STRUCTURAL ANALYIS AND DESIGN OF A BOX CULVERT ALONG ISUANIOCHA, MGBAKWU ROAD IN AWKA-NORTH LOCAL GOVERNMENT AREA, ANAMBRA STATE USING BS-8110

BY

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NAU/2016224005

IN PARTIAL FULFILMENT OF THE REQUIRMENT FOR THE AWARD OF BACHELOR DEGREE IN ENGINEERING (B.ENG) IN CIVIL ENGINEERING.

SUBMITTED TO THE

DEPARTMENT OF CIVIL ENGINEERING

FACULTY OF ENGINEERING

NNAMDI AZIKIWE UNIVERSITY, AWKA

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FEBRAURY 2022

CERTIFICATION

This is to certify that this project work is done by Akpa Nnamdi Oyo, Reg no: 2016224005 has been approved in partial fulfillment of the requirement for the award of bachelor degree in Engineering in civil engineering.

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APPROVAL

This design work has been assessed and approved by the department of Civil Engineering, Nnamdi Azikiwe University, Awka.

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DEDICATION

This project work is dedicated to God Almighty whom his infinite grace and mercy has led me to the successful completion of my project. I equally dedicate this work to my late Dad Oyo Akpa and to my entire family at large for their support and encouragement towards the success of this work.

ACKNOWLEDGEMENTS

The help of God Almighty throughout my life and stay in this university is highly acknowledged.

Many thanks to the entire staff of Civil Engineering Department especially my project supervisor Engr. Prof. C.H. Aginam for his instruction, patience and understanding in guiding me through this project. The effort of our Head of Department, Engr. Dr. C. A. Ezeagu in organizing the department is also highly acknowledged and also my lecturers, Engr. Prof Ekenta, Engr. Prof. Chidolue, Prof. C.M.O Nwaiwu. Prof. N.E. Nwaiwu, Engr. Nwajuaku .A.I, Engr. Dr. Adinna, Engr. Ezewamma, Rev. Dr Nwkaire, Engr. Dr. P. Onodagu, Engr. Dr. Odinka, Engr.B. Njotae, Engr.N. Nwokdiko, I really appreciate your efforts towards impacting me the requisite knowledge needed for my excelling in this department and beyond.

I will also in no small way appreciate my lovely parents, Mr. and Mrs. Akpa for their love, and continuous support throughout my stay in this graeat institution and to my uncle Engr. Nwakpa Oyoyo, you all make me feel blessed for having you all as a family.

All my friends and course mates are well appreciated for their love and support. God bless you all.

SYMBOLS AND ABBREVIATION

Q = Discharge

L = Culvert length

S = Culvert slope

Ke = Entrance loss coefficient

V = Velocity

D = Height box

Dc = Critical depth

N = Manning's roughness coefficient

B = Culvert width

A = Cross sectional area

M = Moment

M_A = Moment at point A

T_{as} = unit weight of asphalt

W = Wheel load

H = Depth of earth fill

∅ = Angle of repose

U.D.L = uniformly distributed load

ϵMA = Summation of moment about point A

E = Modulus of elasticity

I = Second moment of area

Q = Shear force

N = Axial forces

f_{CU} = Characteristics strength of concrete

fs = Service stress in reinforcement (deflection requirement)

As = Area of steel

AS_{prov} = Area of steel provided

As min = Minimum area of steel

M.F = Modification factor

V = Shear stress

Vc = Concrete shear stress

c/c = centre to centre

k_a = Coefficient of active pressure

p_a = Active earth pressure

X = Level arm

M_r = Resisting moment

Mnet = Net moment

q = Earth bearing pressure

N_f = Near face

FHWA = Federal highway authority

RCD = Reinforced concrete design

AASHTO = America Association of state highway and transportation officials

μ = Coefficient of friction

Fo = Factor of safety for overturning

e = Eccentricity

ABSTRACT

This project is the analysis design of Box culvert along Isuaniocha/ Mgbakwu road in Awka North Local Government Area. The sequence of execution commenced with a site investigation, collection of data and information for the hydraulic design, which gives the adequate dimension of the culvert. The hydraulic design was carried out in line with federal highway authority (FHWA) design guidelines. Then, the structural analysis and design was done based on the result of the hydraulic design. The culvert was analyzed and designed as a rigid frame under different load combination of dead load, live load, water pressure and literal earth pressure. The method of analysis used was force method for triple and single cell box culvert was designed for both ultimate limit state and serviceability limit state. The wing walls and headwalls were designed as cantilever retaining walls. Detailing was done according to BS8110. Having satisfied all necessary checks the proposed culvert is fit for construction and will serve its purpose under the worst condition.

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CHAPTER ONE

INTRODUCTION

A culvert is a small opening of less than six meres (6m), provided to allow flow of water pass through the embankment and follow the natural channel. Since culvert pass through the earthen embankment, subjected to same traffic loads as the road carries, the structural elements are required to be designed to withstand maximum bending moments and shear forces.

The structural analysis and design of triple and single cell box culvert is therefore the determination of the stresses and strain developed by the culverts when loaded and providing the appropriate reinforcement members to suit these stresses and strains.

1.1 Statement of Problem

Inaccessibility of Isuaniocha village for conveying goods due to the absence of access road thereby preventing the location of industries in the large expanse of land in the interior parts of Isuaniocha village and thus impeding the development of the area.

1.2 Aim & Objective

The aim of this project is to produce adequate design of a triple and single cell box culvert across a natural drainage channel at Isuaniocha road, Awka North to easy vehicular movement.

The main objective is to provide a design that is structural stable, based upon appropriate hydraulic principles, economy and minimized effects.

1.3 Scope of Study

The study is limited to the hydraulic and structural analysis, design and detailing a triple and single cell box culvert with each cell having an opening dimension of 2.8m X 2.8m and is to be located along Isuaniocha-Mgbakwu road, Awka North local government area based on the data obtained from the MINISTRY OF WORKS, inlet and outlet nomograph will be used for the hydraulic design of the culvert. The structure will be designed using BS8110 and the structural detailing will also be in accordance with BS8110.

The detailing of the hydraulic design is in accordance with Federal Highway Authority (FHWA)

1.4 Significance of the Design

The natural channel at Isuaniocha road has been an impediment to the flow of traffic and pedestrians between Mgbakwu Town, the design of a triple cell box will direct drainage of accumulated stream water runoff from roads.

Thus, this design will help recognize the need to allow natural movement of surface water where fill roads would otherwise adversely affect the natural flow and alter the hydrology of the area.

1.5 Classification Of Culverts

Culverts can be classified as:

i. pipe Culvert
ii. Arch Culvert
iii. Pipe Arch Culvert
iv. Box Culvert
v. Slab/bridge culvert
vi. Metal Box Culvert

1.6 Factors Affecting Choice Of Culverts

Some factor to be considered in choosing the type of culvert to be used are as follows:

i. Road profiles.
ii. Channel characteristics.
iii. Flood damage evaluations.
iv. Construction and maintenance cost.
v. Estimates of service life.

1.7 Material For Construction

The following materials can be used in construction:

i. Concrete (Reinforced and non-reinforced)

ii. Galvanized steel (Smooth and corrugated)

iii. Aluminium (Smooth and corrugated)

vi. Plastic (Smooth and corrugated)

V. High density polyethylene.

1.8 Factors Affecting Choice Of Materials For Construction

The selection of the construction materials for a culvert depends on several factors that can vary considerably with location. They include:

i. Structure strength, considering filling height, loading condition a foundation condition
ii. Hydraulic efficiency, considering manning roughness, cross section area and shape.
iii. Installation, local construction practices ,availability of pipe embedment material and point tightness requirement
iv. Durability, considering water and soil environment (PH and resistivity), corrosion (Metallic coating selection) and abrasion.
v. Cost, considering availability of materials

1.9 Functions of Culvert.

i. Culverts are provided to allow water flow freely across the embankment without causing failure of the road.
ii. They are provided to balance the water level on both sides of embankment during floods.
iii. It provides a platform for road construction to ensure its continuity over the water course without obstructing the flow of the water course.

1.10 Application Of Culverts

Culverts are applied in surface water drainages, stream diversion, conveyor tunnels, storage tanks, pedestrian subways, vertical shafts and walls, vehicle access and cattle creeps, installation of thrust bore techniques and so on.

1.11 Culvert Flow

The flow is usually non-uniform with regions of both gradually varying flow. Outlet velocity can range from 3mls for culverts on mild slope to 9mls for those on steep slopes

1.11.1 Types Of Control Flow

a) Inlet Control:

A culvert flowing in inlet control has shallow, high velocity flow categorized as "supercritical". For supercritical flow the control section is at the upstream and the barrel (inlet).

b) Outlet Control:

A culvert flowing in outlet control will have relatively deep, lower velocity flow termed "subcritical" flow. For subcritical flow the control is at the down stream end of the culvert (the culvert).

1.12 Concrete Box Culvert

One of the most common types of culverts used today is the concrete box culvert. It is made up of cement, fine aggregate, coarse aggregate and water with reinforcement. A box culvert has a top slab, vertical walls and a bottom slab which may or may not be present. When present, it provides a lined channel for the water and a base for the walls. It could also be left out in order to allow the stream crossing to maintain its natural streambed. In such a case the side walls are supported on concrete footings.

The dimensions of the box culvert are determined by the hydraulic requirements, forces it would be subjected to, bearing capacity of in-situ soil and so on. Box culverts are used in a variety of circumstances for both small and large channel openings and are easily adaptable to a wide range of site condition. In cases where the required size of the opening is very large, a multi-cell box culvert can be used. It is important to note that although a box culvert may have multiple barrels (cells), it is still a single structure. The internal walls are provided to reduce the unsupported length of the top slab.

1.13 Types of Construction

There are two main types of concrete box culverts. They are

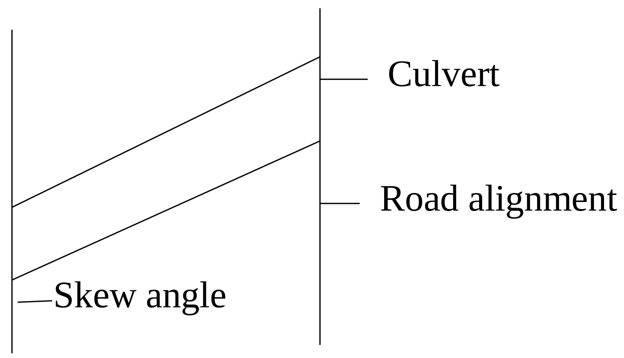
i. **Cost-in-place box culverts:** They are constructed at the site using form work as mould and pouring concrete into it (casting) so that the liquid concrete takes the shape of the mould when it solidifies. Reinforced cast-in-place (CIP) concrete culverts are typically rectangular (box) shaped. The major advantage of cast in place box culverts is that the culvert can be designed to meet the specific geometric requirement of the site.
ii. **Pre-cast box culverts:** They are fabricated in a controlled environment at the factory and then conveyed to the site where they are to be installed. They are designed for various depths to cover and various live loads and are manufactured in a wide range of sizes. Standard box sections are available with spans as large as 3.7 meters. Some advantages of precast box culverts are;

i. High quality
ii. Reduced water control cost

iii. Quick and easy on-site installation
iv. High durability and so on.

1.14 Skew Culvert

Sometimes the road alignment may cross a stream at an angle other than a right angle. In such situation a skew culvert may be provided. Thus



1.15 Cushion

A box culvert can be placed such that the lop slab is almost at road level. Here the culvert is said to be placed within the embankment such that the top slab is few meters below the surface. Such culverts are said to be with cushion. Thus, cushion refers to the material which fills the gap between the top slab and the road surface.

1.16 Advantages Of Box Culverts

i. The box culvert is structurally strong, stable and safe.
ii. It is easy to construct
iii. It can be placed at any elevation within the embankment with varying cushion which is not possible with other types.
iv. It does not require separate elaborate foundation and can be placed on soft soil by providing suitable base slab projection to reduce base pressure within the safe bearing capacity of the foundation soil.
v. Bearings are not needed
vi. It can be conveniently extended in future without any problem of design and/or construction.

CHAPTER TWO

2.1 Literature Review

The design of a culvert is divided into two parts. Namely; the hydraulic design and the structural design. The hydraulic design has to do with the estimation of the culvert size and the choice of culvert shape after due consideration of several factors such as design discharge, allowable headwater depth, slope of stream bed, allowable outlet velocity, and the culvert and son on.

According to Adekola (1990), structural design is concerned with estimating as accurately as possible the loads the structure is likely to be subjected to during service and the use of an appropriate method of analysis to evaluate the reactions and internal stresses generated in the structure after which the dimensions of the component parts of the culvert with appropriate strength grade would be chosen to safely resist such stresses under the worst condition.

2.2 Hydraulic Design

Hydraulic design precedes structural design. According to Ross (1988), “Before any attempt is made to design a culvert for a given site, certain field investigations are necessary. The required information is usually taken along with the survey work. Data relevant to center line location, skew of structure, and suggested possible channel changes are recorded along with the field survey. This data should be sufficient to provide cross section and profile, estimate a roughness factor, run off factor and so on. Hydraulic design determines whether the culvert flow is governed by inlet control or outlet control. According to a research sponsored by the Federal Highway Administration (FHWA), Norman, et al (1985), explained that culvert operation is governed at all times by one of two conditions: inlet control or outlet control. Inlet control is the common governing situation for culvert design characterized by the fact that the tail water or barrel conditions allow more flow to be passed through the culvert than the inlet can accept. The inlet itself acts as a controlling or governing section of the culvert, restricting the passage of water into the main barrel”

The ultimate objective of determining the hydraulic requirements for any highway drainage structure is to provide a suitable structure size that will economically and efficiently dispose of the expected runoff.

Certain hydraulic requirement should also be met to avoid erosion and sedimentation in the system.

The main factors considered in culvert deign are the location of the culvert, economy and the type of flow control.

It is the best to locate the culvert in the existing channel bed such that the center line and slope of the culvert coincides with that of the channel.

The design flow rate is based on the storm with an acceptable return period (frequency), culverts are designed for the peak flow rate.

The control section of the culvert is used to classify different culvert flows. The control section is the location at which a unique relationship exists between the flow rate and the depth flow.

There are two procedures for the hydraulic design of culverts. They are

1. Manual use of inlet and outlet control nomographs and
2. The use of computer programs.

The use of culvert design nomographs requires a trial and error solution, the design procedure uses several design charts and nomographs developed from a combination of theory and numerous hydraulic test results.

In the design of a culvert, the headwater elevations are computed for inlet and outlet controls and the controls with the higher headwater elevation is selected as the controlling condition.

As regards estimation of design flow and flood flow, Bhattachar and Michael (2003) stated that, "No method is available by which the extact amount and intensity of the rain fall in any assigned future period can be predicted precisely. Various methods have been used for estimating floods. Some of them are based on the characteristics of the drainage and others are based on the theory of probabilities applied to the previous known flood data; lastly others are based on a study of the rain fall and run off. Various methods, which are generally used in for determining flood flows, can be classified into the following.

1. Determination by means of empirical formulae
2. Determination by envelop curves.
3. Determination by statistical or probability methods; and
4. Determination by unit hydrograph method.

According to IOWA storm water management manual, (December, 2008), the design procedure that requires the use of inlet and outlet control nomographs is as follows.

Step 1: list of design data

- Q = Discharge (M^3/S)
- L = Culvert length (M)
- S = Culvert slope (M)
- Ke = Inlet loss coefficient
- V = Velocity (M/S)
- TW = Tail water depth (M)
- HW = Allowable headwater depth for the design storm (M).

Step 2: Determine trail culvert size by assuming a trial velocity 0.9-1.5m/s and computing the culvert area, A=Q/V. Determine the culvert diameter (m).

Step 3: Find the actual HW for the trail size culvert for inlet and outlet control.

a) For inlet control, enter-control nomograph with D and Q and find HW/D for the proper entrance type. Compute HW, and if too large or too small, try another culvert size before computing HW for outlet control.
b) For outlet control, enter the outlet-control nomograph with the culvert entrance loss coefficient, and trial culvert diameter.
c) To compute HW, connect the length of the scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. compute the headwater elevation HW from the following equation: HW = H +ho –LS Where ho = ½ (critical depth + D), or tail water depth, whichever is greater.

Step 4: Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control. If outlet control governs and the HW is unacceptable, select a larger trail size and find another HW with the outlet control nomographs.

Step 5: Calculate exit velocity and expected streambed scour to determine if an energy dissipater is needed.

Creamer (2007) stated that “the FHWA has standardized the manner by which culverts are examined and designed. The design approach involves first computing the headwater elevation up stream of the culvert assuming that inlet control governs. The two headwater values are compared and the higher of the two is selected as the basis of the culvert design. The procedure described above is repeated for different types of culvert shapes, sizes and entrance condition. The least expansive culvert that produces an acceptable headwater elevation is typically chosen for the final design”.

2.3 Structural Analysis and Design

2.3.1 Structure of The Box Culvert

A box culvert is made up of several parts. According to Punmia, Jain and Jain (1992), “A box culvert is a continuous rigid frame of rectangular section in which the abutment and the top and bottom slabs are cast monolithic”. Sinha and Sharma (2009) also stated that, “the box is one which has its top and bottom slabs monolithically connected up of the three major elements. Namely: top slab, walls and bottom slab. These element have various loads acting on them.

Structural analysis involves the identification of the loads acting on the culvert as well as their magnitude and direction in order to ascertain their effects on the structure. These effects refer to the bending moments, shear and axial stresses, deflection and so on.

Design is the process of selecting the appropriate dimension of the structural elements with respect to the strength characteristics of the construction material (concrete in this case) that will adequately resist the stresses generated by the loads:

2.3.2 Loading

A box culvert is subjected to various loads. These loads could be vertical or lateral loads which can be classified as dead loads or live loads and can be estimated in the different ways under different circumstances.

According to Reynolds and Steedman (1988), "The loads on a box culvert can be conveniently divided as follows:

1) A uniformly distributed load on the top slab and an equal reaction from the ground below the bottom slab.
2) Concentrated imposed load on the top slab and an equal reaction from the ground below the bottom slab.
3) An upward pressure on the bottom slab due to the weight of the walls.
4) A uniformly distributed horizontal pressure on each wall due to the increase in earth pressure in the height of the culvert.
5) A uniformly distributed horizontal pressure on each wall due to pressure from the earth and any surcharge above the level of the roof of the culvert.
6) The internal horizontal and possibly vertical pressures from water in the culvert.

Where a trench has been excavated in firm ground for the construction of a culvert and the depth from the surface of the ground to the roof of the culvert exceeds, say, three times the width of the culvert, it may be assumed that the maximum earth pressure on the culvert is that due to a depth of the earth equal to three times the width of the culvert. Although a culvert passing under a newly filled embankment may be subjected to more than the full weight of the earth above, there is little reliable information concerning the actual load carried and therefore any reduction in the load due to arching of the ground should be made with discretion. If there is no filling and wheels or other concentrated loads can bear directly on the culvert, the load should be considered as carried on a certain length of the culvert. The concentration is modified if there is any filling above the culvert and, if the depth of the filling is h1, a concentrated load F can be considered as spread over an area of $4h_1$. When h_1 equals or slightly exceeds half the width of the culvert, the concentrated load is equivalent to a uniformly distributed load of $F/4H_1^2$ in units of force per unit area over a length of culvert equal to $2h_1$.

The weight of the walls and top (and any load that is on them) produce an upward reaction from the ground. The weights of the bottom slab and the water in the culvert are carried directly on the ground below the slab and thus do not produce bending moments, although these weights must be taken into account when calculating the maximum pressure on the ground. The horizontal pressure due to the water in the culvert produces an internal triangular or a trapezoidal load if the surface of the water outside the culvert is above the top slab. The magnitude and distribution of the horizontal pressure due to the earth against the sides of the culvert can be calculated in accordance with the formulae given in the table 16-20, consideration being given to the possibility of the ground becoming water logged with consequent increased pressures and possibility of floatation."

Oyenuga (2001) stated that "A box culvert consists of three elements that is, top slab, walls and bottom slab and the loads on each component are as follows:

Top slab: The load will include slab own weight, imposed load and weight of earth fill. In cases where the depth of the earth fill is greater than three times the width of the culvert, the earth load can be assumed to be equal to earth loads of height three times the culvert width. Should be based on tyre width. For a wheel load of height, h, the load should be spread over an area of $4h^2$,that is 2h by 2h. When h equals or slightly exceeds one half of the width of the culvert, the wheel load can be assumed as equivalent to a uniformly distributed load of $W/4h^2$ where W is the wheel load. Wheel loads are given in units of 2.5KN and the most common HB loads are 30 and 45units equivalent to 75KN and 112.5KN respectively. Critical culverts should be designed for 45 units HB loads and the number of wheels that incident on the culverts noted.

Walls: Loads on walls include own weight, effect of active earth pressure, the effect of any the inside wall and the wall should be designed to resist this pressure and assuming no back fill.

Bottom slab: the top slab and its imposed load, the walls and the pressures on them produce an upward pressure (reaction) from the ground and causes moments. The weight of the water in the culvert and the weight of the bottom slab should be considered when determining the maximum pressure on the ground but since they borne by the ground directly, they do not generate moment.

According to ASSHTO specification section 12, "When the depth of fill exceeds 2.4m, live load is ignored."

Punmia, Jain and Jain (1992) stated that, A box culvert is subjected to soil load from outside and water load from inside. The vertical walls are subjected to earth pressures from outside and water pressure from inside. Similarly, the bottom slab will be subjected to soil pressure from outside and water pressure from inside. The top slab will, however, be subjected to embankment weight and traffic loads, if any." The weight of bottom slab of a box culvert will be resisted by equal and opposite soil pressure without bending in the bottom slab.

2.3.3 Factor of Safety

Factors of safety are usually applied to cater for unforeseen circumstances and uncertainties in the calculation method.

LRFD concrete box culverts from Engineering Policy Guide (2007) gives load factors for box culvert design thus.

Load Description	**Load Designation**	**Strength 1**		**Service 1 factor**
		Max. Factor	Min. factor	
Dead load of members	DC	1.25	0.9	1.0
vertical earth pressure	EV	1.30	0.9	1.0
Horizontal earth pressure	*EH(barrel)	1.35	1.0	1.0
	EHH(wings)	1.50	NA	1.0
water pressure	WA	1.0	0.0	1.0
Live load	LL	1.75	0.0	1.0
Dynamic load allowance	IM	1.75	0.0	
live load surcharge	*LS	1.75	1.0/0.0	1.0

` Table 2 Load Factors for Box Culvert Design

The maximum factor should be applied with the maximum equivalent fluid, pressure, and the minimum factor should be applied with the minimum fluid pressure. Live load surcharge, LS, is neglected when live load is neglected.

In this project, a factor of safety of 1.40 and 1.60 will be applied to culvert own weight, wheel load and earth load respectively.

2.3.4 Load Cases

According to Oyenuga (2001), “Two conditions should be considered as follows;

- **a. Culvert Empty**: full load on top of the slab, surcharge load and earth pressure on the walls.
- **b. Culvert full**: minimum load on top of the slab (eg own weight), minimum earth pressure (if possible, none), on walls and maximum lateral water pressure on the walls should the area be water logged, the pressure on the wall will be trapezoidal and they will be upward water pressure (equal to the weight of water above the surface of the top slab) on the top slab and should be taken into consideration”. After analyzing both load cases he stated that, “these loads are less than case 1 loads excepts for the wall loads and practically speaking if the culvert is flooded with water would be on both side of the walls cancelling the net water pressure on the walls. The case 1 loads can therefore be used for design purposes”. In the above equation, “these loads” refer to culvert full while “case 1” refers to culvert empty.

Sinha and Sharma (2009) stated that, “mainly three load cases govern the design. These are given below;

- a. Box empty, live load surcharge on top slab of box and superimposed surcharge load on earth fill.
- b. Box inside full with water, live load surcharge on top slab and superimposed surcharge load in earth fill.
- c. Box inside full with water, live load surcharge on top slab and no superimposed surcharge on earth fill.”

  In this project two load cases will be considered. That is, culvert empty and culvert full as stated by Oyenuga(2001).

2.3.5 Analysis and Design

Box culverts shall be analyzed as closed rigid frames. The dead and superimposed earth loads, the lateral earth pressure, and the live and impact load are to be analyzed separately. The result of these separate loading conditions shall be assembled in various combination to give maximum moment and shear at the critical points. That is, the corners and the positive moment areas. Appropriate load positions shall be used to produce maximum positive and minimum moments. A maximum of one half of the moment caused by lateral earth pressure, including any live load surcharge may be used to reduce the positive moment on top and bottom slabs."

There are several methods of analyzing rigid frames. They include;

1. Displacement method.
2. Method of force.
3. Moment distribution method.

Oyenuga stated that, "a box culvert should be analyzed as a rigid structure with moment occurring at the corners. The hardy cross method of moment distribution is best suited for culvert analysis or the kani's method of moment distribution."

According to AASHTO specification section 12, (1998), Buried structures and tunnel liners, shall be analyzed and designed as rigid frames."

Reynolds and Steedman (1988) stated that, "the bending moments produced in monolithic rectangular culverts may be determined by considering the floor slabs as a continuous beam of four spans with equal bending moments at the end support. But, if the bending of the bottom slab tends to produce a downward deflection, the compressibility of the ground and the consequent effect on the bending must be considered.

Oyenuga (2001), further stated that, "due to the interconnections of the members, the shear in the walls introduces axial forces in the slabs and vice. Hence each element must be designed for moment and axial pull (just as a column that is subjected to bending and axial pull."

In this project, the structural members will be designed as columns subject to bending and axial pull.

2.3.6 Reinforcement

Oyenuga (2001) stresses that the designer should, "should note the u-bars provided at the corner to carter for torsional effects". He also stated that a minimum steel reinforcement of 0.4%bh be provided in the structural members. Where b= 1000mm length of the culvert and h= depth of the member.

2.3.7 Dimensions and specifications

According to AASHTO specification section 12, "the minimum top and bottom slab thickness is 200mm. All cells of multiple cell reinforced concrete box (RCB) culvert shall be the same size. The minimum height of RCB culvert is 1.25 vertical clearance to allow for inspection. Four sided boxes can typically be used for spans up to 3.5m span length from 3.4m to 7.5m are typically bridges using three sided rigid frames."

The thickness of walls and slabs of a box culvert shall be not less than 250mm for members with reinforcement in both faces.

If the top slab is to be used as the road way wearing surface, then it shall have a 50mm-75mm concrete top reinforcement cover. Additionally, the top slab concrete shall be 4500 psi minimum strength, and the top math of reinforcing steel shall be epoxy coated. When the top slab is not the riding surface, the earth cover provided hall be not less than 22.86cm (in addition to paving) at the minimum point.

Construction joints shall be provided at approximately 9m. Expansion joints shall be provided at approximately 27m intervals. Reinforcement shall be stopped two inches clear of joints. Head walls shall be provided at the exposed ends of culverts to retain the earth embankment and to act as edge distribution beams.

In other to provide for the effect of scour, cut- off walls in a minimum of 0.9m deep shall be provided at the exposed end of the culverts.

Wing wall footing shall be set at the elevations of the cut-off walls and securely toed to them with reinforcement."

In this project, the slabs and walls are 350mm thick and the top slab has a concrete cover 50mm although it carries an embankment of 2m.

CHAPTER THREE

3.0 Methodology

The proposed culvert to be designed is located at Awka North Local Government Area of Anambra State. Awka North comprises of several villages but as mandated by this design, focus will be on Isuaniocha along Mgbakwu road. Isuaniocha is a populated area in Awka and its climatic type is tropical savannah, wet with latitude of $6^o16^I\ 20^{II}$ N.

Chainage point for triple cell box culvert = 0+750 m

Chainage point for single cell box culvert = 0+525 m

Length of the road = 16 m

Design discharge (Q) = 62.43 m^3/s

Slope, (S) = 0.0092

The design of a box culvert take into account many different engineering and technical aspects at the site and adjacent areas. The following criteria are considered for box culvert design as applicable.

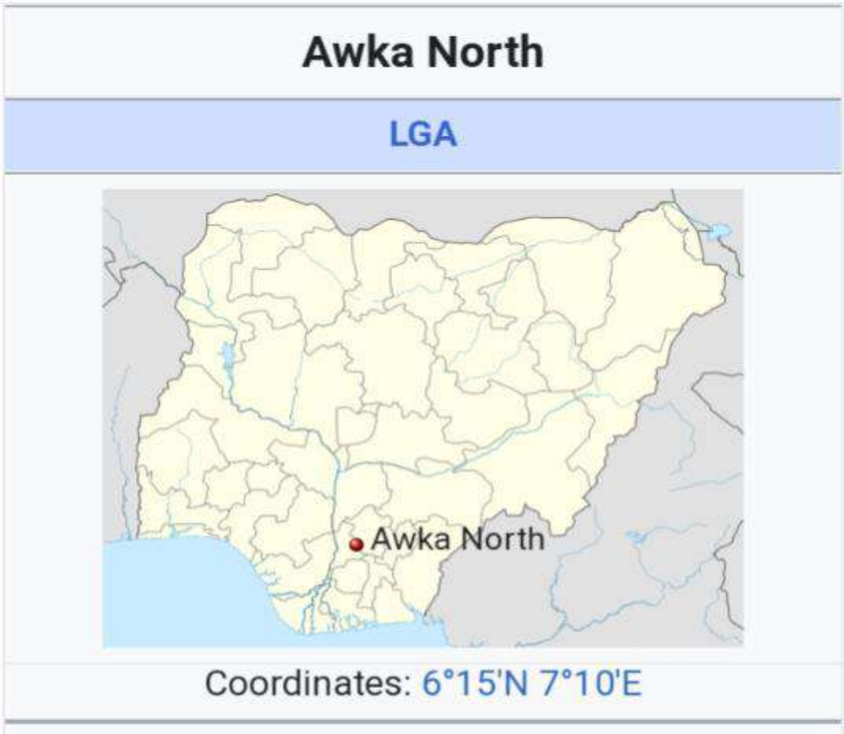


Figure 1: Map of Awka North

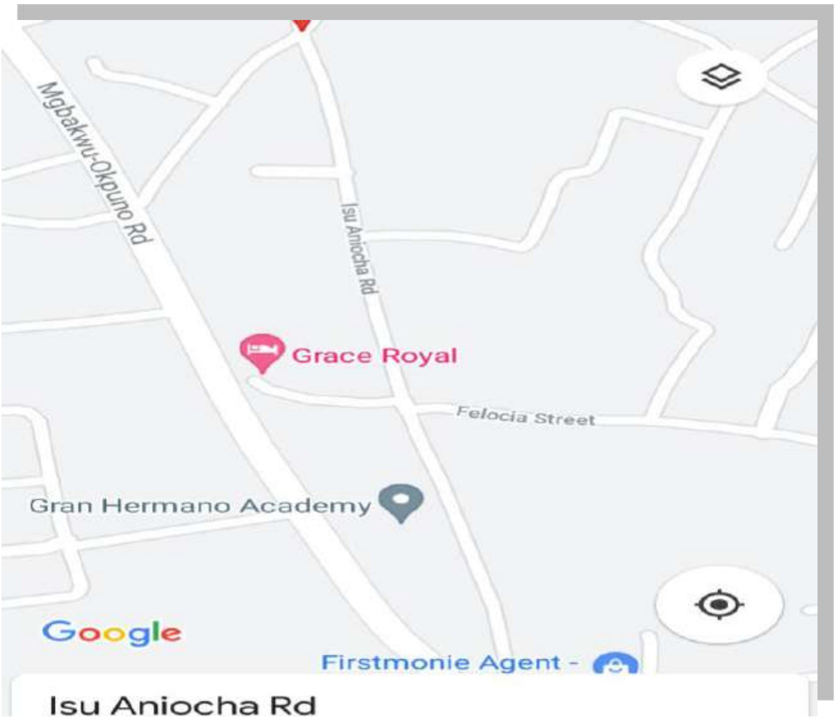


Figure 2: Map showing Isuaniocha Road

3.1 Site Investigation

For a safe and economic design of an engineering project, it is necessary to carry out investigations at the site of the proposed structure. Proper design of a box culvert structure calls for adequate knowledge of the sub-soil and hydraulic condition of the catchment area.

The main objectives of site investigations are:

1. To assess the general suitability of the site for the proposed work.
2. To enable adequate and economic design to be prepared.
3 To foresee and provide against difficulties that may arise during construction due to ground and local conditions.
4 To investigate the occurrence of causes of natural or created changes in the soil conditions and the result arising there from.
5 To get the chainage of the proposed culvert.

3.2 Ground Profile Survey

The design of a culvert must take into account the physical survey of the catchment area of the proposed project, Hydraulic studies of the stream flood, slope, soil type and bearing capacity of the soil.

3.3 The Hydraulic Design

According to Garber and Hoel (2000) 'the ultimate objectives in determining the hydraulic requirements for any highway drainage structure is to provide a suitable structure size that will economically and efficiently dispose of the expected runoff'. Thus, the hydraulic requirement must be met to avoid erosion and sedimentation in the system.

The most appropriate location of a culvert is in the existing channel bed, with centre line and slope of the culvert bed coinciding with that of the channel, therefore parameters used in the hydraulic design (proposed culvert) were obtained from the site investigations and Hydrology and Hydraulics of Box Culvert, alongside the application of information gotten from Ministry of works, Awka.

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MEMBER REF	**CALCULATIONS**	**OUT PUT**
T1.2Hydrology On hydraulics of box culvert	**DESIGN DATA** Chainage point for triple cell culvert = 0 +750 m Chainage point for single cell culvert = 0 + 525 m Design discharge (Q) = 6s2.43 m^3/sec Length of culvert = 16m Slope = 0.0092 Inlet loss coefficient ,Ke = 0.5 Allowable outlet velocity = 2.74m/s Tail water depth, tw = 2.62 m Allowable headwater depth = 3.36 m Manning's roughness coefficient, n = 0.012 Several sizes of culvert were until the size that satisfy the limitations of head water elevation and outlet velocity was formed to be triple cell box culvert with each cell having a dimension of 2.8 x2.8 m. Dimension = 2.8m Width of culvert, B = 8.4m Cross sectional area per cell = 2.8 x 2.8 = 7.84 $\frac{Q}{B} = \frac{62.43\ m,^3/s}{8.4\ m} = 7.43 m,^2/s$ Angle of wing wall flow =30^0 Fig 3.3 inlet control nomograph $\frac{HW}{D}$ = 1.02	

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MEMBER REF	CALCULATIONS	OUT PUT
	Hw =1.02 x 2.8 m = 2.89m Using outlet control nomograph (fig 3.4) Ke= 0.5 Cross sectional area of a single cell = 7.84m^2 Design discharge per cell = $\frac{62.43\ m,^3/s}{3} = 20.81 m,^3/s$ Height = 0.567m Hw = Hw $_+$ $h_o - l_s$ But Hw = $\frac{D_{C+D}}{2}$ or Tw whichever is greater From fig 3.5 critical depth box culvert and $\frac{Q}{B} = 7.432 m^3/s$ Critical depth (DC) = 1.753m Thus Hw = $\frac{D_{C+D}}{2}$ = $\frac{1.753+2.8}{2}$ = 2.277m Tw = 2.62m Tw > hw =2.277 H_o = Tw = 2.62m Hw = 0.569 +2.625 – 16 x 0.000914 = 3.1733m Outlet control Hw > 2.886m (inlet control) Therefore outlet control governs the culvert design Outlet velocity check , V = $\frac{Q}{A} = \frac{62.43}{8.4x\ 2.8} = \frac{2.65m}{s} < 2.74m$/s…………………....ok For minimum flow (Qm) = 24.07m^3/s V_{min} = $\frac{24.07}{8.4\ x2.8} = 1.02m/s$……………………………ok This complies with the recommended non socuring and silt velocity of between 0.914m/s and 3m/s Thus the outlet velocity is ok	Q per cell = 20.81m^3/s Dc = 1.753m Velocity = 2.65m/s V_{min} = 1.02 m/s Provide a triple cell box culvert of 2.8 x 2.8m Each cell

CHAPTER FOUR

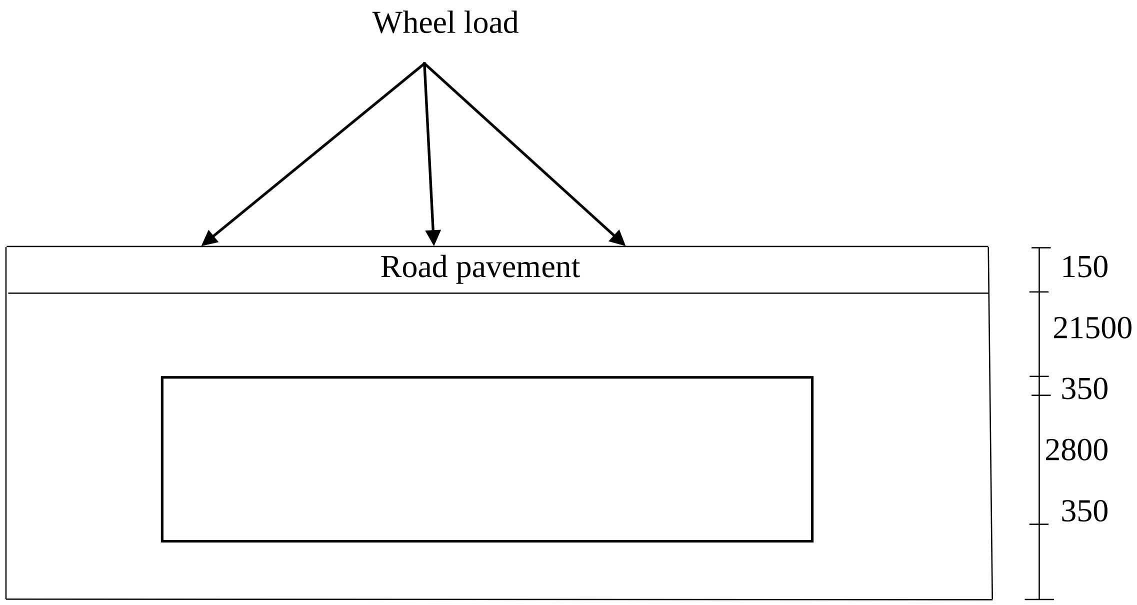
4.0 STRUCTURAL ANALYSIS

Loading Analysis

Having obtained the dimension (size) of the proposed box culvert through the hydraulic design carried out in the previous chapter.

Chainage point for the triple cell box culvert = 0+750 m

The figure below show a section of a triple box culvert with the various loads the culvert are likely to be subjected to.



Chainage point = 0 + 750

Fig 3. Triple cell box culvert

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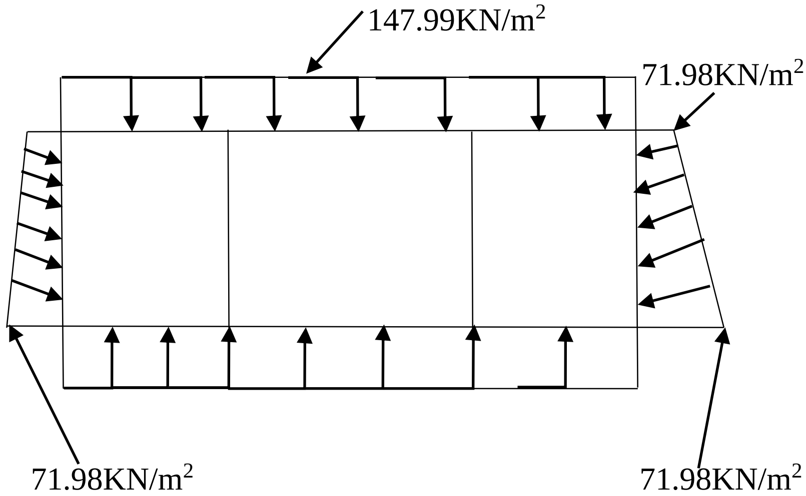
MEMBER REF	**CALCULATIONS**	**OUT PUT**
T.2 Reynold and Steadman	All the design information used for each of the triple cells are applicable for the single cell culvert , CASE 1(culvert when empty) for triple cell box culvert -Top slab **Dead load** Self weight = 0.35 x24x1.4 =11.76KN/m^2 Thickness of road pavement = 0.15m Unit weight of asphalt, vs = 23 KN/m^3 Weight of road pavement = 0.15x23x1.4 =5.52KNm^2 Unit weight of earth fill =18 kN/m^3. Depth of earth fill = 2m, Weight of earth fill = 18 x 2 x 1.6 = 57.67KN/m^2 Total dead load = self weight of slab + weight of road pavement +weight of earth fill = (11.76 +5.52 +57.6)KN/m^2= 74.88KN/m^2 **Live load** Using abnormal HB loading Load per wheel = 112.5KN Since the height of the is more than one half of the culvert,each wheel load is equivalent to $\frac{w}{4h^2} = \frac{112.5}{4\ x\ 2^2} = 7.03\ KN$ Number of wheels that can incident on the culvert = 8(4wheels per axle),since each wheel load is spread over an area of $4h^2$= 4*2^2 = 16m	

MEMBER REF	**CALCULATIONS**	**OUT PUT**
O.V. Oyenuga Pg 335	The area of influence of the four wheel of the first axle will over-lap with the area of influence of those of the second axle. Total live load = 8*7.03*1.3 =73.112KN/m^2 Total u.d.l on top slab of culvert = total dead load + live load. =(74.88 + 73.112) KN/m^2 **Walls:** Earth pressure Angle of response Ø =30 Coefficient of active earth pressure Ka = $\frac{1-\sin Ø}{1+\sin \quad Ø}$ = $\frac{1-\sin 30}{1+\sin \quad 30}$ = 0.333 Earth pressure at any depth = kayz P(2.175) = 0.33 * 18 *2.175* 1.6 = 20.86KN/m^2 Earth pressure at bottom edge of the wall P(5.325) =0.333 * 18 *5.325 *16) = 51.07 KN/m^2 Horizontal surcharge pressure = surcharge due to traffic + surcharge due to pavement = (73.112 + 5.52) 0.33 = 26.18 KN/m^2 Total lateral pressure at top edge of wall =20.86+26.18 =47.04KN/m^2 **Bottom slab** Load from the slab = 147.99KN/m^2 Wall load = weight of each wall per meter x no of walls = (24 x 0.35 x 2.8 x1.4)x 4 = 131.71KN/	Total live load = 73.112KN/m^2 Total slab u.d.l = 147.99KN/m^2 Ka = 0.333

MEMBER REF	CALCULATIONS	OUT PUT
	Distributing weight of wall over bottom slab as UDL=$\frac{131.71\text{KN/m}}{9.8m}$ = 13.44KN/m^2 Total UDL on bottom slab = load from top slab + wall load = (147.99 +13.44)KN/m^2 = (147.99 +13.44) KN/m^2 = 161.43KN/m^2  **CASE 2**:Assuming the culvert is full of water and over flooded to maximum of 2m **Top Slab:** There will be an upthrust underneath the top slab equal to the weight of the water disposed by the 2m depth of culvert embarkment structure water load = 2x 9.81x1.6 = 31.39KN/m^2 walls due to water pressure only at top edge of wall = 2 x 9.81 x 1.6 =31.39KN/m^2	Bottom slab UDL= 161.43 KN

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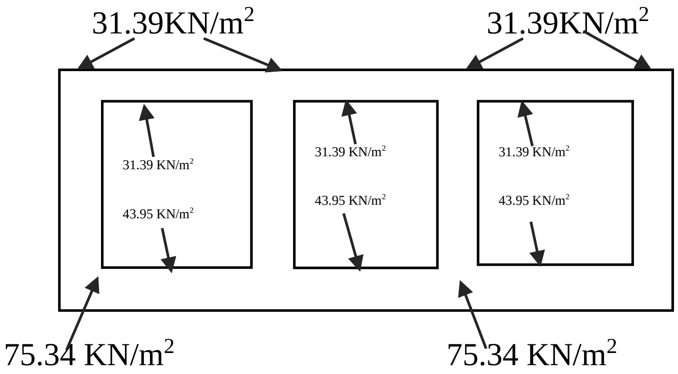
MEMBER REF	CALCULATIONS	OUT PUT

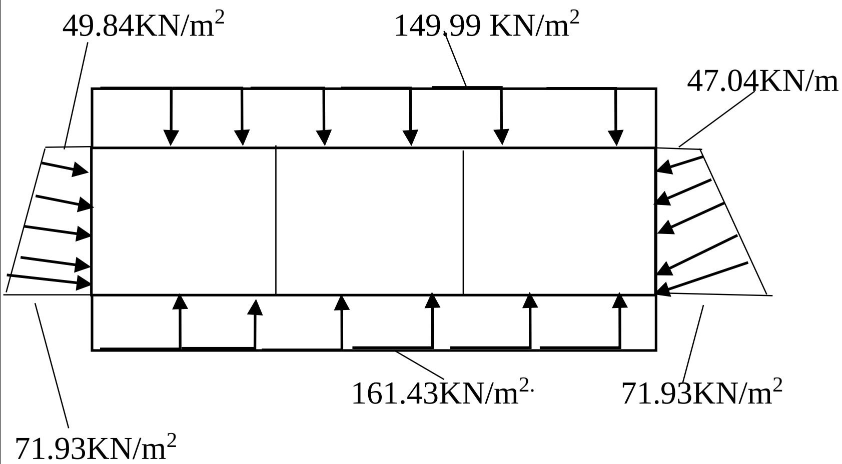
At bottom edge of wall = 48 x 9.81x 1.6 = 75.34 KN/m^2

Bottom slab = 2.8 x 9.81 x 1.6 = 43.95 KN/m^2



The case two loads are less than the case 1 loads ,seeing that the water pressure on the top slab and walls act in the opposite direction to the loads outside .Thereby producing a resultant pressure of less value on the members in practice, there will be water on both sides of the walls at flooding and the net pressure will be zero. On the Bottom slab, the water generates an equal and opposite reaction and no moment is generated. Thus the case 1 loads will be used for the design because they cortical.

4.2 Moment and shear analysis



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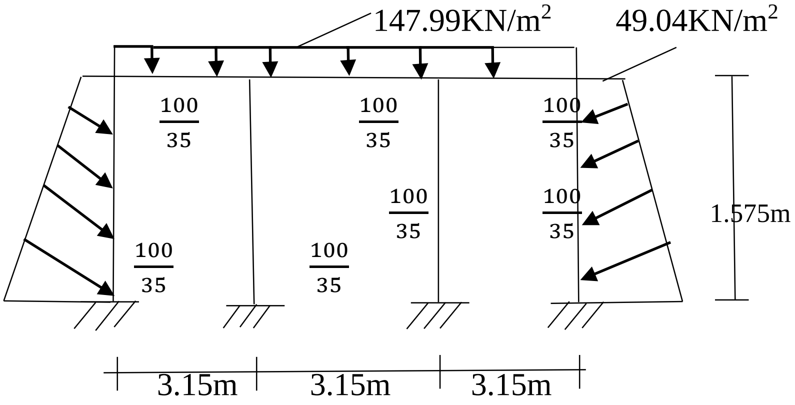
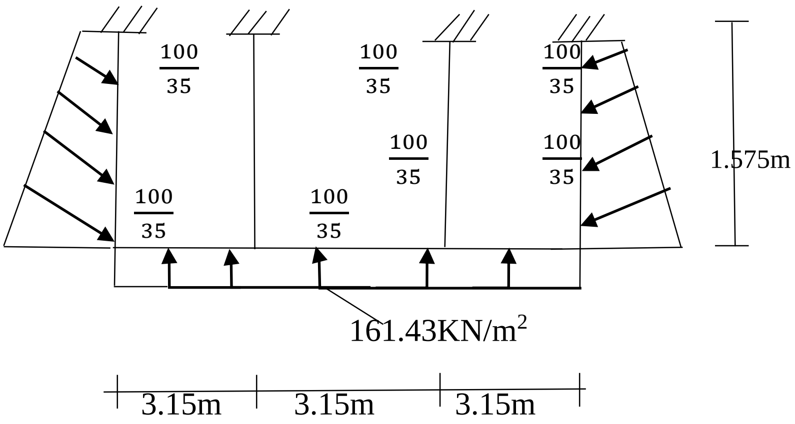
MEMBER REF	CALCULATIONS	OUT PUT
	Using method of force Break the structure into two equal halves across the walls and analyse separately. assuming fixed support at the base	





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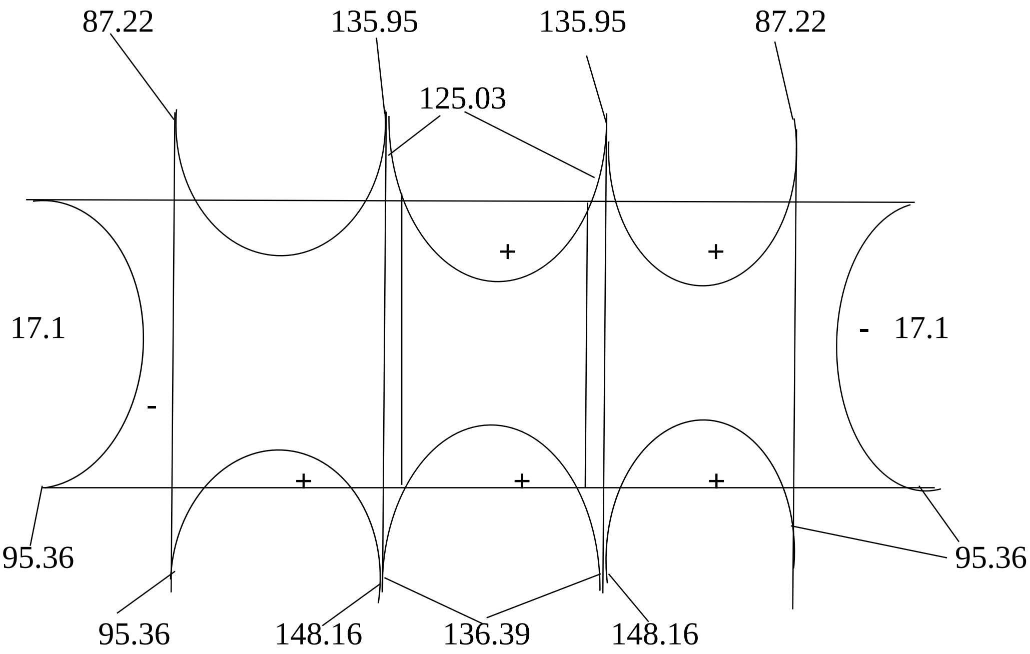
Below depicts Bending moment diagram after all the calculations done



M_{def} all values in KNm

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MEMBER REF	CALCULATIONS	OUT PUT
	Design Of Structural Elements The culvert is design to adequately resist the maximum bending moment and shearing forces. It is subjected to as obtained from the loading analysis in the previous chapter due to interconnections of the members, the shear in the cracks introduce axial forces in the slab and vice versa . Hence each member is designed for the moment and axial forces (just as a column that is subjected to loading and axial pull) From the analysis the following maximum stresses were obtained	

Element	Max support moment (KNm)	Max span moment (KNm)	Max axial pull (KNm)	Max shear (KN)
Top slab	135.95	72.04	1112.02	248.51
Bottom slab	148.19	78.47	131.7	271.01
Side wall	95.36	17.1	239.49	131.01
Internal walls	11.77	9.67	271	11.21

Design information

Client	Mgbakwu ,isuaniocha ,Awka, L.G.A	
Architect,	Akpa Nnamdi Oyo	
Site	Isuaniocha Awka – north, LGA, Anambra state.	
Design engineer	Akpa Nnamdi oyo	

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MEMBER REF	**CALCULATIONS**	**OUT PUT**
Supervising engr	Engr. Prof. Aginam Chukwurah	
Intended of structure	Transportation	
Relevant code	Bs 8110 :part 1:1997 and part 2 :1985	
Design stress	Concrete grade fcu =25N/mm^2 and grade of steel = 460 N/m^2	
Soil condition	Firm gravely laterite clay – 180 KN/m^2	
Concrete cover	50mm	
Several loading	Unit wheel load = 112. 5 KN/m^2	T.10, Reynold and Steedman
	Unit weight of soil cover = 18KN/m^3	
	Angle of internal friction Ø = 30^0	T.17 ,,
	Unit weight of concrete = 24KN/m^3	T.17,
		T.2
Design data	K=$\frac{m}{fcub\ d^2}$ As =$\frac{m}{0.95fylad}$ La = 0.5 +$\sqrt{(0.25-\frac{k}{0.9})} \leq 0.95$ M.f = 0.55 +$\frac{(477-fs)}{(120\left(0.9+\frac{m}{bd^2}\right))}$ h =350mm d = h – c - $\frac{Ø}{2}$ = 350 – 50 - $\frac{16}{2}$ = 292mm $\frac{d}{h} = \frac{292}{300}$ = 0.83mm **Design of Top slab** Support moment M = 135.95KNm N = 112.02KN $\frac{N}{fcubh} = \frac{112.02 \times 10^3}{25\ x\ 1000x350} = 0.01$	

MEMBER REF	**CALCULATIONS**	**OUT PUT**
From design chart 10.2 v.Oyenuga pg 306 T.10.3 RCD Oyenuga pg 344	$\frac{M}{fcubh} = \frac{135.95.\text{x } 10^6}{25\ x\ 1000x350^2} = 0.04$ $\propto = 0.01$ As = 0.0026 x fcubh = 0.0026 x 0.1 x 25 x 1000 x 350 = 2275mm^2 $AS_{min} = \frac{0.4bh}{100} = \frac{0.4 \text{ x } 1000 \text{ x}350}{100}$ = 1400mm^2 Provide Y25 @ 200mm c/c top (As_{prov}=2450mm^2) Provide Y12 @ 150mm c/c distribution	Y25@ 200 c/c top Y12 @ 150mmc/c dist.
	Mid span reinforcement M= 72.04KN/m N = 112.02KN $\frac{N}{fcubh} = \frac{112.02 \text{ x } 10^3}{25\ x\ 1000x350} = 0.01$ $\frac{M}{fcubh^2} = \frac{72.04.\text{x } 10^6}{25\ x\ 1000x350^2} = 0.2$ $\propto = 0.04$	
T.10.2 v.Oyenuga pg 356 T.10.3 RCD v.Oyenuga pg 344	As = 0.0026 x 0.04 x 25 x1000 x350 = 910mm^2 $AS_{min} = \frac{0.4bh}{100} = \frac{0.4 \text{ x } 1000 \text{ x}350}{100}$ = 1400mm^2 Prov Y20 @ 200mm c/c btm (As_{prov}=1570mm^2) Prov Y12 @ 150mm c/c distribution	Y20@ 200 c/c btm Y12 @ 150mmc/c dist

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MEMBER REF	**CALCULATIONS**	**OUT PUT**
	Check For Deflection Fs = $\frac{2}{3} fy \ \frac{\text{ASreq}}{ASprov}$ $= \frac{2}{3}\ x\ 460\ x\ \frac{1400}{1570}\ = 273.46$ M.F = $0.55 + \frac{(477-fs)}{(120\left(0.9+\frac{m}{bd^2}\right))}$ $= 0.55 + \frac{(477-273.46)}{(120\left(0.9+\frac{72.04\ x 10^6}{1000x296^2}\right))}$	Mf = 1.52
	M.F = 1.52 < 2 ………………………..ok Since thee slab is continuous ,the span effective depth ratio is 23 $d_{req} = \frac{span}{23xMF} = \frac{3.15\ X\ 10^3}{230X\ 1.52} = 90.10mm$ $d_{req} < d_{prov}$	Deflection ok
	maximum shear ,v = 248.51 kN but the culvert is chamfered 550mm at the joints. therefore deflection is satisfied Check for Shear is reduced is reduced to V = 243.51 – (0.55 x 147.99) = 167.12KN Shear stress ,V = $\frac{167.12\ x 10^3}{1000x\ 292}$ =0.572N/mm^2 Concrete shear stress $V_c = 0.632(\frac{(100AS)}{bd}^{\frac{1}{3}}(\frac{(400)}{d}^{\frac{1}{4}})$ $= 0.632(\frac{(100x2450)}{1000x292}^{\frac{1}{3}}(\frac{(400)}{292}^{\frac{1}{4}})$ = 0.645N/mm^2 $V < V_c$ ……………………………………..ok	Shear ok

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MEMBER REF	CALCULATIONS	OUTPUT
	5.2 DESIGN OF BOTTOM SLAB	
	Support reinforcement	
	$M = 148.19KNm$	
	$N = 131.7KN$	
	$\frac{N}{f_{cu}bh} = \frac{131.7\times10^3}{25\times1000\times350}$	
	$= 0.02$	
	$\frac{M}{f_{cu}bh^2} = \frac{148.19\times10^6}{25\times1000\times350^2}$	
	$= 0.05$	
chart 10.2 RCD V.O Oyenuga Pg 356	$\alpha = 0.1$	
	$A_s = 0.0026\alpha f_{cu}bh$	
	$= 0.0026\times0.1\times25\times1000\times350$	
	$= 2275\ mm^2$	
	$A_{s\,min} = \frac{0.4bh}{100}$	
	$= \frac{0.4\times1000\times350}{100}$	
	$= 1400\ mm^2$	
T.10.3 RCD Oyenuga Pg 344	provide Y25mm @200mm c/c Btm	Y25/200 c/c Btm each
	provide Y12mm @ 150mm c/c distribution	Y12/150 c/c distribution
	Midspan Reinforcement	
	$M = 78.47\ KNm$	
	$N = 131.7KN$	

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MEMBER REF	CALCULATIONS	OUTPUT

$$\frac{N}{f_{cu}bh} = \frac{131.7 \times 10^3}{25 \times 1000 \times 350} = 0.02$$

$$\frac{M}{f_{cu}bh^2} = \frac{78.47 \times 10^6}{25 \times 1000 \times 350^2}$$

$$= 0.03$$

Chart 10.2 RCD V. Oyenuga Pg 356

$\alpha = 0.06$

$A_s = 0.0026 \alpha f_{cu}bh$

$A_s = 0.0026 \times 0.06 \times 25 \times 1000 \times 350$

$= 1365\ mm^2$

$$A_{s\,min} = \frac{0.4bh}{100}$$

$$= \frac{0.4 \times 1000 \times 350}{100}$$

$$= 1400\ mm^2$$

T.15.3 RCD V. Oyenuga Pg 344

Provide Y20mm @ 200mm c/c Top (A_s prov $= 1570\ mm^2$)

Provide Y12mm @150mm c/c distribution

OUTPUT: Y20/200 c/c Top width; Y12/150 c/c distr.

Check for deflection

$$f_s = \frac{2}{3} f_y \frac{A_{s\,req}}{A_{s\,prov}}$$

$$= \frac{2}{3} \times 460 \times \frac{1400}{1570}$$

$$= 273.46$$

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MEMBER REF	CALCULATIONS	OUTPUT
	$M.f = 0.55 + \frac{(477 - fs)}{\left[120\left(0.9 + \frac{M}{bd^2}\right)\right]}$	
	$= 0.55 + \frac{(477 - 465)}{\left[120\left(0.9 + \frac{78.47 \times 10^6}{1000 \times 292^2}\right)\right]}$	
	$= 1.48 < 2$ . . . . OK	
	Since the slab is continous, the span effective depth ratio is 23	
	$d_{req} = \frac{span}{23 \times m.f}$	
	$d_{req} = \frac{3.15 \times 10^3}{23 \times 1.48}$	
	$= 92.54mm$	
	$d_{req} < d_{prov}$ ∴ deflection is satisfied	deflection OK.
	Check for shear	
	Max. shear $V = 271.05KN$	
	But the culvert is chamfered 550mm at the joints. Thus the maximum shear is reduced to.	
	$V = 271.05 - 0.55(161.43)$	
	$= 182.22KN$	
	Shear stress, $v = \frac{V}{bd}$	

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MEMBER REF	CALCULATIONS	OUTPUT
	$= \frac{182.22 \times 10^3 \ N}{1000 \times 292 \ mm^2}$	
	$= 0.62 \ N/mm^2$	
	Concrete shear stress Vc (permissible)	
	$V_c = 0.632\left(\frac{100A_s}{bd}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}$	
	$V_c = 0.632\left(\frac{100 \times 2450}{1000 \times 292}\right)^{1/3}\left(\frac{400}{292}\right)^{1/4}$	
	$= 0.64 \ N/mm^2$	Shear OK.
	$V < V_c$ - . . . OK	

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MEMBER REF	CALCULATIONS	OUTPUT
	5.3 DESIGN OF SIDE WALLS	
	$M = 95.36$ kNm	
	$N = 237.49$ kN	
	$\frac{N}{f_{cu}bh} = \frac{237.49 \times 10^3}{25 \times 1000 \times 350}$	
	$= 0.03$	
	$\frac{M}{f_{cu}bh^2} = \frac{95.36 \times 10^6}{25 \times 1000 \times 350^2}$	
	$= 0.03$	
Chart 10-2 RCD V. Oyenuga Page 356	$\alpha = 0.05$	
	$A_s = 0.0026\alpha f_{cu}bh$	
	$= 0.0026 \times 0.05 \times 25 \times 1000 \times 350$	
	$= 1137.5 mm^2$	
	$A_{s\,min} = 0.4\% bh = 1400 mm^2$	
T-10-3 RCD V Oyenuga Pg 344	provide Y20mm @200mm c/c both faces ($A_{s\,prov} = 1570 mm^2$)	Y20/200 c/c both faces with
	provide Y12mm @150mm c/c distribution	Y12/150 c/c distrn
	Check for deflection	
	$f_s = \frac{2}{3}f_y \frac{A_{s\,req}}{A_{s\,prov}} = 273.46$ (As before)	
	$M.F = 0.55 + \frac{(477 - f_s)}{\left[120\left(0.9 + \frac{M}{bd^2}\right)\right]} \leq 2.0$	

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MEMBER REF	CALCULATIONS	OUTPUT
	$M.F = 0.55 + \frac{(477 - 273.46)}{\left[120\left(0.9 + \frac{95.36 \times 10^6}{(1000 \times 292^2)}\right)\right]}$	
	$= 1.39 < 2$ . . . ok	
	Since the slab is simply supported the span effective depth ratio is 20.	
	$d_{req} = \frac{span}{20 \times M.F}$	
	$= \frac{3.15 \times 10^3}{20 \times 1.39}$	
	$= 113.3$ mm	
	$d_{prov} > d_{req}$	Deflection ok
	∴ deflection is satisfied	
	<u>check for Shear</u>	
	Maximum shear $V = 131.7$KN	
	As a result of the 550mm chamfer,	
	This shear is reduced to	
	$131.7 - 0.55 \times 47.04 = 105.83$KN	
	Shear stress, $v = \frac{V}{bd}$	
	$= \frac{105.83 \times 10^3}{1000 \times 292}$	
	$= 0.362$ N/mm²	

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MEMBER REF	CALCULATIONS	OUTPUT
	Concrete Shear Stress V_c:	
	$V_c = 0.632\left(\frac{100A_s}{bd}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}$	
	$= 0.632\left(\frac{100 \times 1570}{1000 \times 297}\right)^{1/3}\left(\frac{400}{297}\right)^{1/4}$	
	$V_c = 0.556\ N/mm^2$	
	$V < V_c$	
	∴ Shear is satisfied	Shear OK.

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DATE: JANUARY, 2022 SHEET NO 01 OF 06

MEMBER REF	CALCULATIONS	OUTPUT
	5.4 DESIGN OF INTERNAL WALLS	
	$M = 11.77$ kNm	
	$N = 271.01$ kN	
	$\frac{N}{f_{cu}bh} = \frac{271.01 \times 10^3}{25 \times 1000 \times 350} = 0.03$	
	$\frac{M}{f_{cu}bh^2} = \frac{11.77 \times 10^6}{25 \times 1000 \times 350^2} = 0.004$	
chart 10.2 RCD V. Oyenuga pg 354	$\alpha = 0 \quad \therefore As = 0$ provide nominal reinforcement $A_{c\,min} = 0.4\%bh = 1400 mm^2$	
T 10.3 RCD V. Oyenuga Pg 344	provide Y20 mm @ 200mm c/c both faces ($As_{prov} = 1570 mm^2$) provide Y12 @150mm c/c distribution	Y20/200 c/c both faces walls Y12/150 c/c distrn
	Check for deflection $f_s = 273.46$ (as before)	
Pg347 RCD V. Oyenuga	$M.F = 0.55 + \frac{477 - f_s}{\left[120\left(0.9 + \frac{M}{bd^2}\right)\right]}$ $0.55 + \frac{477 - 273.46}{\left[120\left(0.9 + \frac{11.77 \times 10^6}{1000 \times 350^2}\right)\right]}$ $M.F = 2.18$ use 2	$M.F = 2$

85

DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO: 02 OF 06

MEMBER REF	CALCULATIONS	OUTPUT
	$d_{req} = \frac{Span}{20 \times mf}$	
	$= \frac{3.15 \times 10^3}{20 \times 2}$	
	$= 78.75\ mm$	
	$d_{req} < d_{prov}$	
	∴ deflection is satisfied	Deflection ok
	Shear is obviously satisfied	Shear OK.

86

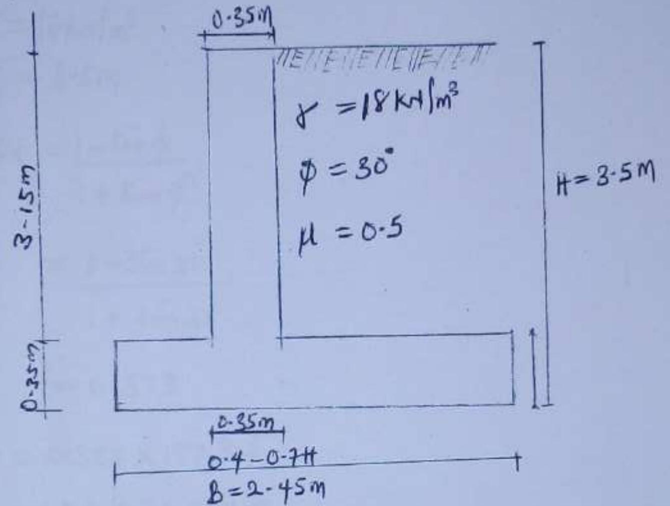
DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANURY, 2022 SHEET NO: 03 OF 04

MEMBER REF	CALCULATIONS	OUTPUT

5.5 DESIGN OF WING WALLS

Proportioning

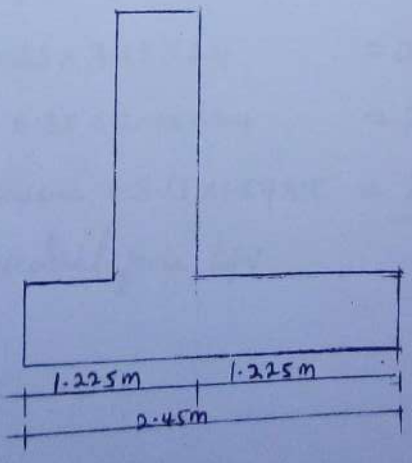


$H = 0.35 + 2.8 + 0.35 = 3.5m$

$B = 0.7H = 0.7 \times 3.5 = 2.45m$

The projection $= 0.16H = 0.16 \times 3.15 = 0.56m$

heel projection $= 2.45 - 0.56 - 0.35 = 1.54m$



DESIGNED BY: AKPA NWAMDI O | REGISTRATION NO: 20162224005

DATE: JANUARY, 2022 | SHEET NO: 02 of 07

MEMBER REF	CALCULATIONS	OUTPUT
	Active pressure	
	$P_a = K_a \gamma Z$	
T7 Reynolds andsteedman	$\phi = 30°$	
	$\gamma = 18 KN/m^3$	
	$Z = 3.5m$	
	$K_a = \frac{1-\sin\phi}{1+\sin\phi}$	
	$= \frac{1-\sin 30°}{1+\sin 30°}$	
	$= 0.333$	
	$P_a = 0.333 \times 18 \times 3.5$	
	$= 20.979 KN/m^2$	
	Earth pressure $P_A = 20.979 \times 3.5 \times \frac{1}{2}$	
	$= 36.713 KN \cdot E_{fh}$	$E_{fh} = 36.713 KN$
	Take Surcharge to be Zero	
	Vertical forces, f_v = These include forces due to	
T.2 Reynolds andsteedman	Wall $= 0.35 \times 3.15 \times 24 = 26.46KN$	
	Base $= 0.35 \times 2.45 \times 24 = 20.58KN$	
	Earth pressure $= 3.15 \times 1.54 \times 18 = 87.318KN$	
	Total vertical force, E_{fv} 134.358 KN	$E_{fv} = 134.358 KN$

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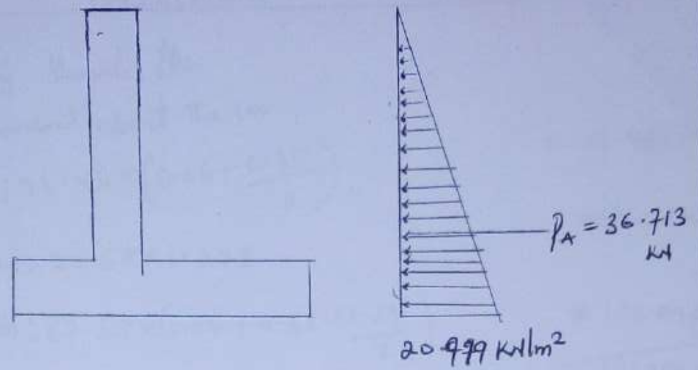
DESIGNED BY: AKPA NNAMDI .O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO: 03 OF 07

REF	CALCULATIONS	OUTPUT
		
	Checks for stability	
	i) check for sliding: Coefficient for friction, $\mu = 0.5$ for sliding, factor of safety $f_s = \frac{\text{Resisting forces (vertical)}}{\text{Sliding forces (horizontal)}} \geq 1.6$	
	Thus $f_s = \frac{\mu \Sigma f_v}{\Sigma f_h}$ $\frac{0.5 \times 134.358}{36.713}$ $f_s = 1.83 \geq 1.6$ ... ok	sliding ok
	ii) Check for overturning Overturning Moment, M_o Moment due to earth pressure $= 36.713 \times \frac{3.5}{3}$ $= 42.83$ kNm	$M_o = 42.83$k

DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224008

DATE: JANUARY, 2022 SHEET NO 04 of 07

MEMBER REF	CALCULATIONS	OUTPUT
	Resisting Moments, M_R	
	Take moment about the toe	
	Wall : $26.46 \times \left(0.56 + \frac{0.35}{2}\right)$ = 19.448 KNm	
	Base : 20.58×1.225 =25.211 KNm	
	Earth : $87.318 \times \left(0.56 + 0.35 + \frac{1.54}{2}\right)$ = 146.494 KNm	
	Total 191.393 KNm	
	factor of safety for overturning, fo	
	$fo = \frac{\text{Resisting moment}, M_R}{\text{Overturning Moment}, M_O} \geq 2.0$	
	$= \frac{191.393}{42.83}$	
	$fo = 4.47 \geq 2$ ... OK	overturning OK
	iii) Check for bearing capacity	
	$M_{net} = \Sigma M_R - \Sigma M_O$	
	$= 191.393 - 42.83$	
	$= 148.563$ KNm	
	Lever arm $\bar{x} = \frac{M_{net}}{\Sigma f_V}$	
	$= \frac{148.563}{134.358}$	
	$= 1.106$m	
	1.106m is within middle third of base 0.82m < 1.106m < 1.63m ... OK	$\bar{x} = 1.106$

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DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 20162240

DATE: JANUARY, 2022 SHEET NO 05 OF 07

MEMBER REF	CALCULATIONS	OUTPUT

Eccentricity, $e = B/2 - \bar{x}$

$$= \frac{2.45}{2} - 1.106$$

$$= 0.119m$$

$$B/6 = \frac{2.45}{6} = 0.408\ m$$

$$0.119m < 0.408m$$

Earth bearing pressure

$$q = \frac{\Sigma fv}{B}\left(1 \pm \frac{6e}{B}\right)$$

$$q_{toe} = q_{max} = \frac{\Sigma fv}{B}\left(1 + \frac{6e}{B}\right)$$

$$= \frac{134.358}{2.45}\left(1 + \frac{6 \times 0.119}{2.45}\right)$$

$$q_{max} = 70.82\ kN/m^2$$

$$q_{heel} - q_{min} = \frac{\Sigma fv}{B}\left(1 - \frac{6e}{B}\right)$$

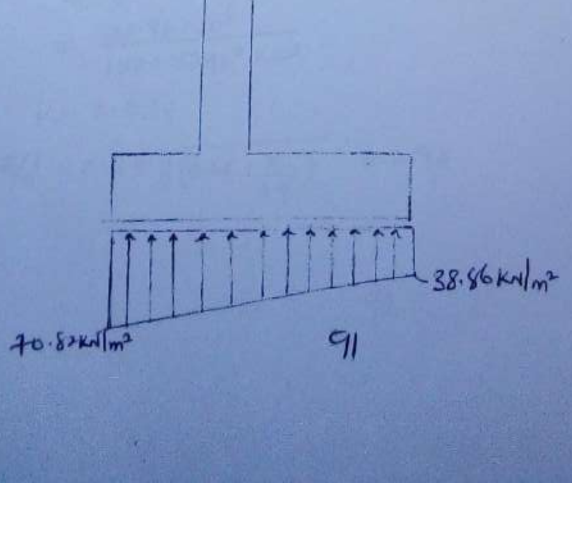
$$= \frac{134.358}{2.45}\left(1 - \frac{6 \times 0.119}{2.45}\right)$$

$$= 38.86\ kN/m^2$$

$q_{min} = 38.86\ kN/m$

$$70.82\ kN/m^2 < 180\ kN/m^2$$

bearing pressure is adequate



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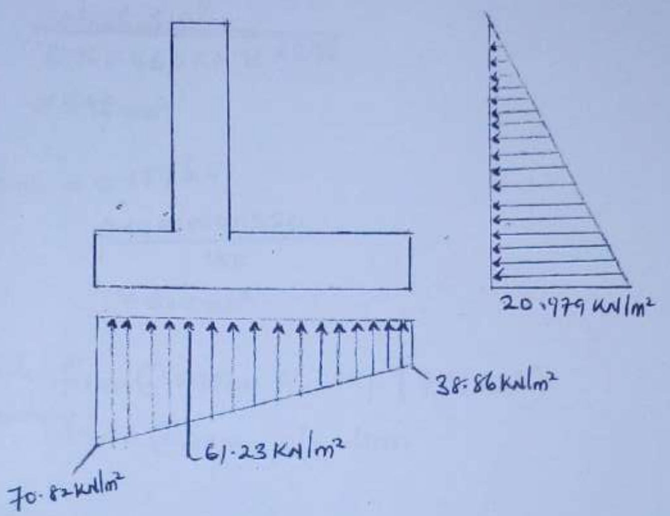
DESIGNED BY: AKPA NNAMDI O.
DATE: JANUARY, 2022

REGISTRATION NO: 2016224005
SHEET NO 06 of 07

MEMBER REF	CALCULATIONS	OUTPUT

5.6 DESIGN OF WALL



Bending Reinforcement

Wall

Horizontal moment $= 36.713\left(0.175 + \frac{3.15}{3}\right)$

$= 44.97$ kNm

Horizontal moment (U.L.S) $= 44.97 \times 1.6$
$= 71.96$ kNm

Horizontal moment = 71.9 kNm (U.L.S)

$h = 350$ mm

$d = 350 - 50 - 10 = 290$ mm

$K = \frac{M}{bd^2 f_{cu}}$

$= \frac{71.96 \times 10^6}{1000 \times 290^2 \times 25}$

$K = 0.034$

$Z/d = 0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)} = 0.95$

92

DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO 28 of 07

MEMBER REF	CALCULATIONS	OUTPUT
	$A_s = \frac{M}{0.95 f_y Z}$	
	$= \frac{71.96 \times 10^6}{0.95 \times 460 \times 0.95 \times 290}$	
	$= 598\ mm^2$	
BS V-Oyenuga pg 315	$A_{s\,min} = 0.15\%\,bh$	
	$\frac{0.15 \times 1000 \times 350}{100}$	
	$= 525\ mm^2$	
10.3 RCD Oyenuga	provide Y16mm @ 275mm c/c N.F (731mm²)	Y16/275 c/c N.F
	Provide Y12mm @ 150mm c/c distr.	Y12/150 c/c distr.
	Base	
	Designed using pressure at Ultimate limit state.	
	Top Reinforcement	
	Heel:	
	Taking moment about the stem centreline for vertical loads and the bearing pressures.	
	$M = 20.58 \times \frac{1.715}{2.45} \times \frac{1.715}{2} \times 1.0 + 87.318 \times \left(\frac{1.54}{2} + 0.175\right) 1.0 - 38.86 \times \frac{1.715^2}{2} - \left(\frac{61.23 - 38.86}{2}\right) \times 1.715 \times \frac{1.715}{3}$	
	$M = 26.76\ kNm$	
	$h = 350mm,\ d = 290\ mm$	
	$K = \frac{26.76 \times 10^6}{1000 \times 290^2 \times 25}$	
	$= 0.013$	

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DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO 09 OF 07

MEMBER REF	CALCULATIONS	OUTPUT
	$Z/d = 0.5 + \sqrt{\left(0.25 - \frac{0.013}{0.9}\right)}$	
	$= 0.95$	
	$A_s = \frac{26.76 \times 10^6}{0.95 \times 460 \times 0.95 \times 290}$	
	$= 222\ mm^2$	
	provide nominal reinforcement $= 0.15\% bh$	
	$= 525 mm^2$	
T.10.3 RCD Oyenuga	provide Y12mm bars @ 200mm c/c top (A_s prov $= 564 mm^2$)	Y12/200 c/c top
	provide Y12 @150mm c/c distribution	Y12/150 c/c distr.
	Toe (bottom reinforcement)	
	$= -20.58 \times \frac{0.735}{2.45} \times \frac{0.735}{2} + 70.82 \times \frac{0.735^2}{2}$	
	$= 16.86 kNm$	
	Moment is smaller than that for heel which required nominal reinforcement	
	Thus.	
	provide Y12mm bars @200mm c/c Bottom (A_s prov $= 566\ mm^2$)	Y12/200 c btm
		Y12/150 c/c distr

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MBER REF	CALCULATIONS	OUTPUT

5.7 DESIGN OF HEADWALL

Design Information:

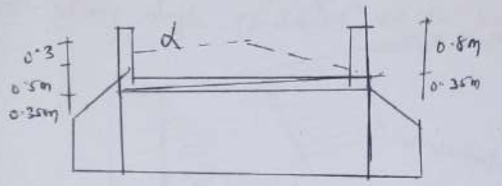
$\gamma = 18 kN/m^3$

$\phi = 30°$

depth of embankment = 2m

Embankment Slope = 1:2

Height of wall = 0.35 + 0.8 = 1.15m



Load Analysis

Coefficient of earth pressure

$$K_a = \cos\alpha \left(\frac{\cos\alpha - \sqrt{(\cos^2\alpha - \cos^2\phi)}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}} \right)$$

$\tan\alpha = 0.5$

$\alpha = \tan^{-1} 0.5$

$= 26.57°$

$$\therefore K_a = \cos 26.57° \left(\frac{\cos 26.57° - \sqrt{(\cos^2 26.57° - \cos^2 30°)}}{\cos 26.57° + \sqrt{(\cos^2 26.57° - \cos^2 30°)}} \right)$$

$= 0.537$

Earth pressure $P_a = K_a \gamma H$

$= 0.537 \times 18 \times 0.5 \times 1.6$

$= 7.73 kN/m^2$

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MEMBER REF | CALCULATIONS | OUTPUT

Surcharge pressure, P_s = (Load on top slab − selfweight of top slab − weight of 0.5m of earth fill) Ka

Weight of 0.5m of earth fill = γz

$= 18 \times 0.5 \times 1.6$

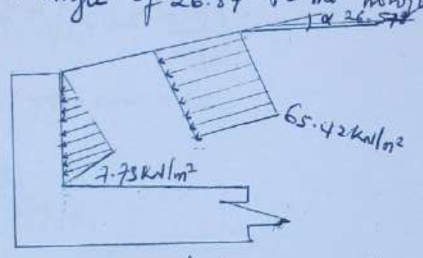
$= 14.4 KN/m^2$

Thus surcharge pressure

$(149.99 - 11.76 - 14.4) 0.537$

$= 65.42 KN/m^2$

As a result of the slope of the embankment, the loads on wall act at an angle of 26.57° to the horizontal. Thus



Resolving the forces to get the horizontal component gives

$P_{ah} = P_a \cos\alpha = 7.73 \cos 26.57° = 6.91 KN/m^2$

$P_{sh} = P_s \cos\alpha = 65.42 \cos 26.57 = 58.14 KN/m^2$

Bending moment = moment due to triangular load + moment due to surcharge pressure.

Moment due to triangular load =

$\frac{1}{2} \times 6.91 \times 0.5 \times \left(0.5 + \frac{0.35}{2}\right) = 1.17 KNm$

Moment due to surcharge pressure

$= 65.42 \times 0.5 \times \left(\frac{0.5}{2} + \frac{0.35}{2}\right)$

$= 14 KNm$

Total design bending moment = $(1.17 + 14) KNm$

$= 15.17 KNm$

ultimate design moment 15.17 KNm

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DESIGNED BY: AKPA NWAMDI REGISTRATION NO: 20K224005

SHEET NO 03 OF 08

DATE: JANUARY, 2022

MEMBER REF	CALCULATIONS	OUTPUT
	DESIGN	
	$M = 15.17\,KNm$	
	Assume 12mm bar as main reinforcement	
	$d = 350 - 50 - \frac{12}{2} = 294\,mm$	
	$k = \frac{M}{bd^2 f_{cu}}$	
	$= \frac{15.17 \times 10^6}{1000 \times 294^2 \times 25}$	
	$= 0.01$	
	$Z/d = 0.5 + \sqrt{\left(0.25 - \frac{0.01}{0.9}\right)}$	
	$= 0.99$ use 0.95	
	$A_s = \frac{M}{0.95 f_y Z}$	
	$= \frac{15.17 \times 10^6}{0.95 \times 460 \times 0.95 \times 294}$	
	$= 124\,mm^2$	
	$A_{s\,min} = 0.15\%\,bh = \frac{0.15}{100} \times 1000 \times 350$	
	$= 525\,mm^2$	
T.10.3 RCD V. Oyenuga Pg 3.44	Provide Y12 @ 175 mm c/c both faces (A_s prov $= 646\,mm^2$) with Y12 @ 150mm c/c distribution	Y12/175 c/c both faces with Y12/150 c/c distr.

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DESIGNED BY:- ARPA NNAMDI O. REGISTRATION No: 2016224005
DATE: JANUARY, 2022 SHEET No 07 of 09

MEMBER REF	CALCULATIONS	OUTPUT
	5.8 DESIGN OF APRON	
	Apron are provided to avoid the occurence of rip rap conditions at the entrance and exit of the culvert base. thus prevent the erosion of the culvert base. The apron terminates at its end with an apron beam which anchors it firmly to the ground - The apron is separated from the culvert frame by a construction joint.	
	DESIGN	
	Apron thickness, $h = 200mm$	
	Minimum area of reinforcement $= 0.15\% bh$	
	Cover to reinforcement $= 50mm$	
	$d = h - c - \phi/2$	
	$d = 200 - 50 - \frac{12}{2} = 144mm$	
	$As_{min} = \frac{0.15bh}{100}$	
	$= \frac{0.15 \times 1000 \times 200}{100}$	
	$= 300 mm^2$	
T.10.3 RCD V. Oyenuga Pg 344	provide Y_{12} @ 275mm c/c Btm ($A_{prov} = 411 mm^2$)	$Y_{12}/275$ c/c Btm with
	provide Y_{10} @ 275mm c/c distribution	$Y_{10}/275$ c/c distr.
	Apron beams	
	let beam breath $= 250mm$	
	beam depth $= 1000 mm$	
	$As_{min} = \frac{0.15bh}{1000} = \frac{0.15 \times 1000 \times 250}{100}$	
	$= 375 mm^2$	
T.10.3 RCD V. Oyenuga Pg 344	provide Y_{12} @ 275mm c/c N.F ($A_{sprov} = 411 mm^2$)	$Y_{12}/275$ c/c N-F with
	provide Y_{12} @ 200mm c/c distribution	$Y_{12}/200$ c/c distr.

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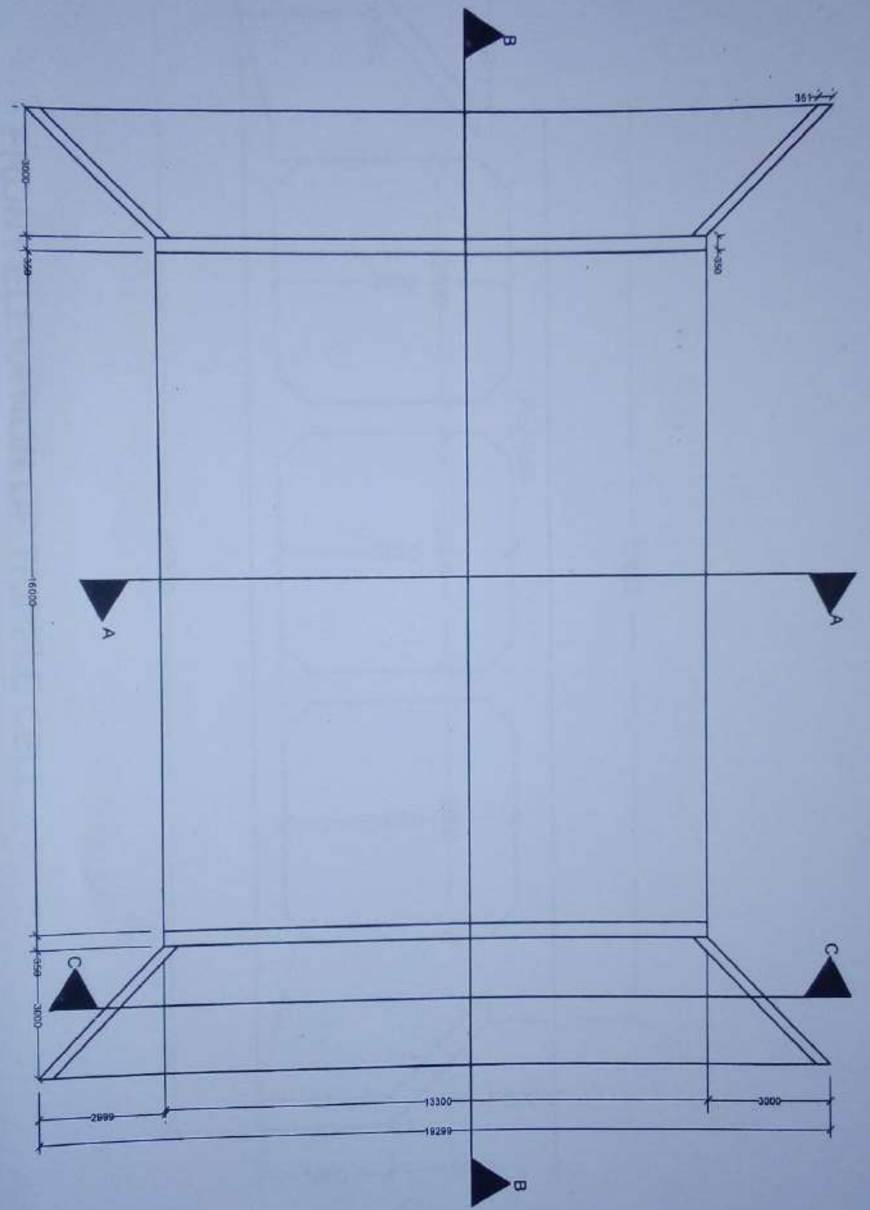
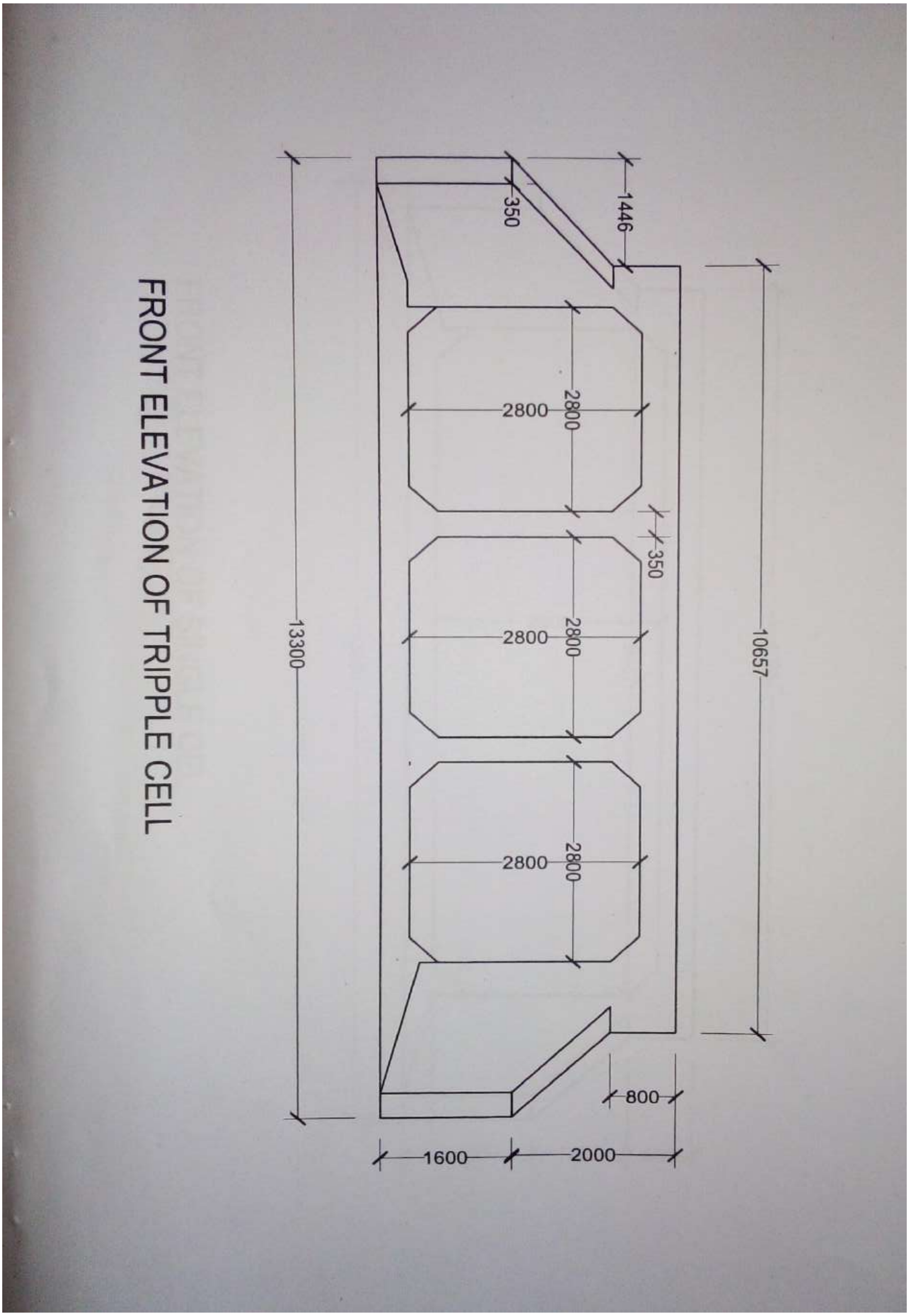
DETAILING

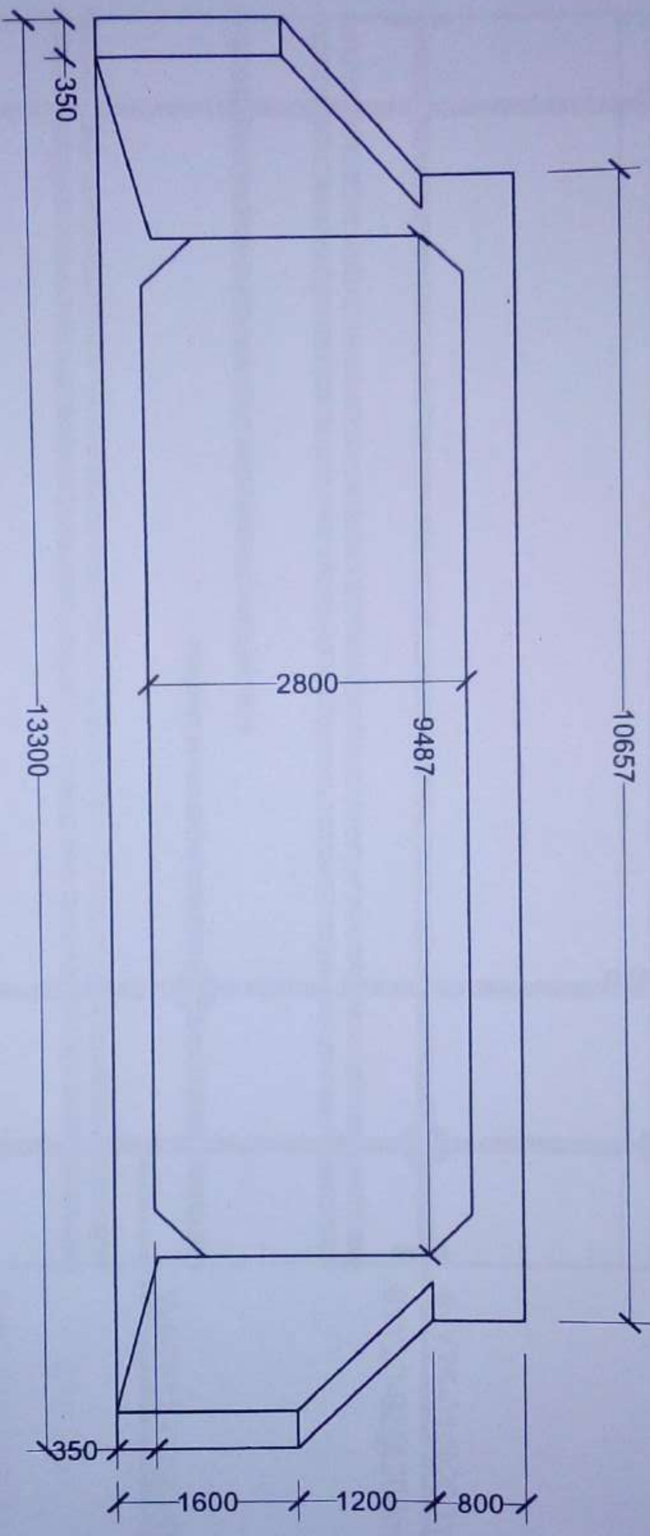


PLAN

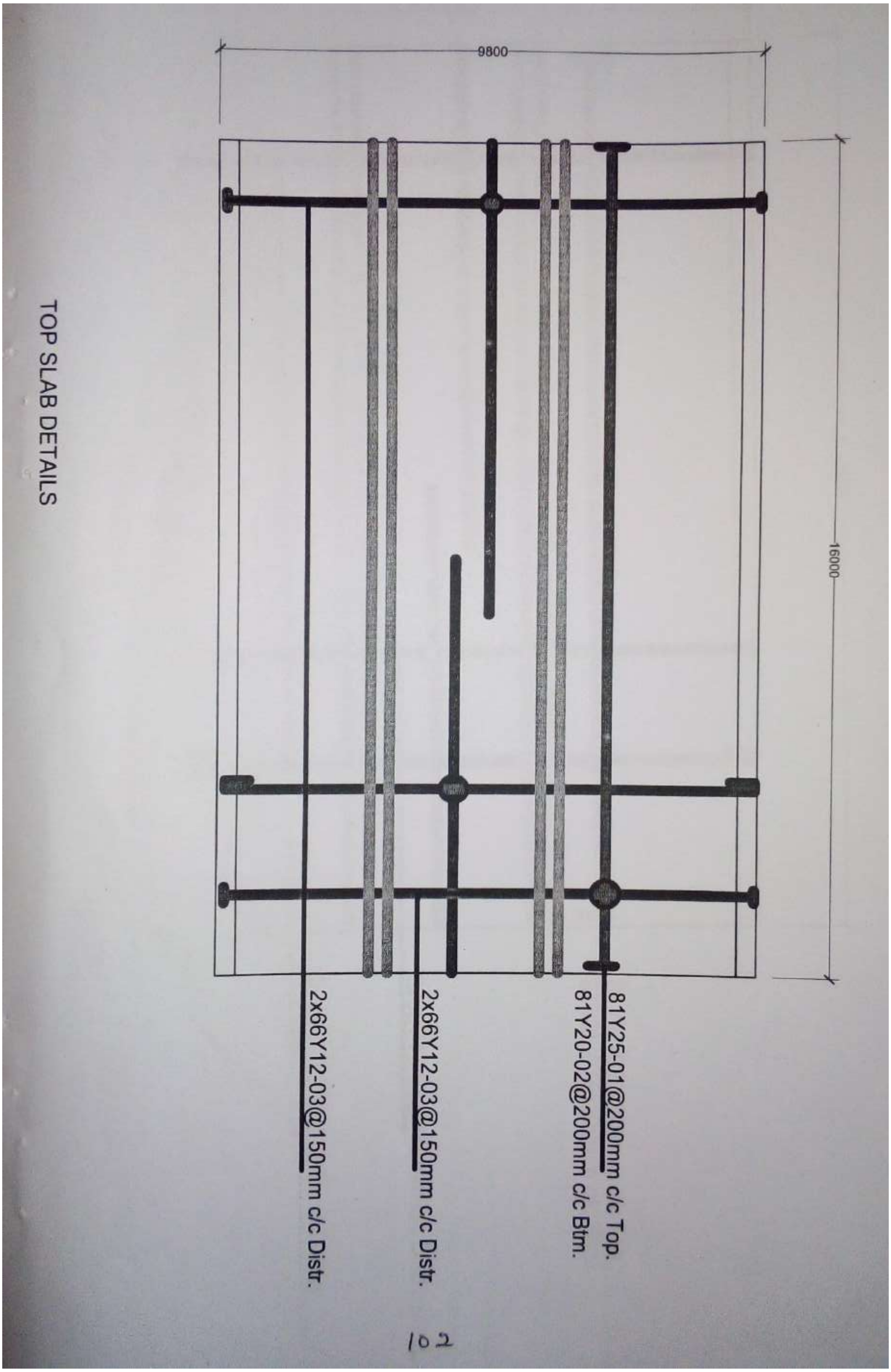
99



FRONT ELEVATION OF TRIPPLE CELL



FRONT ELEVATION OF SINGLE CEL


TOP SLAB DETAILS
9800
16000
81Y25-01@200mm c/c Top.
81Y20-02@200mm c/c Btm.
2x66Y12-03@150mm c/c Distr.
2x66Y12-03@150mm c/c Distr.
102

MEMBER REF	CALCULATIONS	OUTPUT
	Concrete Shear Stress Vc:	
	$V_c = 0.632\left(\frac{100A_s}{bd}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}$	
	$= 0.632\left(\frac{100 \times 1570}{1000 \times 297}\right)^{1/3}\left(\frac{400}{297}\right)^{1/4}$	
	$V_c = 0.556\ N/mm^2$	
	$V < V_c$	
	∴ Shear is satisfied	Shear OK.

DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO: 02 OF 06

MEMBER REF	CALCULATIONS	OUTPUT
	$d_{req} = \frac{Span}{20 \times m_f}$	
	$= \frac{3.15 \times 10^3}{20 \times 2}$	
	$= 78.75\ mm$	
	$d_{req} < d_{prov}$	
	∴ deflection is satisfied	Deflection ok
	Shear is obviously satisfied	Shear OK.

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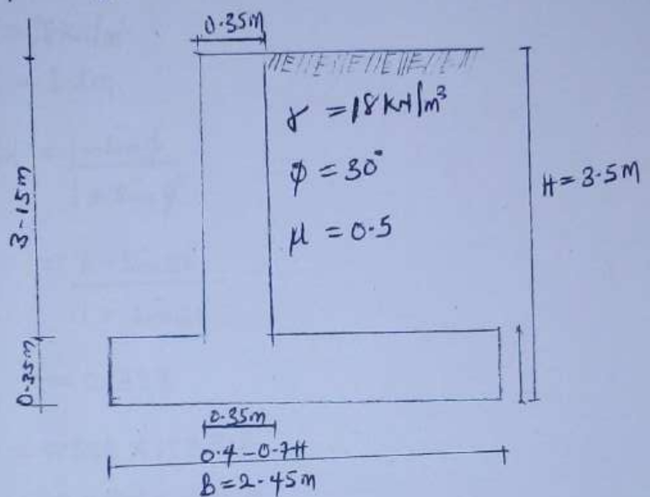
DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 20162248DS

DATE: JANURY, 2022 SHEET NO: 03 OF 04

MEMBER REF	CALCULATIONS	OUTPUT
	5.5 DESIGN OF WING WALLS	

Proportioning

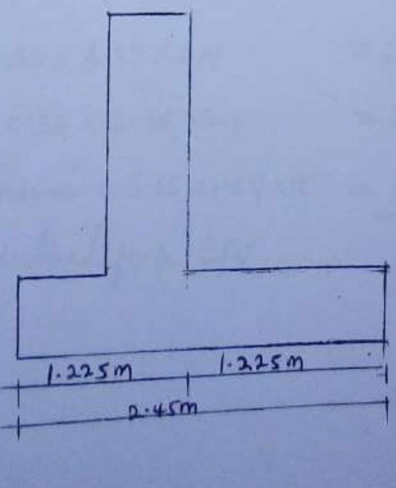


$H = 0.35 + 2.8 + 0.35 = 3.5m$

$B = 0.7H = 0.7 \times 3.5 = 2.45m$

The projection $= 0.16H = 0.16 \times 3.15 = 0.56m$

heel projection $= 2.45 - 0.56 - 0.35 = 1.54m$



DESIGNED BY: AKPA NWAMDI O REGISTRATION NO: 20162240005

DATE: JANUARY, 2022 SHEET NO: 02 of 07

MEMBER REF	CALCULATIONS	OUTPUT
	Active pressure	
	$P_a = K_a \gamma Z$	
T7 Reynolds and steedman	$\phi = 30°$	
	$\gamma = 18 KN/m^3$	
	$Z = 3.5m$	
	$K_a = \frac{1-\sin\phi}{1+\sin\phi}$	
	$= \frac{1-\sin 30°}{1+\sin 30°}$	
	$= 0.333$	
	$P_a = 0.333 \times 18 \times 3.5$	
	$= 20.979 KN/m^2$	
	Earth pressure $P_A = 20.979 \times 3.5 \times \frac{1}{2}$	
	$= 36.713 KN \cdot \Sigma f_h$	$\Sigma f_h = 36.713 KN$
	Take Surcharge to be Zero	
	Vertical forces, f_v = These include forces due to	
T.2 Reynolds and steedman	Wall $= 0.35 \times 3.15 \times 24$ $= 26.46 KN$	
	Base $= 0.35 \times 2.45 \times 24$ $= 20.58 KN$	
	Earth pressure $= 3.15 \times 1.54 \times 18$ $= 87.318 KN$	
	Total vertical force, Σf_v $134.358 KN$	$\Sigma f_v = 134.358 KN$

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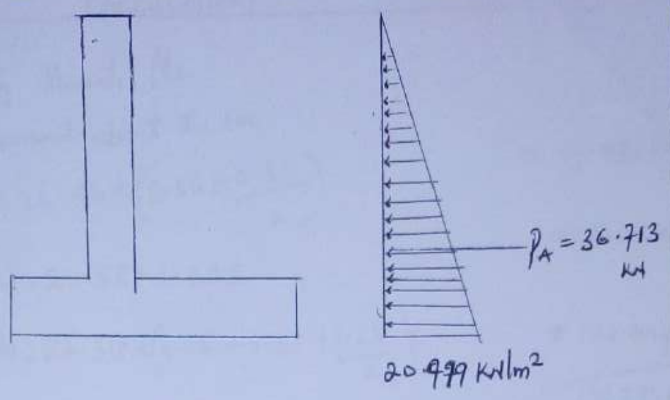
DESIGNED BY: AKPA NNAMDI .O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO: 03 OF 07

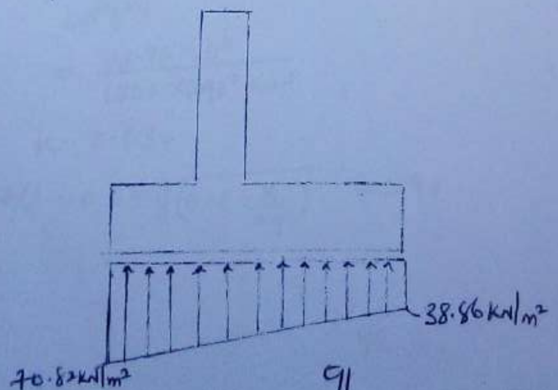
REF	CALCULATIONS	OUTPUT
		
	Checks for stability	
	i) Check for Sliding: Coefficient for friction, $\mu = 0.5$ for sliding, factor of safety $f_s = \frac{\text{Resisting forces (vertical)}}{\text{Sliding forces (horizontal)}} \geq 1.6$	
	Thus $f_s = \frac{\mu \Sigma f_v}{\Sigma f_h}$ $\frac{0.5 \times 134.358}{36.713}$ $f_s = 1.83 \geq 1.6$ .... OK	Sliding OK
	ii) Check for overturning Overturning Moment, M_o Moment due to earth pressure $= 36.713 \times \frac{3.5}{3}$ $= 42.83$ kNm	$M_o = 42.83$

MEMBER REF	CALCULATIONS	OUTPUT
	Resisting Moments, M_R	
	Take moment about the toe	
	Wall: $26.46 \times \left(0.56 + \frac{0.35}{2}\right)$ $= 19.448$ kNm	
	Base: 20.58×1.225 $= 25.211$ kNm	
	Earth: $87.318 \times \left(0.56 + 0.35 + \frac{1.54}{2}\right)$ $= 146.494$ kNm	
	Total 191.393 kNm	
	factor of safety for overturning, fo	
	$f_o = \frac{\text{Resisting moment}, M_R}{\text{Overturning Moment}, M_O} \geq 2.0$	
	$= \frac{191.393}{42.83}$	
	$f_o = 4.47 \geq 2$ ... OK	overturning OK
	iii) check for bearing capacity	
	$M_{net} = \Sigma M_R - \Sigma M_o$	
	$= 191.393 - 42.83$	
	$= 148.563$ kNm	
	Lever arm $\bar{x} = \frac{M_{net}}{\Sigma fv}$	
	$= \frac{148.563}{134.358}$	
	$= 1.106$ m	
	1.106m is within middle third of base	
	$0.82\text{m} < 1.106\text{m} < 1.63\text{m}$ ... OK	$\bar{x} = 1.106$

DESIGNED BY: AKPA NNAMDI O. ' REGISTRATION NO: 20162240

DATE: JANUARY, 2022 SHEET NO 05 OF 07

MEMBER REF	CALCULATIONS	OUTPUT
	Eccentricity, $e = B/2 - \bar{x}$	
	$= \frac{2.45}{2} - 1.106$	
	$= 0.119m$	
	$B/6 = \frac{2.45}{6} = 0.408\ m$	
	$0.119m < 0.408m$	
	Earth bearing pressure	
	$q = \frac{\Sigma fv}{B}\left(1 \pm \frac{6e}{B}\right)$	
	$q_{toe} = q_{max} = \frac{\Sigma fv}{B}\left(1 + \frac{6e}{B}\right)$	
	$= \frac{134.358}{2.45}\left(1 + \frac{6 \times 0.119}{2.45}\right)$	
	$q_{max} = 70.82 kN/m^2$	
	$q_{heel} - q_{min} = \frac{\Sigma fv}{B}\left(1 - \frac{6e}{B}\right)$	
	$= \frac{134.358}{2.45}\left(1 - \frac{6 \times 0.119}{2.45}\right)$	$q_{min} = 38.9$ kN/m
	$= 38.86 kN/m^2$	
	$70.82 kN/m^2 < 180 kN/m^2$	bearing pressure is adequate



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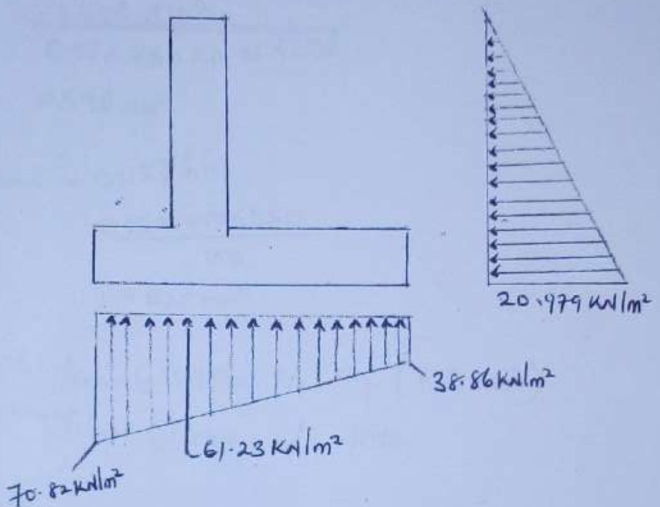
DESIGNED BY: AKPA NNAMDI O.   REGISTRATION NO: 2016224005

DATE: JANUARY, 2022   SHEET NO 06 of 07

MEMBER REF	CALCULATIONS	OUTPUT

5.6 DESIGN OF WALL



Bending Reinforcement

Wall

Horizontal moment $= 36.713\left(0.175 + \frac{3.15}{3}\right)$

$= 44.97\ kNm$

Horizontal moment (U.L.S) $= 44.97 \times 1.6$

$= 71.96\ kNm$

Horizontal moment = 71.9 kNm (U.L.S)

$h = 350mm$

$d = 350 - 50 - 10 = 296mm$

$K = \frac{M}{bd^2 f_{cu}}$

$= \frac{71.96 \times 10^6}{1000 \times 290^2 \times 25}$

$K = 0.034$

$Z/d = 0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)} = 0.95$

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MEMBER REF	CALCULATIONS	OUTPUT
	$A_s = \frac{M}{0.95 f_y Z}$	
	$= \frac{71.96 \times 10^6}{0.95 \times 460 \times 0.95 \times 290}$	
	$= 598\ mm^2$	
BS V-Oyenuga pg 315	$A_{s\,min} = 0.15\% bh$	
	$\frac{0.15 \times 1000 \times 350}{100}$	
	$= 525\ mm^2$	
T10.3 RCD Oyenuga	provide Y16mm @ 275mm c/c N.F ($731 mm^2$)	Y16/275 c/c N.F
	Provide Y12mm @ 150mm c/c distr.	Y12/150 c/c distr.
	Base	
	Designed using pressure at Ultimate limit state.	
	Top Reinforcement	
	Heel:	
	Taking moment about the stem Centreline for vertical Loads and the bearing pressures.	
	$M = 20.58 \times \frac{1.715}{2.45} \times \frac{1.715}{2} \times 1.0 + 87.318 \times \left(\frac{1.54}{2} + 0.175\right) 1.0 - 38.86 \times \frac{1.715^2}{2} - \left(\frac{61.23 - 38.86}{2}\right) \times 1.715 \times \frac{1.715}{3}$	
	$M = 26.76\ kNm$	
	$h = 350mm,\ d = 290\ mm$	
	$K = \frac{26.76 \times 10^6}{1000 \times 290^2 \times 25}$	
	$= 0.013$	

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REF	CALCULATIONS	OUTPUT
	$Z/d = 0.5 + \sqrt{(0.25 - \frac{0.013}{0.9})}$	
	$= 0.95$	
	$A_s = \frac{26.76 \times 10^6}{0.95 \times 460 \times 0.95 \times 290}$	
	$= 222\ mm^2$	
	provide nominal reinforcement $= 0.15\% bh$	
	$= 525\ mm^2$	
10.3 RCD Oyenuga	provide Y12mm bars @ 200mm c/c top (As prov $= 566\ mm^2$)	Y12/200 c/c top
	provide Y12 @150mm c/c distribution	Y12/150 c/c distr.
	Toe (bottom reinforcement)	
	$= -20.58 \times \frac{0.735}{2.45} \times \frac{0.735}{2} + 70.82 \times \frac{0.735^2}{2}$	
	$= 16.86\ KNm$	
	Moment is smaller than that for heel which required nominal reinforcement	
	Thus.	
	provide Y12mm bars @200mm c/c Bottom (As prov $= 566\ mm^2$)	Y12/200 c/c btm
		Y12/150 c/c distr

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MEMBER REF	CALCULATIONS	OUTPUT

5.7 DESIGN OF HEADWALL

Design information -

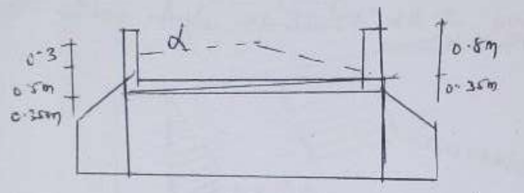
$\gamma = 18 kN/m^3$

$\phi = 30°$

depth of embankment = 2m

Embankment Slope = 1:2

Height of wall = 0.35 + 0.8 = 1.15m



Load Analysis

Coefficient of earth pressure

$$K_a = \cos\alpha \left(\frac{\cos\alpha - \sqrt{(\cos^2\alpha - \cos^2\phi)}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}} \right)$$

$\tan\alpha = 0.5$

$\alpha = \tan^{-1} 0.5$

$= 26.57°$

$$\therefore K_a = \cos 26.57° \left(\frac{\cos 26.57° - \sqrt{(\cos^2 26.57° - \cos^2 30°)}}{\cos 26.57° + \sqrt{(\cos^2 26.57° - \cos^2 30°)}} \right)$$

$= 0.537$

Earth pressure $P_a = K_a \gamma H$

$= 0.537 \times 18 \times 0.5 \times 1.6$

$= 7.73 kN/m^2$

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DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224065

DATE: JANUARY, 2022 SHEET NO: 03 of 04

MEMBER REF	CALCULATIONS	OUTPUT
	$M.F = 0.55 + \frac{(477 - fs)}{\left[120\left(0.9 + \frac{M}{bd^2}\right)\right]}$	
	$= 0.55 + \frac{(477 - 405)}{\left[120\left(0.9 + \frac{78.47 \times 10^6}{1000 \times 292^2}\right)\right]}$	
	$= 1.48 < 2$ . . . . ok	
	Since the slab is continous, the span effective depth ratio is 23	
	$d_{req} = \frac{span}{23 \times M.F}$	
	$d_{req} = \frac{3.15 \times 10^3}{23 \times 1.48}$	
	$= 92.54mm$	
	$d_{req} < d_{prov}$ ∴ deflection is satisfied	deflection OK.
	Check for shear	
	Max. shear $V = 271.51KN$	
	But the culvert is chamfered 550mm at the joints. Thus the maximum shear is reduced to.	
	$V = 271.51 - 0.55(161.43)$	
	$= 182.22KN$	
	Shear stress, $v = \frac{V}{bd}$	

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MEMBER REF	CALCULATIONS	OUTPUT
	$= \frac{182.22 \times 10^3 \ N}{1000 \times 292 \ mm^2}$	
	$= 0.62 \ N/mm^2$	
	Concrete shear stress Vc (permissible)	
	$V_c = 0.632 \left(\frac{100 A_s}{bd}\right)^{1/3} \left(\frac{400}{d}\right)^{1/4}$	
	$V_c = 0.632 \left(\frac{100 \times 2450}{1000 \times 292}\right)^{1/3} \left(\frac{400}{292}\right)^{1/4}$	
	$= 0.64 \ N/mm^2$	Shear OK.
	$V < V_c$ - . . . OK	

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DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016204605

DATE: JANUARY, 2022 SHEET NO: 01 OF 05

MEMBER REF	CALCULATIONS	OUTPUT

5.3 DESIGN OF SIDE WALLS

$M = 95.36$ kNm

$N = 237.49$ kN

$$\frac{N}{f_{cu}bh} = \frac{237.49 \times 10^3}{25 \times 1000 \times 350} = 0.03$$

$$\frac{M}{f_{cu}bh^2} = \frac{95.36 \times 10^6}{25 \times 1000 \times 350^2} = 0.03$$

Chart 10-2 RCD V. Oyenuga Page 356

$\alpha = 0.05$

$A_s = 0.0026 \alpha f_{cu} bh$
$= 0.0026 \times 0.05 \times 25 \times 1000 \times 350$
$= 1137.5\,mm^2$

$A_{s\,min} = 0.4\% bh = 1400\,mm^2$

T-10-3 RCD V. Oyenuga Pg 344

provide Y20 mm @ 200mm c/c both faces
($A_{s\,prov} = 1570\,mm^2$)

provide Y12mm @ 150mm c/c distribution

Output: Y20/200 c/c both faces with Y12/150 c/c distribution

Check for deflection

$f_s = 2/3 f_y \frac{A_{s\,req}}{A_{s\,prov}} = 273.46$ (As before)

$$M.F = 0.55 + \frac{(477 - f_s)}{\left[120\left(0.9 + \frac{M}{bd^2}\right)\right]} \leq 2.0$$

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MEMBER REF	CALCULATIONS	OUTPUT
	$M.F = 0.55 + \frac{(477 - 273.46)}{\left[120\left(0.9 + \frac{95.36 \times 10^6}{(1000 \times 292^2)}\right)\right]}$	
	$= 1.39 < 2$ . . . OK	
	Since the slab is simply supported the span effective depth ratio is 20.	
	$d_{req} = \frac{span}{20 \times M.F}$	
	$= \frac{3.15 \times 10^3}{20 \times 1.39}$	
	$= 113.3\,mm$	
	$d_{prov} > d_{req}$	
	∴ deflection is satisfied	Deflection OK
	<u>check for Shear</u>	
	Maximum Shear $V = 131.7\,KN$	
	As a result of the 550mm chamfer,	
	This shear is reduced to	
	$131.7 - 0.55 \times 47.04 = 105.83\,KN$	
	Shear stress, $v = \frac{V}{bd}$	
	$= \frac{105.83 \times 10^3}{1000 \times 292}$	
	$= 0.362\,N/mm^2$	

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DESIGNED BY: AKPA NNAMDI O. | REGISTRATION No: 20162024005
DATE: JANUARY, 2022 | SHEET NO: 03 OF 05

MEMBER REF	CALCULATIONS	OUTPUT
	Concrete Shear Stress V_c:	
	$V_c = 0.632\left(\frac{100A_s}{bd}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}$	
	$= 0.632\left(\frac{100 \times 1570}{1000 \times 297}\right)^{1/3}\left(\frac{400}{297}\right)^{1/4}$	
	$V_c = 0.556\ N/mm^2$	
	$V < V_c$	
	∴ Shear is satisfied	Shear OK.

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DESIGNED BY: AKPA NNAMDI O. | REGISTRATION NO: 2016224005
DATE: JANUARY, 2022 | SHEET NO 01 OF 06

MEMBER REF	CALCULATIONS	OUTPUT
	5.4 DESIGN OF INTERNAL WALLS	
	$M = 11.77 \, kNm$	
	$N = 271.01 \, kN$	
	$\frac{N}{f_{cu}bh} = \frac{271.01 \times 10^3}{25 \times 1000 \times 350} = 0.03$	
	$\frac{M}{f_{cu}bh^2} = \frac{11.77 \times 10^6}{25 \times 1000 \times 350^2} = 0.004$	
Chart 10.2 RCD V. Oyenuga Pg 354	$\alpha = 0 \quad \therefore A_s = 0$ Provide nominal reinforcement $A_{c\,min} = 0.4\% bh = 1400 mm^2$	
T 10.3 RCD V. Oyenuga Pg 344	provide Y20 mm @ 200mm c/c both faces ($A_{s\,prov} = 1570 \, mm^2$) provide Y12 @ 150mm c/c distribution	Y20/200 c/c both faces width Y12/150 c/c distrb
	Check for deflection $f_s = 273.46$ (as before)	
Pg 347 RCD V. Oyenuga	$M.F = 0.55 + \frac{477 - f_s}{\left[120\left(0.9 + \frac{M}{bd^2}\right)\right]}$ $0.55 + \frac{477 - 273.46}{\left[120\left(0.9 + \frac{11.77 \times 10^6}{1000 \times 350^2}\right)\right]}$	
	$M.F = 2.18$ use 2	$M.F = 2$

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DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO: 02 OF 06

MEMBER REF	CALCULATIONS	OUTPUT
	$d_{req} = \frac{Span}{20 \times m_f}$	
	$= \frac{3.15 \times 10^3}{20 \times 2}$	
	$= 78.75\ mm$	
	$d_{req} < d_{prov}$	
	∴ deflection is satisfied	Deflection ok
	Shear is obviously satisfied	Shear ok.

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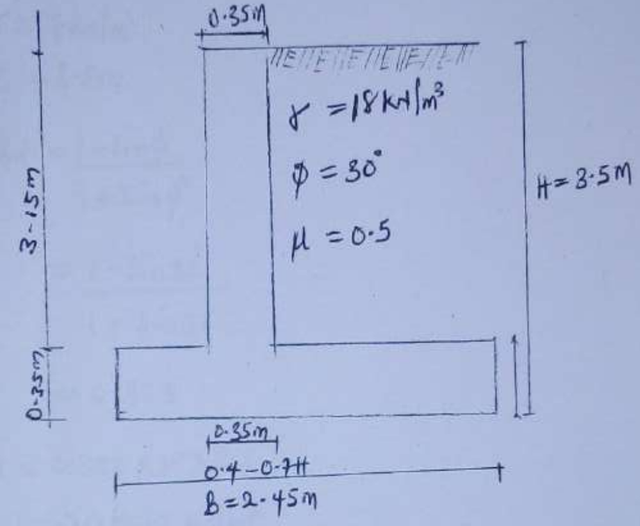
DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO: 01 OF 04

MEMBER REF	CALCULATIONS	OUTPUT

5.5 DESIGN OF WING WALLS

Proportioning

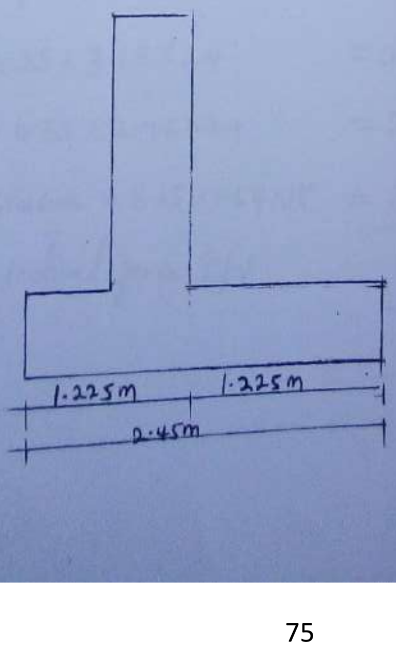


$H = 0.35 + 2.8 + 0.35 = 3.5m$

$B = 0.7H = 0.7 \times 3.5 = 2.45m$

The projection $= 0.16H = 0.16 \times 3.15 = 0.56m$

heel projection $= 2.45 - 0.56 - 0.35 = 1.54m$



DESIGNED BY: AKPA NWAMDI O   REGISTRATION NO: 20162240005

DATE: JANUARY, 2022   SHEET NO: 02 of 07

MEMBER REF	CALCULATIONS	OUTPUT
	Active pressure	
	$P_a = K_a \gamma Z$	
T7 Reynolds and steedman	$\phi = 30°$	
	$\gamma = 18 KN/m^3$	
	$Z = 3.5m$	
	$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$	
	$= \frac{1 - \sin 30°}{1 + \sin 30°}$	
	$= 0.333$	
	$P_a = 0.333 \times 18 \times 3.5$	
	$= 20.979 KN/m^2$	
	Earth pressure $P_A = 20.979 \times 3.5 \times \frac{1}{2}$	
	$= 36.713 KN \cdot Ef_h$	$Ef_h = 36.713$ KN
	Take Surcharge to be zero	
	Vertical forces, fv = These include forces due to	
T-2 Reynolds and steedman	Wall $= 0.35 \times 3.15 \times 24 = 26.46 KN$	
	Base $= 0.35 \times 2.45 \times 24 = 20.58 KN$	
	Earth pressure $= 3.15 \times 1.54 \times 18 = 87.318 KN$	
	Total vertical force, Efv 134.358 KN	$Efv = 134.358$ KN

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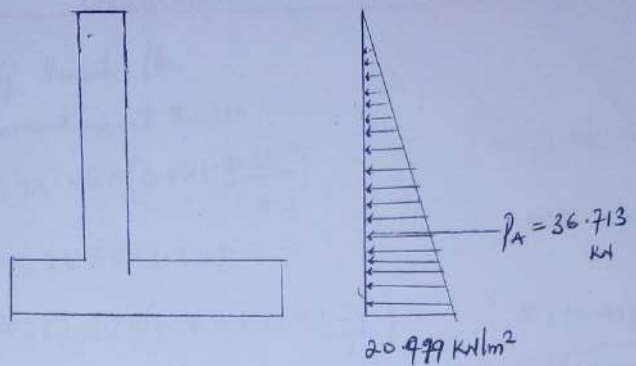
DESIGNED BY: AKPA NNAMDI .O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO: 03 OF 07

MEMBER REF	CALCULATIONS	OUTPUT



<u>Checks for stability</u>

i) <u>Check for Sliding:</u>

Coefficient for friction, $\mu = 0.5$

for Sliding, factor of safety

$$f_s = \frac{\text{Resisting forces (vertical)}}{\text{Sliding forces (horizontal)}} \geq 1.6$$

Thus $f_s = \frac{\mu \Sigma f_v}{\Sigma f_h}$

$$\frac{0.5 \times 134.358}{36.713}$$

$f_s = 1.83 \geq 1.6$ ... ok — Sliding OK

ii) <u>Check for overturning</u>

Overturning Moment, M_o

Moment due to earth pressure $= 36.713 \times \frac{3.5}{3}$

$= 42.83$ kNm — $M_o = 42.83$ kNm

MEMBER REF	CALCULATIONS	OUTPUT
	Resisting Moments, M_R	
	Take moment about the toe	
	Wall : $26.46 \times \left(0.56 + \frac{0.35}{2}\right)$	$= 19.448$ KNm
	Base : 20.58×1.225	$= 25.211$ KNm
	Earth : $87.318 \times \left(0.56 + 0.35 + \frac{1.54}{2}\right)$	$= 146.494$ KNm
	Total	191.393 KNm
	factor of safety for overturning, fo	
	$fs = \frac{\text{Resisting moment, } M_R}{\text{Overturning Moment, } M_O} \geq 2.0$	
	$= \frac{191.393}{42.83}$	
	$fo = 4.47 \geq 2$ ... OK	overturning OK
	iii) Check for bearing capacity	
	$M_{net} = \Sigma M_R - \Sigma M_O$	
	$= 191.393 - 42.83$	
	$= 148.563$ KNm	
	Lever arm $\bar{x} = \frac{M_{net}}{\Sigma fv}$	
	$= \frac{148.563}{134.358}$	
	$= 1.106$ m	
	1.106m is within middle third of base $0.82m < 1.106m < 1.63m$ ... OK	$\bar{x} = 1.106$

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DESIGNED BY: AKPA NNAMDI O.   REGISTRATION NO: 20162240

DATE: JANUARY, 2022   SHEET NO 05 OF 07

MEMBER REF	CALCULATIONS	OUTPUT

Eccentricity, $e = B/2 - \bar{x}$

$$= \frac{2.45}{2} - 1.106$$

$$= 0.119m$$

$$b/6 = \frac{2.45}{6} = 0.408\ m$$

$$0.119m < 0.408m$$

Earth bearing pressure

$$q = \frac{\Sigma fv}{B}\left(1 \pm \frac{6e}{B}\right)$$

$$q_{toe} = q_{max} = \frac{\Sigma fv}{B}\left(1 + \frac{6e}{B}\right)$$

$$= \frac{134.358}{2.45}\left(1 + \frac{6 \times 0.119}{2.45}\right)$$

$$q_{max} = 70.82 kN/m^2$$

$$q_{heel} - q_{min} = \frac{\Sigma fv}{B}\left(1 - \frac{6e}{B}\right)$$

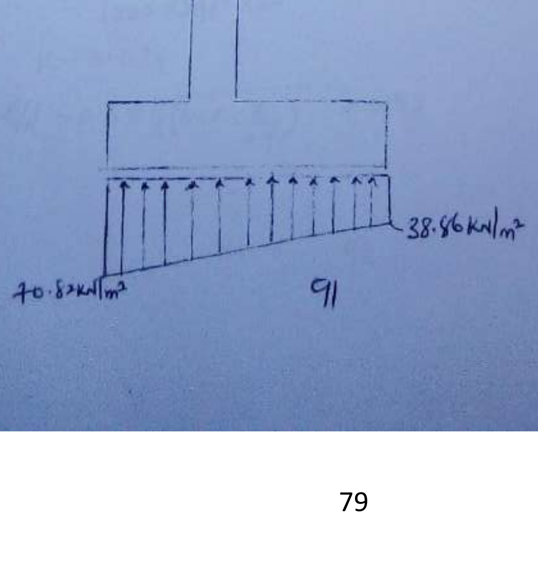
$$= \frac{134.358}{2.45}\left(1 - \frac{6 \times 0.119}{2.45}\right)$$

$$= 38.86 kN/m^2$$

$q_{min} = 38.86$ kN/m

$$70.82 kN/m^2 < 180 kN/m^2$$

bearing pressure is adequate



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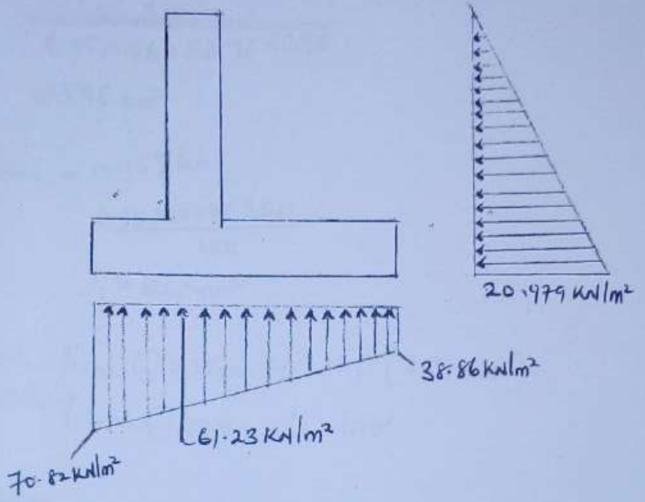
MEMBER REF	CALCULATIONS	OUTPUT

5.6 DESIGN OF WALL



Bending Reinforcement

Wall

Horizontal moment $= 36.713\left(0.175 + \frac{3.15}{3}\right)$

$= 44.97\ kNm$

Horizontal moment (U.L.S) $= 44.97 \times 1.6$
$= 71.96\ kNm$

Horizontal moment = 71.9 KNm (U.L.S)

$h = 350mm$

$d = 350 - 50 - 10 = 290mm$

$K = \frac{M}{bd^2 f_{cu}}$

$= \frac{71.96 \times 10^6}{1000 \times 290^2 \times 25}$

$K = 0.034$

$Z/d = 0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)} = 0.95$

92

DESIGNED BY: AKPA NNAMDI O. REGISTRATION NO: 2016224005

DATE: JANUARY, 2022 SHEET NO 28 of 07

MEMBER REF	CALCULATIONS	OUTPUT
	$A_s = \frac{M}{0.95 f_y z}$	
	$= \frac{71.96 \times 10^6}{0.95 \times 460 \times 0.95 \times 290}$	
	$= 598\ mm^2$	
RCD V-Oyen-uga pg 315	$A_{s\,min} = 0.15\% bh$	
	$\frac{0.15 \times 1000 \times 350}{100}$	
	$= 525\ mm^2$	
10.3 RCD Oyenuga	provide Y16mm @ 275mm c/c N.F (731mm²)	Y16/275 c/c N.F
	Provide Y12mm @ 150mm c/c distr.	Y12/150 c/c distr.
	Base	
	Designed using pressure at Ultimate limit state.	
	Top Reinforcement	
	Heel:	
	Taking moment about the stem centreline for vertical loads and the bearing pressures.	
	$M = 20.58 \times \frac{1.715}{2.45} \times \frac{1.715}{2} \times 1.0 + 87.318 \times \left(\frac{1.54}{2} + 0.175\right) 1.0 - 38.86 \times \frac{1.715^2}{2} - \left(\frac{61.23 - 38.86}{2}\right) \times 1.715 \times \frac{1.715}{3}$	
	$M = 26.76\ KNm$	
	$h = 350mm,\ d = 290\ mm$	
	$K = \frac{26.76 \times 10^6}{1000 \times 290^2 \times 25}$	
	$= 0.013$	

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MEMBER REF	CALCULATIONS	OUTPUT
	$Z/d = 0.5 + \sqrt{\left(0.25 - \frac{0.013}{0.9}\right)}$	
	$= 0.95$	
	$A_s = \frac{26.76 \times 10^6}{0.95 \times 460 \times 0.95 \times 290}$	
	$= 222\ mm^2$	
	provide nominal reinforcement $= 0.15\% bh$	
	$= 525\ mm^2$	
T.10.3 RCD V. Oyenuga	provide Y12mm bars @ 200mm c/c top (A_s prov $= 564\ mm^2$)	Y12/200 c/c top
	provide Y12 @150mm c/c distribution	Y12/150 c/c distr.
	Toe (bottom reinforcement)	
	$= -20.58 \times \frac{0.735}{2.45} \times \frac{0.735}{2} + 70.82 \times \frac{0.735^2}{2}$	
	$= 16.86 KNm$	
	Moment is smaller than that for heel which required nominal reinforcement Thus. provide Y12mm bars @200mm c/c Bottom (A_s prov $= 566\ mm^2$)	Y12/200 c btm Y12/150 c/c distr

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DESIGNED BY: AKPA NNAMDI O | REGISTRATION NO: 2016[illegible]
DATE: JANUARY, 2022 | SHEET NO [illegible] OF 08

MEMBER REF	CALCULATIONS	OUTPUT

5.7 DESIGN OF HEADWALL

Design information:

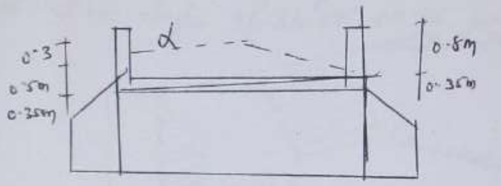
$\gamma = 18 kN/m^3$

$\phi = 30°$

depth of embankment = 2m

Embankment Slope = 1:2

Height of wall = 0.35 + 0.8 = 1.15m



Load Analysis

Coefficient of earth pressure

$$K_a = \cos\alpha \left(\frac{\cos\alpha - \sqrt{(\cos^2\alpha - \cos^2\phi)}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}} \right)$$

$\tan\alpha = 0.5$

$\alpha = \tan^{-1} 0.5$

$= 26.57°$

$$\therefore K_a = \cos 26.57° \left(\frac{\cos 26.57° - \sqrt{(\cos^2 26.57° - \cos^2 30°)}}{\cos 26.57° + \sqrt{(\cos^2 26.57° - \cos^2 30°)}} \right)$$

$= 0.537$

Earth pressure $P_a = K_a \gamma H$

$= 0.537 \times 18 \times 0.5 \times 1.6$

$= 7.73 kN/m^2$

95

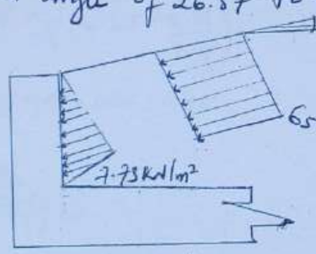
MEMBER REF	CALCULATIONS	OUTPUT

Surcharge pressure, p_s = (Load on top slab − selfweight of Top slab − weight of 0.5m of earth fill) K_a

Weight of 0.5m of earth fill = γz
$= 18 \times 0.5 \times 1.6$
$= 14.4 KN/m^2$

Thus surcharge pressure
$(149.99 - 11.76 - 14.4) 0.537$
$= 65.42 KN/m^2$

As a result of the slope of the embankment - the loads will act at an angle of 26.57° to the horizontal. Thus



Resolving the forces to get the horizontal component gives

$P_{ah} - P_a \cos\alpha = 7.73 \cos 26.57° = 6.91 KN/m^2$

$P_{sh} = P_s \cos\alpha = 65.42 \cos 26.57 = 58.14 KN/m^2$

Bending moment = Moment due to triangular load + moment due to surcharge pressure.

Moment due to triangular load =

$\frac{1}{2} \times 6.91 \times 0.5 \times \left(0.5 + \frac{0.35}{2}\right) = 1.17 KNm$

Moment due to surcharge pressure

$= 65.42 \times 0.5 \times \left(\frac{0.5}{2} + \frac{0.35}{2}\right)$

$= 14 KNm$

Total design bending moment $= (1.17 + 14) KNm$
$= 15.17 KNm$

Output: ultimate design moment 15.17KNm

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DESIGNED BY: AKPA NWAMDI REGISTRATION NO: 20K224005

SHEET NO 03 OF 08

DATE: JANUARY, 2022

MEMBER REF	CALCULATIONS	OUTPUT
	DESIGN	
	$M = 15.17 KNm$	
	Assume 12mm bar as main reinforcement	
	$d = 350 - 50 - \frac{12}{2} = 294 mm$	
	$K = \frac{M}{bd^2 f_{cu}}$	
	$= \frac{15.17 \times 10^6}{1000 \times 294^2 \times 25}$	
	$= 0.01$	
	$Z/d = 0.5 + \sqrt{\left(0.25 - \frac{0.01}{0.9}\right)}$	
	$= 0.99$ use 0.95	
	$As = \frac{M}{0.95 f_y Z}$	
	$= \frac{15.17 \times 10^6}{0.95 \times 460 \times 0.95 \times 294}$	
	$= 124 mm^2$	
	$As_{min} = 0.15\% bh = \frac{0.15}{100} \times 1000 \times 350$	
	$= 525 mm^2$	
T.10.3 RCD V. Oyenuga Pg 3.44	Provide Y_{12} @ 175 mm c/c both faces ($As_{prov} = 646 mm^2$) with Y_{12} @ 150mm c/c distribution	Y_{12}/175 c/c both faces with Y_{12}/150 c/c distr.

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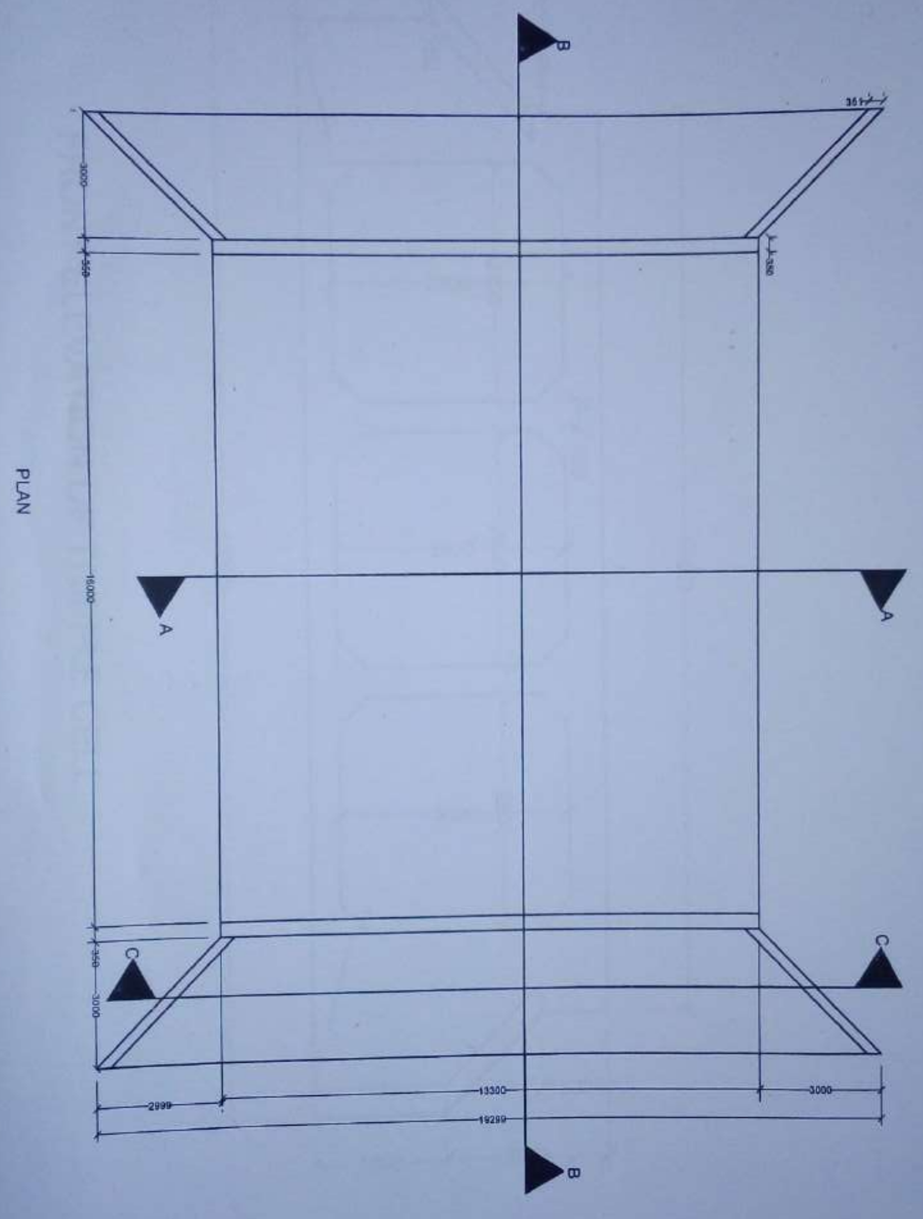
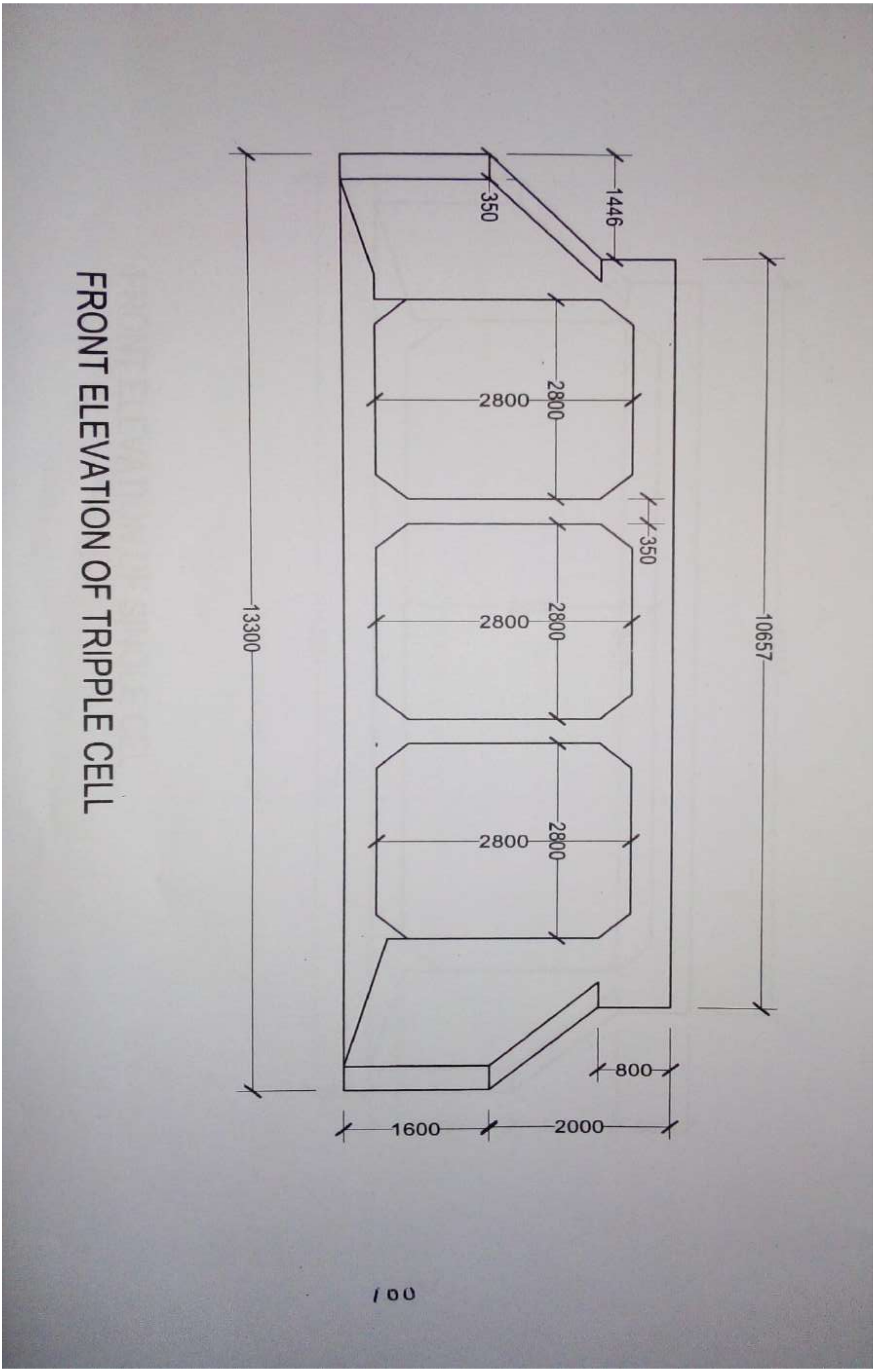
DESIGNED BY: AKPA NNAMDI O. REGISTRATION No: 2016224005
DATE: JANUARY, 2022 SHEET No 01 of 09

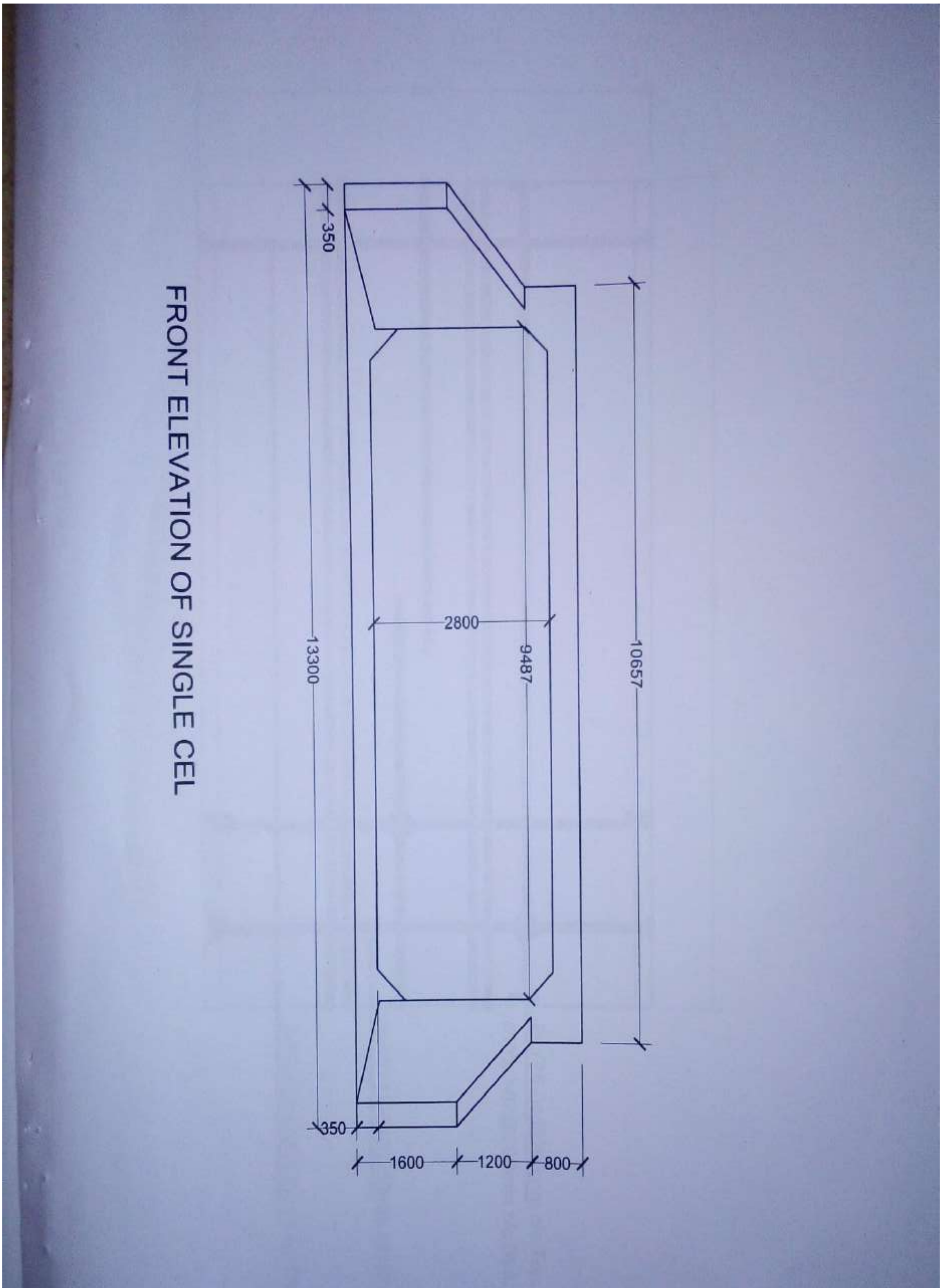
MEMBER REF	CALCULATIONS	OUTPUT
	5.8 DESIGN OF APRON	
	Apron are provided to avoid the occurrence of rip rap conditions at the entrance and exit of the culvert base. thus prevent the erosion of the culvert base. The apron terminates at its end with an apron beam which anchors it firmly to the ground. The apron is separated from the culvert frame by a construction joint.	
	DESIGN	
	Apron thickness, h = 200mm	
	Minimum area of reinforcement = 0.15%bh	
	Cover to reinforcement = 50mm	
	$d = h - c - \phi/2$	
	$d = 200 - 50 - \frac{12}{2} = 144mm$	
	$As_{min} = \frac{0.15bh}{100}$	
	$= \frac{0.15 \times 1050 \times 200}{100}$	
	$= 300 mm^2$	
T.10.3 RCD V. Oyenuga Pg 344	provide Y_{12} @ 275mm c/c Btm ($A_{prov} = 411mm^2$)	Y_{12}/275 c/c Btm with
	provide Y_{10} @ 275mm c/c distribution	Y_{10}/275 c/c distr.
	Apron beams	
	Let beam breath = 250mm	
	beam depth = 1000mm	
	$As_{min} = \frac{0.15bh}{1000} = \frac{0.15 \times 1000 \times 250}{1000}$	
	$= 375mm^2$	
T.10.3 RCD V. Oyenuga Pg 344	provide Y_{12} @ 275mm c/c N.F ($A_{s prov} = 411 mm^2$)	Y_{12}/275 c/c N-F with
	provide Y_{12} @ 200mm c/c distribution	Y_{12}/200 c/c distr.

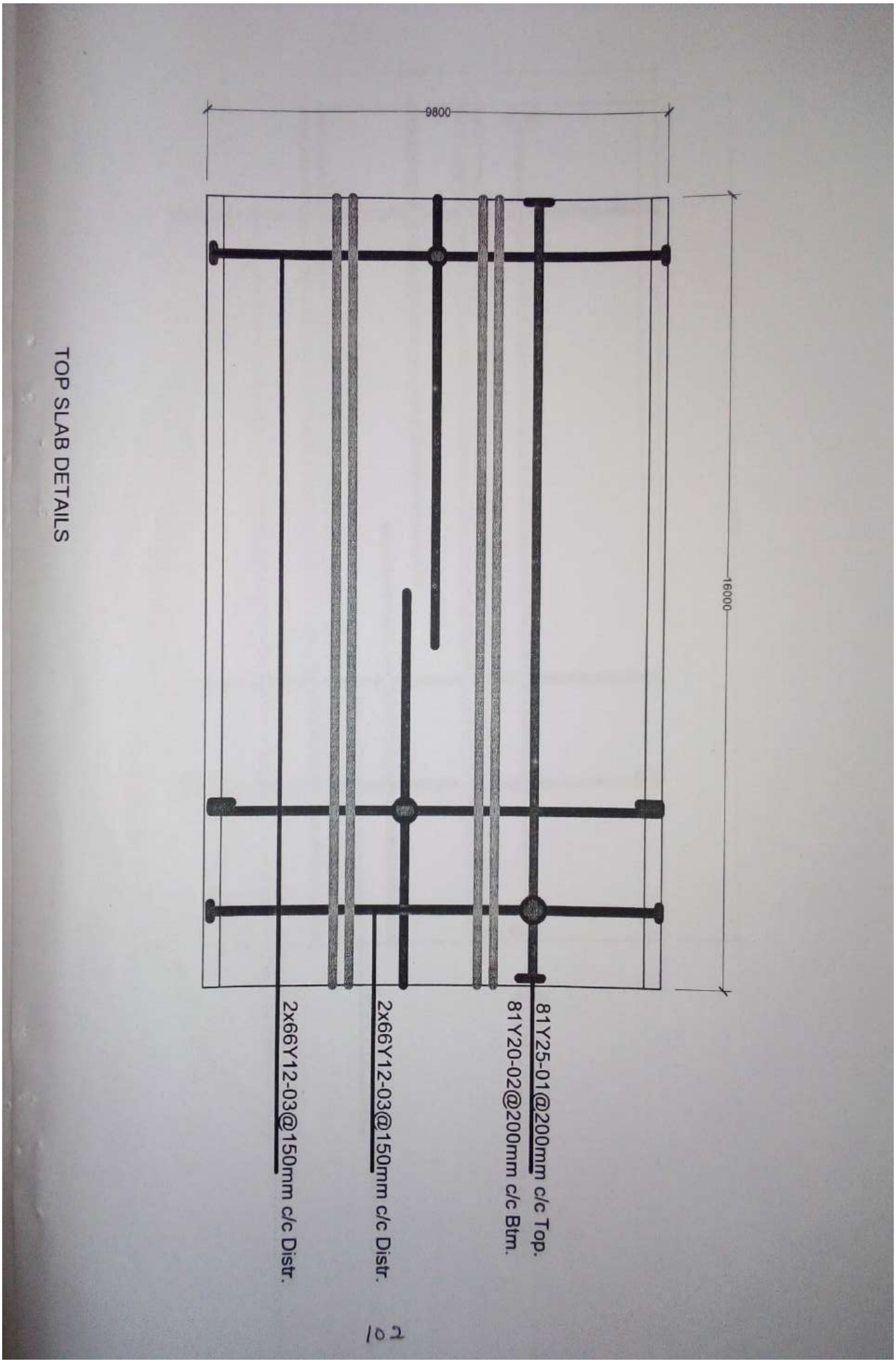
DETAILING

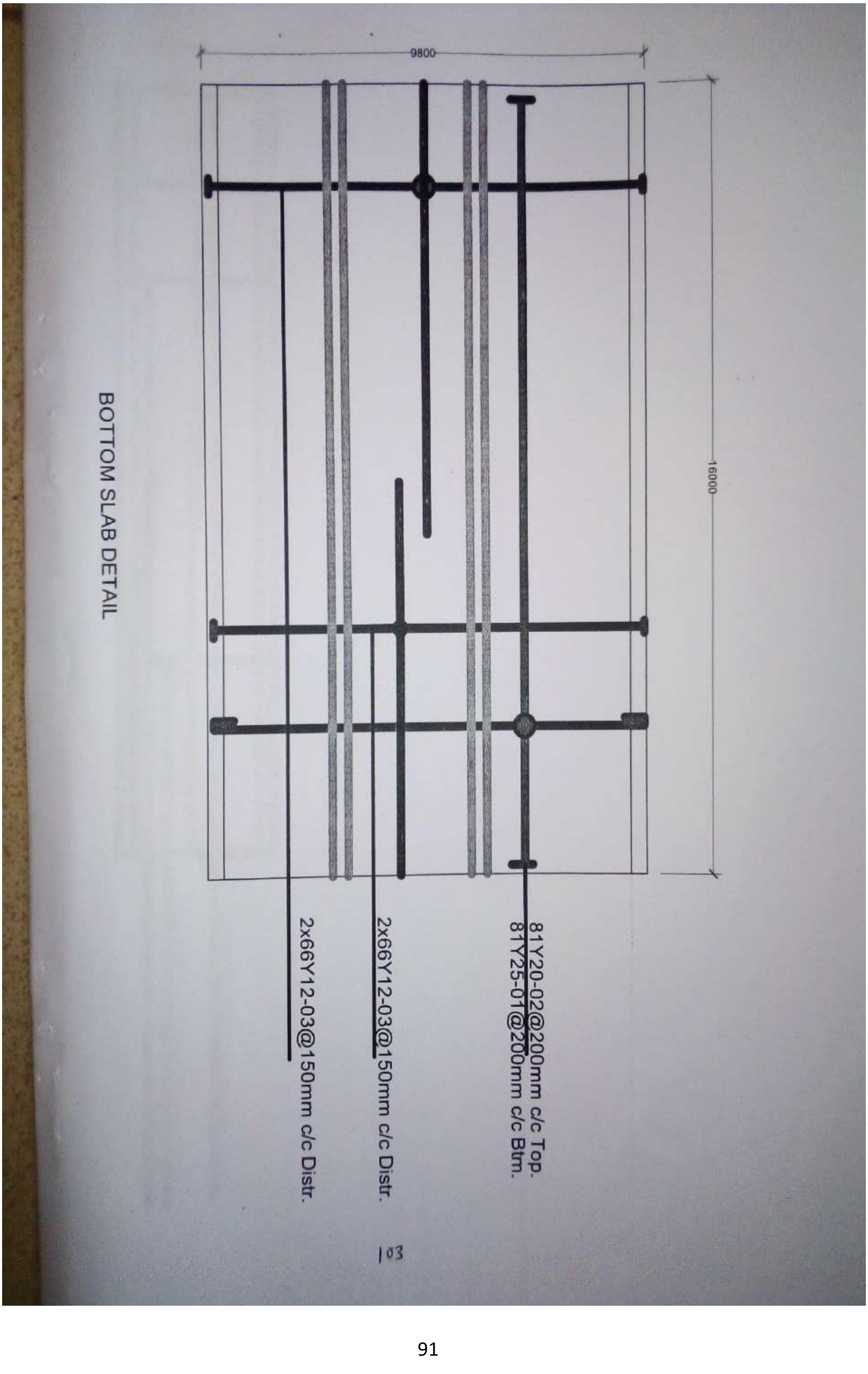


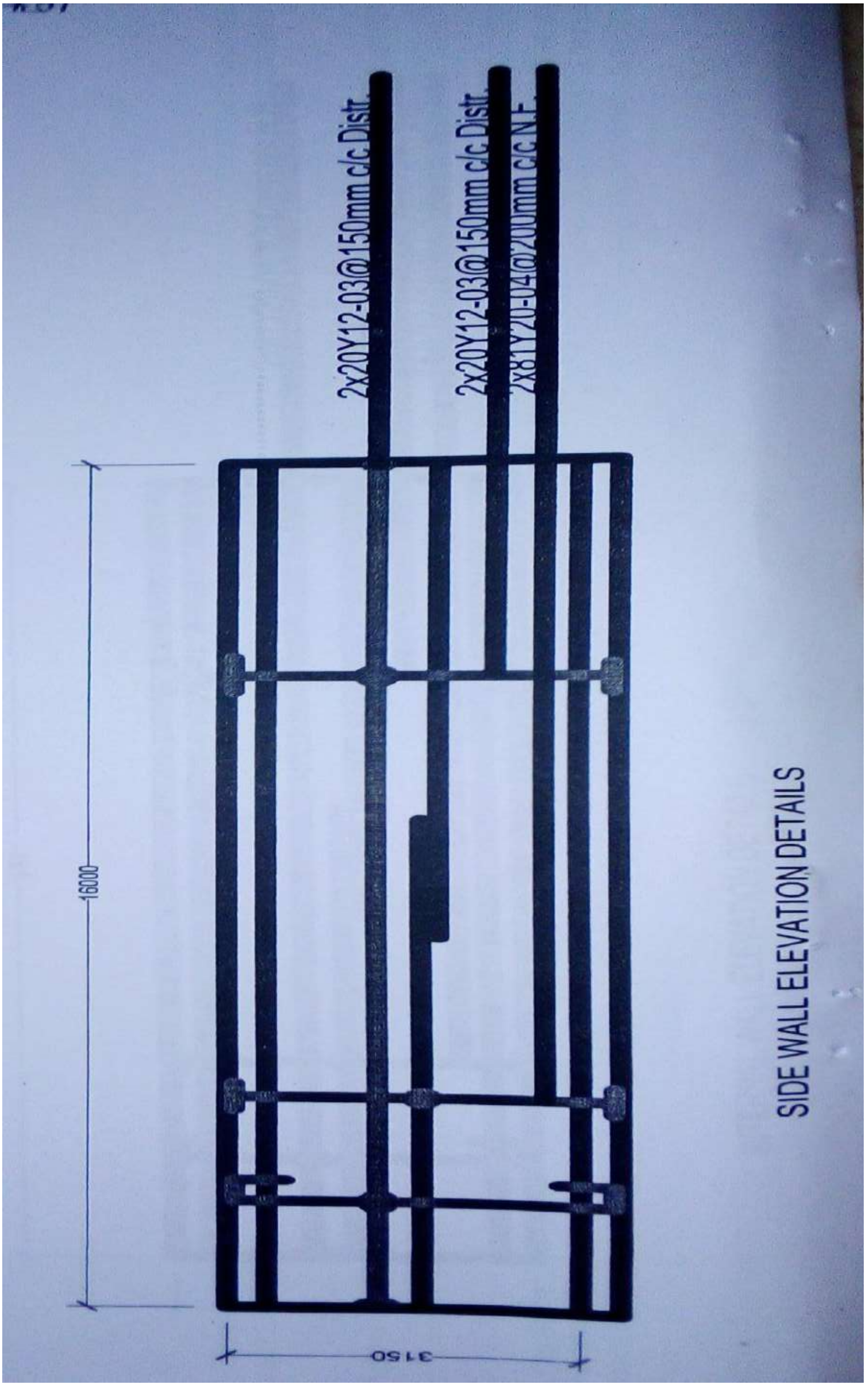
99

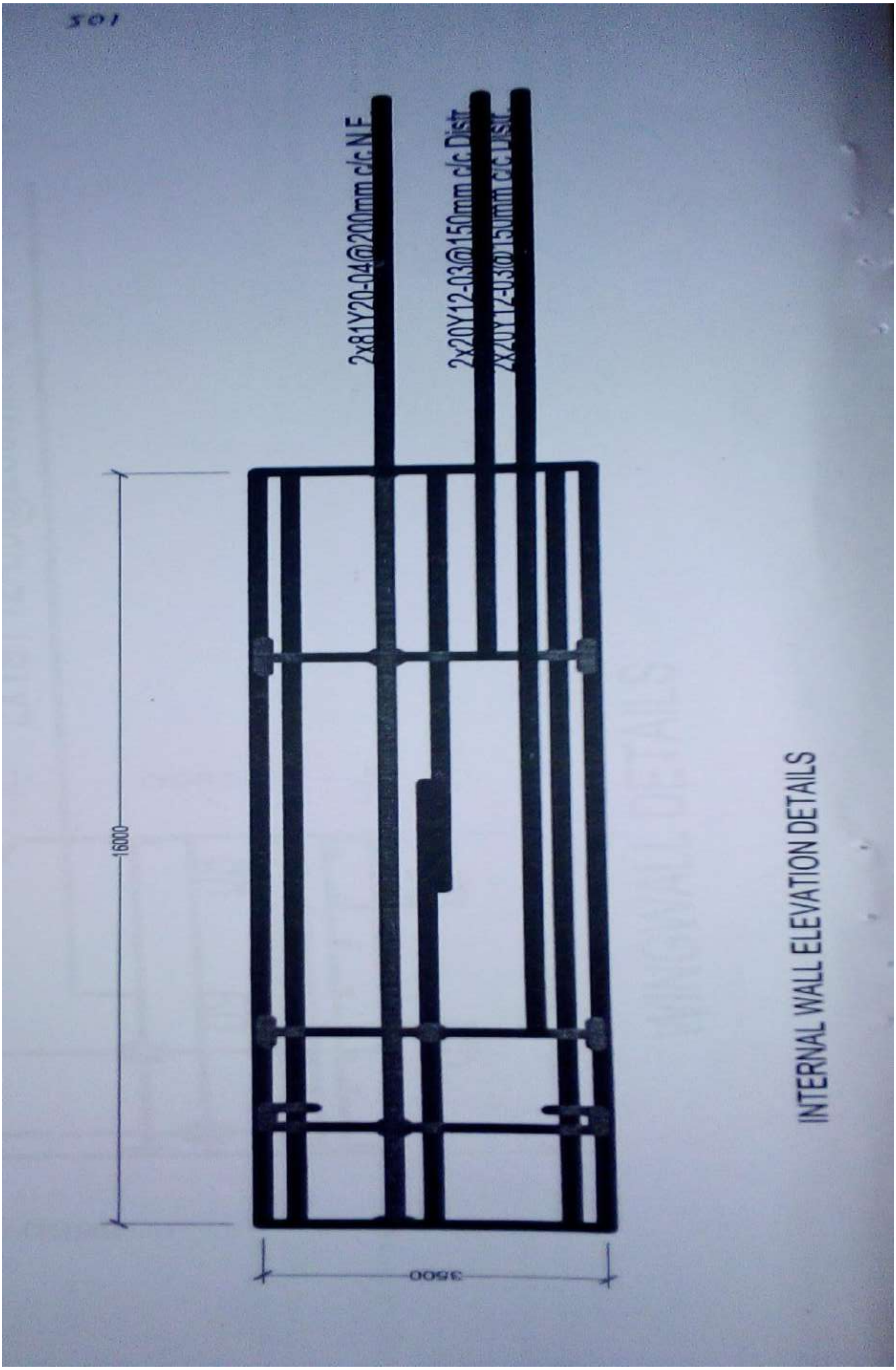

FRONT ELEVATION OF TRIPPLE CELL
13300
10657
1446
350
2800
2800
350
2800
2800
2800
2800
800
1600
2000

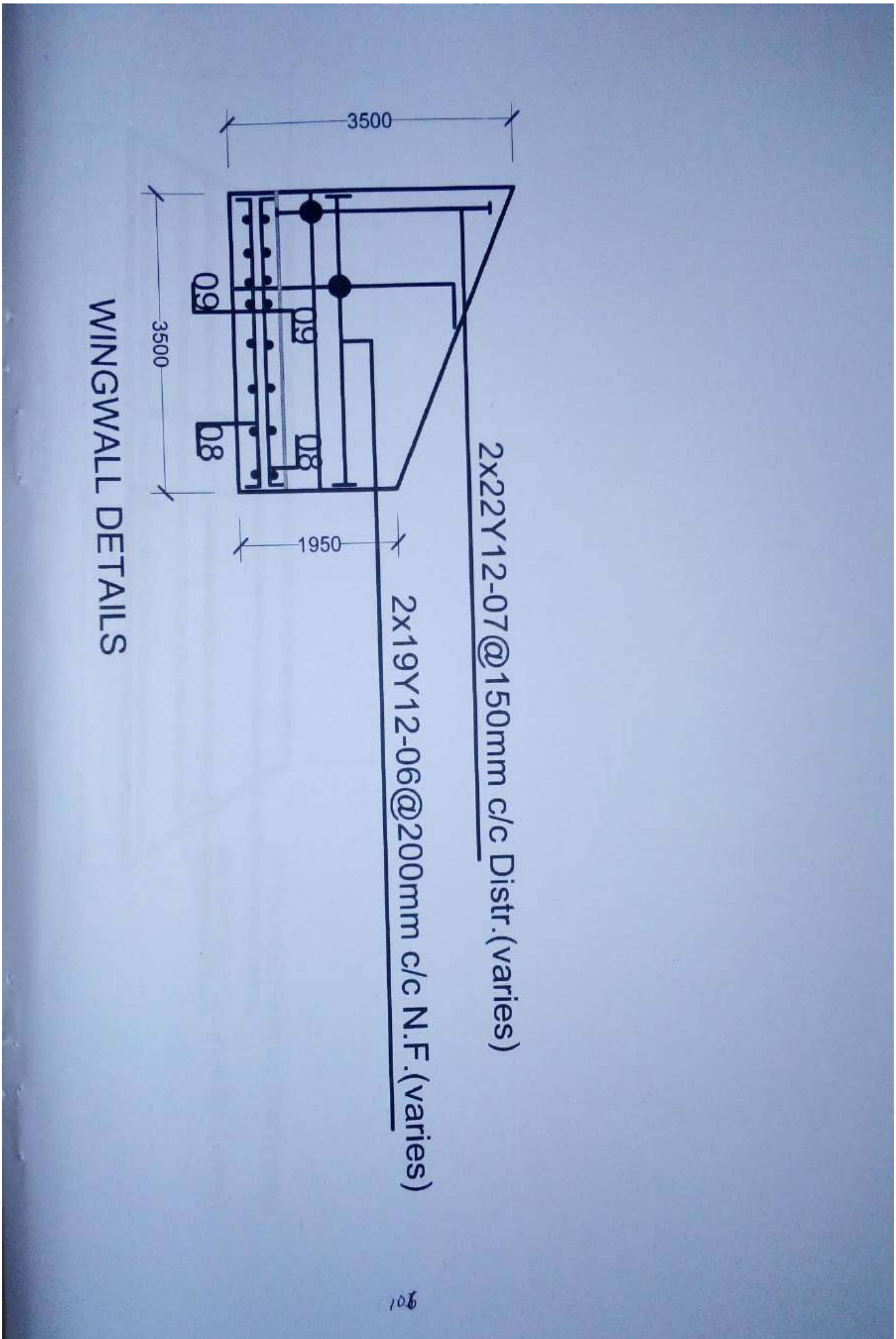

FRONT ELEVATION OF SINGLE CEL
350
13300
2800
9487
10657
350
1600
1200
800

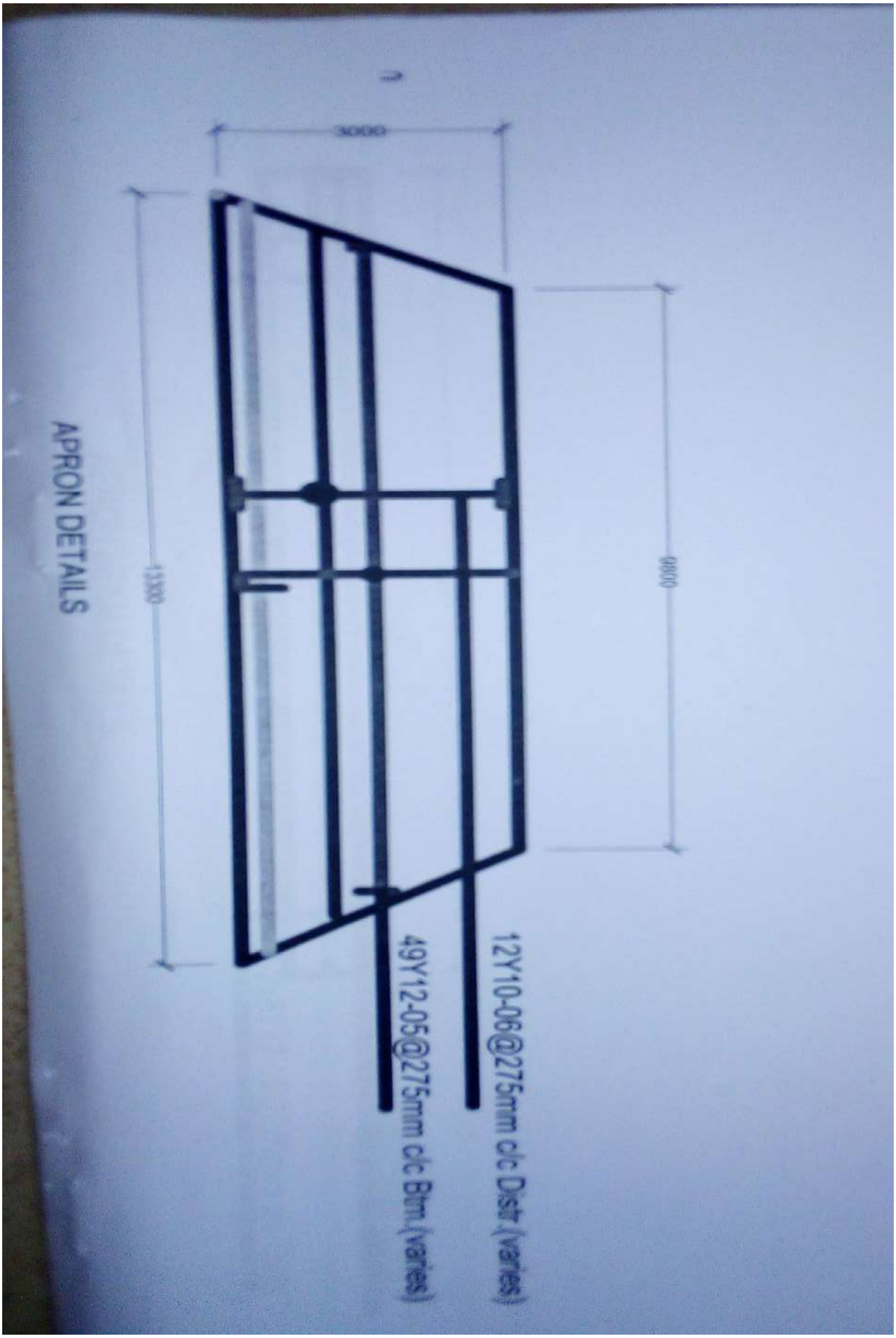

TOP SLAB DETAILS
9800
16000
81Y25-01@200mm c/c Top.
81Y20-02@200mm c/c Btm.
2x66Y12-03@150mm c/c Distr.
2x66Y12-03@150mm c/c Distr.
102

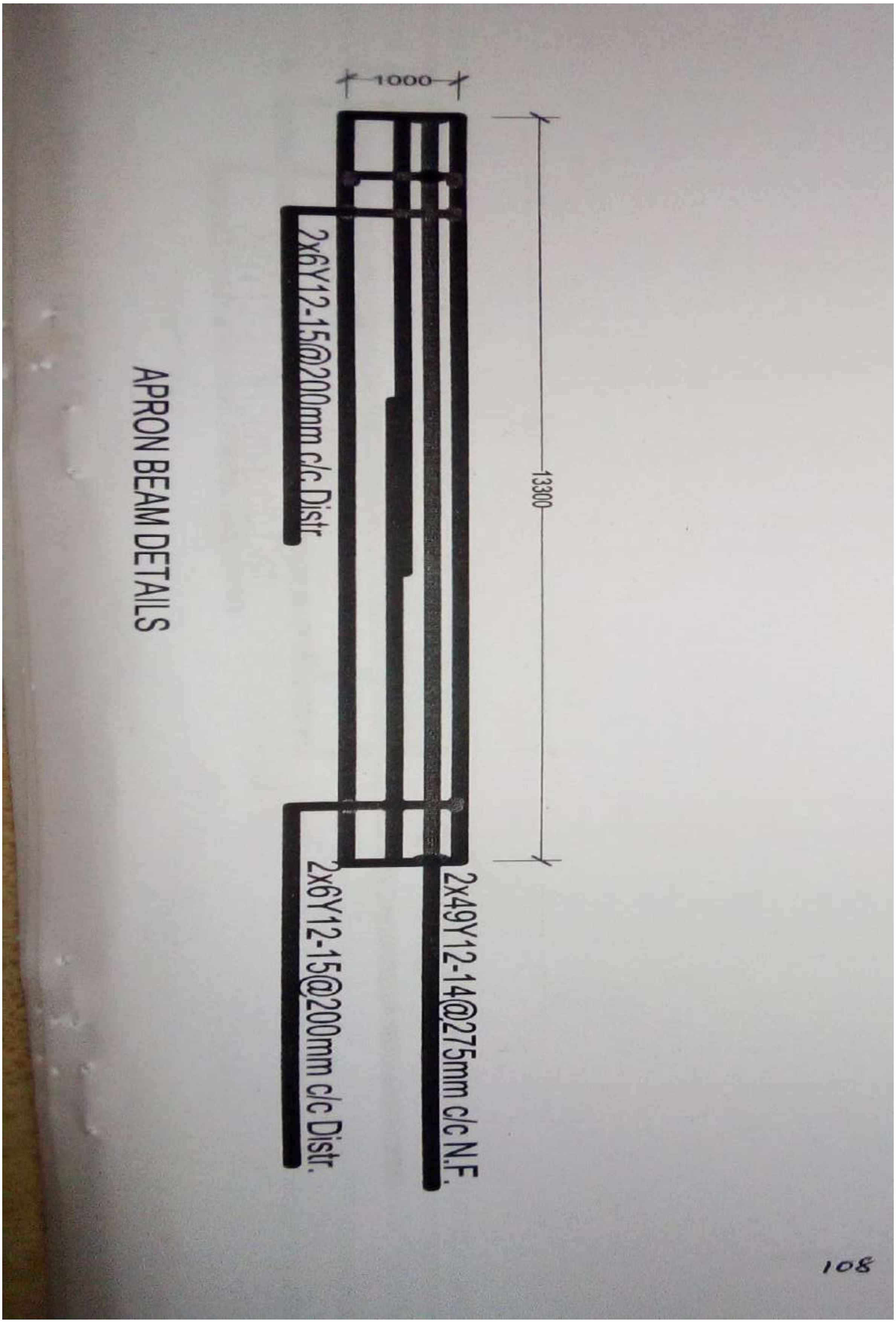

9800
16000
BOTTOM SLAB DETAIL
81Y20-02@200mm c/c Top.
81Y25-01@200mm c/c Btm.
2x66Y12-03@150mm c/c Distr.
2x66Y12-03@150mm c/c Distr.
103

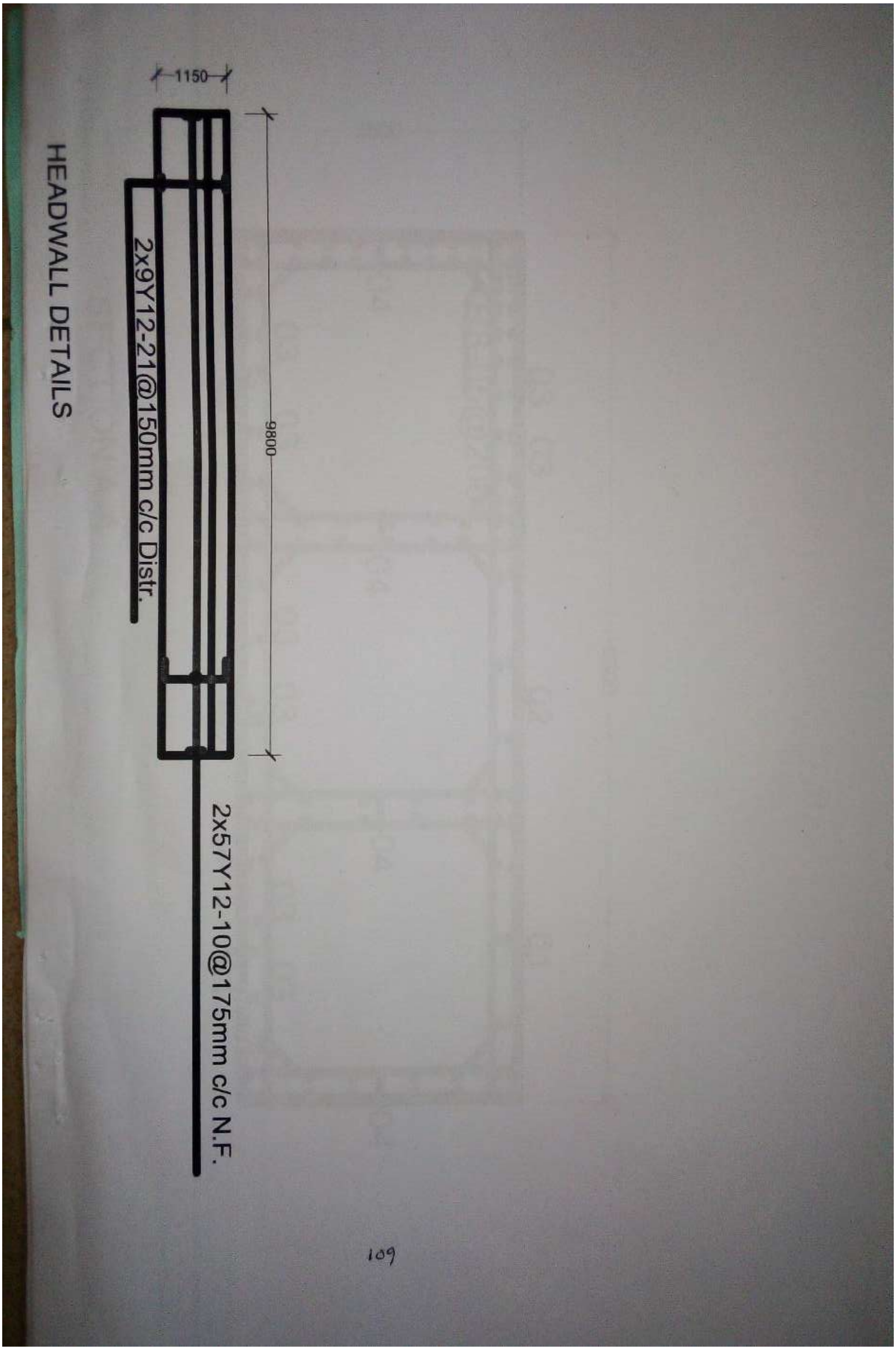

2x20Y12-03@150mm c/c Distr.
2x20Y12-03@150mm c/c Distr.
2x81Y20-04@200mm c/c N.F.
16000
3150
SIDE WALL ELEVATION DETAILS

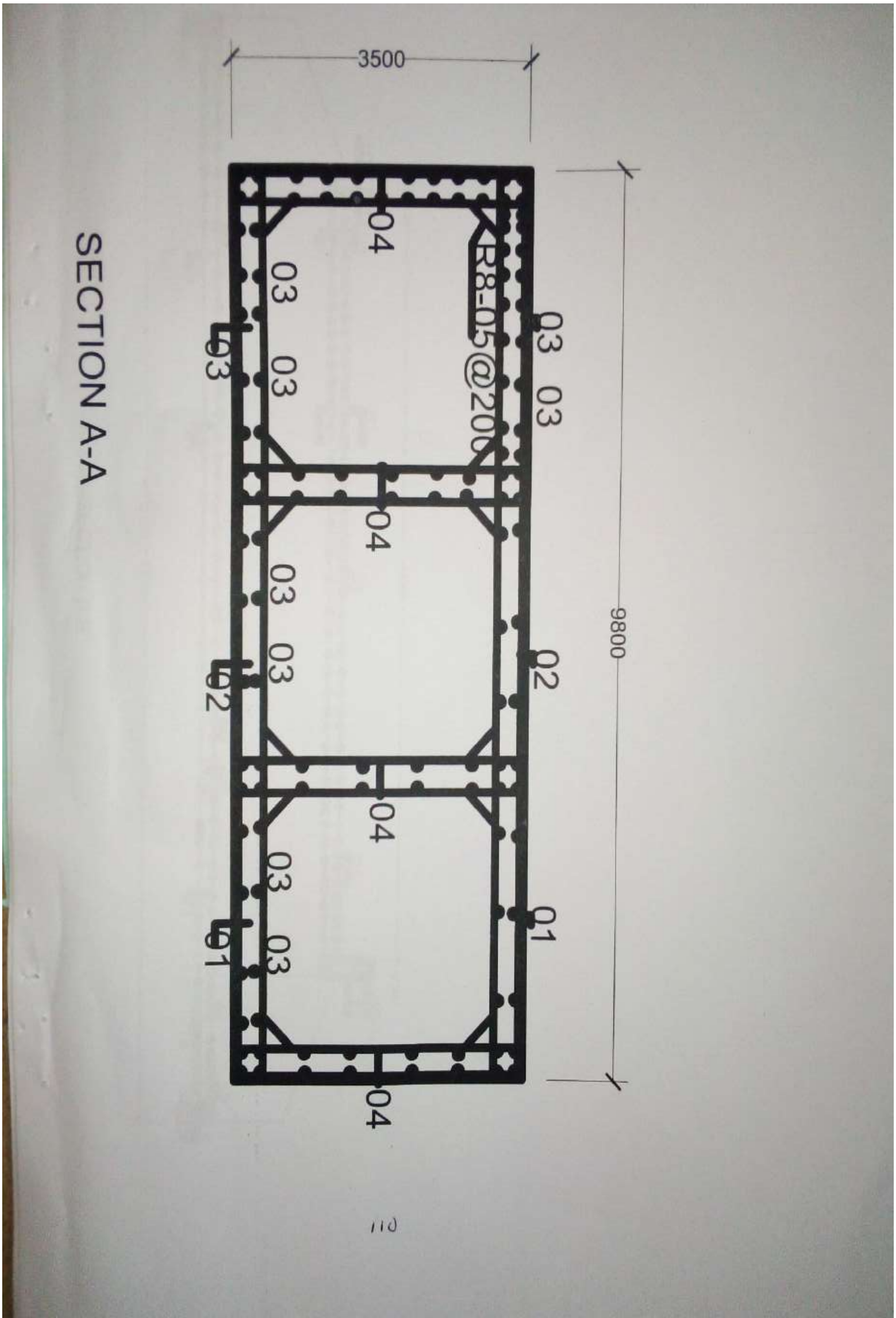

105
2x81Y20-04@200mm c/c N.F
2x20Y12-03@150mm c/c Dist
2x40Y12-03@150mm c/c Dist.
16000
3500
INTERNAL WALL ELEVATION DETAILS

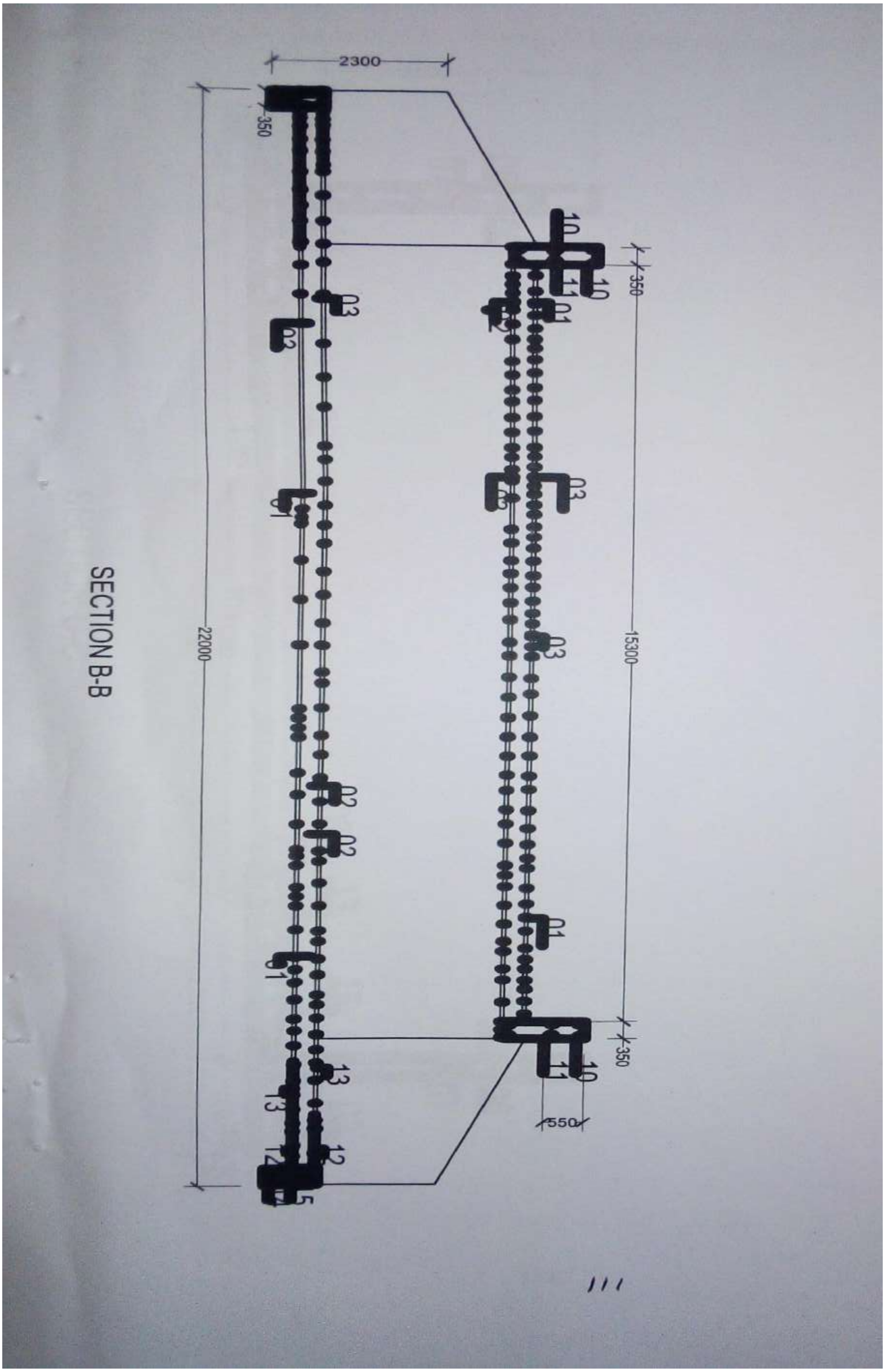

WINGWALL DETAILS
3500
3500
1950
09
09
08
08
2x22Y12-07@150mm c/c Distr.(varies)
2x19Y12-06@200mm c/c N.F.(varies)

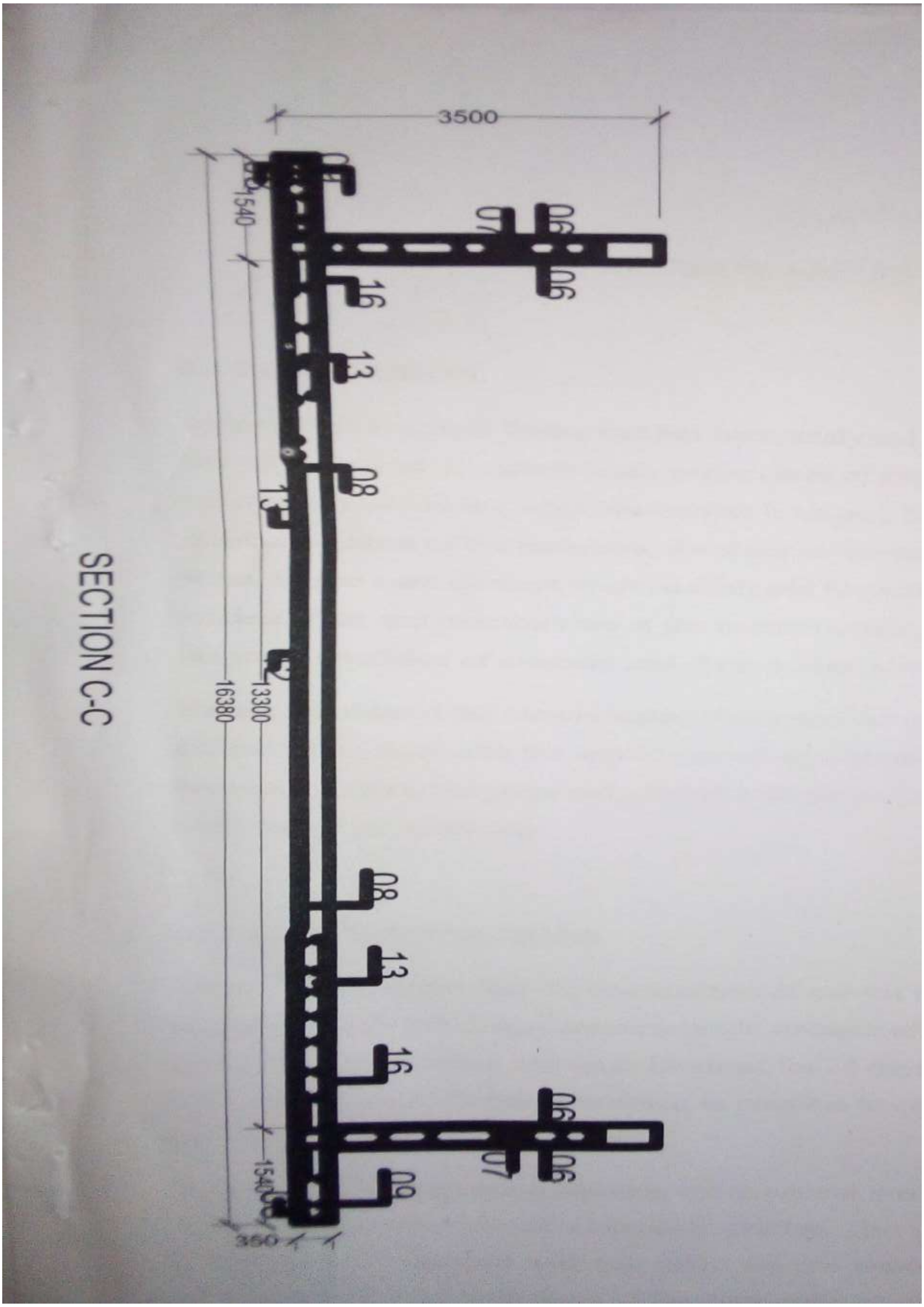

APRON DETAILS
3000
12Y10-06@275mm c/c Distr.(varies)
49Y12-05@275mm c/c Btm.(varies)


1000
1300
2x6Y12-15@200mm c/c.Distr.
2x49Y12-14@275mm c/c N.F.
2x6Y12-15@200mm c/c Distr.
APRON BEAM DETAILS
108


1150
9800
2x9Y12-21@150mm c/c Distr.
2x57Y12-10@175mm c/c N.F.
HEADWALL DETAILS
109


3500
9800
R8-05@200
04
03
02
01
SECTION A-A


SECTION B-B
2300
22000
15300
350
550


SECTION C-C
3500
16380
13300
1540
350

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6.1 CONCULSION

Box culvert is a rigid frame and has been analyzed as such using method of force. It can be seen from the analysis of culvert loads under cases of culvert empty and culvert flowing full that the worst condition occurs when the culvert is empty. It has also been observed that due to the interconnections of the members, the shear in the walls introduce axial forces in the slab and vice versa. Hence each element must be designed for moment and axial pull just like columns. The mildest shear and moments are at the internal walls of the triple and single cell box culvert and the worst condition of moment and shear occurs at the bottom slab.

The schedule of reinforcement suggest that 113 length of Y_{25} is required for main bars and 98 length of Y_{20} is to be required as distribution bars for the triple cell and also 98 length of Y_{20} is required for main bars and 34 length of Y_{12} is required for distribution bars for the single cell

Having considered the factors stated above and more, the culvert has been designed for the ultimate limit state with the application of appropriate factors of safety. And having satisfied all necessary checks, the proposed culvert is fit for construction and is sure to perform optimally under the worst condition.

6.2 RECOMMENDATIONS

I hereby recommend that the construction of culvert be supervised by a qualified engineer. In accordance with the design concrete grade strength of $20N/mm^2$, a concrete mix of 1:2:4 should be used for construction and must be cured for 28 days to allow the concrete attain its maximum strength. Care must be taken as much as possible to ensure that the slope of the culvert aligns with that of the natural stream bed.

For proper interconnection between the structural members of the culvert, the reinforcements of the various members should adequately overlap. There should be at least a 600mm lap of the reinforcement of the wall with that of the top and bottom slab. The reinforcement of the side walls should also lap with those of the wing walls by at least 600mm. there should be at least a 1m lap of the head wall reinforcement and that of the top slab. At the edge, at least a 1m should occur between the headwall reinforcement and the wall reinforcement.

Proper maintenance of the culvert should be carried out by inspecting the culvert regularly to identify potential problems and prevent them, removing accumulated debris from the culvert, resurfacing of spalled areas and so on.

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APPENDICES

BAR BENDING SCHEDULE FOR TRIPLE CELL BOX CULVERT

Bar Mark	Size/Type	Number of each	Number of member	Shape code	Length (mm)	Total length of each (mm)
01	**Y_{25}**	**81**	**1**	**U- Shape**	**16000**	**1296000**
02	**Y_{20}**	**81**	**1**	———	**9800**	**793800**
03	**Y_{12}**	**20**	**2**	———	**9800**	**392000**
04	**Y_{10}**	**81**	**2**	**U-shape**	**3150**	**510300**
05	**Y_{20}**	**49**	**2**	**-----------**	**3500**	**343000**
06	**R_8**	**12**	**1**			

Total length of Y25 = 1296000 mm

Total length of Y20 = 793800+343000 = 1136800 mm

Total length of Y12 = 392000 mm

Total length of Y10 = 510300 mm

Conversion

Y25

1 length =11500

$= \frac{1296000}{11500}$ = 113 lengths

Y20

1 length = 11500

$= \frac{1136800}{11500} = 98$ length

Y12

1lenght = 11500

$= \frac{392000}{11500} =34$ length

Y10

1 length = 11500

$= \frac{510300}{11500} = 44$ length