A PROPOSED DESIGN OF STEEL PAVILION AT GUDU DISTRICT ABUJA F.C.T.

BY

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A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGERIA IN PARTIAL FULFILMENTOF THE REQUIREMENT FOR THE AWARD OF THE POST GRADUATE DIPLOMA IN CIVIL ENGINEERING TECHNOLOGY

MARCH, 2012

DECLARATION

I hereby declare that this report has been composed by me and that it is a record of my own work. It has not been accepted in any previous application for PGD programs. All sources of information are specially acknowledged by means of references.

.... 28/06/2012
....
JOSEPH SHELLENG ONEHI **SIGNATURE/DATE**

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DEDICATION

This write-up is dedicated to Almighty God for his endless blessings and protection towards me in the course of my academic pursuit and to my family present and future

APPROVAL SHEET

This project titled "A Proposed Design of Pavilion at Gudu District Abuja F.C.T.", by Joseph Shelleng Onehi, meets the requirements for the award of Post Graduate Diploma in Civil Engineering and is hereby approved.

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ACKNOWLEDGEMENT

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ABSTRACT

The main purpose of this project is to produce a structural design of steel and reinforced concrete cover pavilion, to virtualizes the aesthetic and safety implication of using steel and reinforced concrete in design. The detail design methods used in this project are according to the British Codes and standards the limit state and Elastic Method of analysis are the basis of the design and the calculations are in S.I. unit all. The design is also buttressed by extracts from CP 110, BS 8110 and BS 5950. The structure has length of 54.94m with total width of 13.53m and a sitting capacity of 1500. The columns are of 3 types which are reinforced with Y20 and Y16 diameter bars, while the beams are of the same size 300 x 600mm exception of the roof beam which is 225 by 225mm in size and reinforced with Y25 and Y20 diameter bars. The slab is designed as stair slab which has 17 panels of 6m by 6m and thread of 800mm, with riser of 300mm and are reinforced with Y10 as the main reinforcement bars and Y10 as the distribution bars. The roof beam 2 is a universal beam, which is bolted to the column C_1 and Stanchion C_2A. The columns rest on foundation footings of 2.45m by 2.45m, 2.5m by 2.5m and 2.6m x 2.6m which are reinforced with Y20 diameter bars. At the end of project work recommendations were made for future work, references are also referenced.

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NOTATION (REINFORCED CONCRETE)

A_S = Area of steel

$A_{sv/c}$ = Area of links

b = Width of Section

b_w = Breadth of web or rib of a member

d = Deflection depth of tension reinforcement

F = Design load

Ft = Tension force

F_{cu} = Characteristic concrete cube strength

F_y = Characteristic strength of reinforcement

F_{yv} = Characteristic strength of links

G_k = Characteristic dead load

h = Overall depth of section in plane of bending

h_f = Thickness of flange

La = Lever arm factor

Le = Effective column length

m = Design moment

Mf = Modification factor

N = Design load of corbel

Q_k = Characteristic live load

RA	=RB	Reaction
S_v	=	Spacing of links along the member
V	=	Shear force
V_c	=	Ultimate shear stress in concrete
W_u	=	Ultimate load per unit length
W_k	=	Characteristic wind pressure
x	=	Neutral axis depth
Z	=	Lever arm
ϕ	=	Diameter of bars

NOTATION (STEEL DESIGN)

A	=	Cross sectional area
A_g	=	Gross sectional area
b_t	=	Tensile stress or bending stress in tension
f_{bc}	=	bending stress in compression
f_{ca}	=	compression stress due to axial load
f_b	=	load on any bolt due to moment

CHAPTER ONE

INTRODUCTION

Every living being needs shelter against rain, sunshine and other adverse climatic conditions. It could be seen that structures play many important roles to the environment where it is located to the populace. These ranges from shelter, aesthetic, and safety purpose to economic consideration.

Having this in mind, there is a need to provide a safe workable and economic structure to meet the above target. To achieve the fundamental aim of this project, the structure in question, that is the spectators cover grand pavilion consists of structural members, like beams, columns, slab, foundation footings etc which must be designed, assembled monolithically to be able to carry, resist and transmit loads to the sub-soil without damage to the structure and users.

Reinforced concrete consists of an assemblage of reinforced concrete members while the steel structure consists of an assemblage of steel sections, which carries all the loads to which these are subjected to without any dynamic effects. The load bearing members consists of beams, slab, columns and foundation footings. In reinforced concrete each of the members consists of two or more reinforced bars embedded in concrete. The steel bars take all the tensile stresses (theoretically) so that cracks that occur in members do not appreciably weaken it. The bars are usually placed in position where they are most effective in resisting the tensile or compressive force induced in the members by imposed load.

A steel structure normally called frame structure may be designed as a combination of steel sections such as beams column and angle channels etc, which are selected from steel profile. They are rigidly connected together to form a monolithically framed structure. Each individual member must be capable of resisting the force action on it.

In design, steel can be designed using both elastic and plastic design method. Elastic method of design is employed.

A limit state may be defined as the state of a structure at which the structure becomes unfit for use. To satisfy the objective of this method of design all relevant limit states should be considered in order to ensure an adequate degree of safety of structure and the serviceability limits. in the design of reinforced concrete, limit state design is employed.

Experience and judgment, which play such an important role in structures receive little attention in technical literature, if experience and judgment are to be real benefit, the designer must learn from the lesson of past failures, unfortunately technical literatures on the failures of past is extremely scarce. Understandably, people do not wish to discuss their mistakes, yet, full discussion of these failures in technical literature could be just as useful as to the profession and discussion of great achievements.

For this reason, it becomes imperative to have a comparative discussion and analysis of structures before the selection of type of structure (steel, concrete, timber etc) methods of design (elastic or limit state design) and layout of structure.

1.1 Aims and Objectives of the Study

Aims

- To design a structure that will be conducive for, 1500 spectators and structurally stable

Objectives.

1. Preparation of architectural drawings
2. The selection of type and layout of structure, the type of structure is selected on the basis of functional service and economic.
3. Determination of service load or analysis of loads, dead and impose loads, roof and wind load, self weight.

4. Determination of internal forces and moments
5. Proportioning of member and connections, for steel members, this will be done with the following criteria in mind, economy, adequate strength, rigidity and ease of connection.
6. After the size of a member has been determined from loads: it is checked for service requirements, such as deflection, shear etc.
7. When the selection properties are finally known, it is necessary to verify or note the assumed weight of the structure correspond to the final weight.
8. Production of structural drawings and typing of the literally work.

1.2 Scope of the Study

The study is limited to the design in reinforced concrete and steel. The use of elastic method of design to BS590 (Steel) and limit state design to BS8110 which is buttressed by extracts from CP110 (reinforced concrete) and BS499 (Steel).

1.3 Significance

Good analysis is based on the accurate anticipation of the behaviours of structure service conditions; the structural designer who performs an analysis without first visualizing the behaviour of forces in the structure is trading on a dangerous ground. It's imperative to have a comparative discussion and analysis of structures before the selection of type of structure, method of design and layout of the structure will be decided upon.

1.4 Function Of The Structure

Pavilion is a building next to sports ground, used by players and people watching (spectators) the game. This definition is based on the location of the structure and its intended use.

As stated earlier in the introductory aspect of the project work, structures play many important roles to the environment where it is located. These range from shelter, aesthetic and safety to economic consideration.

1.5 Design Theory (Elastic Design)

Improved analytical methods and materials of construction have provided a new approach to the design of steel buildings called plastic design. However, it must be emphasized that ultimate strength design can never completely replace design based upon the elastic process (conventional design). Ultimate strength design is based upon unreal loads (increased to allow for a factor of safety) yet in spite of the factor of safety provided, the structure can only perform satisfactorily when the real load on it only remains within the elastic limit of the material if not permanent deformation will occur. (Stanley et al, 1972).

Most structural members of a building are designed to carry a load that will not develop it is ultimate strength under normal service condition. There are so many element of uncertainty both as to building and uniformity in quality of precision in structural design, consequently, some margin of safety are provided by setting allowable working stresses at values well below the ultimate strength.

The factor of safety is defined as the ratio between the ultimate strength and working stress. However, this definition is not wholly satisfactory since failure of a structure members in building actually begins when working stress exceed the yield point or more precisely, elastic limit. The theory of elasticity is the proper basis for design, the factors of safety is based on the difference between maximum actual stress at predicted load and the yield stress that will occur.

The average stress on the gross sectional area of a compression member in steel with a specified minimum yield stress shall not exceed value of P_c (permissible stress (N/mm^2) obtained from table, for any member carrying loads resulting from dead weight with or without imposed loads, the maximum ratio shall not exceed 180 (BS 5950, 1985) clause 14a of BS 5950 states that members subjected to both axial compression and bending stress shall be so proportional that the quantity of $F_c/P_c + F_{bc}/P_{bc} < 1$. The value of λ should not exceed 250 for members resisting self weight and wind loads only.

Elastic design should be carried out under factored load, serviceability load of a building or parts should not impair the strength of efficiency of the structure or its components or cause damage to the building.

1.5.1 Limit State Design

Limit state design refers to design method used in structural enginieering. A limit state is a condition of a structure beyond which if no longer fulfills the relevant design criteria. The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements. A structure designed by limit state design is proportioned to sustain all action, likely to occur during its design life, and to remain fit for use, with an appropriate level of reliability for each limit state.

A limit state is the state at which structure become unfit for use and the design aim is to avoid any such condition being reached during the expected life of the structure.

Three basic methods using factors of safety to achieve safe, workable structure have been developed: they are

- The permissible stress method
- The load factor method and
- The limit state method

Because of some shortcoming from the first two methods the limit state method, it becomes imperative to use the limit state method, because the limit state method overcomes many of the disadvantages of the permissible stress and load factor methods. This is done by applying partial factors of safely, both to the loads and the material strength, and the magnitudes of the factors may varied so that they may be used either with the more elastic stress ranges at working load, the flexibility from development of improved concrete and steel properties.

To satisfy the objective of this method of design all relevant limit states should be considered in order to ensure an adequate degree of safety of the structure and the serviceability limit state. The two principal types of limit state are the ultimate limit state and the serviceability limit state.

1.5.2 Ultimate Limit State

The strength of the structure should be sufficient to withstand the designed load taking into account on the possibility of overturning and buckling. The design strength of material and the design loads should be taken as appropriate for the ultimate limit states. These assessments should ensure that no ultimate limit state is bridged as a result of rapture of one or more critical section by overturning or by buckling caused by elastic or plastic instability. The structure should be designed to support loads caused by normal function there should be a reasonable probability that it will not collapse catastrophically under the effect of misused or accident. No structure can be expected to be resistance to excessive load or forces that will rise due to an extreme cause but it should not be damaged completely.

1.5.3 Serviceability limit state

This ensures behavior of structure under load is satisfactory. Generally, two most important serviceability are deflection and cracking. Other serviceable limit states that may be reached are durability, excessive vibration, fatigue, fire resistance and special circumstances may include earthquake. The relative importance of each limit state would vary according to the nature of the structure the usual procedure is to decide the initial limit state on a particular structure and base the design on this, checks must also be made to make sure that all other limit states are satisfied by the design produced.

1.5.4 Concrete as a Building Material

Reinforced concrete is a strong durable building material that can be formed into many varied shapes and sizes ranging from a simple rectangular column, to a slender curved shell, its utility versatility are achieved by combining the best features of concrete and steel when there are combined, the steel is able to provide the tensile strength and probably some of the shear strength while the concrete, strong in compression, protect the steel to give durability and fire resistance.

1.5.5 Steel as a Structural Material

Steel is one of the most structural material properties of particular importance, its structural usage are enormous; it has a high strength, in addition to its ductility. Ductility is the ability of steel to deform substantially in either tension or compression before failure, other important reaction for its usage include widespread availability, durability particular with a modest resistance to all condition.

Elasticity is the ability of steel to deformed and return to its original shape and size when the forces causing the deformation are removed. A steel with this ability respond elastically. To a greater or lesser extent. Most sold materials exhibit elastic behavior, but there is a limit to the magnitude of the force and accompanying deformation within which elastic recovery is possible for any given material. This limit, called the elast limit is the maximum stress or force per unit area within a solid material that can arise before the onset of permanent deformation.

The ability of a material to deform plastically and to absorb energy with process before fraction is termed toughness. The emphasis of this deformation should be placed on the ability to absorb energy before fraction. Recall that durability is a measure of how much something deforms plastically before fraction. But just because a material is ductile, does not make it tough. The key to toughness is a good combination of strength and ductility

CHAPTER TWO

LITERATURE REVIEW

Theories of design are not found in one literature. There are several design professions institutionalized in different kinds of educational and research context ranging from architecture, engineering and computing. I draw selectively from these fields to build up a frame work that distinguishes between different kinds of structural design that houses spectators. Description of design often hinges on differences between underlying views of Science and Knowledge: positivist science or Constructionism (Dorst and Dikkhuis, 1995) ways of thinking of design range from attempts to build general theories to accounts of particular practices. For Alexander (1971), "the ultimate object of design is form"

An important project report focus is the process of designing. Design can be understood as designers co-creating problems and solutions in an exploratory, iterative process in which problems and solution co-evolve. The literary material used in this report will present design strategies, design requirements and guidelines and also important and relevant ideas related to the project. These literary materials were related on the project topic, mainly focusing on the method, materials, sustainability and innovations proposed for the project.

A structural design of a conference hall, a project report by: Bukumi Adeola, in his report, he stressed that for fire resistance and moreso, because of the area of the structure the roof truss should be designed as steel members and the stair be designed as precast slab to reduce the construction time and for future work, the conference hall sitting capacity be increased from 1000 to 1500.

Buildings for the performing arts: A design and development guide written by: Ian Appleton (1996) focuses on the involvement of the planning, initiation and design of facilities for the performing arts. It includes information requirements and the stages in the development on designing such facility. The literary contains background information about prevailing issues on various building types, and also dealt with considerations of client, consultants, the stages to be achieved, with consideration of the building use.

A structural design of spectators square for Basket ball Court; a project report by: Abdul Adamu (2002), the literary contains recommendation for future work, it focuses on how the project work can be constructed in a very fast way. The spectator square which has a sitting capacity of 500 has a stair slab and the beams which are precast and joined to the columns that are cast in-situ. He recommended that for future work the columns and beams be designed as steel section, and the sitting capacity be increased. Moreso roof cover be provided for the spectator square. Theatre buildings:

A design guide produced by: Association of British Theatre Technicians (1998) took account of the development of new technologies, new form of presentation, changing expectations and the economic and social pressures which required every part of the theatre to be as productive as possible. It focuses on the whole process of planning and designing a theatre buildings giving specific guidance on acoustic and auditorium.

A *structural design of a steel pavilion for children play ground*: A project report by: James Oko (2006), all the structural elements or members are designed as steel members exception of the foundation footing. The pavilion has a sitting capacity of 400 with a riser of 300mm and thread of 600mm. For future work he recommends that the stair be designed as precast slab.

The contract for the construction of spectator's covered grand pavilion at Federal Polytechnic Bauchi was awarded to Sahel Engineering Services Limited in 2005 with interactive Design Consultants firm as the Consultant, under the Education Tax Fund (ETF) intervention for the 14th NIPOGA held in Federal Polytechnic Bauchi. The Spectator's covered ground pavilion is located very close to the school main football pitch; All the beams, columns and slabs are designed as reinforced concrete. The main column is connected to the roof beam at the top or head which is designed as corbel to resist overturning and twisting with the corbel height of 1700mm and length of 110mm which are joined monolithically with cantilever beam and column. The structure has a sitting capacity of 1000, with offices, stores, dressing rooms and conveniences. For future work they advised that the cantilever roof beam be designed as steel members of steel trusses.

3.0 MATERIALS AND METHOD

The use of elastic method of design (steel) and limit state design method (reinforcement concrete) are the basis of the design.

3.1 Steel

In the design of a steel structure, the member used in the design and construction of the structure are based on the elastic method of design to BS5950, the effective loads due to compression, tension, wind pressure and so on and the load that is finally imposed and transmitted to the foundation are determined.

The structural design procedure consists of six principal steps.

- The selection of type and layout of structure, the type of structure is selected on the basis of functional, economic service and the aim of the project work. To design a structure that will be conducive for 1500 spectators.

- Determination of service load, when the general type of structure is chosen or at last several alternatives are defined, a small-scale layout, and the arrangement of members is naturally governed by magnitude of the loads in the members. Wind load 0.86KN/m^2, roof loads (0.75kN/m^2) and self weight. Self weight is based on the steel section selected for tries.

- Internal forces and moments – for statically determinate structure, subjected to static loads, the forces and moments in all the members can

  be computed simply by using conditions of equilibrium, but for statically in determinate structures, it is necessary to make some estimate of the member sizes in order to determine the stresses in the structure.

- Proportioning of member and connections – when internal forces in the members and material are known, the size of each member can be selected with the following criteria in mind.

a. Adequate strength and rigidity

b. Ease of connection and

c. Economy

- Performance under service conditions – after the size of a member has been determined from loads; it is checked for services requirements, such as limiting deflections, undue twisting e.t.c.

- Final review – when the selection properties are finally known, it is necessary to verify or not the assumed weight of the structure correspond to the final weight.

The structure members are chosen from a range of feasible alternative the

structural layout such as spacing of the columns and beams are tentatively chosen, grade 43 Steel is to be used in the design because of the following reasons. The material strength is specified in relation to steel grade. The ultimate strength is dependent on yield stress, stress are given for the grade of steel called S275, S(these were formerly referred to as grades 43.50 and 55). Grade S275 (formerly grade 43) is commonly used, although S355 is popular on large projects where it can offer significant economics. Higher grade are rarely used, except for bridges and special applications.

3.1.1 **Recommendable height and length**

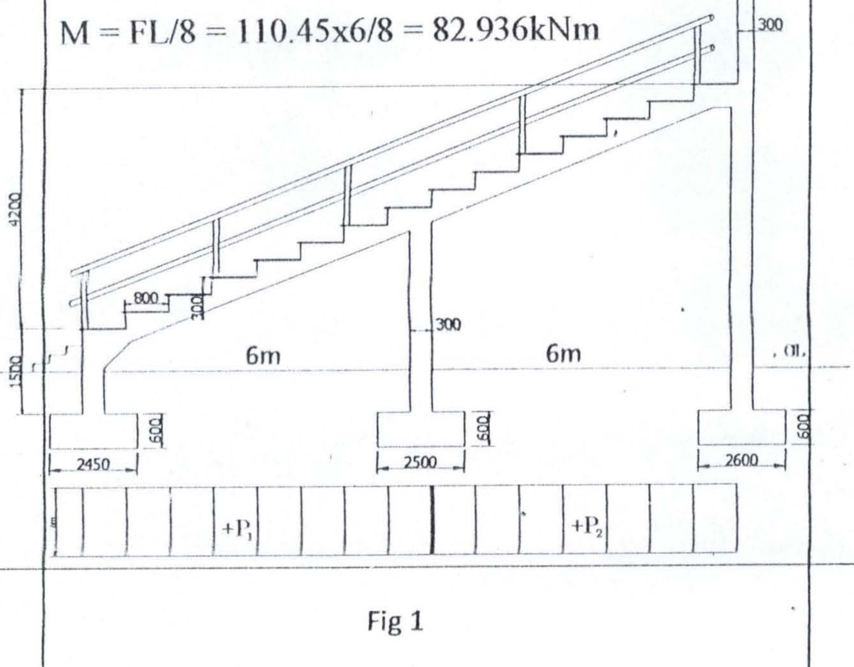
Steel has a reasonable height and length, which can be increased if the need arises either by welding or bolting, can be used in joining as the length of the steel permit

3.2 Reinforce Concrete

Limit state design which is the basis of design of reinforced concrete is employed, this is to BS8110 with little extract from Cp110, and concrete grade is to be used. The methodology involves determining the size of beams, columns and the thickness of the slab, it also involves the determination of self-weight of the structural members, imposed, dead and wind load on the structure, the study of the strength stability of the supporting component and foundation. This is done by computing or determining the minimum bending moment and shear bending moment used in the provision of reinforcement bars. Structural behavior should be given careful attention by assessing the inter connection of the component parts of the structure. The central member problem is to provide an economically competitive structure, which will performs adequately under service condition imposed by working loads

The methodology involves also the determination of the load that will be transmitted to the foundation from the columns. After ascertaining the load on the foundation the stability of the soil is to be determined by the site investigation, appropriate soil test to determine the bearing capacity of the soil. But the already ascertained bearing capacity of the soil will be adopted, because of time factor. The bearing capacity adopted is 200KN/m^2.

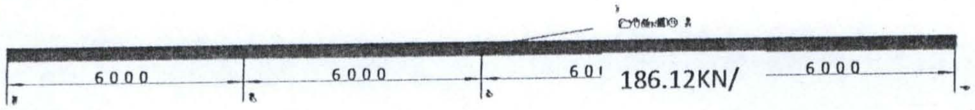
Any part of the structure shall be capable of sustaining the most adverse combination of static and dynamic forces which may reasonable be expected from dead loads and impose load, without the limit state being reached, and the permissible stresses specified being exceeded

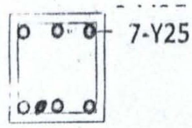
REF	DESCRIPTION/CALCULATIONS	OUTPUT
	3.3.0 (a) Slab Design Analysis The Slab is designed as a stair slab for durability and fire resistance. Assume Section Waist = 250mm, Risers = 300mm, risers =0.3x7=2.1m^2 Tread = 800mm, d = 250 – 25 – 12/2 = 219mm Slop length of stairs $\sqrt{6^2 + 2.1^2} = 6.36$m Considering a 1m width of stairs Weight of waist plus steps = (0.25x6.36+0.8x2.1/2) x 24 = (1.59 + 0.84) 24 Dead Load ⟶ (58.32) KN Finishes Say ⟶ 1KN Total dead load ⟶ 59.32KN Impose Live Load ⟶ 3 x 6 = 18KN Ultimate Load F = (1.4 x 59.32) + (1.6 x 18) F = 110.45kN/m^2 M = FL/8 = 110.45x6/8 = 82.936kNm 	C = 25mm ϕ = 12mm d=219mm fyz=460 Z=0.95d Fcu= 30
	Fig 1	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	$K = M/bd^2fcu = 82.836x10^6/1000x219^2 \times 30 = 0.058$	$K = 0.058$
W.H Mosely J.H.Bungey &Hulse	$Z = 0.093d = 0.93 \times 219 = 203.67$	$Z= 203.67$
	$A_s = M/0.95fyz = 82.836x10^6/0.95x460x203.67 = 930.7mm^2$	
	Provide Y_{12} @ 100mm c/c, $A_s = 1130mm^2$	$A_s = 1130mm^2$
	Distribution bars	
	$A_{smin} = 0.13\%bh = 0.13x1000x250/100 = 325mm^2$	
	Provide Y_{10} @ 200, Area = 393mm	
	(b) Deflection Check	
	Service Stress in tension steel	
	$F_s = 5x40x930.7/8x1130 = 20.6N/mm^2$	
	Modification Factor $M_f = 0.55 + \frac{(477 - F_s)}{120(0.9+m/bd^2)}$	
	$= 0.55 + \frac{477-20.6}{120(0.9+\frac{82.836x10^6)}{1000x219^2}}$	
	$0.55+\frac{456.4}{120(0.9+1.73)}$	
	$0.55+\frac{456.4}{120(2.63)}$	
	$=0.55+\frac{456.4}{315.6}$	
	$=0.55+1.44=1.99<2$	
	Basic Span/Depth Ratio = 20	
	Limiting Span/depth ratio = 1.99x20 = 39.8	
	Actual Span/Depth Ratio = 6360/219 = 29.1	
	Since Limiting Span/depth ratio > actual/depth ratio =39.8 > 29.1	
	Deflection check ok.	
	a. Stair Slab panel '2'	
	Slope length of stairs = $\sqrt{6^2 + 2.4^2} = 6.46m$	
	Weight of waist plus steps =(0.25x6.46+0.8x2.4/2)	
	(0.25x6.46+0.8x2.4/2)24 = 61.8kN	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Finishes = 1kN	
	Total dead load = 62.8kN	
	Live load = 3 x 6 = 18kN	
	Ultimate Load F = (1.4x62.8) + (1.6 x 18)	
	F = 116.72kN/m2	
	M = Fl/8 = 116.672 x 6/8 = 87.54kNm	
	K = M/bd2fcu = 87.54x106/1000x2192x30 = 0.061	
	Z= 0.93d = 0.093 x 219 = 203.67	
	As = M/0.95fyZ = 87.54x106/0.95x460x203.67	
	= 983.55mm2	K = 0.061
	Provide Y12 @ 100 c/c, As = 1130mm2	
	b) service stress in tension steel	Z = 203.67
	fs= $5x40x983.55mm^2/8x1130=20.7\ N/mm^2$	
	$mf = 0.55+\frac{(477-20.7)}{120\ (0.9+m/Bd^2)}$	
	$=0.55+\frac{456.3}{120(0.9+\frac{87.54x10^6}{1000x219^2}}$	
	$=0.55+\frac{456.3}{120(0.9+1.83)}$	
	$=0.55+\frac{456.3}{327.6}$	
	=0.55+1.39=1.94<2	
	Basic span/depth ratio =20	
	Limiting span/depth ratio=1.94x20=38.8	
	Actual span/depth ratio $=\frac{6460}{219}$ = 29.5	
	Since limiting span/depth > actual/depth ratio	
	=38.8>29.5 deflection ok	

	3.4.0 Beam Analysis Beam 2 (B-5) Description: The beam is a continuous beam, it is designed as continuous beam over support <u>Section</u> 500 x 300mm beam H =500, b = 300, d = 500 – 25 – 20/2 – 10 = 455mm Loading S/W = 0.5x0.3x24x1.4 = 5.04kN/M Dead Load from Slab = 0.33nlx = 0.33x87.54x6kN/m =175.08 kN/m	b = 300 H = 500 d = 455

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Total dead load = 180.12kN/m	
	Imposed load =0.33x3x6=6 kN/M	
	Design Load F = 180.12 + 6 = 186.12kN	
		
	<u>Bending moment</u>	
	(a) 1^{st} interior support =mb =mc =md	
	Where F = ql = 186.12 x 6 = 1116.72kN	
	Mb =0.11fl = 0.11 x 1116.72 x6 = 737.041kNm	
	@ <u>Midspan</u>	
	1^{st} and 4^{th} span design as T-Section	
	M_{ab} = 0.09fl = 0.09 x 1116.72 x 6 = 603.03kNm	
	M_{bc} = (Interior span) = 0.07fl=0.07x1116.72 x 6 = 469.02kNm	
	M_{cd} = Mbc and Mab = Mde	
	<u>Sheer Force</u>	
	V_A = 0.45F = 0.45 x 1116.72 = 502.52kNm	
	V_C = 0.6F = 0.6 x 1116.72 = 670.03kNm	
	V_B = 0.55F = 0.55 x 1116.72 = 614.20kNm	
	V_E = 0.45F = 0.45 x 1116.72 = 502.52kNm	
	<u>Main Reinforcement</u>	
	Effective width of flange = bw + 1/5 (0.7L)	
	= 300 + 1/5 (0.7 x 6000) = 1140mm	
	Therefore @ Support 'B', K = M/bd^2fcu	
	K = $603.03x10^6/1140 \times 455^2 \times 30$ = 0.09	K=0.09
	Z = 0.89d	
	Z = 0.89 x 455 = 404.95mm	Z=404.95
	d - z = 455 – 404.95 = 50.05 < hf/2	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Provide transverse steel	
	Area = 0.15hf x 1000/100 = 1.5hf /m length of the beam	
	Neutral axis = d-z/0.45 = 50.05/0.45 = 111.2mm	
	A_s = 603.03x10^6/0.95x460x404.95 = 3407.7mm^2	
	Provide 7 Y_{25} bars, Area 3440mm^2	A_s =3407.7
	(b) Interior Supports – design as a rectangular section	
	M = 0.11 x 1116.72 x 6 = 737.04kNm hogging	
	K = M/bd^2fcu = 737.04x10^6/1140x455^2x30 = 0.1	
	= 0.1 < 0.156	K = 0.1
	Z= d (0.5 + $\sqrt{0.25 - k/09}$	Z= 395.85
	Z = 0.87d = 0.87 x 455 = 395.85mm	
	A_s = M/0.95fyZ = 737.04 x 10^6/0.95x460x395.85m	
	= 4,261mm^2	
	Provide 9Y_{25} Area = 4440mm^2	
	(c) Midspan of 2^{nd} span – design as T-Section	
	K = M/bdf^2fcu = 469.02x10^6/500x455x30 = 0.151	
	Z = d (0.5 + $\sqrt{0.25 - k/09}$	
	Z = 0.79d = 359.45	
	A_s = M/0.95fyZ = 469.02 x 10^6/0.95 x 460 x 359.45	
	= 2,985.87mm^2	
	Provide 7Y_{25} bar, Area = 3440mm^2	Z = 359.45
	d – z = 455 – 359.45 = 95.55 < hf/2	

REF	DESCRIPTION/CALCULATIONS	OUTPUT

Thus, the stress block lies within the flange.

$100\ A_s/\ bwh = 100 \times 3440/300 \times 500 = 2.30$ percent

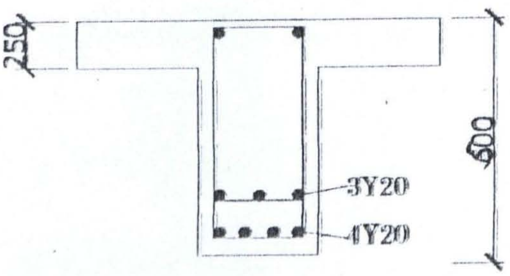
Thus, the Steel percentage is greater than the minimum

specified by the Code.

Transverse Steel in the flange

$1.5hf = 1.5 \times 250 = 375mm^2/m$

Provide R10 bars @ 200mm Centres, Area $393mm^2$



(a) Check max. shear stress

Maximum shear @ face of support is

$V_s = f/2 - wu \times$ support width/2

$= 1116.72/2 - 186.12 \times 0.15$

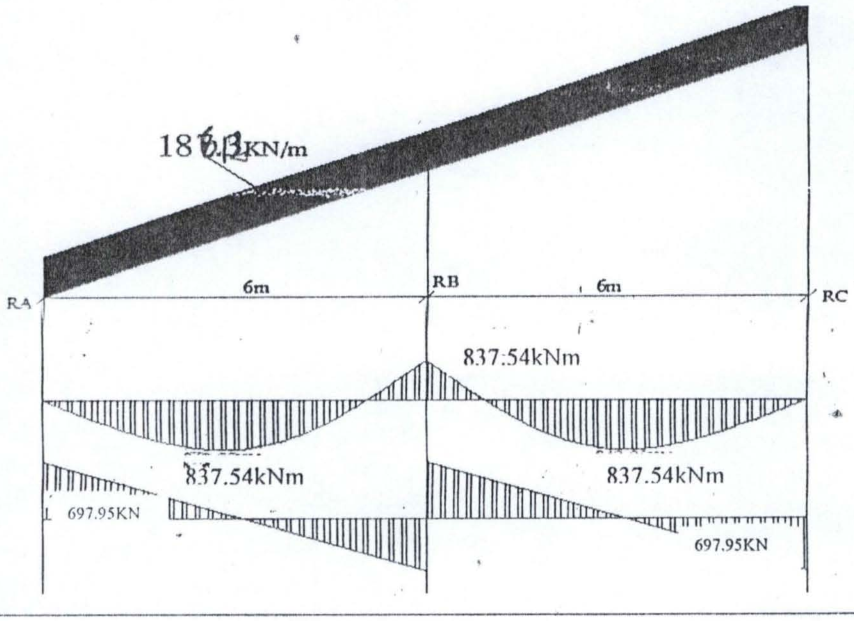
$V_s = 530.44kN$

$V = Vs/bd = 530.44 \times 10^3/300 \times 455$

$= 3.89N/mm^2 < 5N/mm^2$ or $0.8\sqrt{fcu}$

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	(b) Nominal Links	
	$A_{sv}/S_v = 0.4b/0.95fyv = 0.4x300/0.95x250 = 0.51$	
	Xteristics strength of the mild steel links is $f_{yv} = 250N/mm^2$	
	Provide R_{10} links @ 300mm centres	
	$A_{sv}/A_v = 0.52$	
	(c) End Supports	
	Shear distance 'd' from the face is	
	$Vd = 0.45f - wu$ (d + support width/$_2$)	
	$= 502.52 - 186.12 (0.455 + 0.15)$	
	$= 389.92kN$	
	$v = Vd/bd = 389.92 \times 10^3/300x455 = 2.86N/mm^2$	
	$100 A_s/bd = 100 \times 3440/300x455 = 2.52$ therefore	
	From table 5.1, $V_c = 1.04$	
	$A_s/S_v = b(V-V_C)/0.95f_{yv} = 300 (2.86-1.04)/0.95 \times 250=2.30$	$V_c = 0.97$
	Provide R_{12} links @ 100mm Centres	
	Shear resistance of Nominal links + Concrete is	
	$V_n = (\underline{A_{sv}/S_v} \times 0.95f_{yv} + bv_c) d$	
	$= 2.30 \times 0.95 \times 250 + 300 \times 1.04) 455 = 390.5kN$	
	Shear reinforcement is required over a distance 'S' given by	
	$S = V_s - V_n/w_u = 502.52-390.5/186.12$	
	= 0.6m from the face of the support.	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Number of R_{12} links @ 100mm centres required at each end	
	of the beam is $1 + (S/200) = 1 + (1600/200) = 4$	
	<u>Deflection Check</u>	
	Basic ratio $= 26$	
	$100\ A_s/bd = 100 \times 3440/300\times455 = 2.52$	
	Modification Factor $= 1$	
	Max span/depth ratio $= 26 \times 1$	
	Actual span/depth ratio $= 6000/455 = 13.2 < 26$	
	Deflection ok.	
	Shear force @ Support 'C'	
	$V_C = 0.6f = 670.03$kNm	
	$V_s = 670.03 = w_u \times$ support width/$_2$	
	$= 670.03 - 186.12 \times 0.15$	
	$= 642.11$kN	
	$V = V_s/bd = 642.11 \times 10^3/300 \times 455 = 4.70N/m^2 < 5N/m^2$	
	<u>Beam 4</u> (2-2)	
	Design as a continuous beam	
	<u>Section</u>	
	500 x 300mm, h = 500, b = 300 Cover = 25	
	$d = 500 - 25 - 20/2 - 10 = 455mm$	
		C = 25
	<u>Loading</u>	D = 455mm
	Self weight of the beam = 0.5x0.3x24x1.4=5.04kNm	
	Dead Load from slab = 1/3nlx =1/3x87.54x6 =175.08kNm	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Total Dead Load = 180.12 kNm Imposed Load = 1/3 x 3 x 6 (1/3nlx) = 6kNm Design Load 180.12 + 6kNm = 186.12kNm End Condition, $M_A = 0$, $M_C = 0$ $-M_AL_1 - 2\ M_B\ (L_1 + L_2) - M_cL_2 = (WL^3/4)\ x2$ $-24M_B = (186.12x6^3/4)x2$ $-24M_B = 20,100.96$ $M_B\ = 837.54KNm$ Taking moment about 'B' to the left R_A x 6 – 186.12 x 6 x 6/2 = 837.54 R_A = 697.95kN Taking moment about 'B' to the right $6R_C$ – 186.12 x 3 x 6 = 837.54 R_C = 697.95KN $R_A + R_B + R_C$ = 1116.72 x 2 R_B = 837.54KN AB = BC = m = $wl^2/8$ = 837.54kNm Since it has an equal span, it will be better designed as continuous beam. Total ultimate load on a Span F = 186.12 x 6 = 1116.72kN 	

REF	DESCRIPTION/CALCULATIONS	OUTPUT

<u>Bending moment</u>

(a) Midspan of 1st and 2nd, span design as a T-Section

$M = 0.09fl = 0.09 \times 1116.72 \times 6 = 603.03$kNm

Effective width of flange = bw + 0.7L/5

= $300 + 0.7 \times 6000/5 = 1140$mm

Therefore $K = m/bd^2fcu = 603.03 \times 10^6/1140 \times 455^2 x = 0.09$,

$Z = 0.89d = 404.95$mm

Same provision as beam 2

Provide 7 Y_{25} bars, Area 3440mm^2

OUTPUT: K= 0.09
Z = 404.95
As =3440mm^2

<u>Roof beam 2</u>

this beam is not carrying any roof load, it serves as a brace between the columns (C_d)

<u>Loading</u>

Section 225 x 300mm

(1) Self weight of beam = $0.225 \times 0.3 \times 24 = 1.62$kN/m

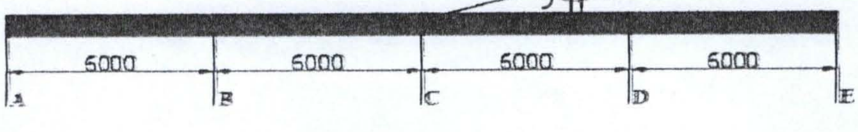
Total dead load gk = 1.62kN/m

Imposed live load = 0.75kN/m

Design Load n = 1.4gk + 1.6qk

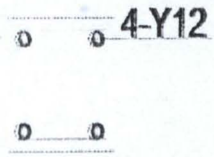
n = $1.4 \times 1.62 + 1.6 \times 0.75$

n = 3.47kN/m



OUTPUT: **d = 257**
C = 25
θ = 16

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	<u>Bending Moment</u>	
	@ 1st Interior Support = $M_B = M_C = M_D$	
	Where $f = ql = 3.47 \times 6 = 20.82$ kN	
	$M_B = 0.11fl = -0.11 \times 20.82 \times 6 \;\; = \;\; 13.74$kNm	
	@ <u>Midspan</u>	
	1st and 4th Span design as T-Section	
	$M_{AB} = 0.09fl = 0.09 \times 20.82 \times 6 = 11.24$kN	
	M_{BC} = (Interior Span) = $0.07fl = 0.07 \text{x} 20.82 \text{x} 6 = 8.74$kN	
	$M_{CD} = M_{BC}$ and $M_{AB} = M_{DE}$	
	<u>Shear Force</u>	
	$V_A = 0.45f = 0.45 \times 20.82 = 9.37$kNm	
	$V_C = 0.6f = 0.6 \times 20.82 \;\; = \;\; 12.49$kNm	
	$V_B = 0.55f = 0.55 \text{x} 20.82 \;\; = 11.45$kNm	
	$V_E = 0.45f = 0.45 \times 20.82 = 9.37$kNm	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	(a) Main Reinforcement	
	Effective width of flange = bw + $^{1}/5$ (0.7L)	
	= 225 + $^{1}/5$ (0.7 x 6000) = 1065mm	
	Therefore @ Support 'B' $K = m/bd^2fcu$	
	$K = 11.24x10^6/1065x257^2x30 = 0.005$	
	Z = 0.99d > 0.95d	K = 0.005 Z = 244.15
	Z = 0.95 x 257 = 244.15mm	
	d – z = 257 – 244.15 = 12.85 < hf/2	
	Provide transverse steel	
	Area = 0.15hf x 1000/100 = 1.5hf /metre length of the beam.	
	Neutral axis = d-z/0.45 = 12.85/0.45 = 28.56mm	
	$A_s = 11.24 \times 10^6/0.95x460x244.15$ = $105.35mm^2$	
	Provide 4 Y_{12} bars, Area = $452mm^2$	
	(b) Interior Supports – design as a rectangular section	
	M = -0.11 x 20.82x6 = 13.74kNM hogging	
	$K = m/bd^2fcu = 13.74x10^6/1065x257^2x30 = 0.065$	
	Z = 0.99d > 0.95d, 0.95 x 257 = 244.15	
	$A_s = M/0.95fyZ = 13.74x10^6/0.95x460x244.15 = 128.8mm^2$	
	Provide 4 Y_{12} bar, Area $452mm^2$	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	(c) Midspan of 2nd span – design as T-Section	
	$K = m/bf^1d^2fcu =$	
	$8.74x10^6/225x257^2x30 = 0.02$	
	$Z = 0.98d > 0.95d$, $0.95 \times 257 =$ 244.15mm	
	$A_s = m/0.95fyZ =$	
	$8.74x10^6/0.95x460x244.15 = 81.92mm^2$	
	Provide $4Y_{12}$ bars, Area $452mm^2$	
	<u>Wind Loading</u>	
	Calculation in conformity with requirements of Cp_3	
	Chapter V part 2, 1970	
	In order to arrive @ the loading to be considered, it is necessary to know the location of the site as well as other conditions for the purpose of the project the following data are made available.	
	Building situated in Abuja, Federal Capital with no unusual topological conditions, building is about 12m wide x 54.95m long x 9m high.	
	W_k = characteristics wind pressure	
	V_s = Design wind speed in m/s	
	V = Basiç wind in m/s	
	S_1 = Multiplying factor relating to topology	
	S_2= Multiplying factor for building = Light above ground b/wind breaking	
	S_3 = Multiplying factor related to life structure for Abuja, basic wind speed (V) = 40-473m/s	
	Dynamic wind pressure = $0.613V^2$	

14.2/1.5 = 9.5

Uniform load on the cantilever roof beam or rafter

9.5 x 7.83 kN/ 14.2 = 5.24 kN/m

<u>Loading of roof beam 1</u>

-Dead load from Trier Section: 254x146x31kg/m (UB)

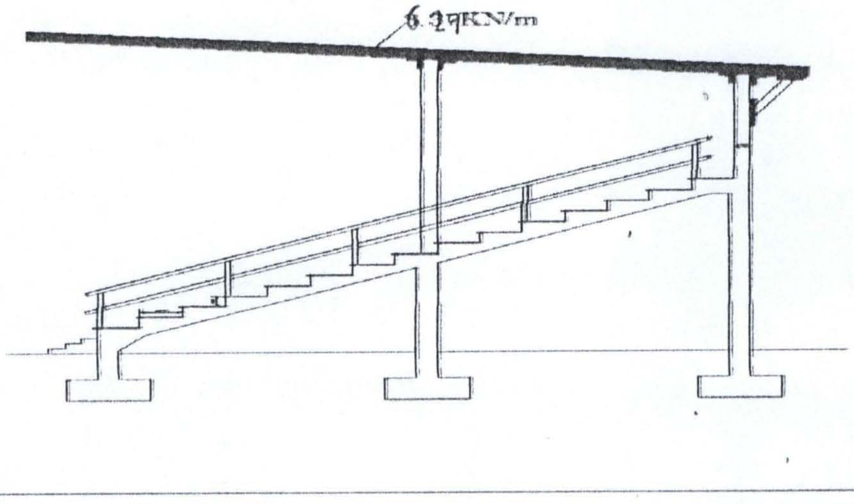
31x9.81/1000 = 0.3kN/m

- load from roof = <u>5.24kNm</u>

-Total dead load = 5.54kN/m

Imposed load (human beings) =<u>0.75kN/m</u>

Design load 6.29kN/m



REF	DESCRIPTION/CALCULATIONS	OUTPUT

Member in bending

For tensile fibre design is satisfactory if ftb is less than

($m/zc \leq pbt$ ($f_{bt} \leq m/2t \leq p_{bt}$)

Similary for compressive fibre, $f_{bc} = m/zc \leq p_{bc}$

Where F_{bt} = tensile stress or bending steel in tension

F_{bc} = bending stress in compression

M = applied moment (due to the force in the Beam)

P_{bt} - Permissible bending stress

Z = Section modules

$F_c/p_c + f_{bc}/p_{bc} < 1$

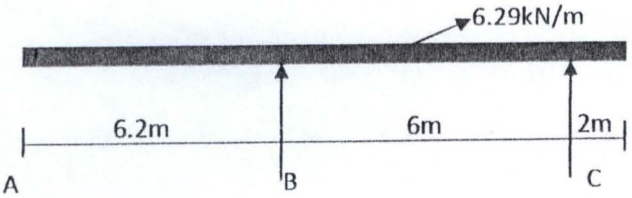
The value of λ should not exceed 250 for member resisting self weight and wind loads only.

Assuming UB of thickness between 10 and 40mm

$P_{bt} = 265/mm^2$, for grade 275, $p_{bt} = 265$

Bs 5950 — Z required is m/p_{bc} or m/p_{bt}

Design



Moment of span AB = $WL^2/2 = 6.29 \times 6.2^2/2 = 1209$kNm

Moment on span BC = $wl^2/8 = 6.29 \times 2^2/8 = 28.31$kNm

Moment of span CD = $wl^2/2 = 6.29 \times 2^2/2 = 12.58$kNm

Since span AB which is the cantilever part is the critical part we will use the moment to design.

Moment of span AB = $wl^2/2 = 6.29 \times 6.2^2/2 = 120.9$kNm

Z required is m/P_{bc} or m/P_{bt}

Therefore $Z = 120.9 \times 10^6/265 =$

$0.45623 \times 10^6 mm^3 = 456.23cm^3$

Provide 305 x 127x 42kg/m (UB)

$Z = 531.2cm^3$

$P_{bt} = m/z = 120.9 \times 10^6/0.5312 \times 10^6 = 227.6N/mm^2$

P_{bt} okay

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	$B = le\ 0.85/r_{min} = 6\ x0.85x10^3/270 = 188.9$	
	D/T = 25.4 P_{bc} = 92 adequate	
	Therefore provide 305 x 127 x 42kg/m	
	Deflection Check	
Steel designers manual	Cantilever beam with uniformely distributed load of total value. Wu = 6.29kN/m	
	Deflection = 306.6mm. T = 12.1mm, t = 8.0mm	
	Txx = 8143cm^4= moment of inertia	
	Zxx = 531.2	
	Actual deflection due to load	
	For the cantilever section 'AB'	Actual δmax 1.77mm
	Actual deflection due to load	
	$\delta max = wl^3/\delta EI = 6.29x10^6x(6.2x10^3)^2/8x2.1x10^5x8143x10^4 = 1.77mm$ Allowable deflection δmax = L/180 =$6.2x10^3/180$ = 34.4mm 1.77 < 34.4mm deflection okay	Allowable δmax = 34.4mm
	For the supported span, 'BC' actual deflection due to load $\delta cal = 5ql^3/384EI = \frac{5x6.29x10^6x(6.2x10^3)^2}{384x2.1x10^5x8143x10^4} = 0.184mm$ Since 1.77> 0.184, use 1.77 for checks	
	Check for Shear	
	Maximum shear force V = wl_2, 6.29 x 6.2 = 39.0KN	
	The Shear force f_v should not be greater than the Shear Capacity P_v, given by $P_v = 0.6fy\ A_V$ in which A_V is the. Shear area taken as tD, where D = overall depth t = The Web thickness $P_v = 0.6x250N/mm^2x306.6mm\ x\ 8mm$ P_v = 367.920kN = 367.9kN > 39.0kN $T_{va} = 30.0x10^3/306.6x8 = 15.9/mm^2 < 0.7f_y$ for Steel with $f_y = 250mm^2$	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Design of Column member $C_2^{\wedge}$ (Structure)	
	Ag required = P/Pc, F_{ca} = P/Ag ≤ P_c	
	Ag = Gross Sectional area	
	P_c depends on slenderness ratio	
	λ= le/r_{min} where le = effective col.length	
	For trial section, limits of slenderness ratio are	
	λ max = 180 for vertical loads only	
	λ max= 250 for vertical and wind loads	
	Loading	
	-Load from room beam 1 (6.29x3 +6.29x6) =56.61kN	
Table 16 BS 5950	S/W of the trial section 203x203x46kg/m,	
	46x9.81x 5.5/1000 = 2.5kN	
	Total design load 59.11kN	
	For trial section λ_{max} = 180 for vertical loads only	
	From Table P_b = 48N/mm^2 hinged top and bottom	
	Ag = 59.11x10^3/46N/mm^2 =1282mm^2 = 12.326m^2	
	Provide 203 x 203 x 46kg/m (Ag = 58/8cm^2)	
	λ = 5500/51.1 = 108.6, P_c = 107N/mm^2	
	F_{Ca} = 59.11 x10^3/58.8x10^2. 10.06N/mm^2 < PC ok	
	Ag,. (Goss Sectional Area)	
	a) 3.5 Design of bracing	
	In order to preserve the stability of the structure, horizontal as well as diagonal bracing have to be provided. In this project, the structure is designed into part and each component is analysed and respectively sized.	
	b) Horizontal bracing (HB_1)	
	Horizontal bracing will be in compression, members selected for trial section (100 x 75 x 15.4kg/m Angle) length of horizontal bracing 6m, =15.4x9.81x6/1000 = 1.0KN	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Force from cantilever beam	
	Assume (10% of the load) = 5.911kN	
	Total Load 6.911kN	
	Net Area = $4.283 \times 10^3 / 265 N/mm^2 = 26.10mm^2 = 2.61cm^2$	
	Provide (75 x 50 x 6kg/m) Area = $7.19cm^2$	
	c) <u>Connection of Horizontal bracing</u>	
	Using gusset plate angle of 8mm thick (i.e. minimum required thickness for all exposed structures). Thus, use 8mm thick gusset plate for the connection of horizontal brace to roof beam 1.	
	(200 x 200 x 8mm gusset angle plate) with a 6mm fillet weld. The horizontal bracing will be at the interval of 3m.	
	d) <u>Design of black bolt</u>	
	Bolts for the connection (Assume 24mm)	
	Shear capacity $tvf\pi d^2/4 = 80 \times 3.142 \times 24^2/4 = 36.2kN/bolt$	
	t_{vf} = max. permissible shear stress in fastener	
	No. of M_{24} bolts = fv/shear = 1.2x36.2/36.2 = 1.2=2	
	Thus use 2 No. of M_{24} block bolts	
	Use the same for diagonal bracing DB_1 connecting Roof beam 1 to Roof Beam 2 which has a length of 2.82m	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Design of base plate for the connection of Roof beam 1 to Column C_1 and Column C_2	
	Axial load from roof beam ⟶ 56.6kN	
	Let assume the size of base plate material since the size of the Cantilever beam 1, stanchion C_2 are 305 x 127, and 203 x 203) Assume a plate of 600 x 305 x 18mm.	
	Check for the thickness of the plate.	
	Thickness of base plate	
	$t = \sqrt{3w/P_{bc}}\ (a^2 - b^2/4)$	
	where t = thickness of the base plate	
	w = pressure on the underside of the base	
	a = greater projection of base plate beyond Col In mm	
	b = smaller projection of the base plate beyond col. In mm	
	Pbc = permissible bending stress in the slab 785	
	$w = 56.61 \times 10^3/305 \times 600 = 0.31 N/mm^2$	
	a = 600-305/2 = 147.5, b = 0	
S.K Duggal	$t = \sqrt{3 \times 0.31/185}\ (147.5^2 - 0^2/4)$	
	$t = \sqrt{105.37} = 10.46$	
	Use 12mm thick plate	
	Provide 600x305x12mm gusset plate	
	It will be welded to the face of the stanchion with 6mm fillet weld.	
	Let Provide 20mm diameter bolts, gross diameter 24mm	
	Strength of bolt in single shear	
	$t \wedge f \pi D^2/4 = 80 \times 3.142 \times 24^2/4 = 36$	

REF	DESCRIPTION/CALCULATIONS	OUTPUT

Where $V_s = V_C S_2 S_3$

For this particular structure

$S_1 = 1.0$, $S_3 = 1.0$ (corresponds to an excess in speed occurring once in 50 yrs)

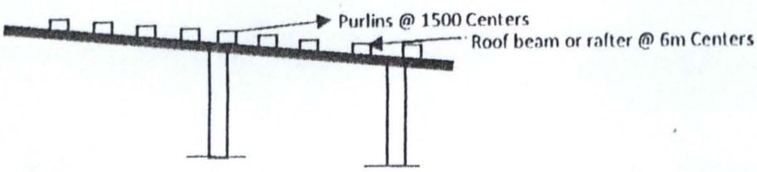
Description	Hm	S_2	$V_S = VS_2Y_3$	W_k KN/m^2
Roof	9 – 12	0.794	37.56	0.86

Dynamic wind pressure W_K

$0.613\,(47.3 \times 0.794)^2 = 0.86KN/m^2$

Design of cantilever beam (Roof beam 1) inclined to an angel of 10^o

Analysis of the cantilever beam



-purlins 100 x 75 x 15.4kg/m @ 1500 centres

Corrugated Aluminium roofing sheet

<u>Loading on Purlin</u>

S/w of Aluminium- 2.44x9.81x1.5/1000 =0.0360kN/m

S/w of purlin =15.4x9.81/1000 = <u>0.15kN/m</u>

Total dead load 0.186kN

Imposed Load

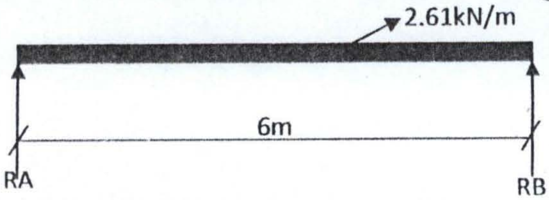
Roof without access except for maintenance = $0.75kN/m^2$

Total imposed load = 0.75 x1.5 1.125kN/m

Wind load from wind analysis <u>$=0.86kN/m^2$</u>

Design load 2.171x1.2 $=2.61kN/m^2$

Since the cantilever roof beam or rafters are @ 6m spacing



RA = RB = wl/2 = 2.61 x 6 /2 = 7.83kN

The No. of purlins on the cantilever roof beam or rafter.

REF	DESCRIPTION/CALCULATIONS	OUTPUT

Fd = load on any bolt due to moment

D = diameter of bolt

T= thickness of gusset plate

Strength in bearing of bolt

0.8fbDt =0.8x200x24x12 = 46.1kN

Safe load = 36.2kN

No. of bolt required56.61/36.2 = 1.6 = 2 bolts

Provide also gusset angles with welded connection having a length of 200mm between intersections roof beam 1,column 1 and column2A

L = 200mm, Try 200 x 200 x 24

Area = $90.6cm^3$

$\delta_v = 3.90$

$L/\delta v = 200 \times 0.85/3.9 \times 10 = 4.4$ $P_c = 85$

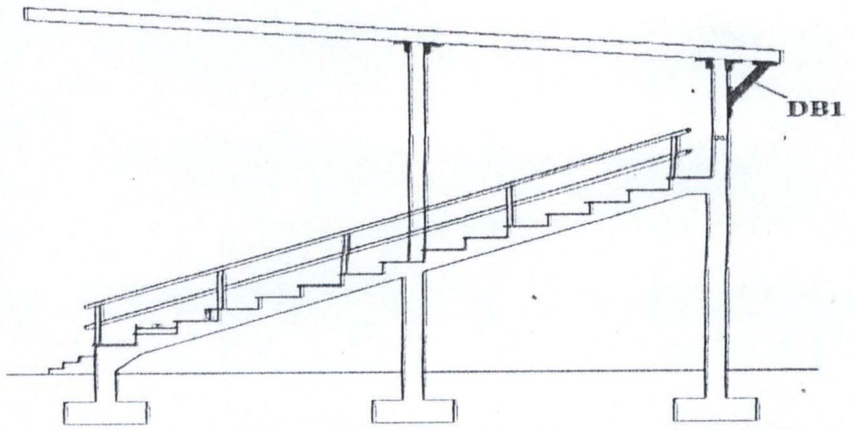
Allowable stress = $0.85 \times 85 = 65N/m^2$

Actual stress = $56.61 \times 10^3/90.6 \times 10^2 = 6.25N/m^2$

Adequate

Vertical diagonal (DB$_1$) bracing

Diagonal bracing will be in tension, force on the diagonal member = 47.785kN



Max Force in diagonal bracing =2.83 x 56.61/2 = 80.1kN

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Where 2.83m is the length of the DB_1	
	Net Area = 80.1x $10^3/265 = 3.023cm^2$	
	Try 254 x 102 x 22kg/m (UB) = (Area = $28.4cm^2$)	
	Check	
	$f_{bt} = 80.1x\ 10^3/28.4 \times 10^2 = 28.2N/mm^2$ o.k.	
	<u>Connection of the DB_1 to the Cantilever beam (1) and the Column C_1</u>	
	Use 12mm plate as calculated before, welded to the force of diagonal brace (DB_1) and bolted with 22mm diameters block bolt.	
	3.6 <u>Design of Column C_2 and C_3 (Concretè)</u>	
	<u>Durability and fire resistance</u>	
	Nominal Cover for mild exposure = 25, 30mm cover to main reinforcement, fire resistance exceeds 1 hour.	
	1. Column C_3 (300 x 300mm)	
	b = 300, h = 300, d = 300 – 30- 20/2 - 10 = 250	
	d/h = 250/300 =0.83	
	Column C_3 is axially loaded Column, = 1500mm	
	Effective Column height Le = B_{lo}	
	1.2 x 1500 =1.800m	
	Le/h = 1800/300 = 6 < 10 Design as short column	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	<u>Loading</u> Load from Beam 3 = 186.12 KN/m Load from Beam 4 = 186.12kN/m 372.24 x 6 = 2,233.4/2 = 1116.72kN Self weight of column = 0.3^2 x 1.5 x 24 x 1.4 = 4.53kN Total Load 1121.24kN A_{sc} = N – 0.4fcuA_c/0.8f = 1121.24 x 10^3 - (0.4 x 30 x 300^2)/0.8 x 460 A_{sc} = 418.40 /368 = 121.060mm^2 Provide 4 Y_{20} bar, Area = 1260mm^2 Minimum design moment = 0.05Nh M = 0.05 x 1121.24 x 0.3 = 16.82KN/m N/bh = 1121.24 x 10^3/300 x 300 = 12.45N/mm^2 M/bd^2 = 16.82 x 10^6/300 x 300^2 = 0.62N/mm^2 Area of minimum reinforcement is A_S = 1% of cross sectional area of column = 1/1000 x 300 x 300 = 900mm^2 Provide 4Y_{20} bar, Area = 1260mm^2	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	<u>Spacing of Links</u> Minimum size 1/4 x 20 bars (A_{sc} prov) =1/4 x 20 = 5 < 6 Spacing, 12 x 20 = 240mm Provide R_{10} @ 200 c/c Check for minimum and maximum area of steel $100A_s/A_{col}$ = $100X1260/300^2$ =1.4 > 0.4 Area o.k. <u>Column C_2 (300 x 300mm)</u> B = 300, h = 300, d = 300 - 30 -- 25/2 - 10 = 247.5mm Column C_2 is axially loaded column,l0 = 2400mm Effective Column height le = B_{lo} 1.2 x 2400 = 2.88mm Le/h = 2880/300 = 9.6 < 15 Design as a short column <u>Loading</u> Load form Beam 2 & 4 = 186.12 x 2 x 6/2 = 1116.72kN Self weight of Column = 0.3^2x2.88x24x1.4 = 8.71kN Load form stanchion C_2 = <u>59.11KN</u> Total designed load 1184.50KN	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Reinforcement $A_{SC} = N - 0.4f_{cu}\ A_S/0.8fy$ $A_{SC} = 1184.50 \times 10^3 - (0.4x30x300^2)/0.8x460 = 283.91mm$ Minimum design moment = 0.05Nh M= 0.05 x 1184.50x 300/1000 = 17.77KN/m $N/bh = 1184.50x\ 10^3/300 \times 300 = 13.16.00N/mm^2$ $m/bh^2 = 17.77x10^6/300 \times 300^2 = 0.66N/m^2 = 0.66N/m^2$ Area of minimum reinforcement is 1% of cross-Sectional area of column= 1/100 x 300 x 300 $= 900mm^2$ Provide $4Y_{20}$ bars, Area $= 1260mm^2$ Design of Column C_{1A} Section = 300 x 300mm b = 300, h = 300mm d = 300-30 - 20/2-10 = 200 Column is axially loaded Column, lo = 2.4m Effective Column length le = B_{lo} 1.2 x 2.4 = 2880 Le/h = 2880/300 = 96 < 10 Loading Load from cantilever beam = 6.29 x (3+ 2) = 31.25KN Load from Roof beam 2 = 3.47 x 6 = 20.82kN Self weight of the Column=0.3x0.3x24x1.4x2.4 = 7.26kNm Total Design Load 59.53kNm	C = 30 $\phi = 20$

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	<u>Reinforcement</u> $A_{SC} = N - 0.4f_{cu}\ A_c/0.8fy\ \ A_{SC}$ $= 59.530x10^3 - (0.4x30x300)/0.8 \times 460 = 2773mm^2$ Minimum design moment = 0.05Nh $M= 0.05 \times 2773mm^2 \times 300/100 = 41.60N/mm^2$ $m/bh^2 = 41.60x10^{6-}/300x300^2 = 1.54N/mm^2$ $N/bh = 2773 \times 10^3/300^2 = 30.81N/mm^2$ Area of minimum reinforcement is 1% of cross-Sectional area of column $1/100x300x300=900mm^2$ Provide $4Y_{20}$ bars, Area $= 1260mm^2$ <u>Design of Column C_1</u> Section = 300 x 600mm b = 300, h = 600mm d = 600-30 – 25/2-10 = 547.5mm Column is axially loaded Column, lo = 4.5m Effective Column length Le = B_{lo} 1.2 x 4.5 = 5.400m Le/h =5400/600 = 9 < 10 <u>Loading</u> Load from beam 1 & 4=186.12x2x6/2 = 1116.72kN/m -Self weight of Column = 0.6x0:3x4.5x24x1.4=27.21kN/m Load from Column C_{1A} = <u>59.11kNM</u> Total Design Load 1203.04kNm	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	<u>Reinforcement</u>	
	$A_{SC} = N - 0.4f_{CU}\ A_C/0.8fy\ -0.4fcu$	
	$A_{SC} = 1203.04x10^3 - (0.4x30x600x300)/0.8x460\text{-}0.4x30$	
	$A_{SC} = N - 2627.58mm^2$	
	Minimum design moment = 0.05Nh	
	M= 0.05 x 1203.04x600/1000 = 36.10kN/m	
	N/bh = 1203.04 x 10^3/600x300 = 6.68N/mm^2	
	$M/bd^2 = 36.10x10^6/600x300^2$ = 0.67N/mm^2	
	Area of minimum reinforcement is1% of cross sectional area of column = 1/100x300x600=1800mm^2	
	Provide 6 Y_{25} bars, Area = 2950mm^2	
	<u>Spacing of links</u>	
	Minimum size =1/4 x 25 = 6.25 > 6	
	Maximum spacing = 12 x 25 = 300	
	Provide R_{10} @ 250c/c	
	<u>Foundation Design</u>	
	Description: Foundation is a sub-structural member supporting the entire super-structural member i.e. columns, beams and slabs and transmitting the loads to the soil below. In this particular design the type of foundation to be used are the isolated pad foundation	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	1. Foundation type C, BTC (Isolated pad footing) Self weight of the base = 5% of the total column load or multiply by a factor of 0.5 (assumed) <u>Loading</u> Load from Column (unfactored load) C_3 N = 1121.23kN, 5% of 1121.23 = 56.06 Total Load 1177.3KN Permissible soil bearing capacity (pressure), $Ps = 200KN/m^2$ Column Size (300 x 300mm) $a_1 = 300mm$, $a_2 = 300mm$, cover C = 40mm Depth of base h = 600 (assumed) d = 600 – 40 – 20/2 = 550mm Required base area = column load x $1.0/P_{net}$ = 1177.3x10/200 = 5:9m Try base area 2.45 x 2.45 = $6m^2$ Base Area $A_b = 6.0^2$ (b) For the ultimate limit state Column design axial load 1.4Gk + 1.6Qk = 1177.3kN Earth pressure = $1177.3/2.45^2 = 196.14kN/m^2$	

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	(c) At the Column face Shear stress V_C = N/(col. Perimeter x d) $1177.3 \times 10^3/1200 \times 550 = 1.8 N/m^2 < 0.8\ f_{cu}$ (d) punching Shear Critical perimeter = col. Perimeter +8 x 1.5d =4 x 300 + 12 x 550 = 7800mm Area within perimeter = $(300 + 3d)^2 = 3.8 \times 10^6 mm^2$ Therefore, punching shear force V =196.14 $(2.45^{2-}3.8)$ = 432.00kN Punching Shear Stress V = V/perimeter x d $432.00 \times 10^3/7800 \times 550 = 0.10 N/mm^2$ From Table 5.1 this ultimate Shear Stress is not excessive, therefore h = 600mm will be suitable (e) Bending reinforcement At the column face, which the critical section M = (196.14x2.45x1.075)x1.075/2 = 278.00kNm For the concrete $M_u = 0.156 fcu\ bd^2$ $M_u = 0.156 \times 30 \times 2450 \times 550^2 \times 10^{-6} > 278.00 kNm$ $K = 278.00 \times 10^6/2450 \times 550^2 \times 30 = 0.013$ La = 0.95, Z = 0.95 x 550 = 527.25mm $A_s = m/0.95fyZ = 278.00 \times 10^6/0.95 \times 460 \times 527.25$ $= 1206.6 mm^2$	

REF	DESCRIPTION/CALCULATIONS	OUTPUT

Provide 10 T_{20} bars @ 200, Area 2510mm

$100\ A_s/bh = 100 \times 2510/2450 \times 600 = 0.14 > 0.13$

As required by code

The spacing of 200mm is ok, as the max. spacing is 750mm as provided by the code. (BS8110)

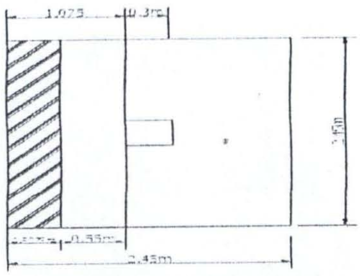
(f) final check of the punching shear

$100\ A_s/bd = 100 \times 2510/2450 \times 550 = 0.19 < 0.4 N/mm^2$

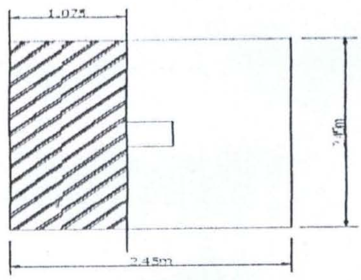
V_c = ultimate shear stress for a concrete strength of $30 N/mm^2 = 0.37\ N/m^2$

Punching Shear Stress is outside the area, therefore a 600mm thick pad is adequate

(g) Shear Stress



(a) Shear



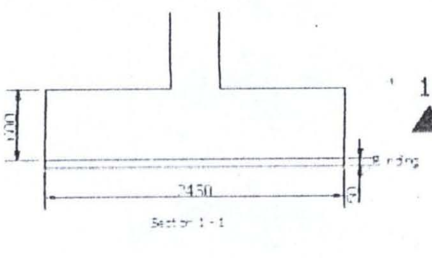
(b) Bending

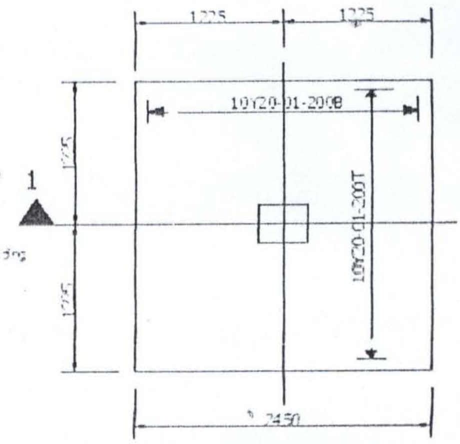
At critical section for shear, 1.0d from the col. Face

$V = 196.14 \times 2.45 \times 0.5375 = 258.3KN$

$v = V/bd = 258.3 \times 10^3/2450 \times 550 = 0.192 < 0.4$

Therefore the section is adequate



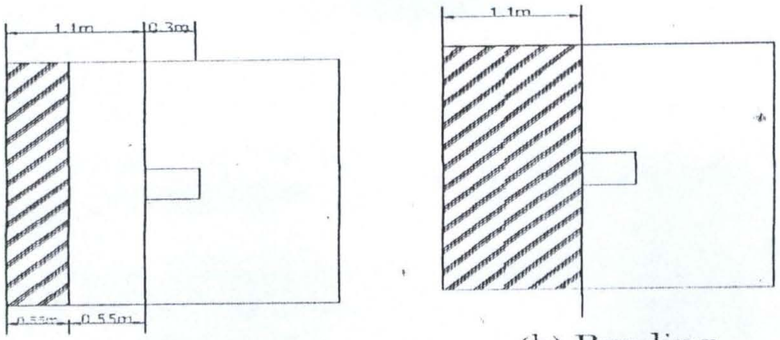


<table>
<tr><th>REF</th><th>DESCRIPTION/CALCULATIONS</th><th>OUTPUT</th></tr>
<tr><td></td><td><u>Base type B, BTB</u>
(2) Foundation type: BTB (Isolated pad footing)
Self weight of the base =5% of the total column load
-Load from Colum C_2 (unfactored load) 1184.50kN
-Selfweight weight = 59.23kN
Total Load = 1243.73kN
Col. Size (300 x 300)
Cover = 40mm
d = 600 – 40 – 20/2 = 550mm
Permissible soil bearing capacity (pressure)
$P_{net} = 200kN/m^2$
Required base = 1243.73 /200 = 6.22^2
Try base area $A_b = 2.5^2$
(b) for the ultimate limit state
Col. Design axial load
Earth pressure -= 1243.73 /2.5^2 = 199.0kN/m^2
(c) At the column face
Shear stress V_C = N/col. Perimeter x d
1243.73 x 10^3/1200 x 550 = 1.88N/m^2 < $0.8\sqrt{fcu}$
(d) Punching Shear
Critical perimeter = Col. Perimeter + 8 x 1.5d
= 4 x300 + 12 x 550
= 7800mm
Area within Perimeter $(300 + 3d)^2 = 3.8 \times 10^6 mm^2$
Therefore punching shear force V = 199.0 (2,5^2-3.8)
= 487.55kN</td><td>ϕ = 20
C = 40
h = 600
d = 550</td></tr>
</table>

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	$487.55 \times 10^3 / 7800 \times 550 \quad = 0.11 N/mm^2$	
	From table 5.1, this ultimate shear stress is not excessive,	
	therefore h = 600mm will be suitable.	
	(e) Bending reinforcement	
	At the col. Face which is the critical section	
	$M = (199.0 \times 2.5 \times 1.10) \times 1.1/2 = 300.99kNm$	
	For the concrete	
	$M_U = 0.156$ fcu bd^2	
	$M_U = 0.156 \times 30 \times 2500 \times 550^2 \times 10^{-6}$	
	$= 3539.25 > 300.99kNm$	
	$K = 300.99 \times 10^6 / 2500 \times 550^2 \times 30 = 0.013$	
	$La = 0.95, Z = 0.95 \times 550 = 527.25$	
	$A_S = m/0.95fyz = 300.99X10^6/0.95X460X527.25$	
	$= 1306.33mm_2$	
	provide $10T_{20}$ bars @ 200, area 2510mm	
	$100 A_s/bh = 100 \times 2510 \times 2500 \times 600 = 0.17 > 0.13$	
	As required by code	
	the spacing of 200mm is o.k.	
	f) Final check of the punching shear	
	$100A_s/bd = 100 \times 2510/2500x550 = 0.18 < 0.4N/m^2$	
	Punching Shear Stress is outside the area	
	Therefore 600mm thick pad footing is adequate	

REF	DESCRIPTION/CALCULATIONS	OUTPUT

g)



(a) Shear (b) Bending

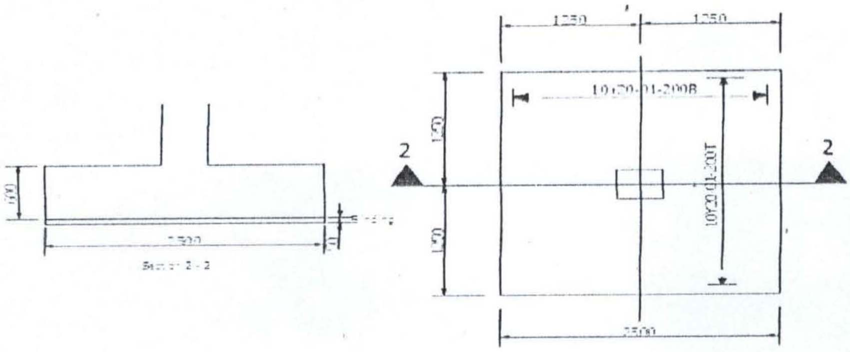
@ Shear

At critical section for shear, 1.0d from the Col. Face

V = 199.00x 2.5 x 0.55 = 273.63

$v = V/bd = 273.63 \times 10^3 / 2500 \times 550 = 0.2 < 0.4$

Therefore the section is adequate



Base Type A, BTA

(3) Foundation type: BTA (Isolated pad footing)

Selfweight of the base = 5% of the total column load

-Load from Column C_1 (unfactored load) 1203.04kNm

-Self weight of base = 60.15kN

-Total load = 1263.20kN

Permissible Soil bearing capacity (pressure), PS = $200kN/m^2$

Column size (300 x 600)

$a_1 = 300$, $a_2 = 600mm$, Cover = 40mm

Depth of base h = 600 (assumed)

D = 600 – 40 – 20/2 = 550mm

REF	DESCRIPTION/CALCULATIONS	OUTPUT
	Required base area = Col. Load x 1.0/Pnet	
	= 1263.2/200 = 6.32	
	Try base area 2.6 x 2.6 = 6.76^2	
	Base Area Ab = 6.76^2	
	b) for the ultimate limit state	
	Col. Design axial load	
	1.4Gk + 1.6Qk = 1263.2	
	Earth pressure = $1263.2/2.6^2$ = $186.86kNm^2$	
	c) At the column face	
	Shear stress V_C = N/(Col. Perimeter x d)	
	$1263.2 \times 10^3/180 \times 550 = 1.31N/m^2 < 0.8\sqrt{fcu}$	
	(d) Punching Shear	
	Critical Perimeter = Col. Perimeter + 8 x 1.5d	V =442.24kN
	= 1800 + 12 x 550	
	= 8400mm	
	Area with perimeter = (300 + 3d) (60 + 3d)	
	$= 4.39 \times 10^6$mm	
	Therefore	
	Punching Shear Force V = $186.86 (2.6^2 - 4.39)$	
	= 442.24kN	
	Punching Shear Stress V_S = V/perimeter x d	
	$= 442.24kN \times 10^3/8400 \times 550 = 0.1N/mm^2$	
	From table 5.1 The ultimate shear stress is not excessive	
	therefore h = 600mm will be suitable	

REF	DESCRIPTION/CALCULATIONS	OUTPUT

(b) Bending reinforcement

At the column force which is the critical section

$M = (186.86 \times 2.6 \times 1.15) \times 1.15/2 = 321.26kNm$

For the Concrete

$M_U = 0.156\ fcu\ bd^2$

$M_U = 0.156 \times 30 \times 2600 \times 550^2 \times 10^{-6}$

$= 3680.82 > 321.26kNm$

$K = 321.26\ kNm \times 10^6/2600 \times 550^2 \times 30 = 0.014$

$La = 0.95,\ Z = 0.95 \times 550 = 527.25mm$

$A_s = M/0.95fyZ = 321.26 \times 10^6/0.95 \times 460 \times 527.25$

$= 1394.31mm^2$

Provide 12 T_{20} bars @ 200 centers, Area $2510mm^2$

$100\ A_s/bh = 100 \times 2510/2600 \times 600 = 0.16 > 0.13$

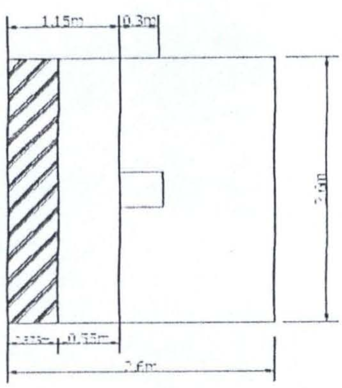
(c) Final check of the punching shear

$100\ A_S/bd = 100 \times 2510/2600 \times 550 = 0.18 < 0.4N/mm^2$

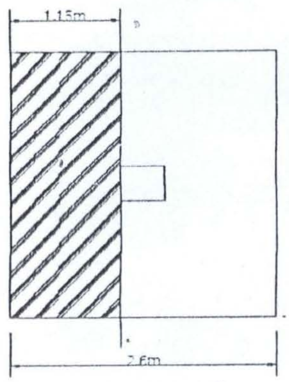
V_C = ultimate shear stress for a concrete strength of

$30N/m = 0.37N/m^2$

Punching Shear Stress is outside the area, therefore a 600mm thick pad is adequate.



(a) Shear



(b) Bending

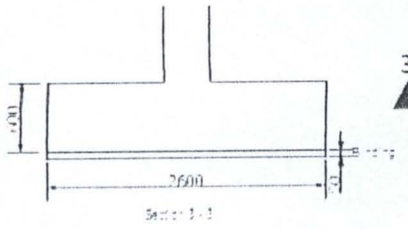
REF	DESCRIPTION/CALCULATIONS	OUTPUT

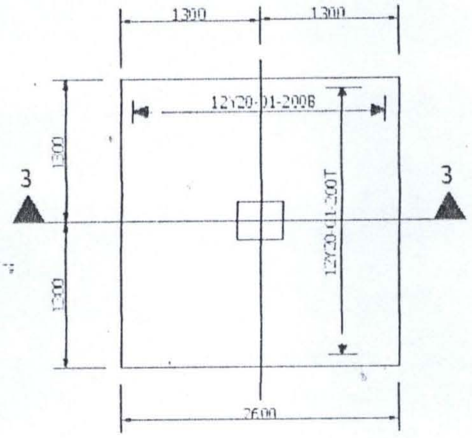
At critical section for shear 1.0d from col. Face

$V = 193.0 \times 2.6 \times 0.575 = 297.5\text{kN}$

$V = v/bd = 297.5 \times 10^3/2600 \times 550 = 0.2 < 0.4$

Therefore the section is adequate.





CHAPTER FOUR

4.0 RESULT AND DISCISSION

4.1 RESULT

The spectators pavilion is designed in accordance with BS8110 for reinforced concrete and steel sections in accordance to BS5950. The structural area is chosen based on the sitting capacity of 1500. The slab which is designed in form of stair slab has 17 square equal panels of 6m by 6m with waist of 250mm, thread and a riser of 800mm and 300mm which is reinforced with Y12 diameter bars as the main reinforcement bars and Y10 diameter bars as the distribution bars, this were chosen based on the analysis, dead and imposed load.

Reinforce concrete beam design consists primarily of producing member details which will adequately resist the ultimate bending moment, shear forces and tensional moments. A 500 x 300mm beam size is chosen because of the high moment in the midspan and shear force check. This also accounts for the high reinforcement of Y25mm diameter reinforcing bars. A steel section member were chosen as the cantilever beam because of ease of construction

In the foundation design, isolated pad foundations were designed because they are spaced at distances apart.

4.2 DISCUSSION

Steel structures are compound of elements which are rolled to a basic cross section in a mill and worked to a desire size and form in a fabricating shop or site. A significant difference between steel and reinforced concrete construction is that the designer has more control over the shape of reinforced concrete element for building a steel structure. The designer is normally compelled to use standard rolled sections. In steel structure buildings, the various elements should be compactable at the joints, if a large member of different shapes and sizes of element are designed or chosen, it will be practically difficult to fit the member and connections will be problem.

It is axiomatic that the designer of any engineering undertaking ought to have a fair understanding of the nature of the materials he or she proposes to use. Steel structure when placed in exposed conditions are subjected to corrosive, therefore, they require frequent painting.

The design of steel section is governed by cross sectional and section modulus, also by availability of the section in the market which becomes a major consideration. Another factor governing the choice is the ease with which the section can be connected. The issue of choice of section , connection and stability of the structure makes it difficult to bring out the best aesthetical nature of the structure because of section which are rolled into different shape, this makes it difficult to machined some part of the structure to the desired shape, but in reinforce concrete design , the designer has more control over the shapes, because of the nature of the materials that make –up the reinforced concrete structure. The finishing of structural steel consist of the rolling of the steel shapes, the fabricating of the shapes for the particular job (including cutting to the proper diameter and punching the hole necessary for field connection) and their erection, very rarely will a company perform only one or two of them which makes it difficult to get the real architectural shapes required and expensive if gotten.

The designer needs to keep in mind the factor that can lower cost without sacrifice of strength.

From the design work and comparison made, it has shown that the size of structural element of steel is smaller compare to that of reinforced concrete. This is due to the height strength of steel per unit weight this determines the choice of using steel structure as a stanchion in column (C_{2A}) more so, the properties of steel do not change appreciately with time as those of a reinforced concrete structure. I.e. steel members can withstand test of time. These facts are of great importance for long span building and tall building of this kind.

CHAPTER FIVE

5.0 CONCLUSSION AND RECOMMENDATION

5.1 CONCLUSSION

The aim of the structural designer is to produce a safe and economical structure to meet certain functional and aesthetical requirement. In other to achieve this goal the designer must have a thorough knowledge of the properties of materials, structural behaviour, structural analysis and mechanics and the correlation between the layout and the function of a structure.

The type of structure is selected on the basis of functional economic service and aesthetic requirement. Similarly, other considerations such as customer's wishes, designer's preferences or establishment precedents dictate the type of structures to be adopted. It is often necessary to investigate several layouts and final choice is made only after several comparative designs are fairly well advanced.

Many people who are superstitious do not discuss flat tyres or make their will because they fear they would be tempting fate. These same people would probably not care to discuss the subject of engineering failure. Despite the prevalence of these superstitions there is a need for awareness of the items which have most frequently caused failure in the past, failure is more important than study of past success. Benjamin Franklin supposedly made the observation that "a wise man learns from failure than from success", for this reason, it becomes important to have a comparative discussion and analysis of structures before the selection of type of structure (Elastic, timber, reinforced concrete etc.) method of design (Elastic or limit state design) and layout of structure.

In the cause of this project work some problems were stated around which the project work centered. These problems form the economic and safety implication of using steel design and reinforced concrete design for the structure (cover grand pavilion)

5.2 RECOMMENDATION

The design of structure involves the planning of the structure for specific purpose, proportioning of members to carry load in more economic manner and consideration for erection at site, first the structure should serve the purpose for which it is intended and this is achieved by proper functional planning.

Secondly, it should have adequate strength to withstand direct and induce force to which it may be subjected during its life span. An inadequate assessment of forces and there effects on the structure may lead to excessive deformation and its failure. Therefore, the design of structure include functional planning, acknowledgement of various forces strength of materials and the design method. In addition, the structure should be economically safe to erect.

An attempt has been made in this project to compare reinforce concrete design and steel to virtualize the economic and safety implication of using steel design and reinforced concrete design for the structure. Base on the design work and findings there is need for comparative work in order therefore to make structure economic and safe.

The designer needs to keep in mind the factor that can lower cost without sacrifice of strength. For future work, I therefore recommend that the structure (cover grand pavilion) should be designed in such a way that all the beams, columns be designed as steel section (members) while other structural element be designed as reinforced concrete.

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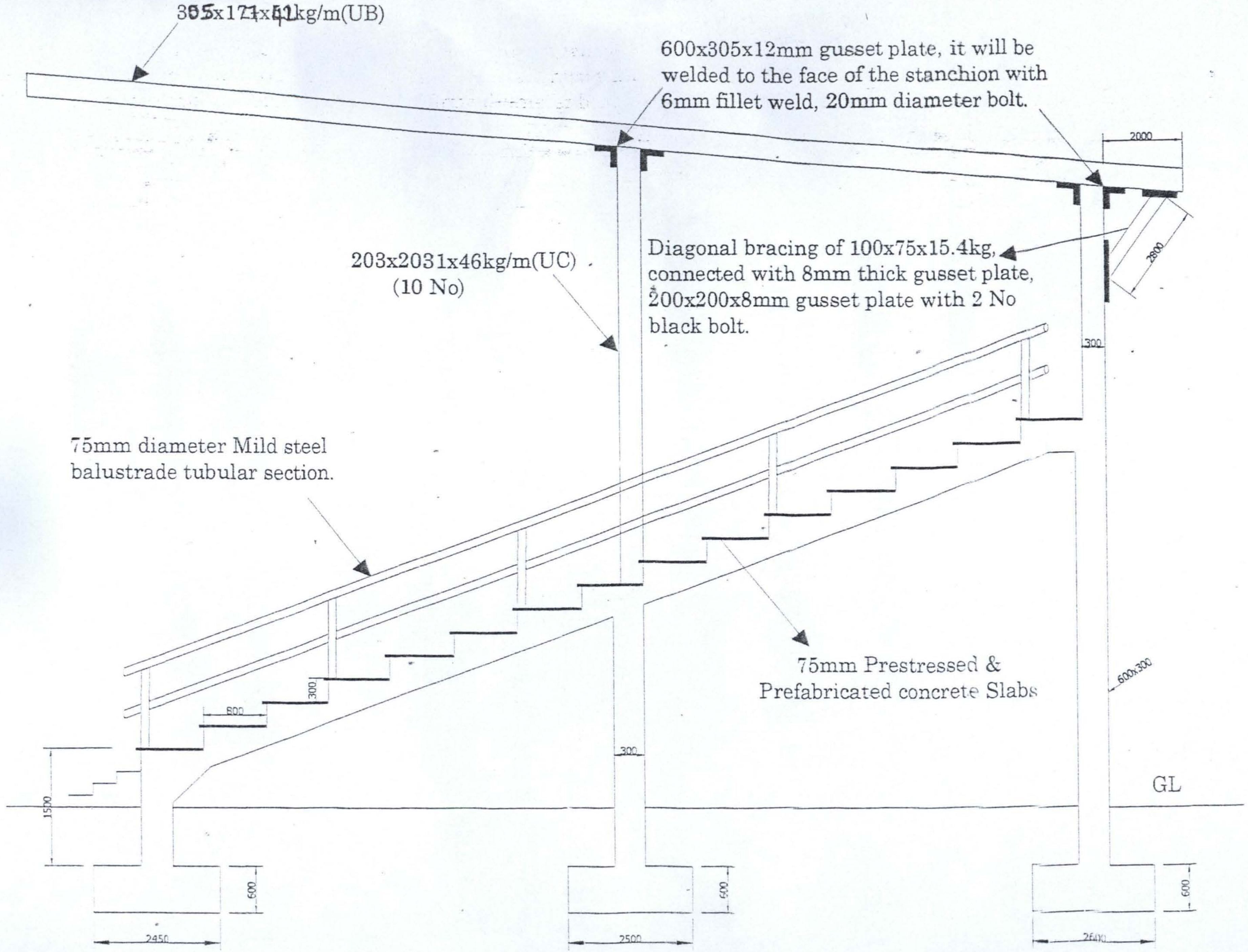
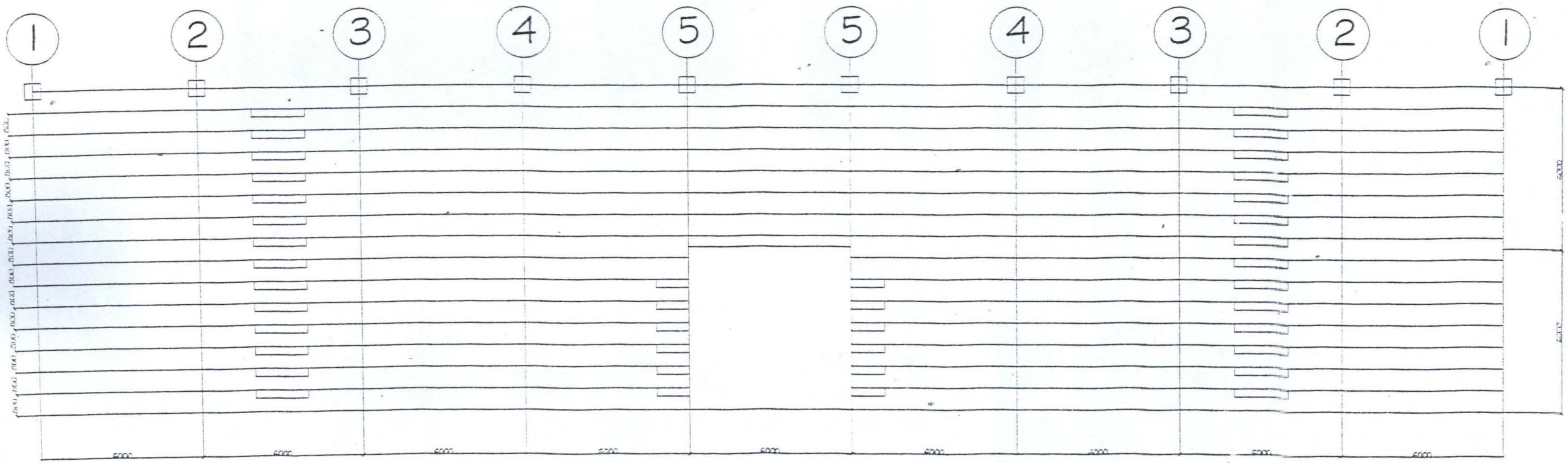
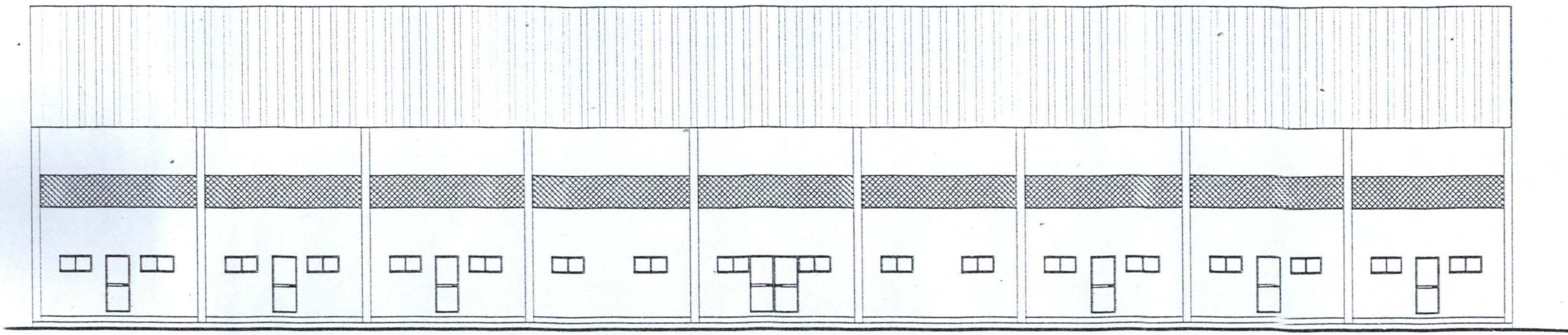

305x171x41kg/m(UB)
600x305x12mm gusset plate, it will be welded to the face of the stanchion with 6mm fillet weld, 20mm diameter bolt.
2000
203x2031x46kg/m(UC)
(10 No)
Diagonal bracing of 100x75x15.4kg, connected with 8mm thick gusset plate, 200x200x8mm gusset plate with 2 No black bolt.
2800
300
75mm diameter Mild steel balustrade tubular section.
75mm Prestressed & Prefabricated concrete Slabs
600x300
300
800
300
GL
1500
600
600
600
2450
2500
2600

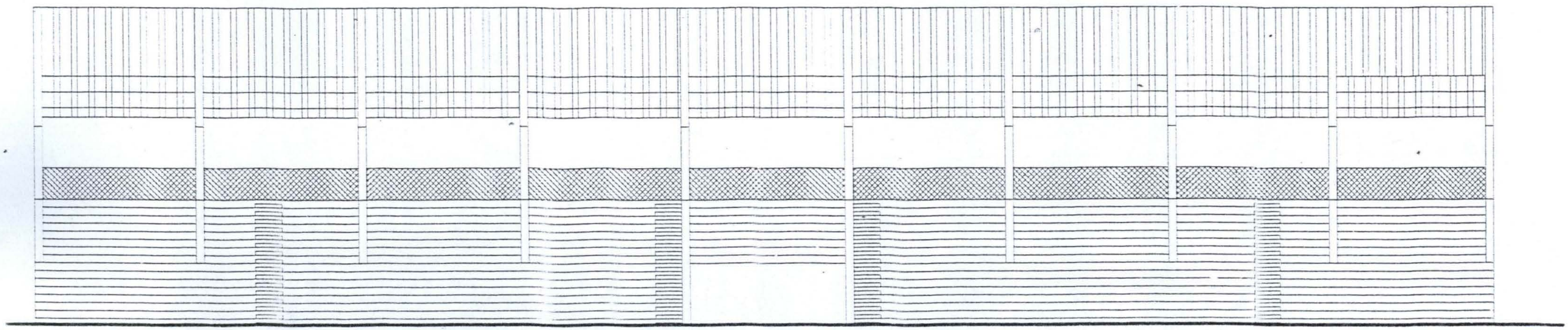


1
2
3
4
5
5
4
3
2
1
6000
6000
6000
6000
6000
6000
6000
6000
6000
6000
6000
6000

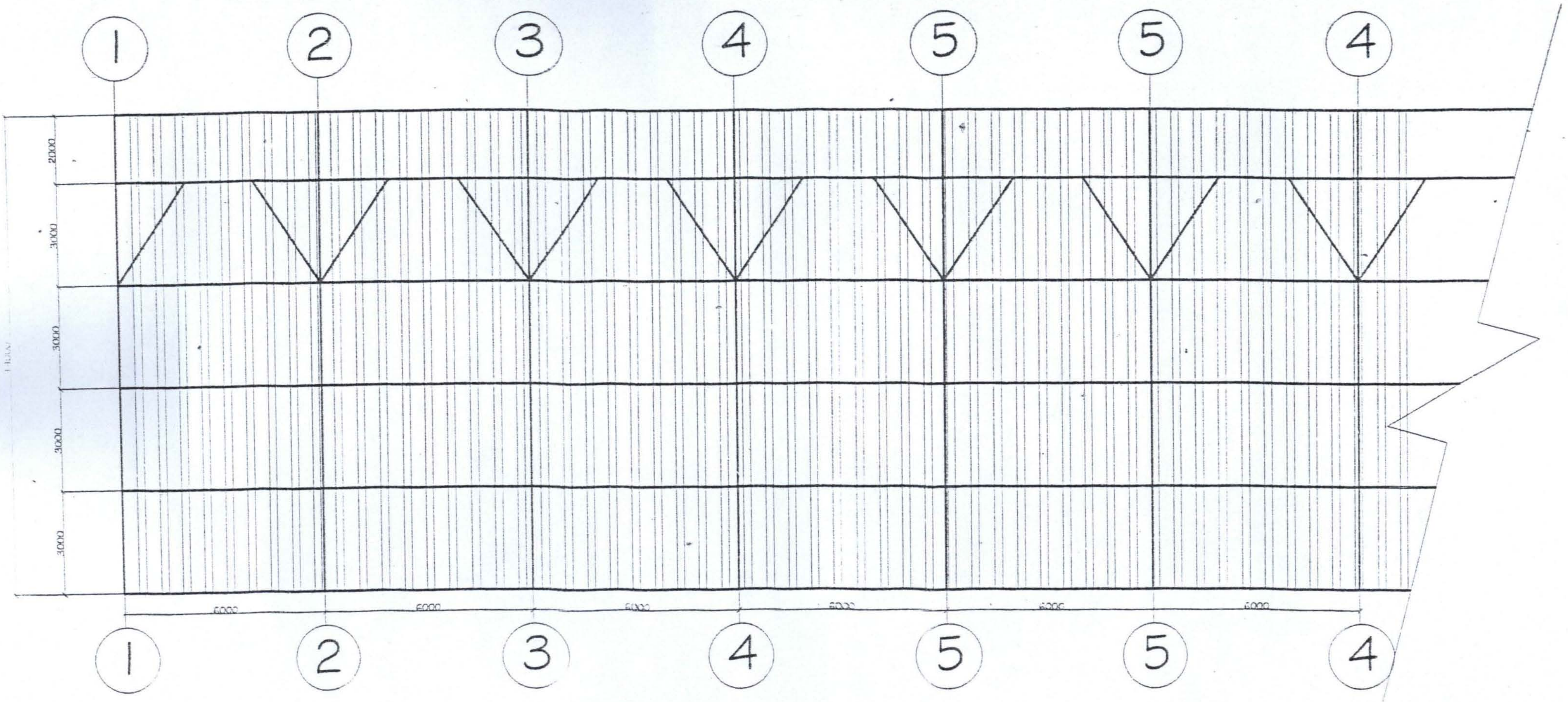
FLOOR PLAN



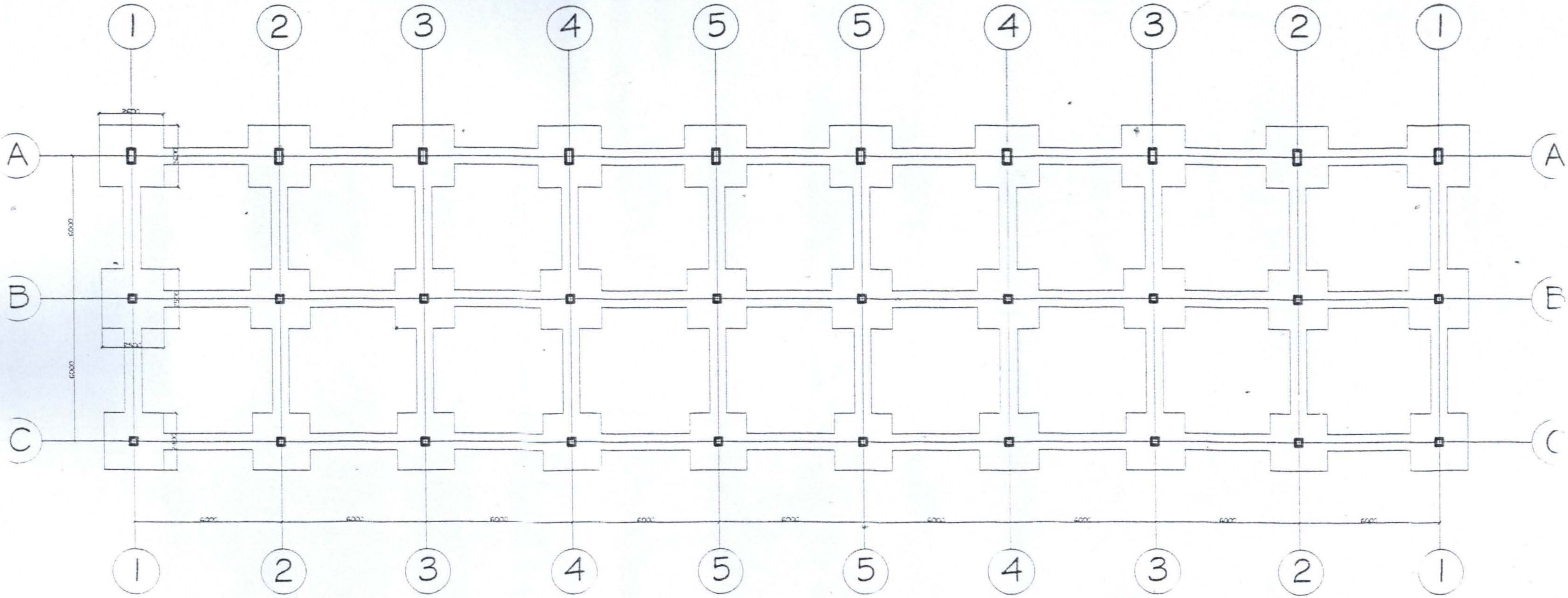
REAR ELEVATION



FRONT ELEVATION


1
2
3
4
5
5
4
2000
3000
3000
3000
3000
6000
6000
6000
6000
6000
6000
1
2
3
4
5
5
4

ROOF PLAN



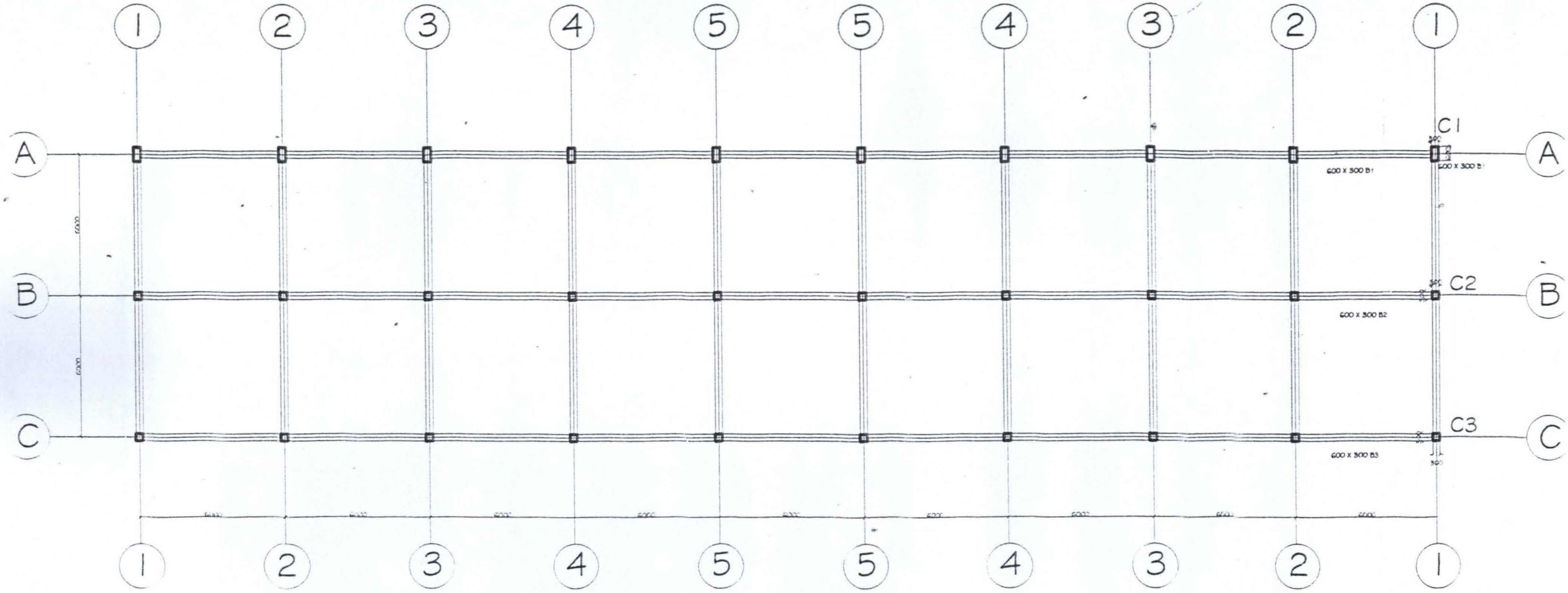
BASE TYPE

BTA = 10 NUMBER
BTB = 10 NUMBER
BTC = 10 NUMBER

COLUMN

C1 = 10 NUMBER
C2 = 10 NUMBER
C3 = 10 NUMBER

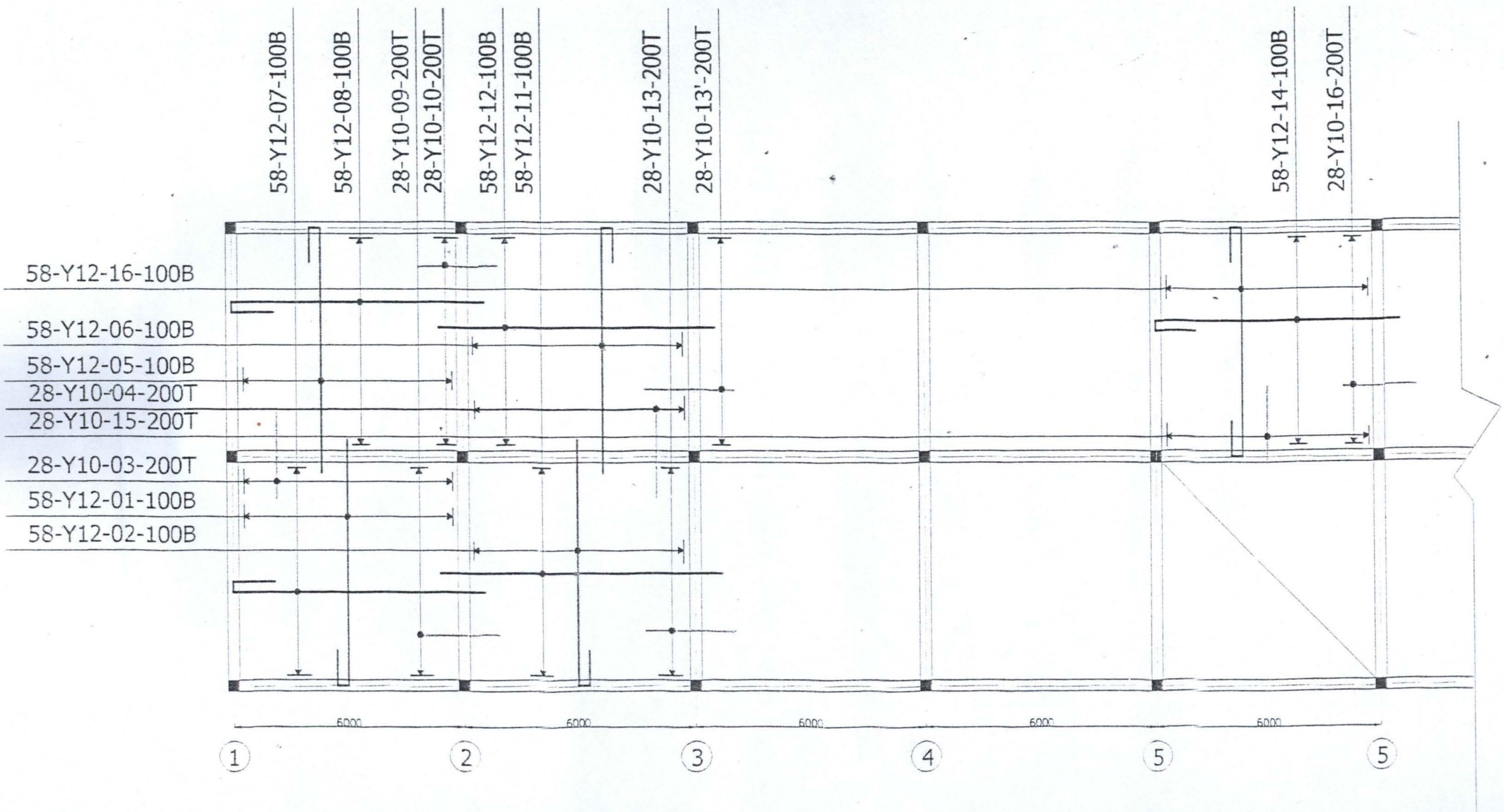
FOUNDATION FOOTING



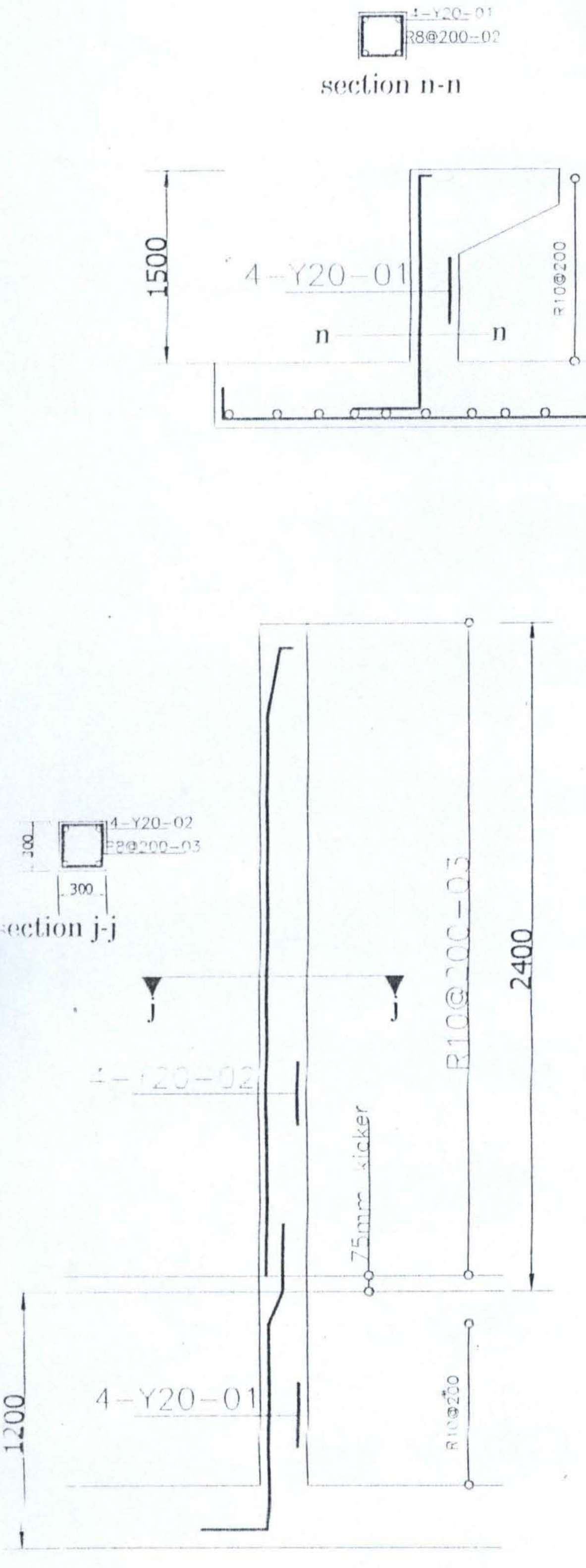
COLUMN

C1 = 10 NUMBER
C2 = 10 NUMBER
C3 = 10 NUMBER

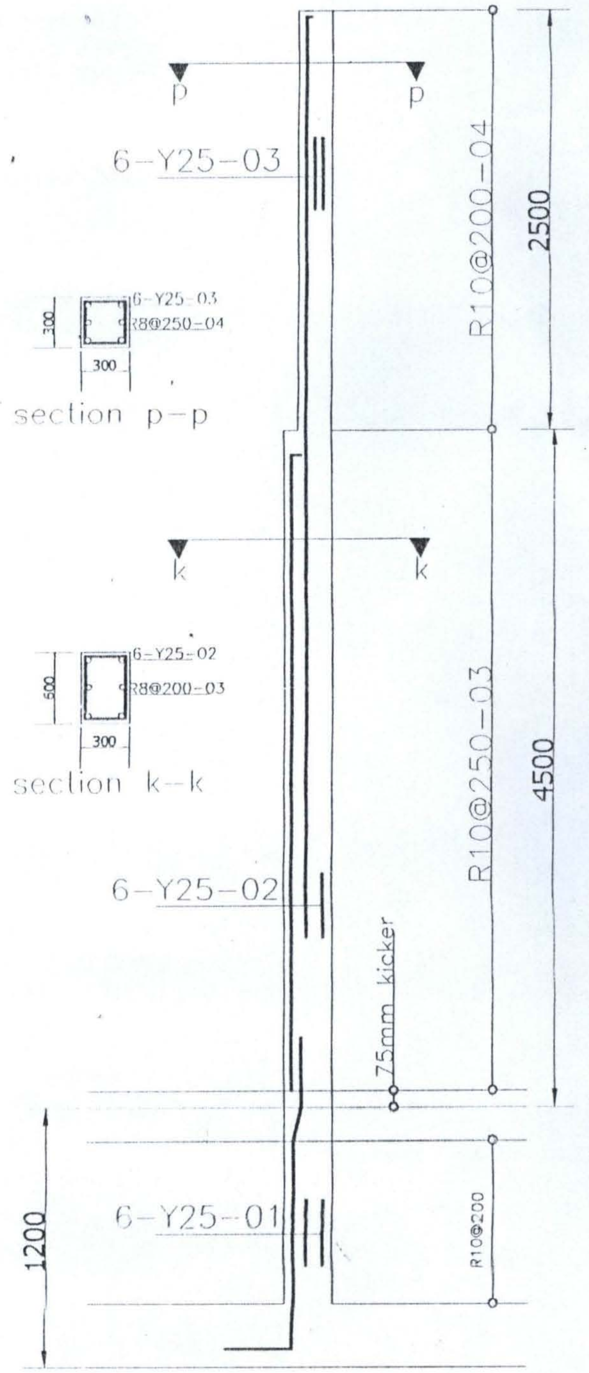
BEAMS



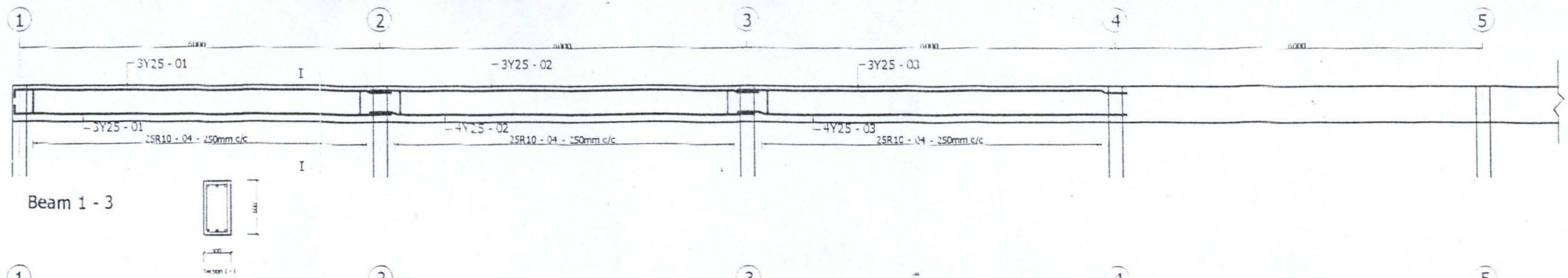
SLAB DETAILING



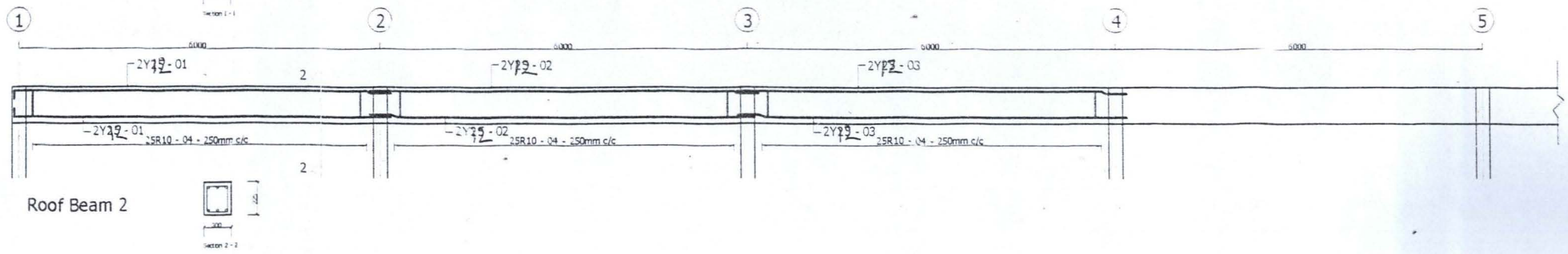
Column C2
(10 No)



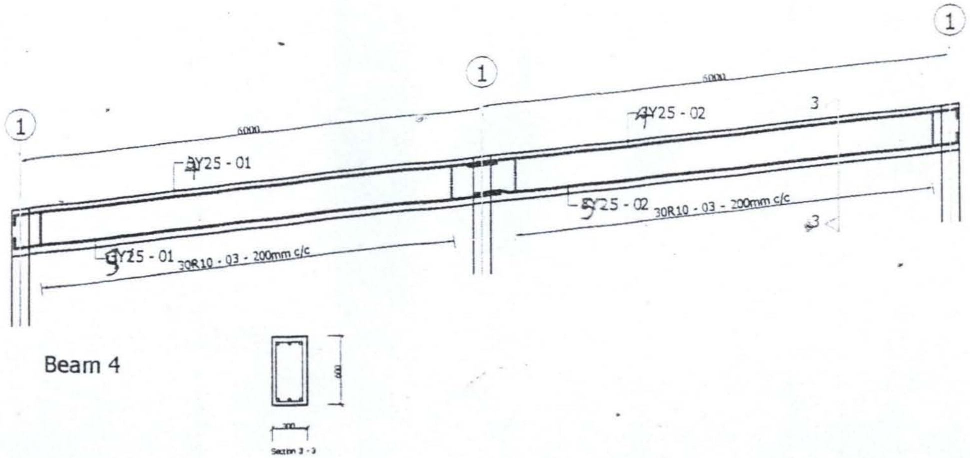
Column C1 & C1A
(10 No)



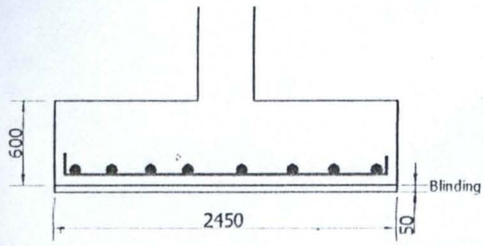
Beam 1 - 3



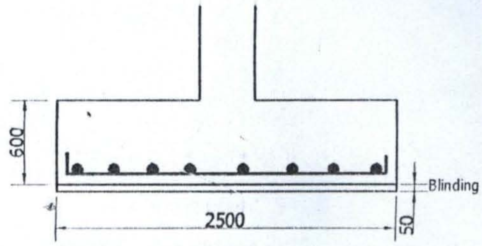
Roof Beam 2



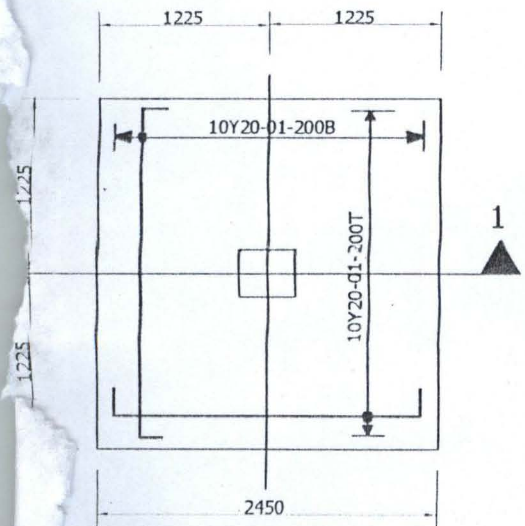
Beam 4



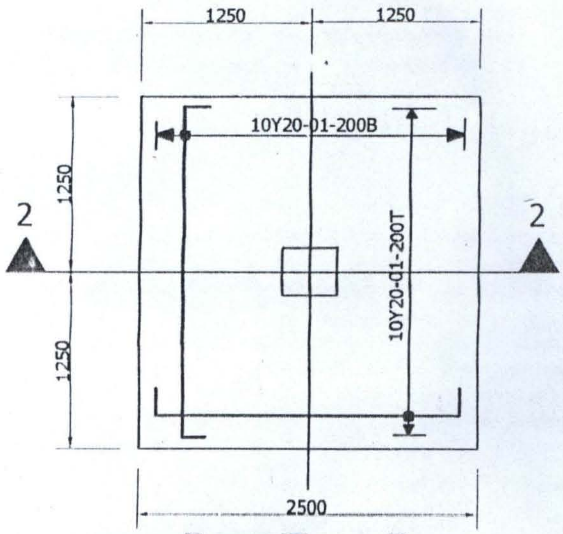
Section 1 - 1



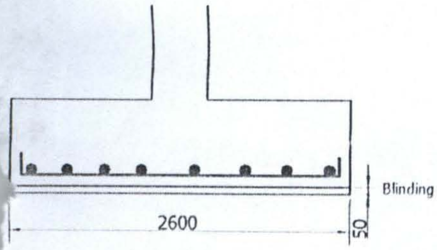
Section 2 - 2



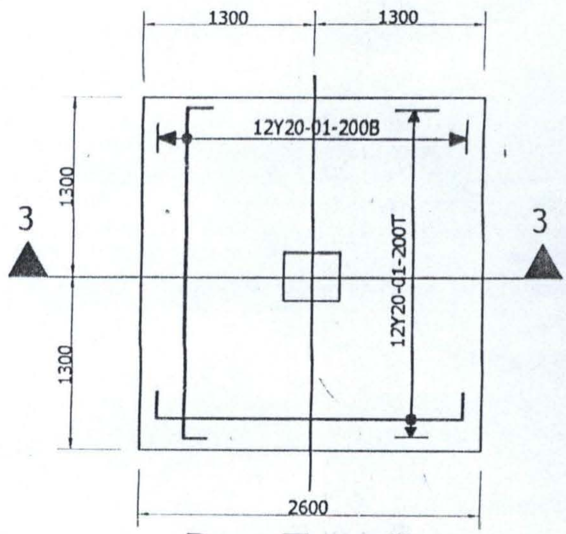
Base Type A
(10 No)



Base Type B
(10 No)



Section 3 - 3



Base Type C
(10 No)