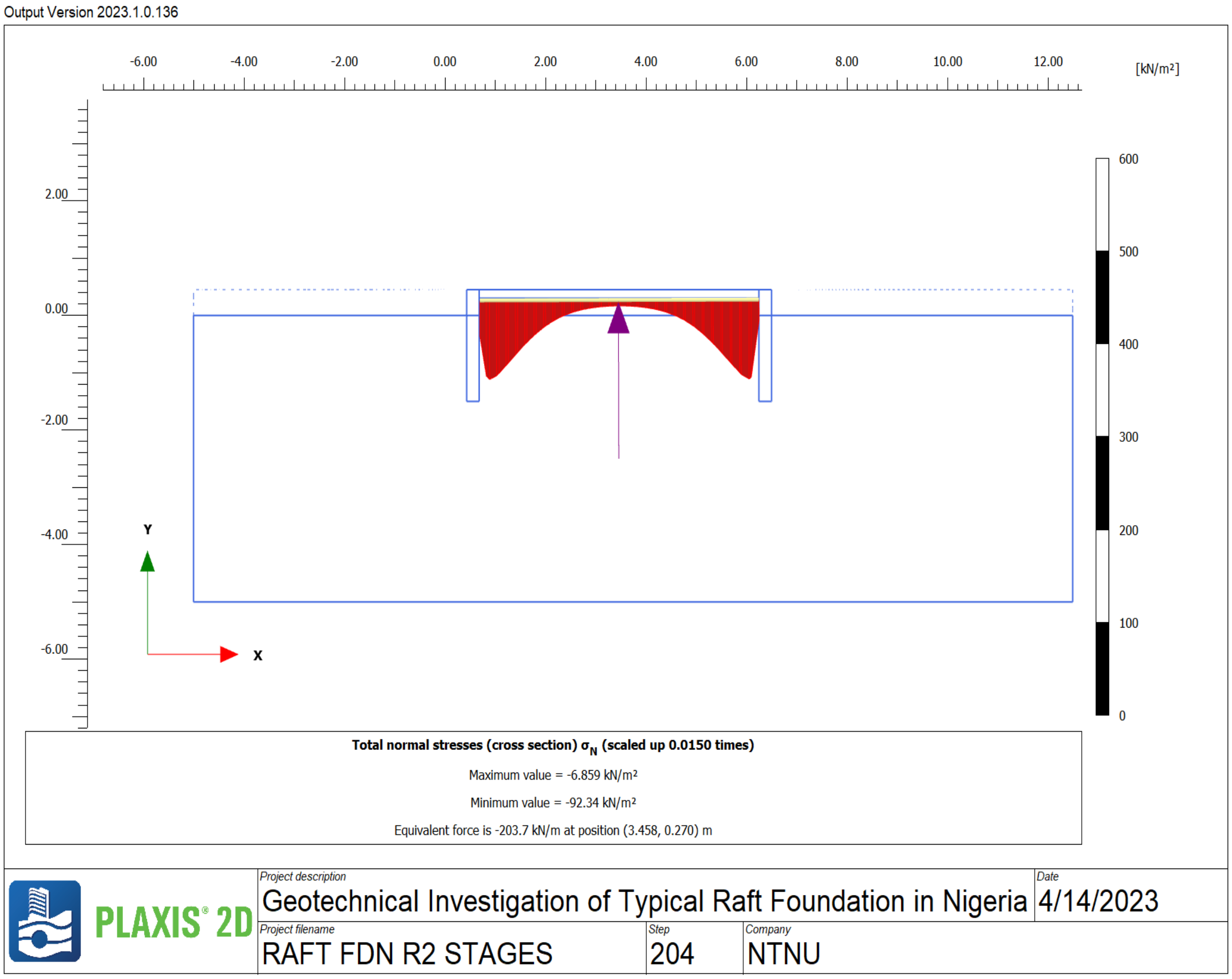

FIGURE 4.3.13: TOTAL NORMAL STRESS IN THE CONCRETE SLAB WHEN THE BEAM HAS REDUCED STIFFNESS

Figure 4.3.13 indicates the result of when the beam stiffness is reduced, this situation is aimed to simulate the effect of a dummy beam as earlier mentioned. This situation is required to clarify the concern of some engineers who opined that beams are just edge stiffeners. Meanwhile, from the result in Figure 4.3.13, there are concentrated stresses at the edges of the slab and less stress at the mid-span of the slab. The concentrated stresses are connected to a dummy beam placed at the edges.

Also, it was therefore noted in the same figure that the load carried by the slab increased from 73.97kN/m to 203.7kN/m which suggests now that all loads are now transferred to the slab. However, the total load supported by the slab should have been an algebraic sum of the beam and slab loads (60.95kN/m + 73.97kN/m = 134.92kN/m) from Figure 4.3.11 and 4.3.12. Meanwhile, the load in the slab (for the case of a dummy beam) is higher than the sum of the beam and slab load for some reason.

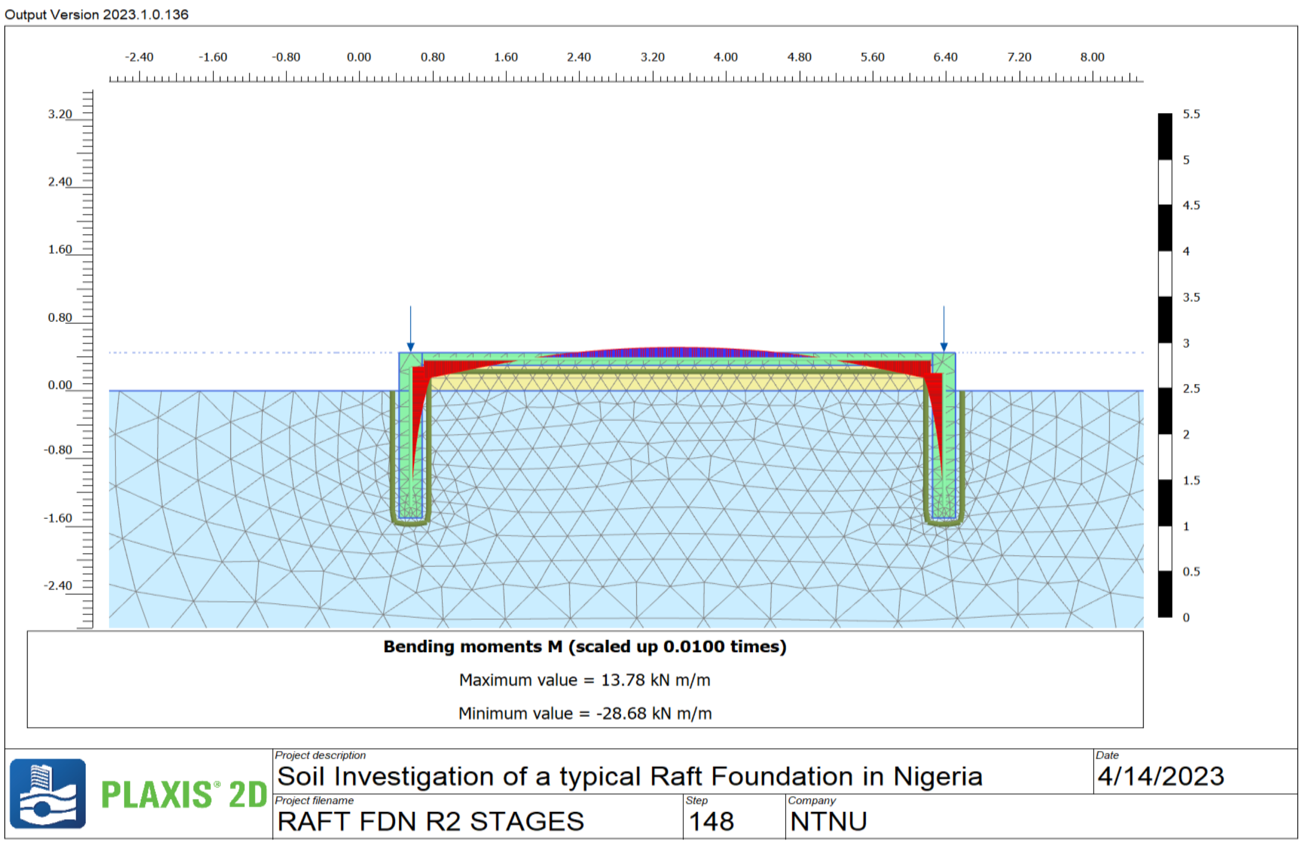

FIGURE 4.3.14: BENDING MOMENT IN THE RAFT FOUNDATION

The above is the bending moment diagram (but in addition with the shear force diagram) is normally used in providing reinforcements for bending and shear in the substructural elements. However, this would not be discussed further because it is not the focus of the research. Just for clarity that the maximum bending moment, in this case, is 28.68kNm/m and the minimum bending moment is 13.78kNm/m.

CHAPTER FIVE

5.0 Conclusion

The conclusions are extracted from the discussion of the results in Chapter Four and provide solutions to the project research questions stated in Chapter 1. The project research questions are quoted in italics and the findings of each are discussed afterwards.

- *Aim and Objective 1: To investigate how the loads from the superstructure are mobilizing contact stresses at slab level and beam level. The calculated contact stresses are then to be compared with the allowable pressure (bearing capacity) recommended in the geotechnical design report;*

Figure 4.3.8 and Figure 4.3.9 clearly show that there is higher contact pressure at the beam founding depth than at the slab level. The allowable pressure at the bottom of the beam is therefore mobilized before the pressure under the slab however, infill of the compacted materials can increase the load carried at the slab level.

- *Aim and Objective 2: To investigate how allowable bearing capacity vary with depth;*

This can be demonstrated from Figure 4.3.9 where is clearly shown that the contact pressure at the beam founding depth exceeds the recommended value 75kPa in the soil report whilst the pressure in the slab zone is within the recommended value. To equally support this from the soil report, it was observed that the bearing capacity is obtained from the correlation of SPT number of blows and bearing capacity. The number of blows has a direct proportionality to depth hence the bearing capacity under the beam is expected to be higher than the bearing capacity beneath the slab (this intuitively supports the forgoing argument).

- *Aim and Objective 3: To investigate how larger allowable bearing capacity at beam foundation depth than at slab level depth may influence the foundation design*;

This depends on the design criteria employed by the structural designers. If the beam and the slab connections are flexible, which means that there is the possibility of independent settlement, then the beam would be designed independently to carry all the foundation loads. The design can be tailored to be similar to a wall design where the connecting slabs can settle independently (that is the slab can be regarded to be on grade-this means concrete slab that is placed on earth or soil) through the joint. In contrast, when the connection is rigid, then the settlement is monolithic which then activates the slab to carry quite a large amount of loads once the whole building settles since it is now monolithic (this is the situation in this project work).

- *Aim and objective 4: To investigate the effect of settlement beneath the beam and how it transfers load to the infill materials under the slab.*

Figures 4.3.8 and 4.3.10 discussed in Chapter 4 show the pattern at which the whole structure settles since the building is monolithic, no differential settlement is expected. Albeit stresses are higher beneath the beam compared to the slab.

From the discussion under Figure 4.3.8 in Chapter where the settlement shows that the whole structure settles monolithically. This settlement compressed the infill material within the slab region which causes redistribution of stresses under the slab and results in the slab carrying extra loads. This load in the slab makes the central part of the slab to bulge.

Likewise, Figure 4.3.3 shows the idealized settlement pattern (free-body diagram) between the beam and slab. Figure 4.3.3 supports the above argument that is bulging of the central part of the slab. The central part of the slab settles less than the beam.

- Other findings during the investigation are as follows (although some findings have been exhaustively explained in Chapter 4):

I. The connection between the beam and slab can be enhanced by using a stiff member which could be achieved by placing adequate reinforcement at the joint. Embedded steel section (UB, UC, or welded pipes) could also be recommended however, the moment connection or rigid connection should be guided by following the recommended design procedures:

II. In the absence of ensuring a rigid connection between the beams and slabs, there may likely be a disconnect at the joint due to high shear force caused by the column loads (it is about 170kN/m in this case) placed directly on the raft beam. If the beam and slab are separated, then the beam in this situation would however act like a wall. The consequence of the beam acting like a wall would make a drastic change to the design because the beam was never designed as a wall. Structurally, a beam bends longitudinally hence the main reinforcement is placed horizontally while walls have an in-plain bending thus the main reinforcement is placed vertically. The beam is also not designed as a standalone but instead as an integral part of the raft slab (beam and slab foundation). Ordinarily, the stress under the beam is remarkably high when the beam and the slab are monolithic, the stresses under the beam would be much higher when there is a disjoint between the slab and the beam.

III. The Plaxis model also reveals that the beam is not redundant as some engineers may think but carries substantial loads/stresses as demonstrated in Chapter 4. The beam has some advantages as explained earlier and should not be completely removed. In a situation where value engineering is paramount, a toe beam (low depth or perimeter beam are usually referred to as toe beam in Nigeria) or tapered slab are highly recommended because of the stresses at the corners.

5.1 Recommendation

The quality of the result obtained is highly dependent on the quality of the input data hence it is recommended that there should be several quality checks in the input data. The quality checks should cover several laboratory tests and curve fitting using any good software that can curve fit laboratory tests like Plaxis. Several laboratory tests/experiments would likely prevent the use of correlation curves or empirical formula to determine unknown soil parameters, as this could be the source of errors.

It is also recommended that the filling material should also be extended to when sharp sand is used as a filling. Using sharp sand as a filling material would have a different laboratory result as compared to the lateritic fill material, then it would be highly recommended to explore the differences and similarities in the results of using sharp sand as an infill material.

Likewise, it is not practical to conclude the percentage of stress, or the load-carrying capacity of the beam based on one material and one site situation hence several tests and situations/variants are recommended before concluding.

Finally, there is a need to study further the flexible and rigid connections between the slab and the beam and establish the effect of having either a rigid or flexible connection.

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APPENDIX 1: SOIL REPORT

SUB-SOIL INVESTIGATION REPORT

FOR

A PROPOSED RESIDENTIAL BUILDING

AT

POLICE COLLEGE IKEJA, LAGOS STATE, NIGERIA.

THE AIG NIGERIA POLICE COOPERATIVE SOCIETY MULTI-PURPOSE SOCIETY LIMITED RESIDENT

APEX GEOSERVICES & GEOTECHNICS

APEX BN 2450347

September 2022

2				
1				
0	16/09/2022	Sub-Soil Investigation Report	A.A	S.A
Rev	Date Issued	Description	Reviewed By	Approved By

Address:11AhmedJimoh Close, Surulere, Lagos State
Mobile: 07038122555

REPORT CERTIFICATION

This report has undergone in-house quality control checks and has been certified. We appreciate the opportunity given to us to be of service to you on this project. Please, do not hesitate to call on us for any further clarifications.

Distribution:

This report is distributed as follows:

The AIG Nigeria Police Cooperative Society Multi-Purpose Society Limited Resident	2 copies
Apex Geoservices & Geotechnics	1 copy
Report issued	16th September 2022

Stamped By

Engr. Taiwo TIFASE
Civil Engineer

Issuing Authority

Azeez AKINYEMI
Technical Director
07038122555

SYNOPSIS

Client:	The AIG Nigeria Police Cooperative Society Multi-Purpose Society Limited Resident
Project:	Sub-Soil Investigation
Contractor:	Apex Geoservices & Geotechnics
Location:	Police College Ikeja, Lagos State,Nigeria.
Date:	September 2022
Water Level:	Surface
Accessibility:	Site is motorable with good access roads
Drainage:	Not established
Soil Types:	CLAY and SAND
Discussion of Results:	Test results show that the top soil will support Pad foundation for gate house, fence, generator house and A-floor (A Bungalow). For any structure more than one floor, **Raft** foundation should be considered and founding depth placed between **1.00m** and **2.00m** with bearing capacities ranging from **54.0kN/m^2** to **81.0kN/m^2**. The consistent increase in bearing capacity makes the site suitable for **RAFT** foundation considering oncoming loads. NOTE: Considering the recommended depth above, RAFT FOUNDATION should only be considered for any design that is not more than 3-floors(2-storey building). For any design that is more 3 floors, Deep (PILE) FOUNDATION should be considered.

Recommendation: Concluding from above, Light structures can be placed on pad foundation while for a structure that exceeds one floor, Raft foundation should be considered and the founding depth placed between **1.00m** and **2.00m** with bearing capacities between **42.5kN/m²** and **75.6kN/m²**.

In-situ bored pile should be tipped at the depth of **-18.0m** with a bearing capacity of **259.2kN/m²** to achieve the computed carrying capacities tabulated above.

P.S: For a structure that is more than Three floors (2-storey building), Pile foundation should be considered and the pile foundation should be terminated at 18mts depth and the pile diameter should vary between 450mm and 500mm with bearing capacity of 259.2kN/m²

The Engineer is at liberty to choose any diameter that is suitable for his design.

The summary above should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section and the report must be read fully to gain comprehensive understanding of the items contained therein.

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ABBREVIATIONS

SPT:	STANDARD PENETRATION TEST
CPT:	CONE PENETROMETER TEST
QC:	CONE RESISTANCE
BH:	BOREHOLE
N-VALUE:	NUMBER OF BLOWS

1. INTRODUCTION

Apex Geoservices & Geotechnics was contracted by The AIG Nigeria Police Cooperative Society Multi-Purpose Society Limited Resident to carry out a detailed sub-soil investigation on its proposed development at Police College Ikeja, Lagos State,Nigeria.

The purpose of the sub-soil investigation and the attendant report is to:

- Determine the sub-soil and surface/groundwater conditions of the designated locations.
- Evaluate the subsoil stratigraphic sequence, geotechnical/engineering properties of the sub-soil and the subsequent effect on foundation design and construction.
- Carry out laboratory tests, analyze the data obtained and write a report with recommendations on the fit-for purpose of foundation for the structure.

Site visit was carried out in June 2021. Mobilization and site operations were carried out in June 2021 using a percussion drilling machine to drill one (1) geotechnical boreholes.

This report presents a detailed description of the field activities, results of in-situ tests carried out at the site location and laboratory analysis of recovered samples. It represents only the actual spots examined.

2. SITE DESCRIPTION AND ACCESSIBILITY

The site is located at Police College Ikeja, Lagos State,Nigeria. The site is motorable with good access roads.

3. SCOPE OF WORK

The study involves determining the type, nature and mechanical strength and properties of substance materials at the site through the performance of geotechnical boreholes.

The scope of work involves the following:

- Drill One geotechnical boreholes.
- Recover soil samples for laboratory analysis.
- Execute Two CPT to the refusal of cone penetration and/or anchor pull.
- Carry out detailed laboratory analysis on recovered samples.
- Write a report detailing findings of the soil investigation.

All field results and daily reports were logged at the end of each operation day.

3.1 DESCRIPTION OF WORK

3.1.1 CPT Test

This is a direct sounding test which gives a continuous record of variation of penetration resistance with depth. It involves pushing a 3.57cm base diameter cone, with and apex angle of 60° attached to cylindrical rodsthroughthesubsurfaceataspeedof2cm/sec.

The Cone Penetrometer Test (CPT) was performed using a 2.5-ton capacity machine. The equipment, hydraulically operated, is furnished with a cone penetrometer with a cross-sectional area of 10cm^2 and an apex angle of 60°. As the cone was advanced through the sub-surface, the tip of the cone measured the resistance of the underlying soil strata as ***cone resistance (qc)***.

All the dial gauges used in carrying out the tests were calibrated prior to the commencement of the test and hence the results obtained are reliable. The test was terminated when the machine had achieved its maximum capacity and could no longer penetrate. The penetrometer test readings were taken at intervals of 20cm and the readings are presented in graphical form in the appendix section of this report. Groundwater level was also monitored through the CPT rods during investigation.

3.1.2 GEOTECHNICALBOREHOLETEST

The boreholes were executed using a steel cable attached to a shell and sinker bar powered by a percussion drilling machine. The machine was equipped with in-situ standard penetration test (SPT) accessories. Sampling and in-situ testing were carried out progressively with the advancement of the borehole through the overburden as follows:

- Disturbed samples were taken within the sediments at regular interval of0.75m and at change of strata as deemed necessary. This is considered representative of the materials encountered in the course of drilling.
- Standard penetration tests (SPT) were carried out at 3.0m interval to determine penetration resistance in cohesionless strata. The test involved obtaining the number of blows (N-values) through a penetration of 305mm (1foot) by a 65kg hammer falling through a height of 75cm.Materials from the split spoon sampler in the standard penetration test (SPT) and cutting shoe of the 100mm sampler were also taken as disturbed samples.

3.2 TOPOGRAPHY AND DRAINAGE

Generally, the topography of the site is plain.

3.3 GROUNDWATER LEVEL

Groundwater level fluctuates with seasonal changes. The water levels observed at the borehole points was at -8mts during the time of this investigation.

4. STRATIGRAPHY

4.1 GEOLOGIC SETTING

The area under investigation lies within the alluvial deposits of the South-West Nigeria formed during the Quartenary period. It is an integral part of the Dahomey Basin. This alluvial deposit is underlain by the coastal plain sand of the Eocene and its main features are clays and sands. The general depositional sequence in the basin is as follows:

TABLE 4-1: DEPOSITIONAL SEQUENCE OF DAHOMEY BASIN

Name of Formation/Sediment Type	Description	Age (Oldest at the bottom)
Benin Formation	Clays and Sands	Miocene to Pliocene to Recent
Ameki Formation	Clayey Sandstone and sandy Claystone	Miocene
Ilaro Formation	Sands	Upper Eocene
Oshosun Formation	Mudstones and Claystones	Mid Eocene
Imo (Akinbo) Formation	Shale	Lower to Middle Eocene
Ewekoro Formation	Limestone	Upper Paleocene
Araromi Formation	Shales, Siltstones, interbeds of Limestones and Sands	Maestrichtian to Paloecene
Afowo (Abeokuta) Formation	Sands and Sandstone	Maestrichtian

4.2 LOCAL GEOLOGY

The local geology of the site under investigation consists of recent sedimentary deposits of clay, sand and gravel.

4.2.1 SUB-SOIL CONDITION

***From -0.00m to -0.75m**: is made up of Dark Silty CLAY.*

***Below -0.75m to -3.00m**: is made up of Reddish Lateritic CLAY with N.S.P.T value of 6.*

***Beneath -3.00m to -7.50m**: is made up of Reddish Brown Hard Lateritic CLAY having N.S.P.T values ranging from 8 to 11.*

***Below -7.50m to -11.25m**: is made up of Reddish Brown Very Hard Lateritic CLAY with N.S.P.T values ranging from 14 to 17.*

***Beneath -11.25m to -15.75m**: is made up of Reddish Lateritic Hard CLAY with Traces of SAND having N.S.P.T values ranging from 19 to 22.*

***From -15.75m to -17.25m**: is made up of Reddish Brown Lateritic Sandy CLAY.*

***Below -17.25m to -19.50m**: is made up of Brownish Red Lateritic Medium Dense SAND with N.S.P.T values of 24*

***Underneath -19.50m to -21.00m**: is made up of Brownish Grey Medium Dense SAND.*

***Finally, from -21.00m to -25.50m**: is made up of Whitish Medium Dense SAND with N.S.P.T values ranging from 26 to 28.*

5. FOUNDATION DISCUSSION AND RECOMMENDATION

5.1. DESIGN DETAILS

Result discussion and analysis shall be based on empirical interpretation and correlation of the test data and our wide experience on similar jobs in this part of the country.

The foundation type to be chosen for a particular structure depends largely on the following:

i. Loads to be transmitted.
ii. Receiving soil strata.
iii. Settlement should neither cause any damage nor interface with the function of the structure.

The choice between shallow foundation and deep foundation can be arrived at after careful consideration of the following elements.

i. The magnitude of the transmitted loads from the stratum.
ii. The soil nature.
iii. The economic aspect of the element of the foundation work.
iv. Problems concerning foundation construction.

5.2. FOUNDATION RECOMMENDATION

5.2.1. SHALLOW FOUNDATION

The type of foundation that can be adopted in any engineering construction is usually determined by the kind of structure to be supported, designed load and soil conditions encountered.

Test results show that the top soil will support Pad foundation for gate house, fence, generator house and A-floor (A Bungalow). For any structure more than one floor, **Raft** foundation should be considered and founding depth placed between **1.00m** and **2.00m** with bearing capacities ranging from **54.0kN/m^2** to **81.0kN/m^2**. The consistent increase in bearing capacity makes the site suitable for **RAFT** foundation considering oncoming loads. NOTE: Considering the recommended depth above, RAFT FOUNDATION should only be considered for any design that is not more than 3-floors(2-storey building). For any design that is more 3 floors, Deep (PILE) FOUNDATION should be considered.

See table below for allowable bearing capacities for the top **5.00m.**

TABLE 5-1: ALLOWABLE BEARING CAPACITIES TO-5.00M

Depth(m)	Bearing Capacity(KN/m²)
1.00	54.0
2.00	81.0
3.00	94.5
4.00	121.5
5.00	135.0

5.2.2. <u>DEEP FOUNDATION</u>

As stated above, Deep (pile) foundation should be considered for any structure that exceed 3-floors(2-storey building), In-situ bored pile can be considered. The pile diameter recommended should vary between **450mm** and **500mm** and should be designed to be tipped at a depth of-**18.0m** with a bearing capacity of **259.2kN/m²** across the length of the structure.

Table 5-1 below shows the recommended pile diameters and their corresponding working loads at **-18.0m**.

TABLE 5-2: PILE SIZES AND WORKING LOADS AT -18.0 METERS

Depth (m)	Diameter (mm)	Base Resistance (kN)	Positive Skin Friction (kN)	Negative Skin Friction (kN)	Safe Working Load (kN)	Ultimate Working Load (kN)	Factor of Safety
-18.0	450	548.90	542.94	70.70	340,38	1021.14	3.0
-18.0	500	677.58	603.26	78.56	400.76	1202.28	3.0

5.3. SUMMARY AND CONCLUSION

Concluding from above, Light structures can be placed on pad foundation while for a structure that exceeds one floor, Raft foundation should be considered and the founding depth placed between **1.00m** and **2.00m** with bearing capacities between **42.5kN/m²** and **75.6kN/m²**.

In-situ bored pile should be tipped at the depth of **-18.0m** with a bearing capacity of **259.2kN/m²** to achieve the computed carrying capacities tabulated above.

P.S: For a structure that is more than Three floors (2-storey building), Pile foundation should be considered and the pile foundation should be terminated at 18mts depth and the pile diameter should vary between 450mm and 500mm with bearing capacity of 259.2kN/m²

The Engineer is at liberty to choose any diameter that is suitable for his design.

5.4. GENERAL

We strongly recommend that the design and construction of the foundation and all earthworks be executed in accordance with good engineering practice as embodied in recognized Codes of Practice such as the British Standard Institutions BS. 8004: 1986.Code of Practice for Foundations and BS.6031: 1981.Code of Practice for Earthworks.

APPENDIX A: BOREHOLE LOGS AND CPTS

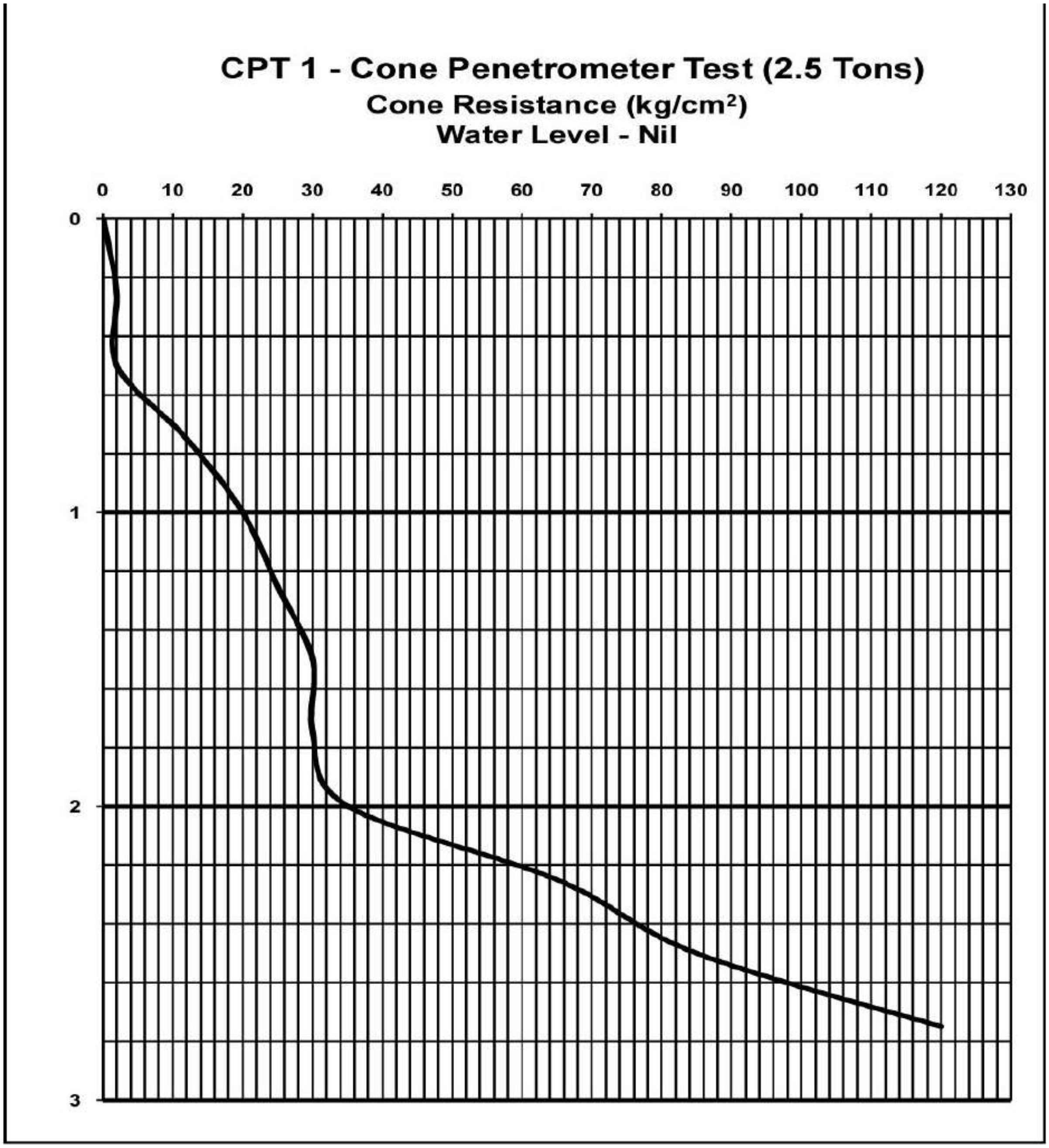
Client:	The AIG Nigeria Police Cooperative Society Multy-Purpose Society Limited Resident
Project	Proposed Development
Location:	Police College Ikeja, Lagos State,Nigeria
Date:	Sep-21
Borehole No:	BH-1

Depth (m)	Sample	Sample Description	NSPT VALUE (0–70)
0.00	D	Dark Silty CLAY	
0.75	D		
1.50	S	Reddish Lateritic CLAY	6
2.25	D		
3.00	S		8
3.75	D		
4.50	D	Reddish Brown Hard Lateritic CLAY	
5.25	D		
6.00	S		11
6.75	D		
7.50	S		14
8.25	D		
9.00	S	Reddish Brown Very Hard Lateritic CLAY	15
9.75	D		
10.50	S		17
11.25	D		
12.00	S		19
12.75	D	Reddish Lateritic Hard CLAY With Traces Of SAND	
13.50	D		
14.25	D		
15.00	S		22
15.75	D	Reddish Brown Lateritic Sandy CLAY	
16.50	D		
17.25	D		
18.00	S	Brownish Red Lateritic Medium Dense SAND	24
18.75	D		
19.50	D	Brownish Grey Medium Dense SAND	
20.25	D		
21.00	S		26
21.75	D		
22.50	D		
23.25	D	Whitish Medium Dense SAND	
24.00	S		28
24.75	D		
25.25	D		

Key

D	DISTURBED SAMPLE
S	STANDARD PENETRATION TEST

FIGURE 5-1: BOREHOLE LOGS


CPT 1 - Cone Penetrometer Test (2.5 Tons)
Cone Resistance (kg/cm²)
Water Level - Nil
0
10
20
30
40
50
60
70
80
90
100
110
120
130
0
1
2
3

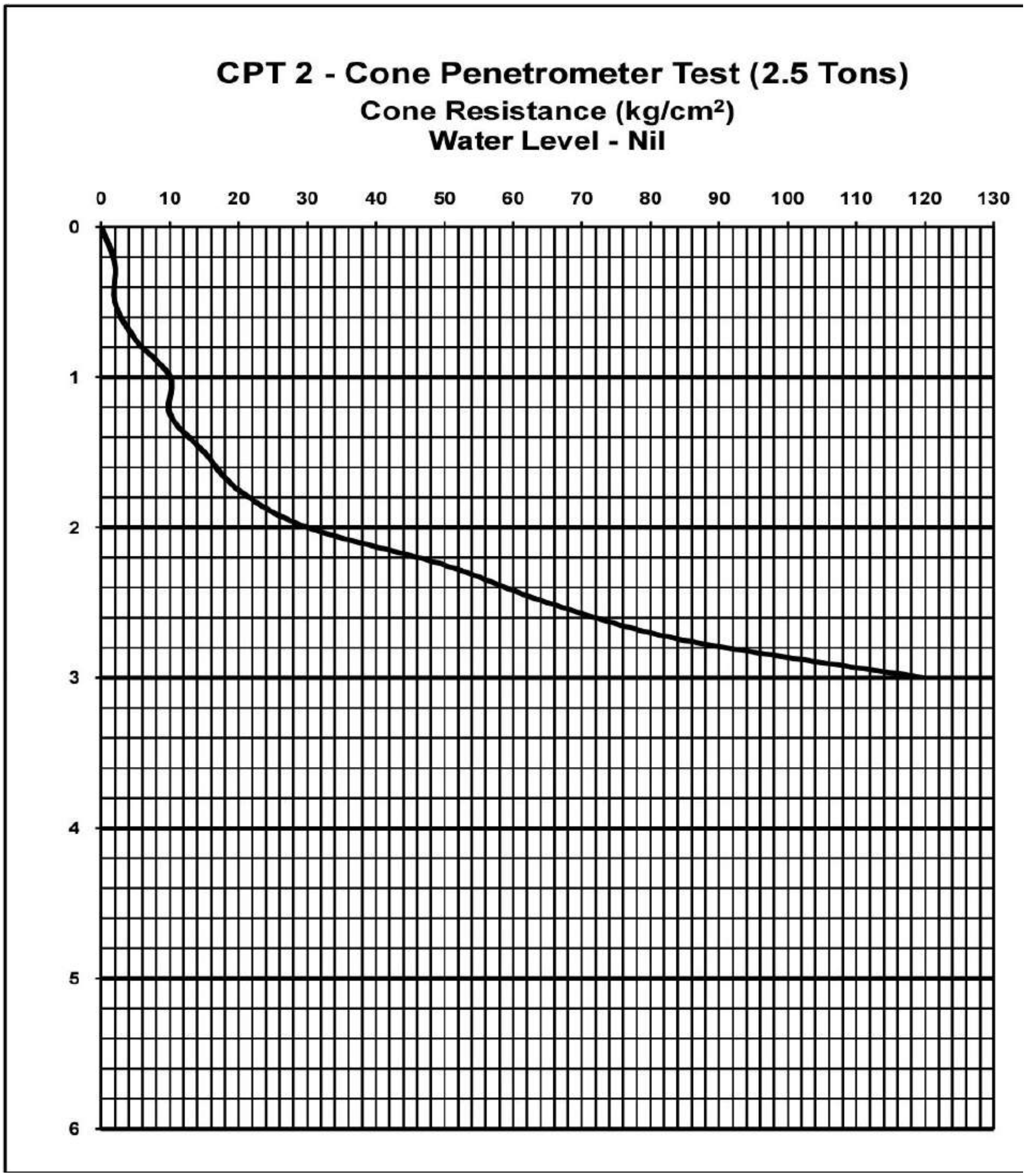
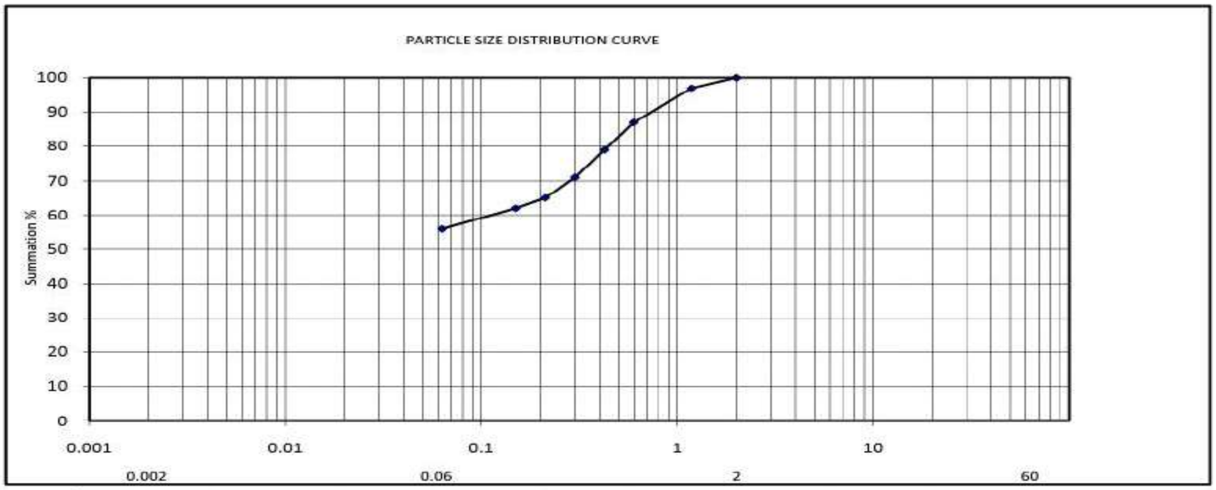


Figure 5-2: CPTS Logs

APPENDIX B: LABORATORY ANALYSIS

OPTIMAL GEMS RESOURCES LIMITED LABORATORY
PARTICLE SIZE DISTRIBUTION GRAPH

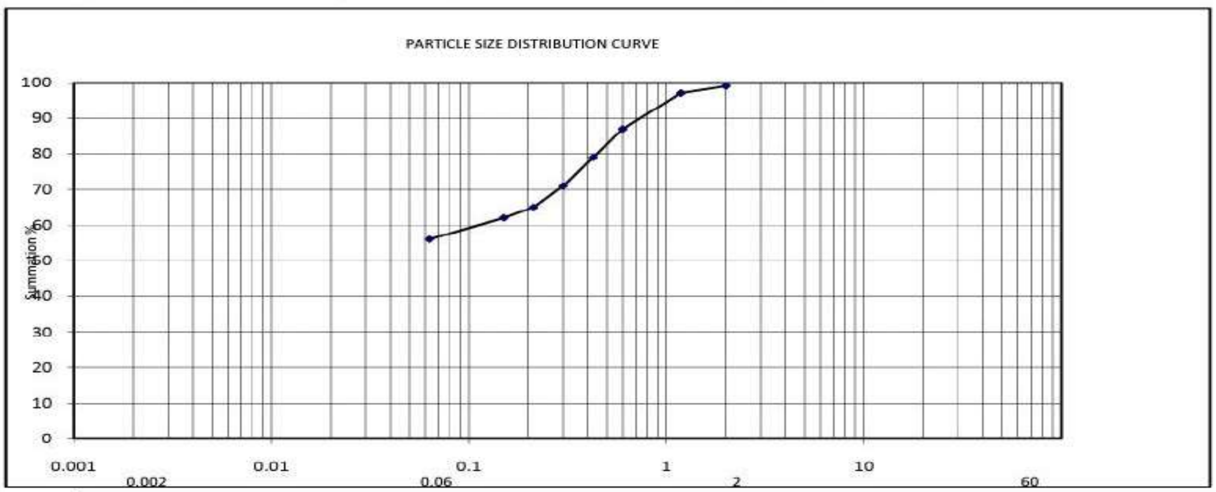
PROJECT: PROPOSED BUILDING DEVELOPMENT **DATE:** Sep-22
LOCATION: Police College, Ikeja, Lagos State.
BH/SAMPLE NO: 1/13 **DEPTH:** 9.0m



Clay Fiction ←	Silt Fraction			Sand Fraction			Gravel Fraction			Boulders →
	fine	med	cse	fine	medium	coarse	fine	med	cse	

Particle Size in mm

PROJECT: PROPOSED BUILDING DEVELOPMENT **DATE:** Sep-22
LOCATION: Police College, Ikeja, Lagos State.
BOREHOLE/SAMPLE NO: 1/21 **DEPTH:** 15.0m



Clay Fiction ←	Silt Fraction			Sand Fraction			Gravel Fraction			Boulders →
	fine	med	cse	fine	medium	coarse	fine	med	cse	

Particle Size in mm

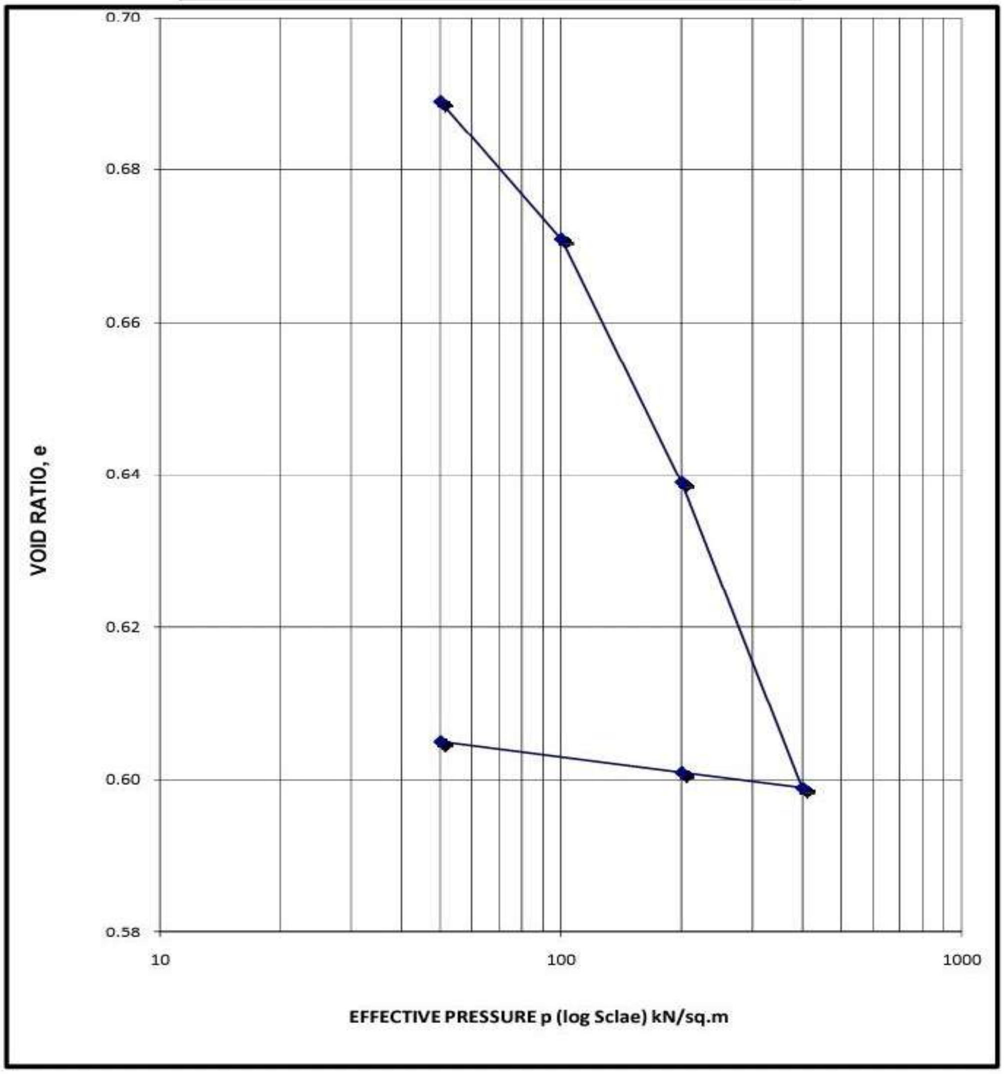
CONSOLIDATION TEST

(e-log p curve)

CONTACT: **PROPOSED BUILDING DEVELOPMENT AT POLICE COLLEGE, IKEJA,LAGOS STATE.**

SAMPLE NO.: **1/3** **DEPTH** **1.50m**

INITIAL MOISTURE CONTENT	=	19.0%
INITIAL WET DENSITY	=	1.89 Mg/M^3
INITIAL VIODS RATIO	=	0.709

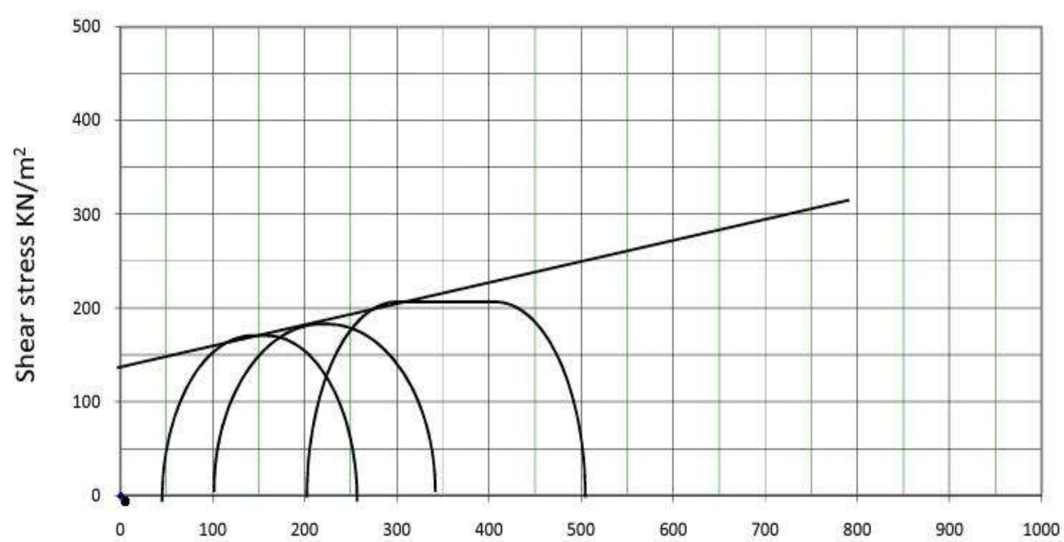


QUICK UNDRAINED TRIAXIAL - COMPRESION TEST **MOHR CIRCLES DIAGRAM**

CONTRACT: **PROPOSED BUILDING DEVELOPMENT AT POLICE , COLLEGE, IKEJA, LAGOS STATE.**

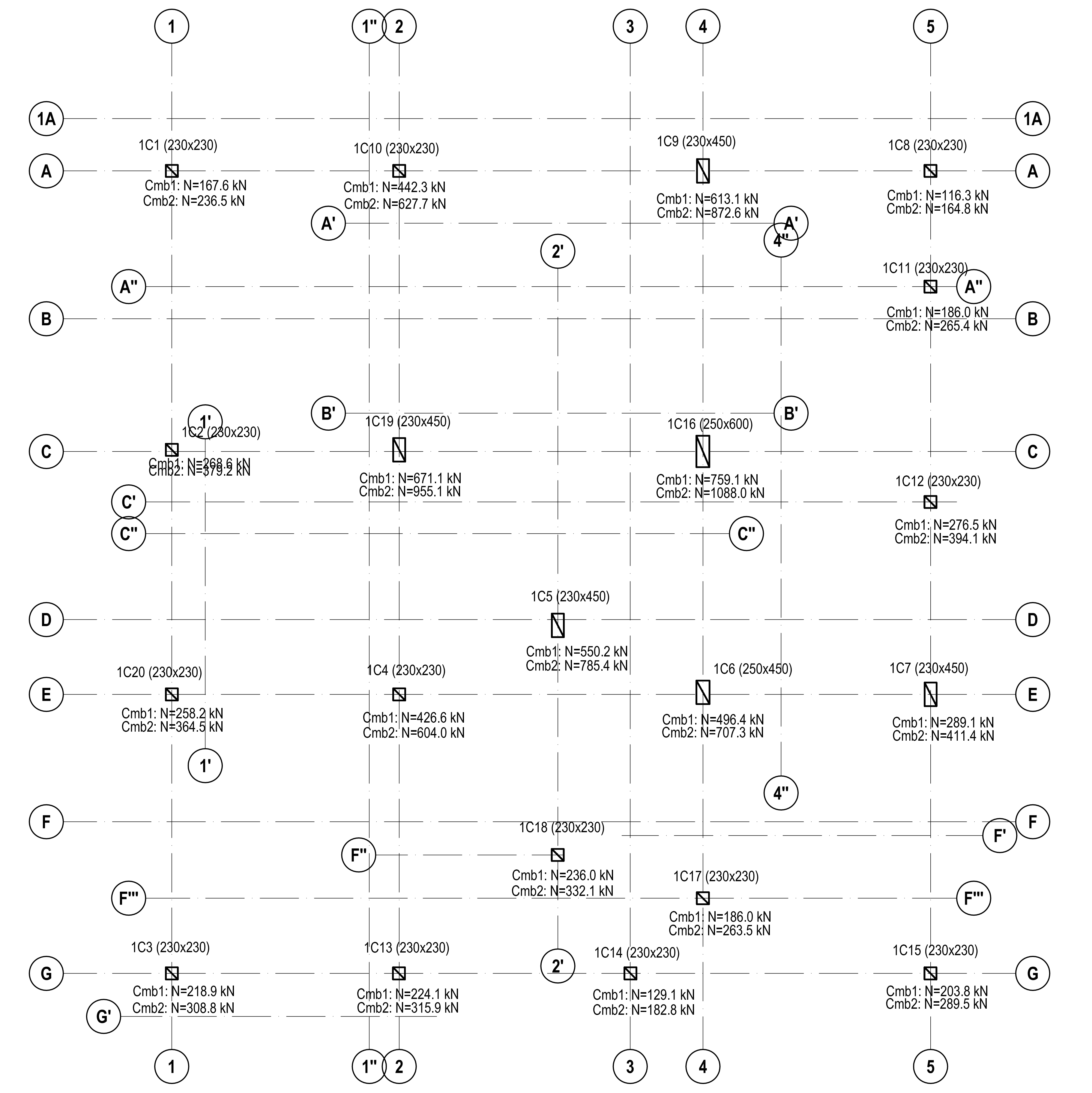
BH NO: 1/3 **DEPTH:** 1.50m

Undrained Cohesion (Cu):	= 67.0kN/m²
Undrained Angle of Friction (Ø)	= 12.0 Deg



APPENDIX C: PICTURES

APPENDIX 2: COLUMN LOADS



Project Title
PROPOSED DEVELOPMENT LAGOS STATE.

Client
MUSA

Consultant

Consultant
MIBBS CONSULT (Civil Structural & Project Mgt.)
16, Chief Mike Close, Ajao Estate,
Lagos
Lagos State.
Email; musasegun45@gmail.com
Tel; +2348034223279

Drawing Title
LOAD TAKE DOWN
Drawing No. PRB-L-01
Rev
-

Job No	
Date	06/16/22
Drawn By;	TOM
Checked By	MUSA
SCALE	

Seal / Stamp ©
Approver
Comments APPROVAL

12/06/2023 8:58:23 pm

052

APPENDIX 3:OEDOMETER / CONSOLIDATION

$G_s = 2.59$
$V_s = 36.911$
$V_v = 33.012$
$e_o = 0.894$
$F = 0.102$

UNIVERSITY OF LAGOS
Civil Engineering Department
Soil Mechanics Labouratory

CONSOLIDATION TEST

Operator
Date
Specimen No

Site
Borehole No.
Sample No.
Depth of Sample

Date													
Applied Pressure kN/m²													
Mean daily Temperature (°C)													
Time (t)	√t 1	Clock Time 2	Dial Reading 4	Deflection (mm) 8	Clock Time 16	Dial Reading	Deflection (mm) 32	Clock Time	Dial Reading	Deflection (mm)	Clock Time	Dial Reading	Deflection (mm)
0	—	25	77	160	281								
16 sec	20	20	157	267	415		447	375					
36 sec	21	71	152	269	418								
1 min	22	72	153	270	421								
2¼ "	22	73	154	271	424								
4 "	23	74	155	272	427								
6¼ "	23	74	156	273	430								
9 "	23	75	156	274	433								
12¼	24	75	157	275	435								
16	24	75	157	276	438								
25	24	76	158	277	440								
36	24	76	158	278	442								
49	25	76	159	279	444								
64	25	77	159	280	445								
81	25	77	160	281	446								
100		77	160	281	447								
121							375	290					
144													
1 day													
A	0.050	0.154	0.320	0.562	0.894		0.250	0.580					
B	18.550	18.446	18.280	18.038	17.706		17.850	18.020					
C	18.600	18.550	18.446	18.280	18.038		17.706	17.850					
D	0.050	0.104	0.166	0.242	0.332		—	—					
Δw	0.108	0.226	0.182	0.134	0.094		—	—					
Δe	0.005	0.016	0.033	0.057	0.091		0.077	0.059					
e	0.889	0.823	0.840	0.783	0.692		0.769	0.828					
w	9.3	8.0	7.9	6.8	5.8								

0.714 0.750

$\omega = 80$
$\phi = 9$

Curtsy Emma Igbokhazia
Msc Civil 2001/2002

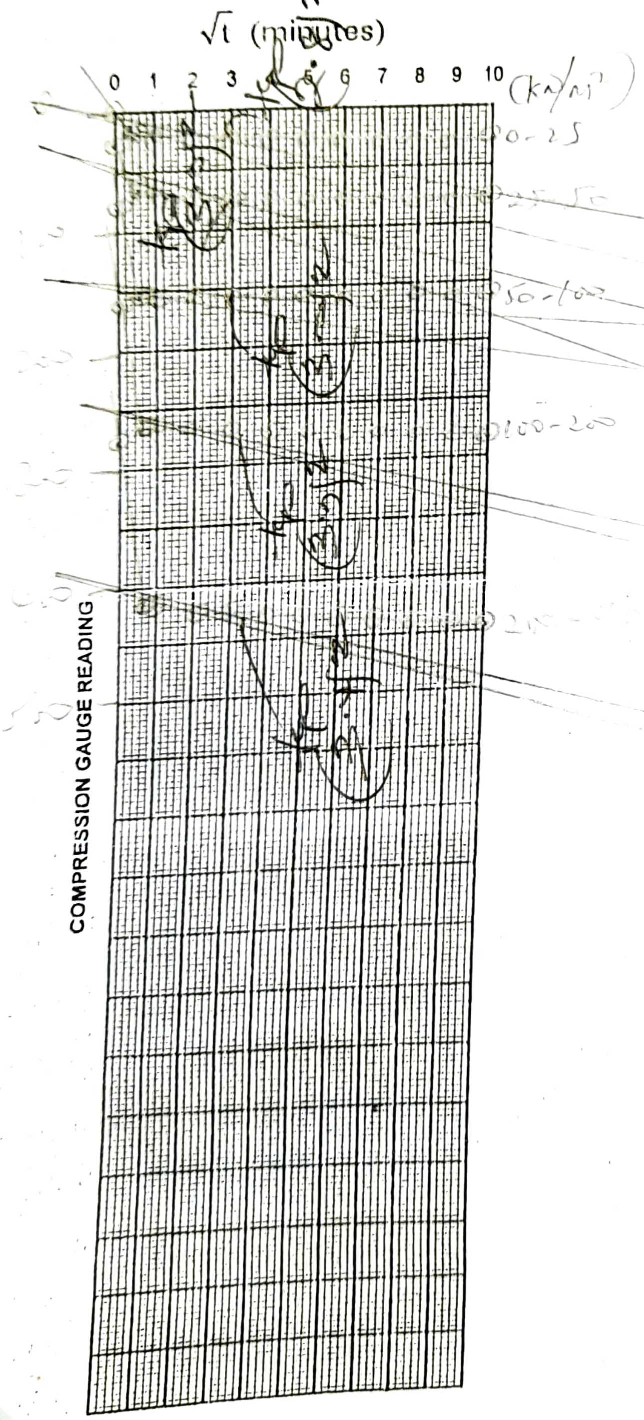
SOIL MECHANICS LABORATORY

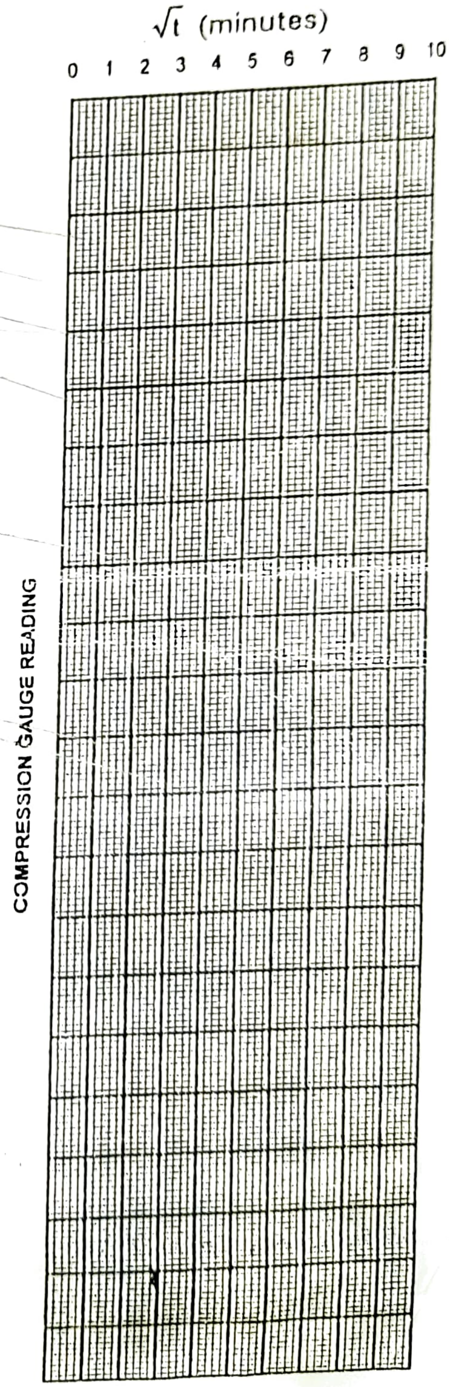
NAME:.. MATRIC NO:..

DEPT:.................................. GROUP:.................................. DATE:........................

SQUARE ROOT OF TIME FITTING METHOD

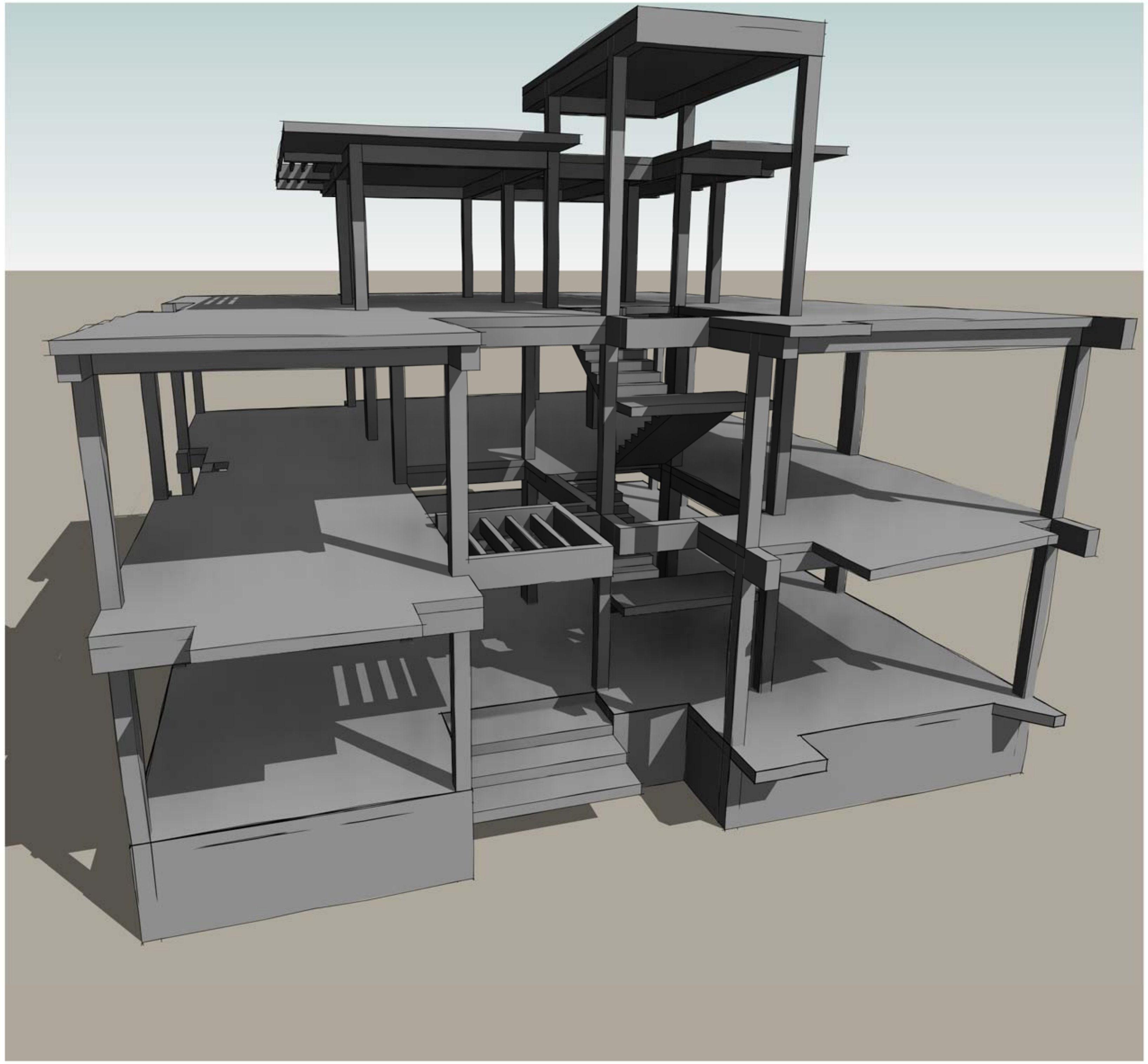
BOREHOLE/SAMPLE NO:.. DEPTH:..





APPENDIX 4: GENERAL ARRANGEMENTS DRAWINGS

PROPOSED RESIDENTIAL DEVELOPMENT



(STRUCTURAL DRAWINGS FOR CONSTRUCTION)

Client

POLICE AIG

Consultant

MIBBS CONSULT.

MIBBS CONSULT.
(Civil Structural & Project
13, Chief Mike Close, Ajao
Estate

C

September 2022

GENERAL

1. Refer to Design Development Report for a description of the Structure, the Design Criteria and the Design Loads.

2. Refer to the Design Development Report for a description of the structural behaviour which may impact on construction.

3. Structural Drawings are to be read in conjunction with all relevant Architectural and Services Drawings.

4. Any discrepancies shall be referred to the PM for a decision before proceeding with work.

5. All dimensions are in mm UNO.

6. All levels are in metres. refer to Architect's drawings
for Datum Level.

7. No dimensions are to be obtained by scaling from drawings.

8. All columns are central on grid UNO. Where vertical structure locations and dimensions are established at a level they are defined at the levels above this UNO.

9. All materials and workmanship shall be in accordance with the current British Standards and Codes of Practice.

10. Where proprietary products are specified, the Contractor may substitute alternative materials with equal performance, dimensions and specification, provided that he gains approval from the PM for such substitution.

11. Temporary loads imposed during construction are to be agreed with the PM, and are to be within the limits of the permanent loads.

12. The Contractor shall be responsible for the design, fabrication, erection and removal of all temporary works and shall provide all temporary bracing and propping necessary to maintain stability during construction.

EARTHWORKS

1. For Existing Ground level refer to Architect's drawings

2.Excavation & Filling to be in accordance with the National Building Specification (NBS).

3. Refer to the Structural Drawings for excavation requirements for the substructure.

4. Bulk excavation generally in existing Remediated Fill. All material to be retained and stockpiled on site , for later use in landscape works. Topsoil to be stockpiled separately.

5. The use of site-won fill beneath civil or structural works is at the discretion of the Contractor, subject to satisfying the specification requirements for fill materials.

6. Stability of stockpiled fill and safe clearance to work operations is the responsibility of the Contractor.

FOUNDATIONS

1. Raft Foundations are to be adopted for the structure

2. A competent geotechnical engineer or engineering geologist is to be employed by the Contractor to inspect the formation of every foundation immediately prior to concreting.

3. Loose or unsatisfactory fill is to be removed. Backfill below specified founding level shall be mass concrete grade C16/20.

4. All exposed formations shall be kept dry and shall be immediately protected from softening by blinding of the surface.

5. Design chemical class for foundations is to be confirmed

CONCRETE

1. All Concrete Works shall comply with the National Structural Concrete Specification, Third Edition, published by the BCA.

2. Designed Concrete Grades:

- Blinding C8/10
- Mass concrete C16/20
- Pad, strip, and raft foundations, including upstands C20/25
- Lower Ground bearing slabs including monolithic retaining walls and upstands C20/25
- Superstructure C20/25 UNO

5. Concrete Finishes

Floor finishes are to be confirmed

6. Reinforcement

Refer to the Reinforcement Estimate for Tender purposes.

a) Reinforcement to be B500B to BS4449: 2005,
characteristic strength 500N/mm².

b) Mesh reinforcement to be B500 to BS4483: 2005.

c) The Contractor is responsible for providing all stools and spacers required for adequately supporting the reinforcement cages.

7. The Contractor is to provide sleeves/box outs for services openings as shown on the services drawings and services builderswork drawings. Do not cut reinforcement that may conflict without approval.

SETTING OUT

1. For Grid Setting out refer to Architectural Drawings

SITE AND GROUND CONDITIONS

1. Refer to Ground Investigation Report for Ground Conditions, when this becomes available.

SUBSTRUCTURE WATERPROOFING, & INSULATION

1. To Be provided at the next stage of design

HOLES, PLINTHS, UPSTANDS INSERTS AND FIXINGS

1. The Contractor is responsible for co-ordination and checking all service openings through the structure.

2. No holes other than those shown on the structural drawings are to be formed in the structure without approval.

3. Holes of maximum dimension equal to or less than 200mm x 200mm are not shown on the structural drgs. Refer to Architectural Drawings and approved builders work drawings. Openings, pockets etc larger than 200mm shall not be placed in the structure unless specifically detailed on the Structural Drawings, without prior approval of PM.

4. The Contractor is responsible for co-coordinating and checking all fixings to the structure at sub-contract package interfaces. Refer to Sub-contractors drawings.

5. For details of all other cast-in elements such as ducts, pipes, lightning protection and lift shaft inserts, refer to the Services Drawings.

6. The metal, glass and other cladding systems are not shown on the structural drawings. For details refer to the Architectural Drawings & relevant sub-contractors drawings.

STRUCTURAL STEELWORK

1. All steelwork is to comply with the National Steelwork
Specification for Building Construction, 5th Edition, published by BCSA/SCI.

2. Shop drawings are to be issued, for examination by the PM, at least three weeks prior to commencement of fabrication.

3. No steelwork shall be fabricated until all review comments on the workshop drawings have been resolved to the PM's satisfaction.

4. All dimensions and levels (including HD Bolts as constructed) affecting new steelwork shall be checked on site and incorporated in workshop drawings.

5. All reactions where shown are in kN UNO.

6. All steel shall be Grade S275 UNO.

7. All Light Gauge Steel Framing to be in accordance with specialist manufacturer's standard details. Additional notes below apply only to rolled steel.

8. All plates shall be 10mm thick UNO.

9. All bolts shall be M16 UNO. No steel to steel connection shall have less than 2 M16 bolts UNO.

10. M12 and smaller bolts shall be Grade 4.6 UNO. M16 and larger bolts to be Grade 8.8 UNO.

11. All bolts to be snug tightened UNO. All bracing connections to be 'non-slip' and employ HSFG bolts.

12. All welds are to be a minimum of 6mm FW UNO.

13. All Butt Welds are full penetration UNO.

14. All Fillet Weld sizes are leg length UNO.

15. Non- destructive testing of welds shall be carried out in accordance with the Specification.

16. Fabricate beams with rolling bow upwards.

17. All HD Bolts shall be commercial bolts or be made from bars with a minimum yield stress of 250 N/mm2.

18. Grout around foundation bolts and under column base plates is to have a minimum characteristic strength as defined in BS5328 at 28 days of not less than 40 N/mm2. Non-shrink grout such as Conbextra GP by Fosroc Ltd. or similar shall be used.

19. The Contractor is responsible for ensuring that all holes and fixings at interfaces into cladding and joinery are provided in the steelwork. Refer to Sub-Contractors drawings

FIRE AND CORROSION PROTECTION TO STEELWORK

1. Fire Protection to be confirmed.

2. Corrosion Protection - to be confirmed

HEALTH AND SAFETY

1. Refer to the Schematic Design Report for an appreciation of the design principles and design criteria for the structure, prior to proceeding with the works.

2. Refer to Designers Risk Assessment Sheets for any particular procedures to be addressed by the Contractor in Method Statements.

3. Refer to H&S Pre-Construction Information.

4. Stability during Erection

Stability during construction is the Contractor's responsibility and is to be assured by his attention to the design of the permanent frame.

7. Construction Loading

Temporary loading requirements of the Contractor are to be within the permanaent design allowances described in the Schematic Design Report, unless otherwise agreed.

8. Effects of Movements & Tolerances.

Refer to Schematic Design Report

ABBREVIATIONS

GENERAL

BOS	Bottom of Steel
BS	British Standard
CC	Centre to Centre
CL	Centreline
CONC	Concrete
CJ	Construction Joint
CONT	Continuous
DIA	Diameter
DRG	Drawing
EF	Each Face
EGL	Existing Ground Level
EJ	Expansion Joint
FS	Far Side
FFL	Finished Floor Level
FGL	Finished Ground Level
GL	Grid Line
HORIZ	Horizontal
Lwt	Lightweight
MAX	Maximum
MC	Main Contractor
MEP	Mechanical, Electrical & Public Health
MIN	Minimum
NS	Near Side
Nwt	Normal Weight
NWC	Normal Weight Concrete
NTS	Not To Scale
No.	Number
OA	Overall
PM	Project Manager
RAD	Radius
SIM	Similar
SOP	Setting Out Point
SSL	Structural Slab Level
SYM	Symmetrical
TBC	To Be Confirmed
TOC	Top of Concrete
TOS	Top of Steel
TYP	Typical
UOB	Underside of Base
UNO	Unless Noted Otherwise
VERT	Vertical

GROUND SLAB CONSTRUCTION

CJ -	Construction Joint
IJ -	Isolation Joint
MJ -	Movement Joint
SCJ -	Saw Cut Joint

STEEL

BW	Butt Weld
FW	Fillet Weld
FPBW	Full Penetration Butt Weld
HD Bolt	Holding Down Bolt
XOX	Hexagonal Round Bolt
HSFG	High Strength Friction Grip
O/O	Out Of
PLT	Plate
SS	Stainless Steel
ASB	Asymmetric Beam
CHS	Circular Hollow Section
PFC	Parallel Flange Channel
RHS	Rectangular Hollow Section
RSA	Rolled Steel Angle
RSC	Rolled Steel Channel
RSJ	Rolled Steel Joist
SHS	Square Hollow Section
UKB	Universal Beam
UKC	Universal Column

SYMBOLS & NOTATION

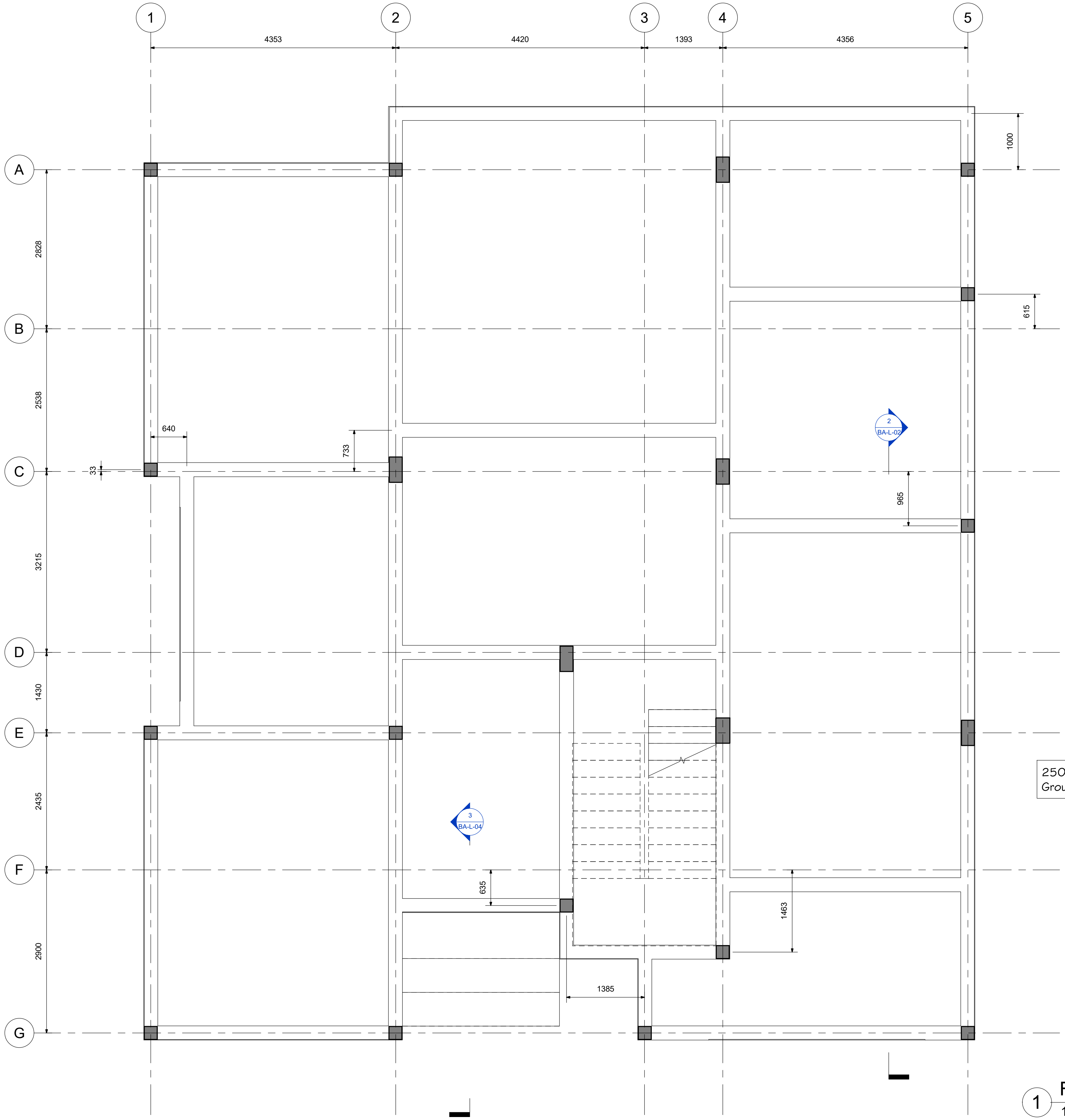
0.000 Level m

Project Title PROPOSED RESIDENTIAL DEVELOPMENT	Client POLICE AIG	Consultant	Consultant MIBBS CONSULT (Civil Structural & Project Mgt.) 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	Drawing Title GENERAL NOTES Drawing No. BA-L-01 Rev	Job No Date 10/07/22 Drawn By; Author Checked By Checker SCALE 1 : 20

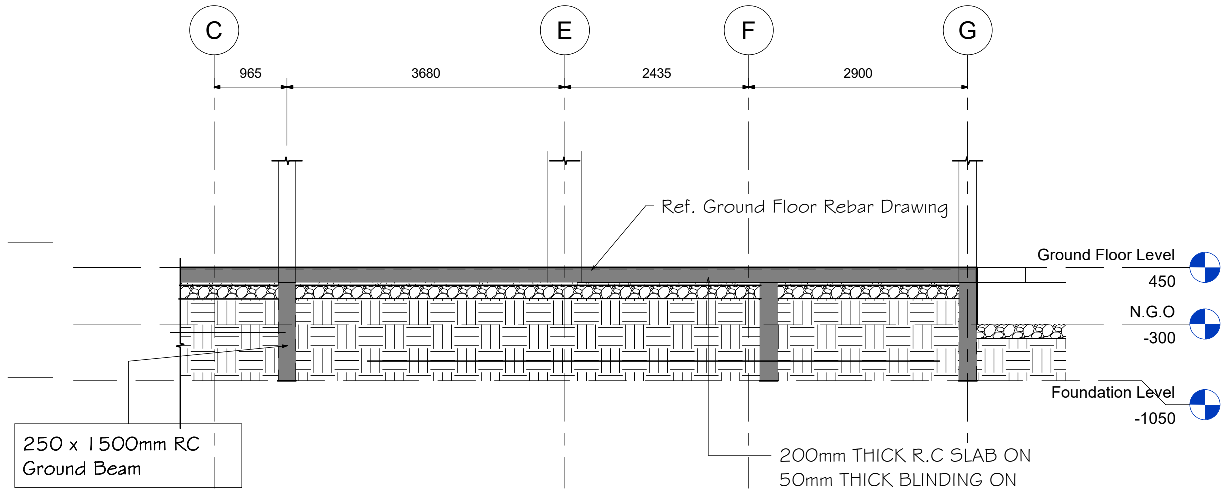
Seal / Stamp
Approver
Comments CONSTRUCTION

September 2022

18/06/2023 11:15:27 pm



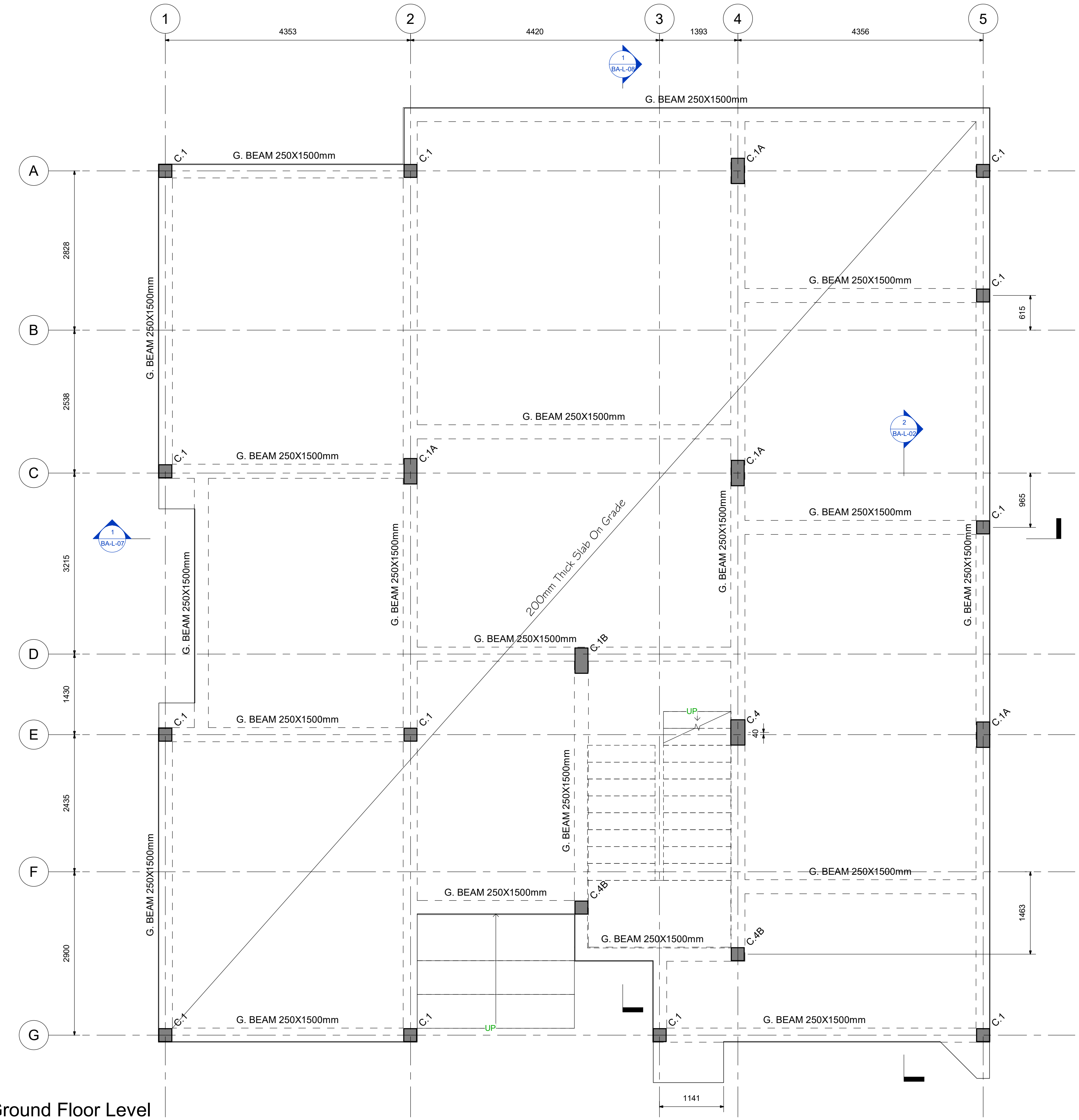
Foundation Level
1 : 40



Section 1
1 : 50

Project Title	Client	Consultant	Consultant	Drawing Title	Job No		Seal / Stamp
PROPOSED RESIDENTIAL DEVELOPMENT	POLICE AIG		MIBBS CONSULT (Civil Structural & Project Mgt.) 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	FOUNDATION LAYOUT	Date	09/30/22	Approver
				Drawing No. BA-L-02	Drawn By;	DARE	
				Rev .	Checked By	MUSA	
					SCALE	As indicated	Comments CONSTRUCTION

September 2022



Ground Floor Level

1 : 40

GROUND FLOOR COLUMN SCHEDULE			
TYPE	SIZES	No.THUS	CONC. GRADE
C.1	230X230mm	12	C25
C.1A	450X230mm	4	C25
C.1B	450X230mm	1	C25
C.4	250X450mm	1	C25
C.4B	230X230mm	2	C25

Grand total: 20

Project Title: PROPOSED RESIDENTIAL DEVELOPMENT

Client: POLICE AIG

Consultant

Consultant
MIBBS CONSULT (Civil Structural & Project Mgt.)
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Lagos
Email; musasegun45@gmail.com
Tel; +2348034223279

Drawing Title: GROUND FLOOR LAYOUT

Drawing No. BA-L-03

Rev .

Job No	
Date	09/30/22
Drawn By;	DARE
Checked By	MUSA
SCALE	1 : 40

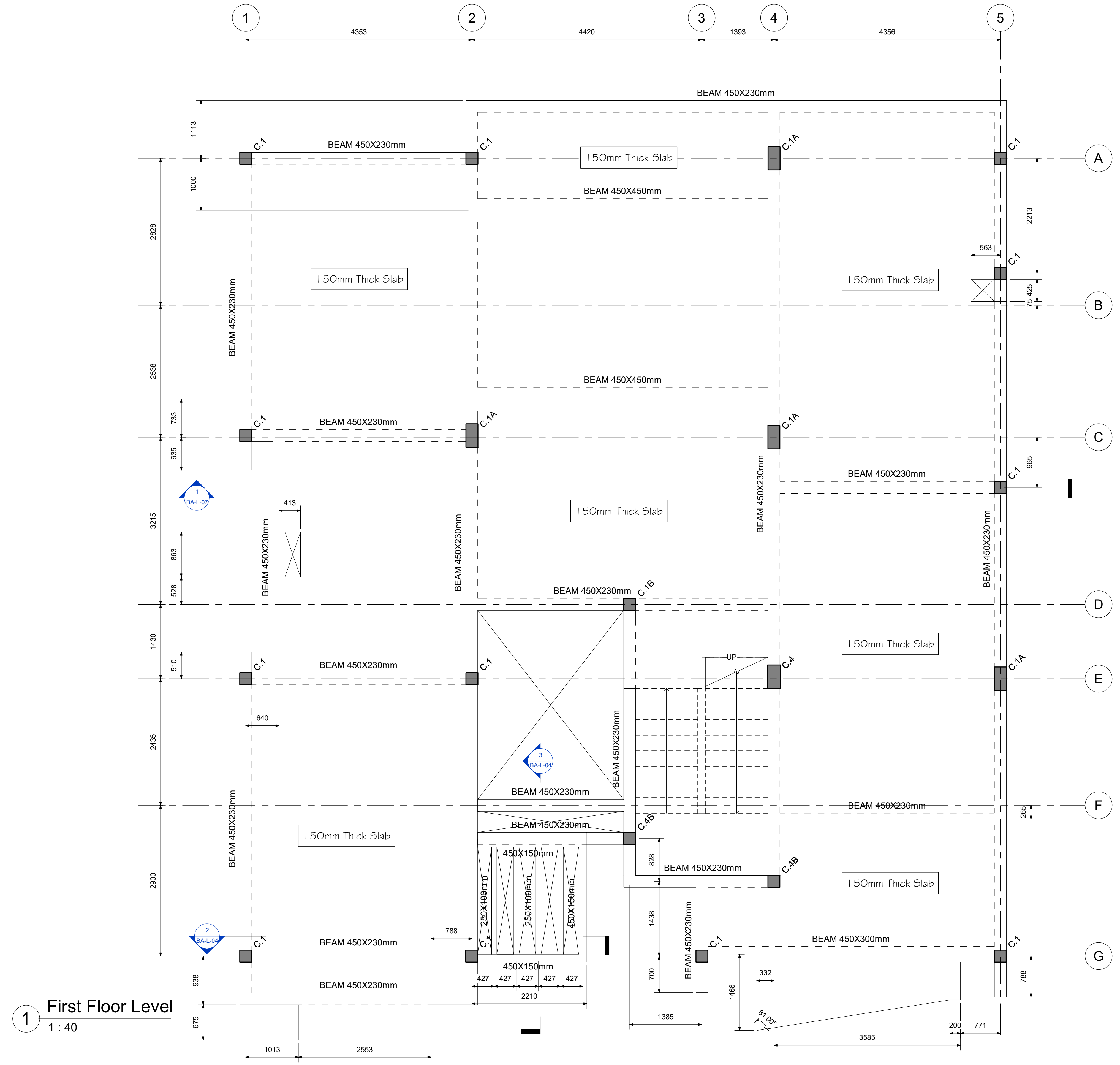
Seal / Stamp

Approver

Comments: CONSTRUCTION

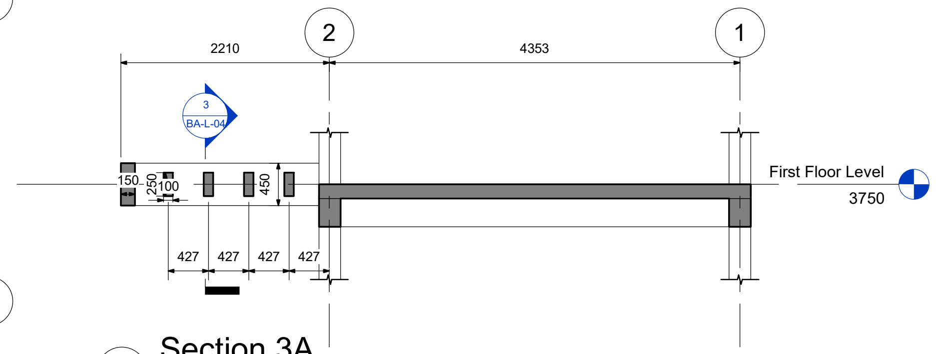
September 2022

19/06/2023 11:15:28 pm

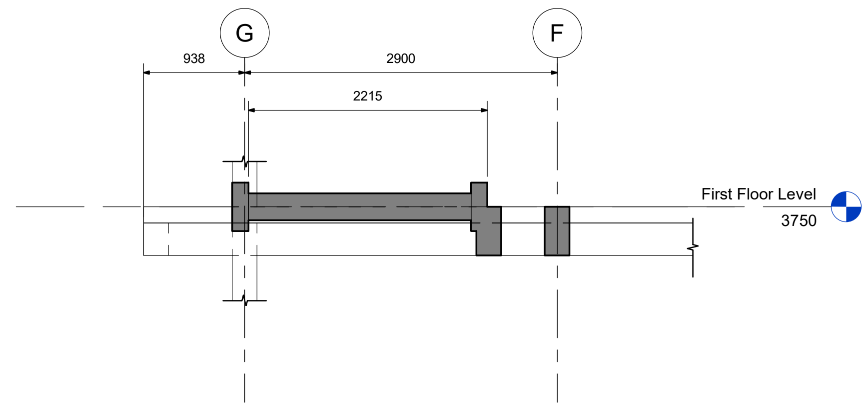


First Floor Level
1 : 40

FIRST FLOOR COLUMN SCHEDULE			
TYPE	SIZES	No.THUS	CONC. GRADE
C.1	230X230mm	12	C25
C.1A	450X230mm	4	C25
C.1B	230X230mm	1	C25
C.4	250X450mm	1	C25
C.4B	230X230mm	2	C25
Grand total: 20			



Section 3A
1 : 40



Section 3B
1 : 35

Project Title
PROPOSED RESIDENTIAL DEVELOPMENT

Client
POLICE AIG

Consultant

Consultant
MIBBS CONSULT (Civil Structural & Project Mgt.)
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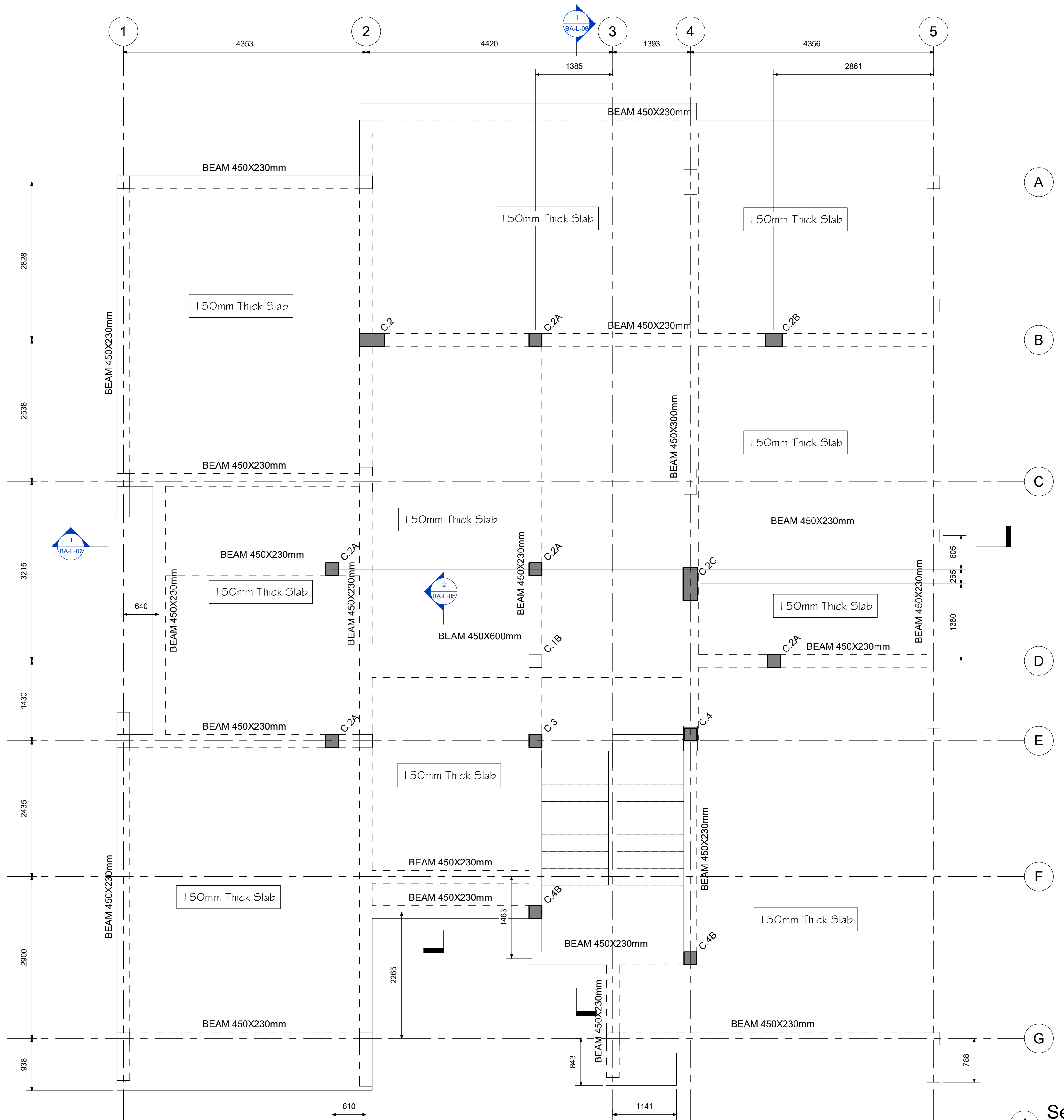
Email; musasegun45@gmail.com
Tel; +2348034223279

Drawing Title
FIRST FLOOR LAYOUT
Drawing No. BA-L-04
Rev

Job No
Date 09/30/22
Drawn By; DARE
Checked By MUSA
SCALE As indicated

Seal / Stamp
Approver
Comments CONSTRUCTION

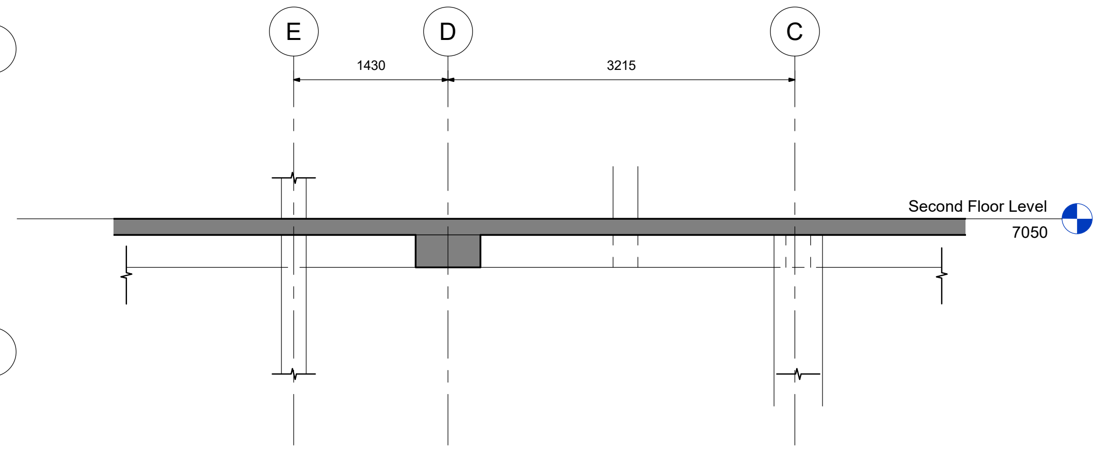
September 2022



1 Second Floor Level
1 : 40

SECOND FLOOR COLUMN SCHEDULE			
TYPE	SIZES	No.THUS	CONC. GRADE
C.2	450X230mm	1	C25
C.2A	230X230mm	5	C25
C.2B	300X230mm	1	C25
C.2C	250X600mm	1	C25
C.3	230X230mm	1	C25
C.4	230X230mm	1	C25
C.4B	230X230mm	2	C25

Grand total: 12



2 Section 5
1 : 35

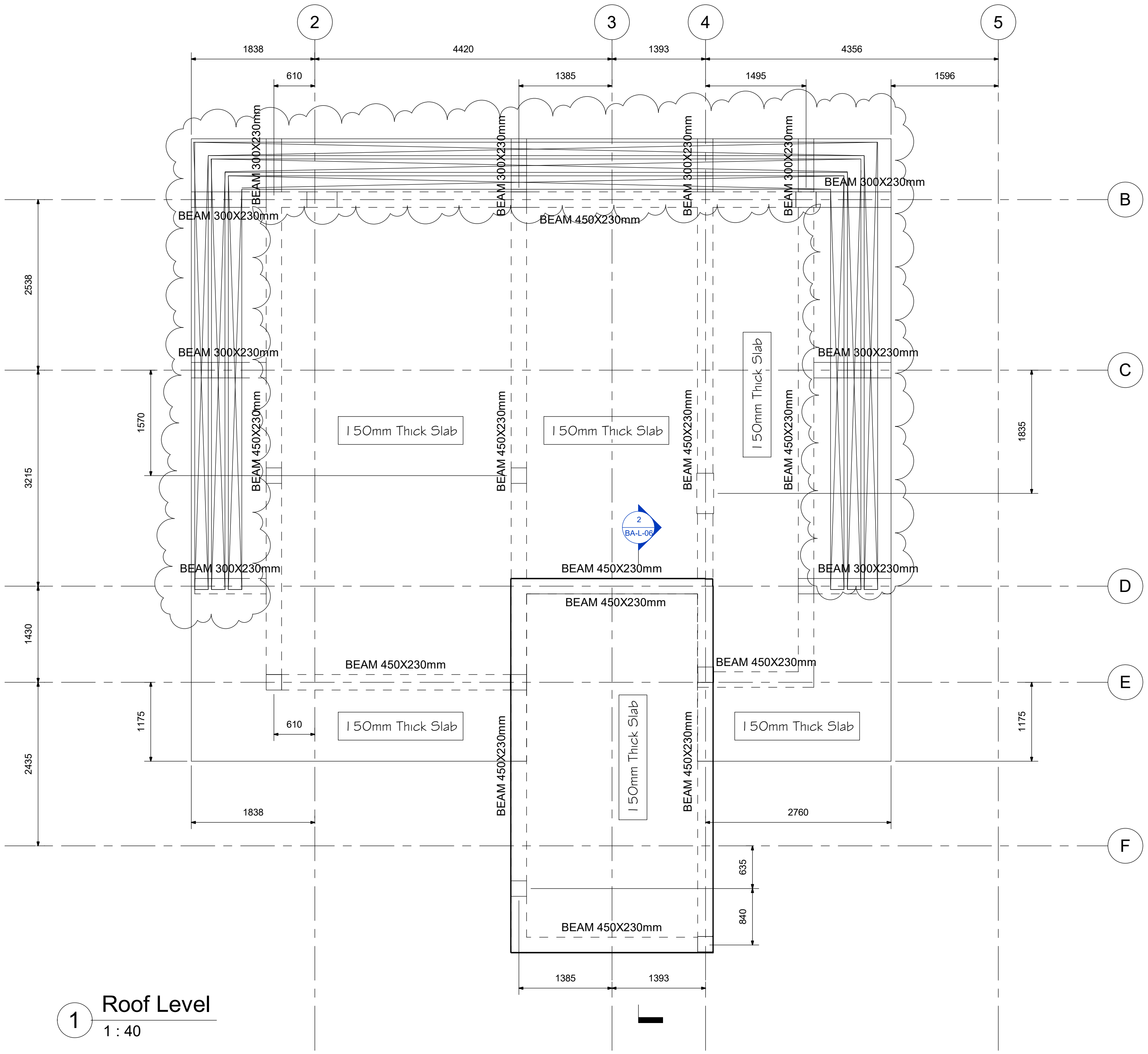
Project Title PROPOSED RESIDENTIAL DEVELOPMENT	Client POLICE AIG	Consultant	Consultant MIBBS CONSULT (Civil Structural & Project Mgt.) 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	Drawing Title SECOND FLOOR LAYOUT Drawing No. BA-L-05 Rev .	Job No Date 09/30/22 Drawn By; DARE Checked By MUSA SCALE As indicated

Seal / Stamp

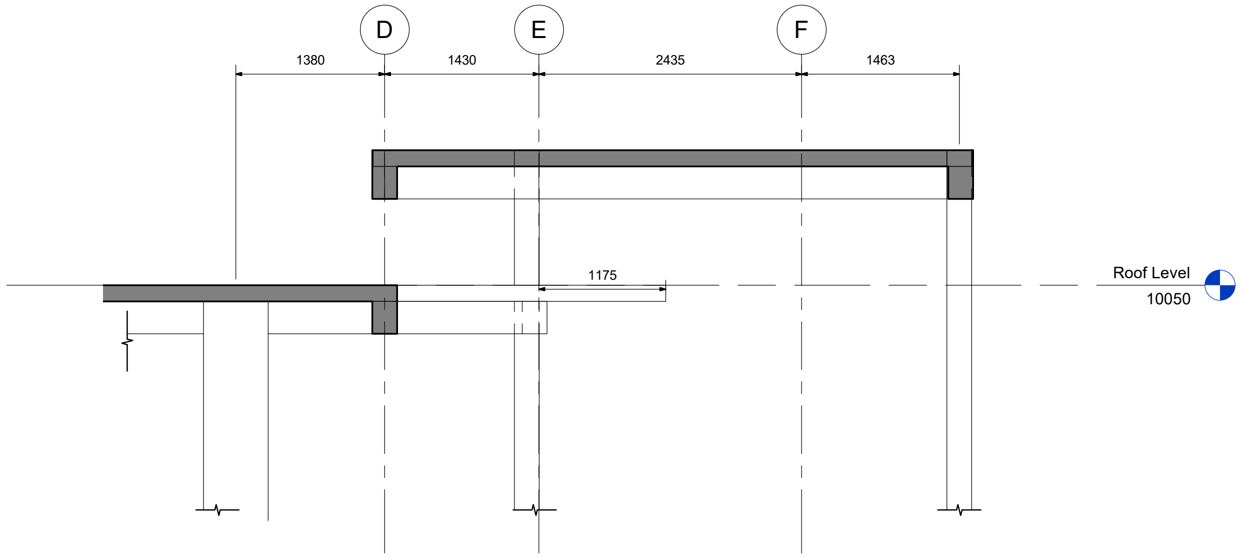
Approver

Comments CONSTRUCTION

September 2022



1 Roof Level
1 : 40



2 Section 4
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Project Title PROPOSED RESIDENTIAL DEVELOPMENT	Client POLICE AIG	Consultant	Consultant MIBBS CONSULT (Civil Structural & Project Mgt.) 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	Drawing Title ROOF LAYOUT Drawing No. BA-L-06 Rev 1	Job No Date 09/30/22 Drawn By; DARE Checked By MUSA SCALE As indicated

Seal / Stamp
Approver
Comments CONSTRUCTION

September 2022

A1 A B C D E F G H I J K L M N O P

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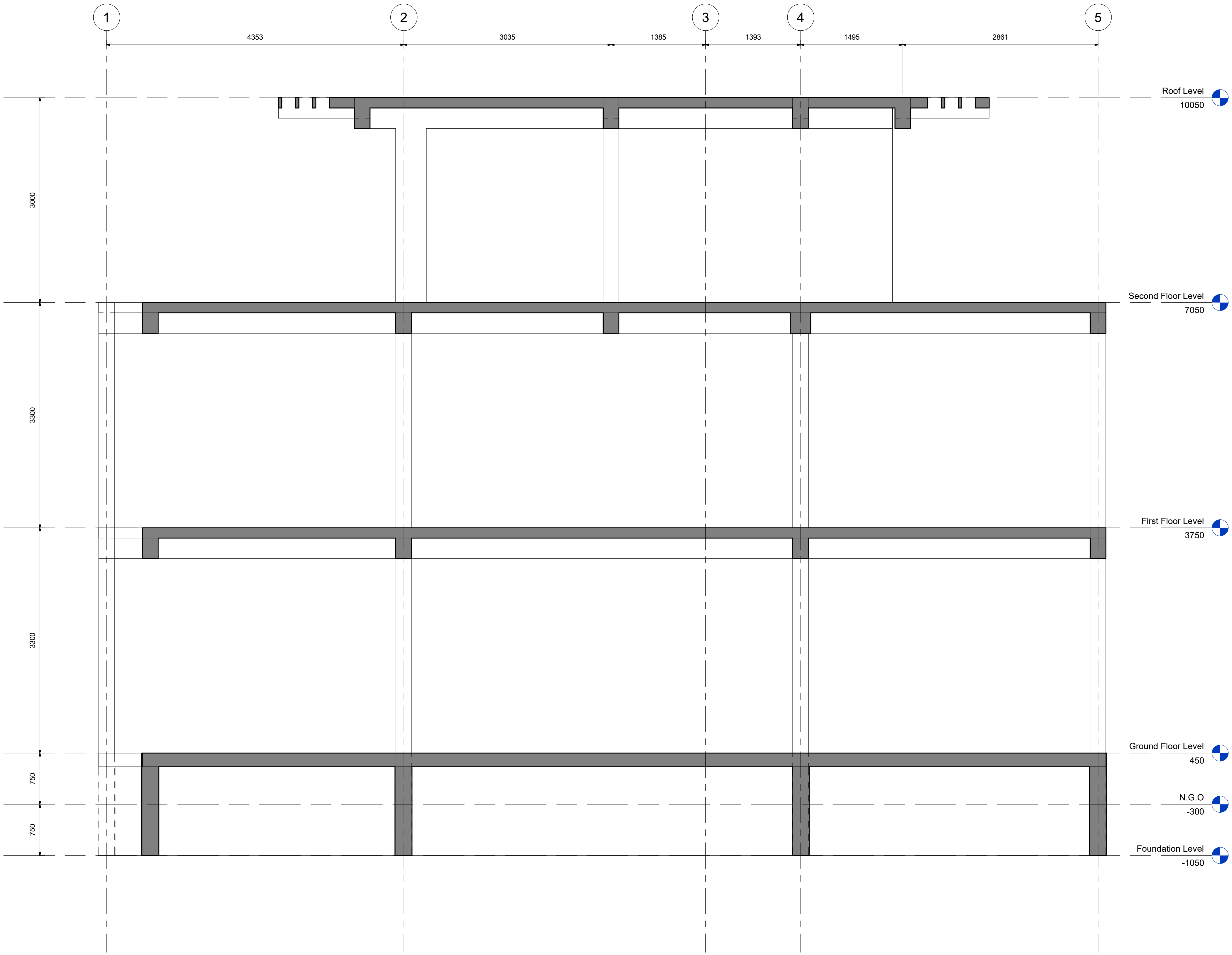

1 Section X-X
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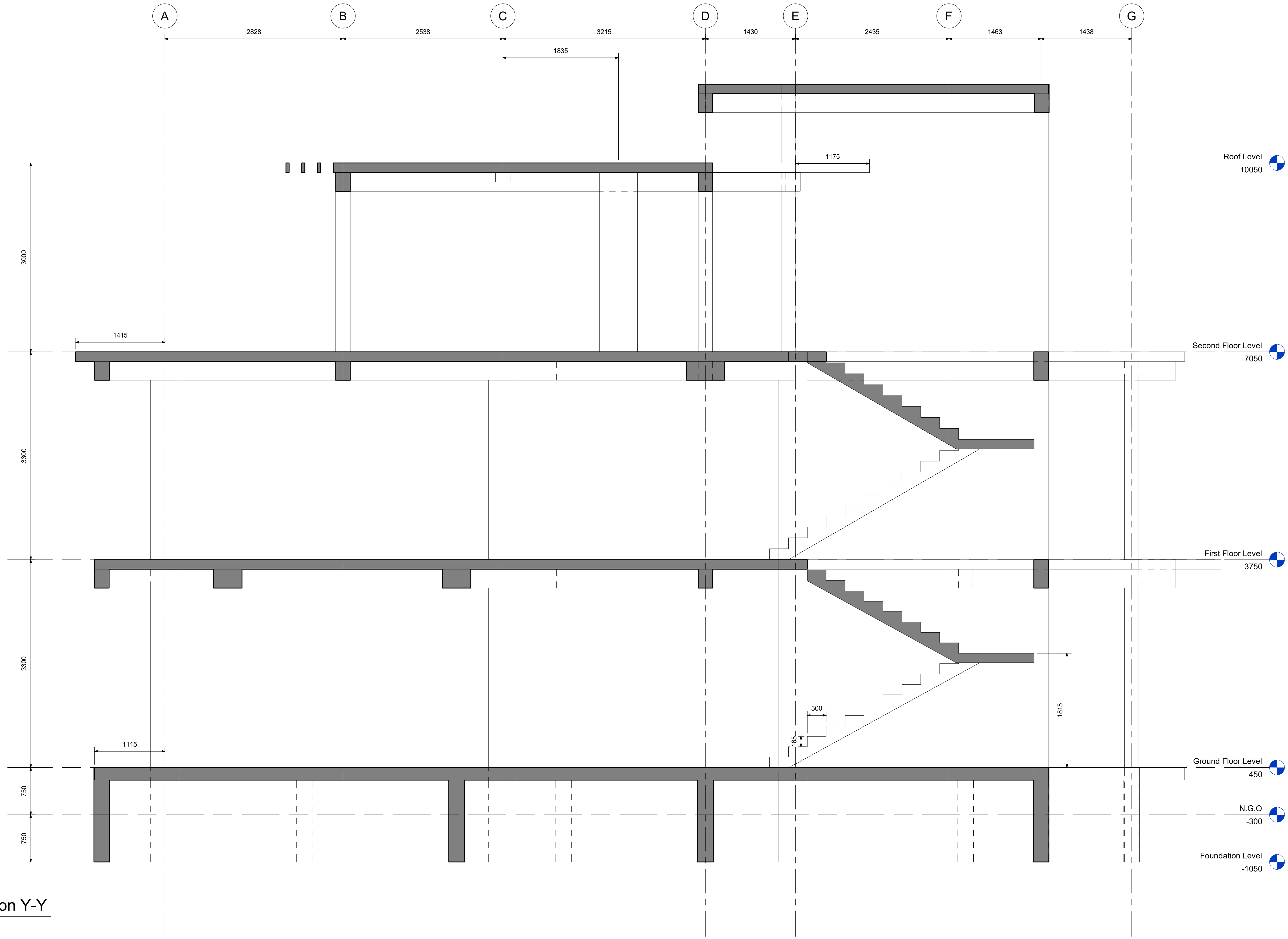
Project Title	Client	Consultant	Consultant	Drawing Title	Job No		Seal / Stamp
PROPOSED RESIDENTIAL DEVELOPMENT	POLICE AIG		MIBBS CONSULT (Civil Structural & Project Mgt.) 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	SECTION 1	Date	09/30/22	Approver
				Drawing No. BA-L-07	Drawn By;	DARE	
				Rev .	Checked By	MUSA	
					SCALE	1 : 30	Comments CONSTRUCTION



Section Y-Y
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Project Title PROPOSED RESIDENTIAL DEVELOPMENT	Client POLICE AIG	Consultant	Consultant **MIBBS CONSULT (Civil Structural & Project Mgt.)** 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	Drawing Title **SECTION 2** Drawing No. BA-L-08 Rev .	Job No Date 09/30/22 Drawn By; DARE Checked By MUSA SCALE 1 : 30	Seal / Stamp Approver Comments CONSTRUCTION

September 2022

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A1

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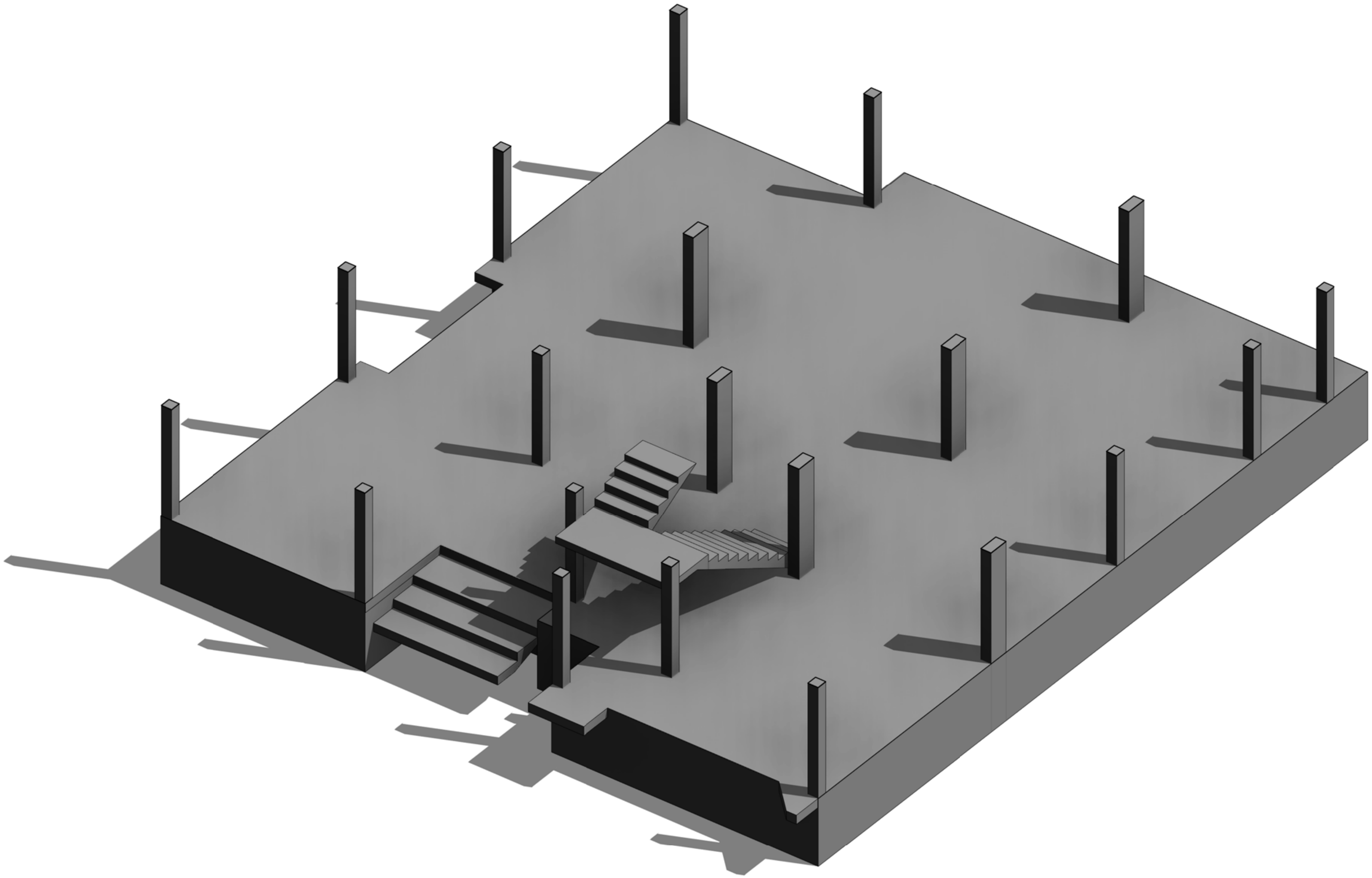

1 GROUND FLOOR 3D VIEW

Project Title	Client	Consultant	Consultant	Drawing Title			Seal / Stamp
PROPOSED RESIDENTIAL DEVELOPMENT	POLICE AIG		**MIBBS CONSULT (Civil Structural & Project Mgt.)** 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	**3D VIEW 1**	Job No		Approver
					Date	09/30/22	
				Drawing No. BA-L-09	Drawn By;	DARE	
				Rev .	Checked By	MUSA	
					SCALE		Comments: CONSTRUCTION

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19/06/2023
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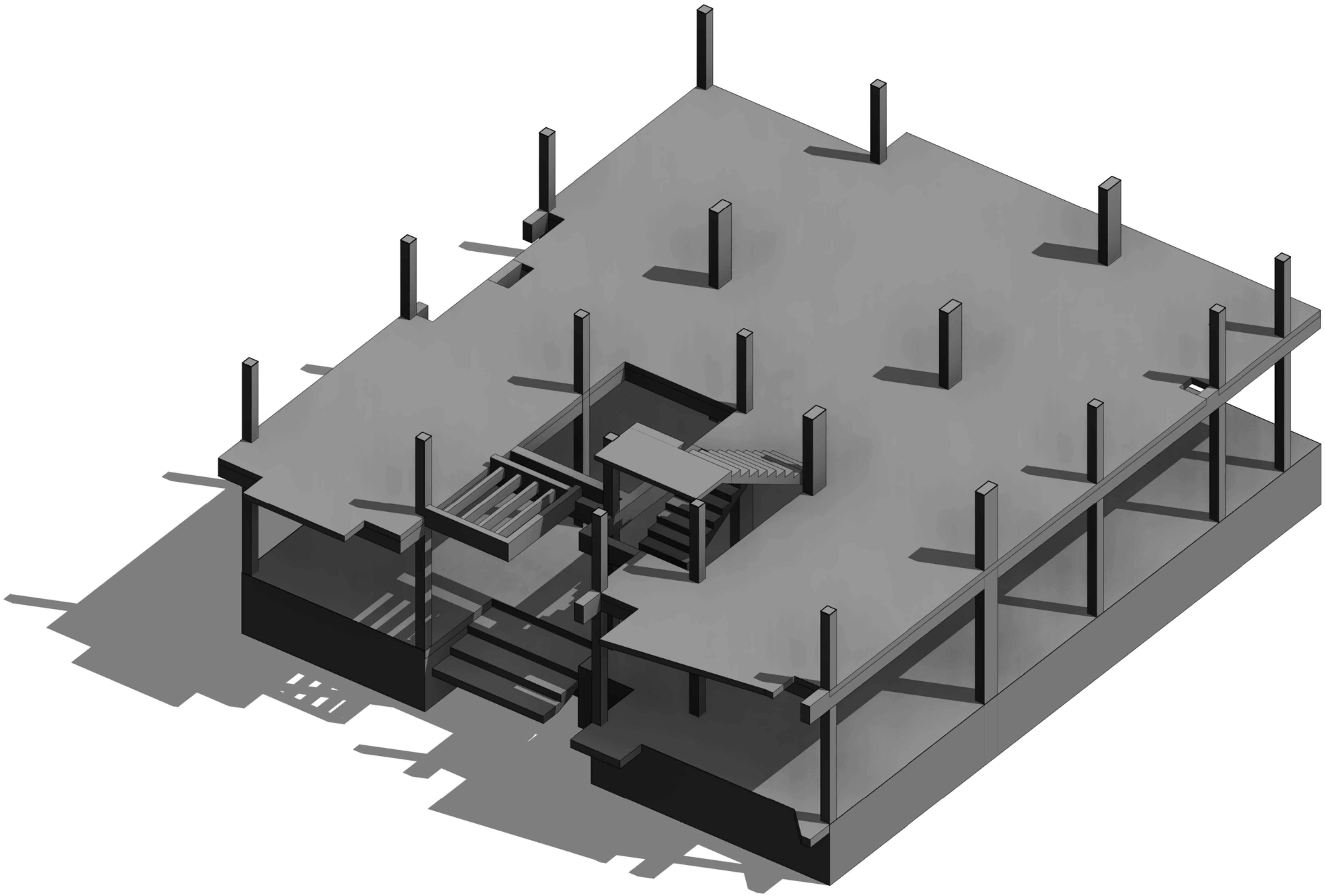

1 FIRST FLOOR 3D VIEW

Project Title PROPOSED RESIDENTIAL DEVELOPMENT	Client POLICE AIG	Consultant	Consultant **MIBBS CONSULT (Civil Structural & Project Mgt.)** 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	Drawing Title **3D VIEW 2** Drawing No. BA-L-10 Rev .	Job No Date 09/30/22 Drawn By; Author Checked By Checker SCALE	**Seal / Stamp** © Approver **Comments** CONSTRUCTION

September 2022

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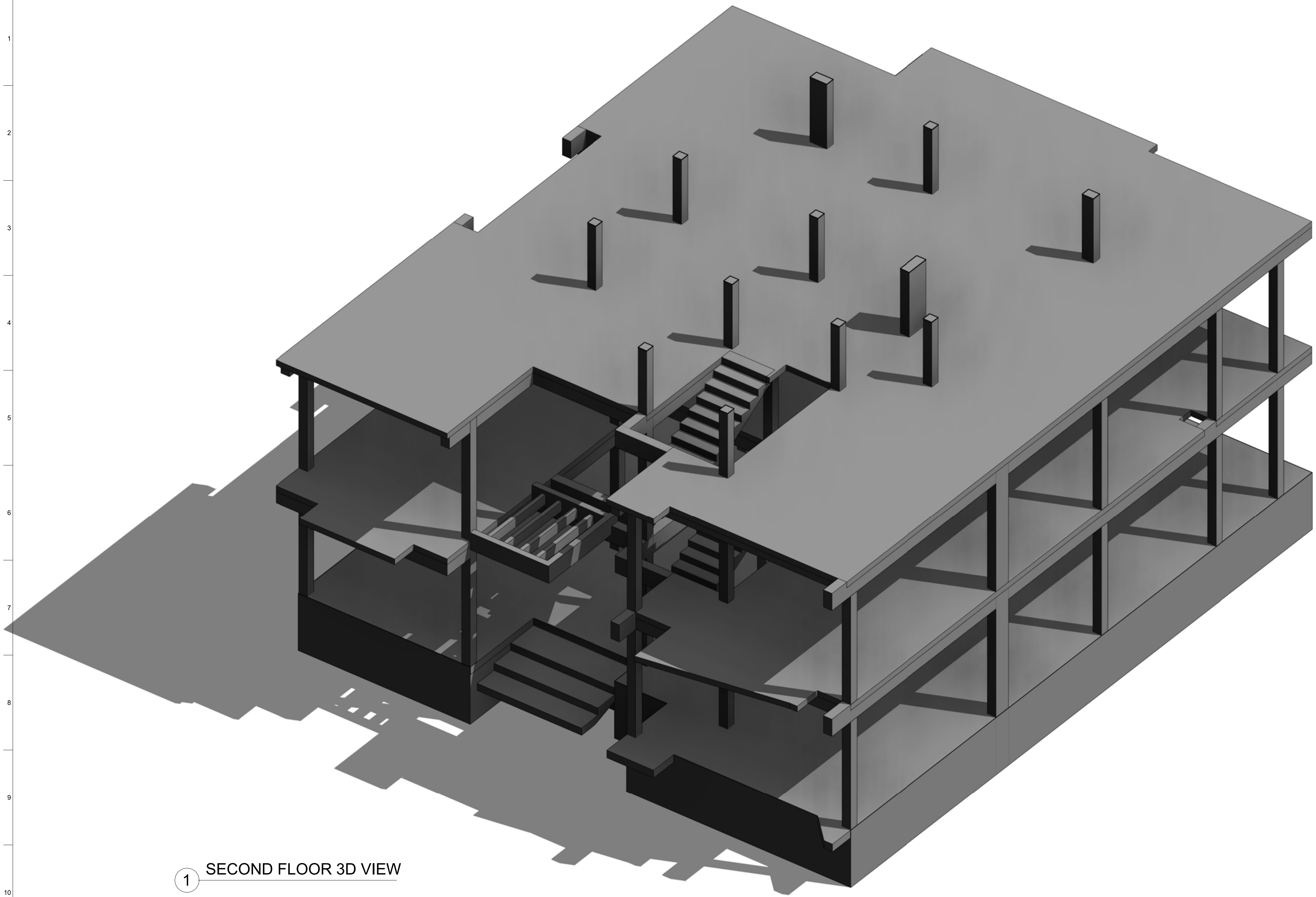

1 SECOND FLOOR 3D VIEW

Project Title	Client	Consultant	Consultant
PROPOSED RESIDENTIAL DEVELOPMENT	POLICE AIG		**MIBBS CONSULT (Civil Structural & Project Mgt.)** 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279

Drawing Title	Job No		Seal / Stamp
3D VIEW 3	Date	09/30/22	Approver
	Drawn By;	Author	
Drawing No. BA-L-11	Checked By	Checker	
Rev .	SCALE		**Comments** CONSTRUCTION

September 2022

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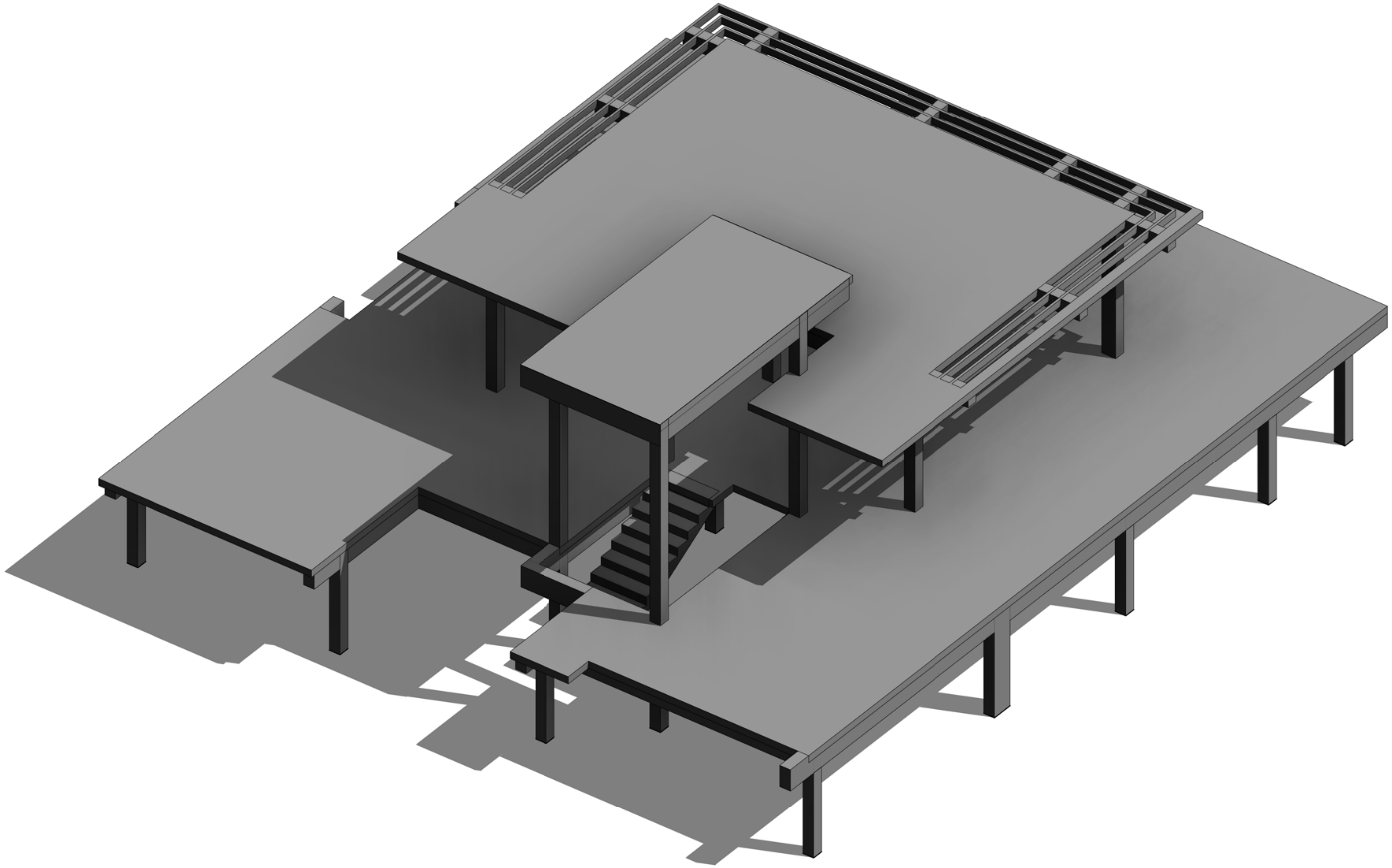

1 ROOF LEVEL 3D VIEW

Project Title	Client	Consultant	Consultant	Drawing Title	Job No		Seal / Stamp
PROPOSED RESIDENTIAL DEVELOPMENT	POLICE AIG		MIBBS CONSULT (Civil Structural & Project Mgt.) 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279	3D VIEW 4	Date	09/30/22	Approver ©
				Drawing No. BA-L-12	Drawn By;	Author	
				Rev .	Checked By	Checker	
					SCALE		Comments CONSTRUCTION

September 2022

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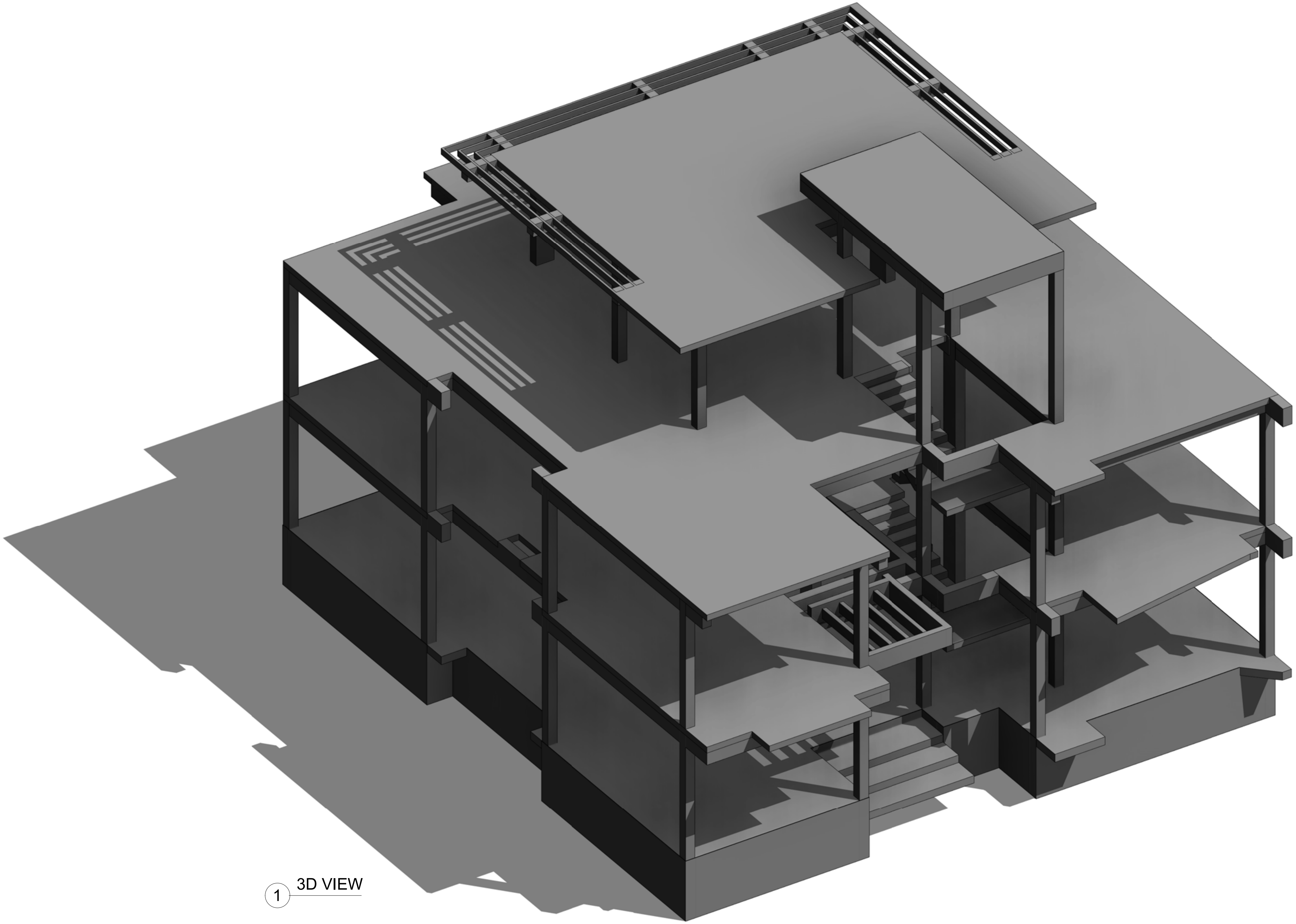

1 3D VIEW

Project Title PROPOSED RESIDENTIAL DEVELOPMENT	Client POLICE AIG	Consultant	Consultant **MIBBS CONSULT (Civil Structural & Project Mgt.)** 13, Chief Mike Close, Ajao Estate, Lagos Email; musasegun45@gmail.com Tel; +2348034223279

Drawing Title **3D VIEW 5**	Job No		Seal / Stamp	©
	Date	09/30/22	Approver	
	Drawn By;	DARE		
Drawing No. BA-L-13	Checked By	MUSA		
Rev .	SCALE		Comments	CONSTRUCTION

September 2022