C O M P A R A T I V E D E S I G N F O R A T H R E E S T O R E Y H O S P I T A L B U I L D I N G I N C O M P O S I T E A N D R E I N F O R C E D C O N C R E T E C O N S T R U C T I O N S

BY

ADAMU BALA BAKOURI

A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, AHMADU BELLO UNIVERSITY, IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF MASTER OF SCIENCE IN CONSTRUCTION TECHNOLOGY.

DEPARTMENT OF BUILDING
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D E C L A R A T I O N

I declare that to the best of my knowledge
this type of work has not, previously, been presented
to any University for the award of any kind of degree.

Sign:.....16/2/84...
A. B. BAKOURI

To

HAFSATU and KAMAL

Man, often-times takes refuge in the belief that what he doesn't know is perhaps not good enough and therefore not worth knowing. Man should maintain good respect for what he does not know and what he does not understand.

F O R W A R D

The design procedures in this thesis are based on the British Code of Practice CP 110. The structural use of concrete, 1972, on one hand, and the Draft Code of Practice of British Standard, Composite Construction in Building, in preparation, on the other hand. Both Codes of the British Standard Institution.

The Composite Code, based on plastic method of designed, is as recommended and published in the Constrado Monograph Series. Presently, no codes of practice for composite columns or frames are available, but all the same a few number of buildings are being designed compositely, such as those at the Universities of Warwick and Cambridge. This shows that none availability of code of practice cannot be a reason for rejecting or impracticalising a published research work since sound and economic design depends more on professional competence and responsibility of engineering practice than on code.

It is hoped that the draft code on composite construction will be blessed within a foreseable future so that practical application of the method could be exploited more confidently.

ZARIA.
February, 1989

A C K N O W L E D G E M E N T

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Numerous friends of mine whose names are not mentioned my sincere salutation goes to them.

A B S T R A C T

A three story hospital building has been designed in composite method and in reinforced concrete one. A critical comparative study between the two methods shows that design by composite method is easier, less complex and takes shorter time. Composite method also resulted in shallower beam depth. Two identified disadvantages of composite method is that , constructionally, it resulted in heavier structural elements and ultimately increasing the overall weight of the hole building. Secondly, that the method resulted in a more costly structure.

The report concludes that for a lightly loaded building and for a building involving not more than three stories the reinforced concrete method is more advantageous. Composite construction can only compete with the reinforced concrete one in heavily loaded buildings and in high-rise buildings. The greater is the number of stories involved, the more economical is composite constructional method.

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N O T A T I O N S

COMPOSITE:

b_f - Steel flange width

t_f - Steel flange thickness

d_w - Web depth

t_w - Web thickness

A_r - Area of steel reinforcement

fyr - Characteristic strength of steel reinforcement

A_s - Area of structural steel

f_y - Characteristic strength of structural steel

f_{cu} - Concrete grade

Y_1 - Depth of Neutral Axis

M_p - Plastic moment of resistance

A_c - Area of Concrete

A_{sc} - Area of structural steel in compression

M_{pn} - Negative plastic moment of resistance

F_r - Force in reinforcement

f_{rd} - $0.87xf_{yr}$

f_{sd} - $0.87xf_y$

d_{wc} - $d-h-t_f$

d_r - Depth of steel reinforcement from surface of concrete

M_{ps} - Plastic moment of resistance of steel alone

Z_p - Sectional modulus of steel

K - Moments ratio

P_d - Design load of shear connectors

P_k - Characteristic strength of connectors

N_u - Column squash load

r - Radius of gyration

q - Concrete contribution factors

I - Moment of inertia

REINFORCED CONCRETE

M_u - Ultimate moment

F - Ultimate load

V - Shear force

L - Member length

h - Depth of member

l_{eff} - Effective length of member

b - Bredth of member

G_k - Dead load

Q_k - Imposed load

A_s - Area of steel

A_{sc} - Area of steel in compression

f_{cu} - Grade of concrete

f_y - Characteristic strength

b_f - Beam flange

h_f - Depth of beam flange

I_{NA} - Moment of inertia

K - Member stiffness

Z - Lever arm

d - Effective depth

A_{sv} - Area of shear links

S_v - Spacing of shear links

b_t - Thickness of beam rib

V - Shear stress

U_{ST} - Total bond stress

f_{bs} - Local bond stress

f_{ul} - Ultimate local bond stress

M_t - Total moment in column

M_i - Additional moment in column

d_c - Concrete cover to reinforcement

c - Eccentricity of load

P_u - Upward Soil pressure

C_p - Critical perimeter

P_{gr} - Ground pressure

L_o - Distance between column centres

CHAPTER ONE

I N T R O D U C T I O N

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1.1. GENERAL

The term composite refers to a system by which different materials are brought together in a structural unit for the purpose of achieving a structure which can withstand an applied load. This includes the use of structural steel and concrete, steel and timber, glass reinforced with wires, and steel bars and concrete in reinforced concrete. Technically, however, reinforced concrete is not regarded as a composite material.

In this work the composite construction considered is that between structural steel and concrete. The elements considered are steel beam interacting with reinforced concrete slab and encased universal column.

1.2. COMPOSITE CONSTRUCTION.

This method has been in existence since early 1920's. Comprehensive international research works covering the period from early experiments were published between 1960 and 1970. Early research works were concentrated on composite beams [2,7]. Later developments showed that composite columns are also possible by encasing a steel section in concrete as opposed to filling steel tube with concrete [5,9].

While the most common form of composite system is that in which solid web steel beams support concrete slabs, other types are also used. These include steel beams with an open web joist, a castelled beams, a truss or a box beam. The concrete slab may rest on top flanges of steel beams or may enclose fully or partially the steel beams. The only requirement is that composite action must be ensured either by means of natural bond between the two materials or by means of suitable shear connectors.

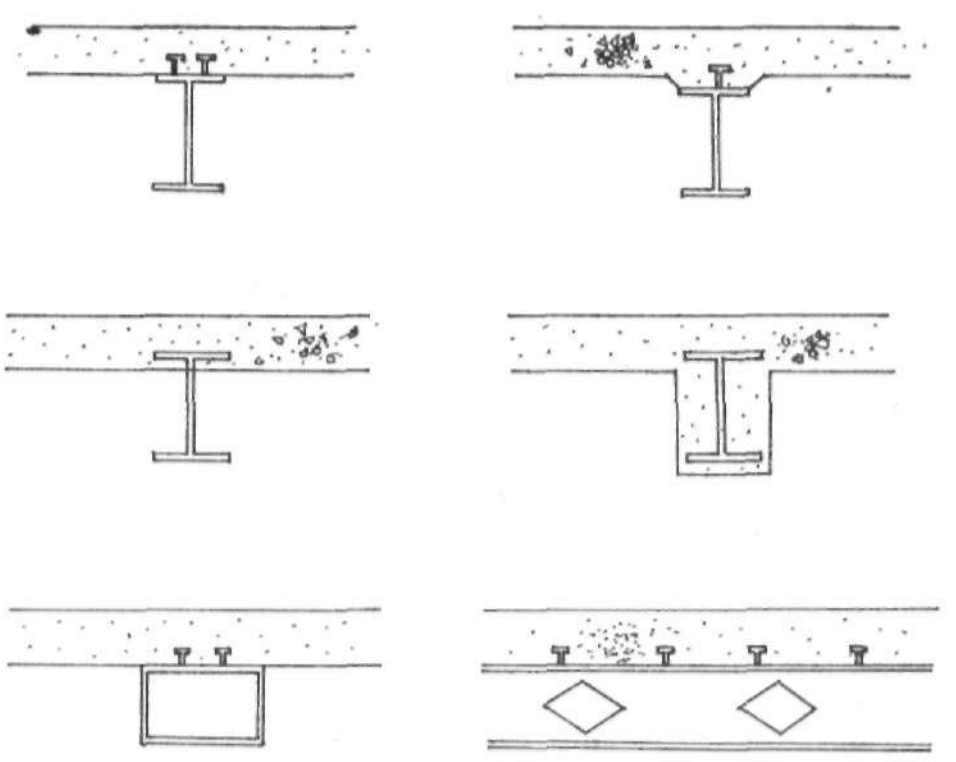


FIG. 1.1. Types of composite beams.

...../3

Comparative studies, in composite and steel structure, conducted in Europe and the United States for medium span beams showed greater saving in composite structure. The saving, though in weight, may not necessarily mean reduction in overall cost. However, a comparative study in Japan, for a continous composite bridge beam of long span showed economy in weight and in overall cost[2].

1.3. REINFORCED CONCRETE CONSTRUCTION.

The British standard codes for reinforced concrete are the CP 114 and CP 110. The structural use of concrete in Building - CP 114, published in 1969 is based on the working stress method and load factor method. In this design method stresses are kept within the elastic range of materials.

Shortcomings observed in the earlier codes led to the publication of CP 110 - the structural use of concrete, in 1972. This method is based on the principles of limit state Design. It is believed that structures designed to this new code of parctice will have better performance than those designed to the earlier codes. The purpose of design, here, is to achieve acceptable probabilities that a structure will not reach a limit state. The aim is to avoid any such condition that may make the structure unfit for use. The two principle types of limit state are the ultimate limit state and serviceability limit state. These are discussed in detail in chapter two.

1.4. DESIGN THEORIES.

In any particular loaded beam, if the load system were increased gradually, deformation increases, with a corresponding increase in stress. This continues until "yielding" occurs at the extreme fibres of the weakest point of the beam. Before yielding the beam is said to be in its elastic range. After yielding, the structure is still capable of withstanding further increase in load. Therefore deformation would considerably increase without further increase in stress until the beam ultimately collapses. At this stage, before failure, the beam is said to be in its plastic state.

The object of elastic design theory is to calculate maximum stresses occuring in a structure and keep these within the limits of "working stresses", in tension and compression. Working stresses are the yield stresses divided by a factor of safety.

Stresses in the plastic region are assumed constant. When the whole cross-section at any particular point in a structure becomes plastic, a plastic hinge would develop. A number of plastic hinges are required to cause complete collapse of a structure.

The number of plastic hinges required depends on the condition of fixity in the structure. One plastic hinge is required for a simply supported beam, three for a built-in-beam, and more for a rigid frame, etcetra. Plastic collapse is assumed to occur at a multiple of "working load". "Collapse load" is a product of working load and "load factor". In plastic design load factor replaces normal factor of safety.

The tolerance of a structure to applied load, at the point just before collapse is called its "ultimate strength". Plastic design theory is, therefore, sometimes referred to as ultimate strength design theory. In this design theory the object is to calculate plastic moment of resistance and to ensure that the maximum normal bending moments do not exceed it.

1.5. AIMS AND OBJECTIVES.

Various authorities highlighted about the advantages composite structures have over steel structures. The aims and objectives of this thesis are:

i. To find out the advantages, if any, a composite structure may have over reinforced concrete structure.

ii. To bring out those factors that contribute to these advantages, or otherwise.

A three storey hospital building has been chosen. The means by which the objectives are to be reached are as follows:

i. To design the building compositely, by plastic method, and in reinforced concrete method, by CP 110.

ii. To make a comparison between the two designs in terms of member size, ease of design, total building weight and in terms of construction economy.

1.6. SCOPE AND LIMITATIONS.

The structual elements designed for the two construction methods are beams and columns. For the limited application of composite construction method the rest of the structural elements are designed by reinforced concrete method. These are slabs, staircase, foundation bases and ramps.

In both designs the structure is considered as framed. Therefore, in the composite construction design a rigid beam-column connection is considered in the shorter span, while in the longer span individual beam spans are simply connected to columns.

In the composite design no checks for stresses in beams are considered. This is because in composite building structures no limits are set for stresses in reinforcement or in concrete. Also no wind loading is considered in both designs. This is because NCP 1. Part 3. Suggest that wind loading could be neglected in an H-shaped building in which the clear space between the parallel blocks is less than four times the width of the smaller of the blocks.

1.7. METHODOLOGY.

The sturctural design is done according to the requirements of relevant codes of practice in common use and the drafts code for composite construction. These are:

1. The structural use of concrete. CP 110.
2. Nigerian standard conde of practice NCP. 1. part 3: Loading.
3. The use of structural steel In Building B.S. 449. Part 2.
4. Draft code for composite construction in Building (By plastic method).

For each of the two design methods a brief discussion on the subject is carried out and followed by a complete design process. Design of beam-column connections, in the composite design, is carried out according to BS 449. The effect of concrete towards further fixity of joints is considered negligible.

CHAPTER TWO

COMPOSITE AND NON-COMPOSITE CONSTRUCTION

2.1. COMPOSITE CONSTRUCTION.

There are two methods by which composite structures are designed. These are the elastic and the plastic design methods.

2.1.1. ELASTIC DESIGN METHOD.

This method is based on elastic theory. It suggests that stresses so calculated in a structural member must not exceed specified values. The values are the working stresses for steel and concrete given in BS 153 and CP 114, respectively. This method is based on the following assumptions:

i. the shear connection between steel beam and concrete slab is sufficient to ensure full interaction between the two materials;

ii. both steel and concrete are linearly elastic;

iii. tensile strength of concrete is neglected.

It is further assumed that the composite member remains elastic, at working load, until steel yield point is reached. After yielding of the steel the structure is assumed to have collapsed.

2.1.2. PLASTIC DESIGN METHOD.

Plastic method of desgin permits failure to occur at some specified load which is a multiple of working load. Plastic hinges are considered to be failure points.

For plastic failure to occur, sufficient number of plastic hinges must be formed. At failure there is a reduction or complete removal of composite action between steel and concrete (failure of shear connectors). This method assumed that:

i. plane section remains so after bending;

ii. steel has already yielded;

iii. tensile strength of concrete is neglected;

iv. beam-slab slip is neglected.

Plastic method ensures that working bending moment does not exceed the plastic moment of resistance. Otherwise the member sections have to be reselected.

2.1.3. CONSTRUCTION METHODS.

The code for composite construction, CP 117 takes into account the method of construction. There are two methods of construction. These are the propped and unpropped methods. Propped construction method is that where the steel beam is supported at intervals before the concrete slab is casted. The main difference is in their approach to dead load calculations.

In the propped method of construction the dead load comprising the weights due to steel member, formwork, concrete slab, floor screed and internal walls are considered together at the same time.

In the unpropped method, however, the dead load is considered in two parts. The first part consists of the weights of steel member, formwork and concrete slab, while the second part is made up of the weights of floor screed and internal walls, which are added after the composite member has attained its full strength. These two parts are considered at different stages of design.

Live and wind loads are also considered though wind load rarely influence the design of composite member. Other loadsdue to shrinkage and creep of concrete and temperature changes are also considered, though they play very insignificant role in composite members in building.

In the plastic method of design only dead and live load are considered. These are added together and multiplied by a plastic load factor of 1.7 to give the design load.

2.1.4. COMPOSITE BEAM.

The interaction between steel section and concrete slab is ensured by means of small pieces of steel bars or shapes, known as studs or shear connectors. The studs are welded to the top flanges of the steel beam and then embedded in the concrete slab. This system is termed 'uncased' composite beam. Whereas, when the steel section is embedded, wholey, in concrete it is know as 'cased' composite beam. This does not require shear connectors.

Deflection of composite beam is checked by limiting its span-effective depth ratio as recommended by BS. 449. In the design of continuous composite beams by plastic method both positive and negative plastic moments of resistance have to be determined. Whenever slab reinforcement continuous past a support it is also considered to contribute to increasing the negative plastic moment of resistance. In a continuous beam individual beam spans are considered separately. In rigidly jointed frame individual beam spans are treated as built-in beam.

For plastic collapse of a built-in beam three plastic hinges are required, one near the middle of span and one at each end. The development of the relevant plastic hinges results whenever the structure is loaded such that its plastic moment of resistance is exceeded at its weakest points. Plastic hinges first form near supports, at the negative moment region. Therefore, the ultimate strength of composite beam near support has far more influence than near mid-span region.5 For this, simple plastic theory can only be used safely for beams in which large rotations can occur in negative moment regions without loss of flexural strength. Thus, the suitability of a beam for plastic design must be based on the behaviour of the plastic moment region.

It is evident that since at the negative moment region concrete is assumed cracked, the ultimate strength of structural steel is the controlling factor. So the suitability

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for plastic design is based on steel yield stress. This led to freezing the limitations on slenderness ratios for webs and compression flanges of unstiffened steel member, in a continuous beam. This is in order to check web and flange buckling.

Thus, for applicability of plastic theory limits on slenderness ratios are as shown below for the corresponding steel grades[5].

For steel grade 50;

$b_f/t_f \leq 12.4$

$d_w/t_w \leq 58\ (1-1.4c)$; with a lower limit of 36

For steel grade 43;

$b_f/t_f \leq 16.8$

$d_f/t_f \leq 70\ (1-1.4c)$; with a lower limit of 43.

Where in both cases $C = A_r.f_{yr}/A_s.f_y$

2.1.4.1. BEAM DESIGN PROCEDURE

The design procedure for a composite beam by simple plastic theory is presented for a given bending moment and vertical shear.

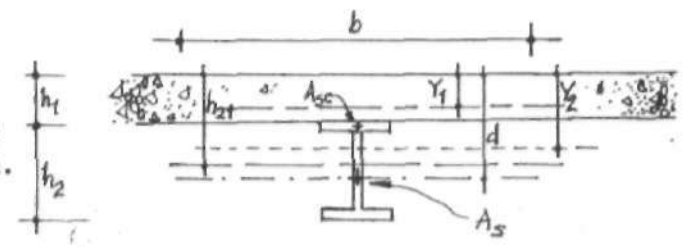


FIG. 2.1. A typical section of composite beam.

1. From the span-depth ratio find h_1+h_2 (h_1, the depth of slab being frozen already, by architect).

2. Assume the lever arm $(h_1+h_2)/2$ and hence estimate the area of steel required, A_s from;

$0.87A_s.f_y(h_1+h_2)/2 = M.$ ----------(2.1.)

3. Check that $0.4\ bh_1.f_{cu}$ exceeds $0.87A_s.f_y$. If not the plastic nuetral axis will be in steel , and A_s as given from 2, above, will then be too small.

4. Select a steel section and check the shear capacity of its web, $0.87A_w.f_y/1.732$ exceeds F.

5. Calculate plastic moment of resistance, M_p, as follows and check that it exceeds, or at best equal to M.

   i. If 3, above is satisfied, the plastic neutral axis is in slab at depth Y_1, found from $0.4bY_1f_{cu} = 0.87A_sf_y$ ------(2.2.)

   ii. If the centroid of the steel section is at depth, d, then $M_p = 0.87A_sf_y\ (d-0.5Y_1)$. ----------(2.3.)

   iii. If 3, above is not satisfied, the plastic neutral axis is at depth Y_2, in steel and $0.4bh_1f_{cu} + 1.7A_{sc}f_y = 0.87A_sf_y$. ----------(2.4.)

   iv. This gives A_{sc} and hence h_{2_1}, and $M_p = 0.87A_sf_y(d-0.5h_1)-1.74A_{sc}.f_y(h_2-0.5h_1)$ ----------(2.5.)

When the beam is subjected to negative bending moment the negative plastic moment of resistance, M_{pn} is calculated as follows:

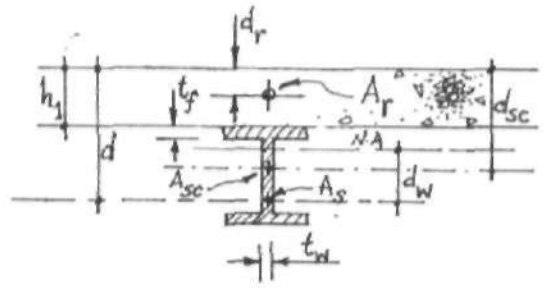


FIG. 2.2. A cross-section of composite beam in a negative moment region.

v. Longtitudinal equilibrum is satisfied by $F_r = A_r.f_{rd}$, from which d_{wc} is given by $2d_{wc}.t_w.f_{sd}=A_r.f_{rd}$ ----------(2.6.)

vi. If d_{wc} is less than, or equal to, $d-h_1-t_f$, the result is correct, and $d_{sc}= d-0.5d_{wc}$. ----------(2.7.)

vii. If d_{wc} is greater than $d-h_1-t_f$ the neutral axis lies in the top plange, then assume $d_{sc}=h_1+t_f$. ----------(2.8.)

viii. At any rate $M_{pn}=M_{ps}+F_r(d_{sc}-d_r)$, where the plastic moment of resistance of steel section, $M_{ps}=0.87 \times Z_p \times f_y$ ----------(2.9.)

It is normally not economical to provide enough slab reinforcement, A_r, so as to make negative plastic moment of resistance, M_{pn}, large enough to satisfy requirement. So the plastic analysis requires a preliminary assumption of the ratio of M_p to M_{pn}. In beams for building this ratio, K, falls between 0.5 and 0.7. If the beam is continuous at both ends $(1+k)\ M_p=wl^2/8$ and if it is continuous at only one end $j=(1/k)[(1+K)^{\frac{1}{2}}-1]$, and $M_p=0.5wj^2.l^2$

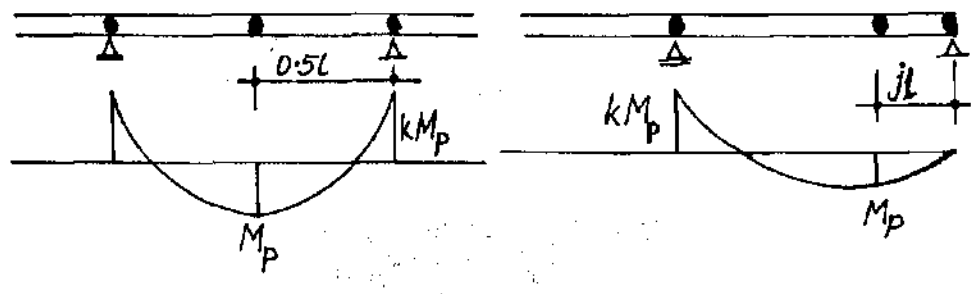


FIG. 2.3. Bending moment diagram at collapse.

2.1.5. SHEAR CONNECTORS.

The problem of connecting the concrete slab to the steel beam is solved by means of shear connectors. These are pieces of steel welded to the flange of the steel beam. There are various types of connectors, the commonest type being the headed stud connectors. These range in diameter from 13 to 25mm, and in length from 65 to 100mm. The maximum shear force that can be resisted by a stud is relatively low, about 150KN. Other types of connectors with higher strength include bars with hoops, tees with hoops, horse shoes and channel.

A shear connector, whatever its detailed make-up, has to perform the functions of transferring shear between steel and concrete, that is to limit slip at the interface, and secondly prevent seperation of the steel and concrete, that is to prevent uplift. For any type of connector it must be assumed that composite interaction is solely achieved by means of the connectors alone and that any bond that may exist between steel and concrete is ignored. In order to ensure that slab is adequately tied down to the steel beam, whatever the form of the shear connector, it should project at least 25mm above the plastic neutral axis.

The design of shear connectors should be based on their total ultimate capacity[7]. On this design basis the longitudinal pitch of the connectors is constant throughout the length of the beam. However, if the beam is subjected to hagging moment it is necessary to sub-divide the span of the beam, with each portion maintaining a constant connector pitch. CP 117: Part 1 recommends that the maximum spacing should not be greater than four times the slab thickness, nor greater than 600mm.

In order to design a shear connector it is necessary to know the location of the plastic neutral axis, so that the height of the connector could be established by ensuring that its height is at least 25mm above the neutral axis. The diameter of the type of the connector has to be selected. And by checking the table given in CP 117: PArt 1, the corresponding value for the characteristic static strength of the connector will be known. This table is reproduced below:

TABLE 2.1. Typical values for a charact. static strength for st

HEADED STUDS		STATIC STRENGTH(KN) FOR CHARACTERISTIC CONCRETE CUBE STRENGTHS(N/mm^2)			
DIA.(mm)	HEIGHT(mm)	20	30	40	50
25	100	139	154	168	183
19	100	90	100	109	119
13	65	42	47	52	57

If the plastic neutral axis is in slab, the longitudinal force in steel, F, at yield, is

$$F = 0.8A_s.f_y \qquad \text{----------(2.10.)}$$

This is the total longitudinal shear resisted by the connectors at half span. The design load of a connector, P_d, is equal to characteris/ strength, P_k, multiplied by 0.80 (an eighty per cent design)[5]. and divided by a factor of safety (1.15)

tic static

$$P_d = 0.8P_k/1.15 \qquad \text{----------(2.11.)}$$

The number of studs, N, required is given by

$$N = F_y/P_d \qquad \text{----------(2.12.)}$$

This number is that required at half span. The strength of connectors in the negative moment regions is taken as 20 per cent less than in positive moment regions. So, in the negative moment region number of connectors is

$$N = 1.25F_r/P_d \qquad \text{----------(2.13.)}$$

Studs could either be placed singly or in pairs. In this design stud connectors of diameter 19mm and a height of 100mm with a characteristic strength of 100 KN for a concrete grade 30 is to be used.

2.1.6. COMPOSITE COLUMNS.

The design of a composite column is, in fact, a method of determining whether a proposed/ member is strong enough for its intended purpose. Initially, composite columns are designed by cased-strut method of BS 449. The column is assumed to resist axial load, while

the moment it resists is as a result of eccentric loads.

A more recent method of column design is the ultimate-strength design method of Basu and Sommerville[5,22]. The method is applicable to pin-ended uniform composite members, having cross-section symmetrical about two perpendicular axes, and subjected to a combination of axial and end moments.

Composite column designed by the limited frame method is assumed loaded from full beam loads on all the beams above the column length. Alternate beams attached to both ends of the column are partially loaded with elastic moments due to dead loads only, while the other beams are loaded fully with plastic moments. In this regard all members attached to both ends of the column are assumed fixed at their other ends[5,6]. In the design of composite column it is assumed that the effective length ratio of the column may not be reduced below unity due to restraint from composite beams, and that the whole out of balance end moment must be shared between the upper and lower column lengths. It is accurate enough to assume that these shares are equal unless there is a sudden change of cross section when the shares are in proportion to the elastic stiffnesses.

In the ultimate strength desing method each axis of bending has the following properties:

The ultimate axial load (squash load) is;

$$N_u = 0.87A_s.f_y + 0.87A_r.f_r + 0.45A_cF_{cu} \quad \text{----------(2.14.)}$$

The radius of gyration is;

$$r^2 = (2501_s + 0.241_c.f_{cu})/N_u \quad \text{----------(2.15.)}$$

The concrete contribution factor;

$$q = 0.45A_cf_{cu}/N_u \quad \text{----------(2.16.)}$$

And the plastic moment of resistance of the section depends on the position of neutral axis.

Thus for major axis bending;

$$Y_1=ht_w/[2t_w+0.46b(f_{cu}/f_s)]; \qquad \text{----------(2.17.)}$$

Provided Y_1 is greater than d_c+t_f

$$M_p=[0.5A_s(h-Y_1)-A_f(d_c-d_{wc})-d_{wc}t_c(Y_1-d_{wc})]f_s+0.87A_rf_r(0.5_h-d_r) \qquad \text{---------(2.18.)}$$

And for minor axis bending the neutral axis almost always intersects the flanges.

Thus:

$$Y_1=(A_s+4t_f.d_c)/[4t_f+0.4h(f_{cu}/f_s) \qquad \text{----------(2.19.)}$$

$$M_p=[0.5A_s(b-Y_1)-2(Y_1-d_c)t_fd_c]f_s+0.87A_r.f_r(0.5b-d_r) \qquad \text{----------(2.20.)}$$

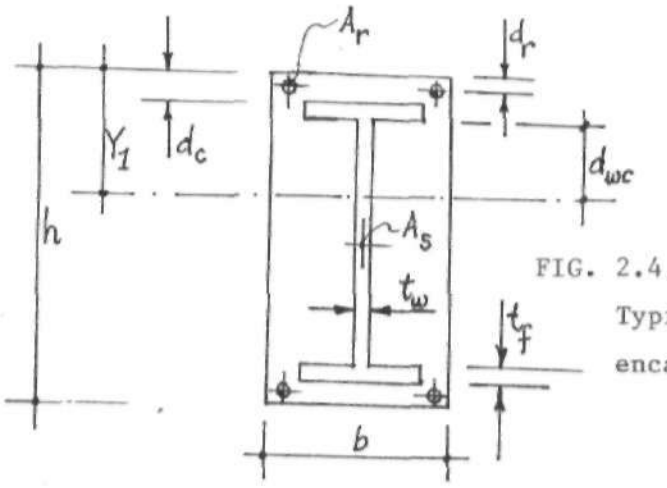


FIG. 2.4.
Typical section of encased composite column.

In this design method the plastic moment of the section must be greater than, or equal to, the greater of the two out-of-balance end moments. Secondly, that the ratio of axial load, N, to ultimate load N_u is less than i_p, as shown in the following design. Other design requirements are that the minimum concrete cover to structural steel section is 50mm and that wire binders must be provided at a pitch of not more than 150mm.

In the following design rigid beam-column connection is considered (the same has been considered during beam design). The rigidity of the connection is in the major axis of the column while in the minor axis there exist simple joint. Composite columns to which beams are rigidly connected are analysed by limited frame method in which all beams above the column are fully loaded, while those connected to its ends are alternately loaded with dead load only.

Because of the timber pitched roofing, there are no composite beams at roof level, except those supporting the parapet walls, and in the minor axis of the building. Frame F_{1A} is designed to represent frames F_1 and F_4. Frame F_2 also represents F_3.

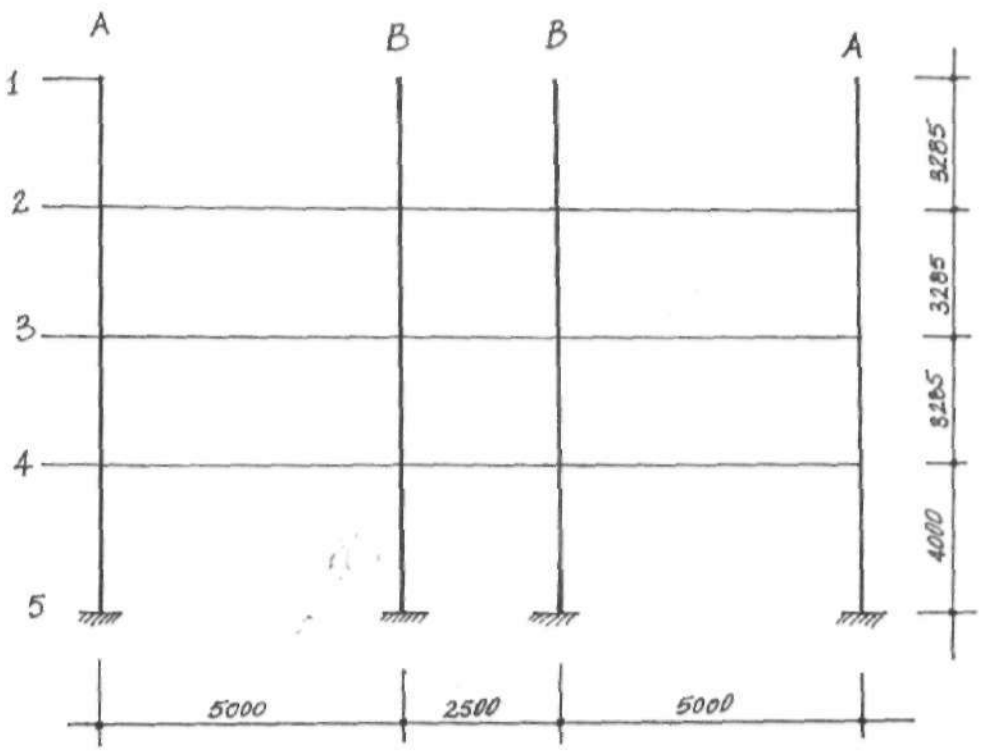


FIGURE 2.5. A typical frames F_1 and F_4.

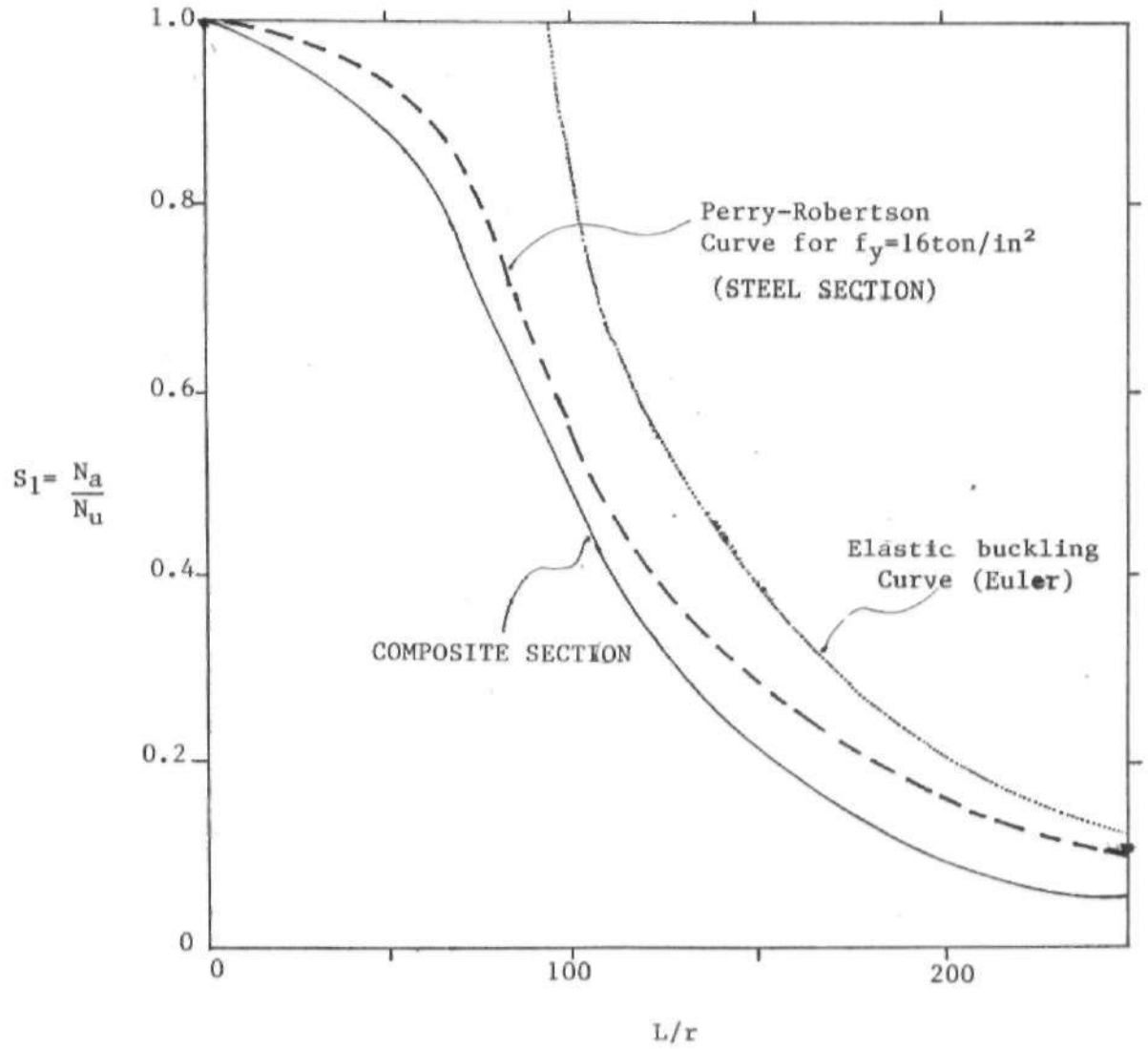


FIG. 2.6. COLUMN BUCKLING CURVE

2.1.7. BEAM - COLUMN CONNECTIONS.

The behaviour of composite frame at ultimate limit state (Load factor method) is strongly influenced by the desing of its beam-column joints. There are three possible types of connections between beams and columns. These are simple, rigid and semi-rigid. Simple connection assumes that columns are continuous, passed the joint, and beams are attached to the column faces.

Rigid connection assumes that beams are continuous passed joints. It is therefore necessary to ensure that perfect rigidity exists at the joints, so that moment transfer between members is possible.

Semi-rigid connection is where there only exist partial rigidity at joints. This is obtained by providing additional top reinforcement to the slab in a frame with simple connections.

Unless individual members are connected, correctly, to each other the intended joint may not be achieved. The frame may not, therefore, function perfectly. Connections are provided by means of riveting, bolting, welding or a combination of these. Care must be exercised in selecting the most suitable from these.

The desing of beam-column connections for this project is done based on the provision of B.S. 449, Part 2, 1969. Also shop welding and site bolting are assumed at the fabrication and construction stages, respectively.

2.1.7.1. RIGID CONNECTIONS

Rigid beam-column connections suggest that the joint is capable of transfering moment stresses between beams and columns. This type of connections is achieved by providing end plates to beams and these bolted to column flanges.

For the purpose of design rotation between beam and column is assumed to take place about a point adjacent the underside of beam bottom flange. This is to determine the size of weld, end plate thickness as well as diameter and number of bolts required to resist the tensile forces created by the joint bending moment[8]. It is also assumed that weld round the top flange and web down to a distance of 80 millimeters resists bending moment. While weld below this point resists vertical shear forces only.

2.1.7.2. SIMPLE CONNECTIONS

In this type of connection it is assumed that bending stresses in beams are not transferred to column. Conditions for simple connections are that top longitudinal reinforcement in slab should consist of small diameter bars. These are to control concrete cracking only. Steel beams should not have end plates or top cleats. Bolts to web cleats should not be close tolerance or friction grip bolts. Large diameter slab top reinforcement should not go beyond column faces[5].

In this project beams in the minor axis are assumed simply connected. The web and seating cleats are of unequal angle 75 x 50 x 8 millimeters. A clear space of 5 millimeters is allowed between the column web and beam flanges.

2.1.8. REINFORCED CONCRETE SLAB.

A slab is a horizontal structural element whose direction of loading is transverse to the direction of span and can be analysed and designed in the same manner as concrete beams. A slab whose ratio of longer to shorter spans is less than two (2) is considered to span in both directions. Its elastic analysis is, therefore, simplified by use of co-efficients given it tables 12 and 13 of CP 110. Section 3.4.3.3 of the code gives loadings on beams supporting a two-way slab.

In a composite structure concrete slab has two main functions: To transfer superimposed loads and slab finish loads to the beams and to act as a flange to the composite T-beams. The design of slab is independant of of composite structure is design either to CP 114 or to CP 110.

2.2. REINFORCED CONCRETE CONSTRUCTION.

The code of practice, CP 110, defines a limit state as a state in which a structure, or part of it, ceases to fulfill its intended function. It is further suggested that the aim of design is to produce a structure that will meet, the requirement of safety, durability and economy.

That is to guarantee that the structure will not become unfit for use within a foreseable future. Thus, the aim of a limit state design.

There are two principal types of limit state. These are the ultimate limit state which is the state when the structure collapses, and the serviceable limit state, which relates to deflection, cracking, vibration, fire, durability and fatigue.

The design is based on ultimate limit state and checks carried out on serviceability limit state. The ultimate limit state relates to structural strength, while serviceability limit state controls thickness of reinforcement concrete cover, minimum distance between bars, minimum and maximum amount of reinforcement required maximum distances of secondary bars and links, et-cetra.

2.2.1. CHARACTERISTIC STRENGTH/LOADS

The characteristic material strength is that value of strength below which test sample results are unlikely to fall. When a large number of sample is taken it is unlikely that more than 5 per cent of the result will fall. Mathematically:
Characteristic strength is equal to mean material strength minus 1.64 of standard deviation. Design is based upon characteristic strength.

Characteristic load is that value of load which has an accepted probability of not being exceeded during the life span of a structure. In practice the design load may be larger than the characteristic load. Each characteristic load (dead, imposed or wind) is therefore, multiplied by a partial factor of safety and these are added to give a design load.

2.2.2. PARTIAL FACTORS.

These are applied to loads in order to take care of possible errors and inaccuracies that may result for a number of unforeseen circumstances.

TABLE 2.2. Patial factors of safety.

LOAD COMBINATION	LIMIT STATE					
	ULTIMATE			SERVICEABILITY		
	DEAD	LIVE	WIND	DEAD	LIVE	WIND
DEAD AND LIVE	1.4(1.0)	1.6	-	1.0	1.0	-
DEAD AND WIND	1.4(0.9)	-	1.4	1.0	-	1.0
DEAD,LIVE & WIND	1.2	1.2	1.2	1.0	0.8	0.8

2.2.3 REINFORCED CONCRETE BEAMS

Most reinforced concrete beams form part of a floor system in which the slab provides the flange for the beam section.

Beam cross-sectional dimensions are determined in relation to approximately estimated bending moment and shear force and to serviceability limit state requirement. The beam breadth should not be greater than half the beam depth.

Beam dimensions are calculated using:

Approximate ultimate load; $F=1.4G_k+1.6Q_k$ ------(2.21.)

Approximate ultimate moment; $M_u=0.1xFxL^2$ ------(2.22.)

Approximate shear force; $V=0.5xFL$ -------(2.23.)

Actual beam depth is calculated from:

In relation to bending; $h \geq (5.2 \text{ to } 7.0)(M_u/f_{cu}.b_f)^{\frac{1}{2}}$ ----------(2.24.)

In relation to shear; $h \geq (2.4 \text{ to } 2.8)(V/f_{cu \text{ shear}})^{\frac{1}{2}}$ ----------(2.25.)

In relation to serviceability requirement;

$d \geq L/26 \times 0.77$ ----(assuming 3% reinforcement) ----------(2.26.)
th

The effective bread/of a flanged beam, b_f, should not exceed the lesser of:

i. the actual beam spacing;

ii. web width plus E.A.

E= 0.2 for a T-beam, or

0.1 for an L-beam.

A= distance between points of zero moments, i.e. 0.7 of effect. span for continous beam or the effective span of simple beam.

2.2.4. REINFORCED CONCRETE COLUMNS.

Columns are primarily compression members. They transfer loads from structural beams and slabs to the foundation. Columns may also have to resist bending forces in addition to direct load. The relationship between column height and its cross-sectional dimensions is important in determining whether a column should be considered as short or slender. A column is termed slender when its slanderness

ratio is greater than 12, otherwise it is termed short column.

Knowledge of the stiffness of any column is vital in column design. This is because the ability of a column to resist bending depends on its flexural rigidity, which is a factor/stiffness ratio.

Sections 3.5.7.1, 3.5.7.2 and 3.5.7.3 of CP 110 give methods of designing slender columns, bent about minor axis, major axis and biaxially, respectively. However, section 3.5.7.3 is used because the columns are biaxially bent. Total moments are found from:

$$M_{tx}=M_{ix}+(Nh/1770)(l_{ex}/h)^2(1-0.0035l_{ex}/h) \quad \text{--------(2.27.)}$$

$$M_{ty}=M_{iy}+(Nb/1750)(l_{ey}/b)^2(1-0.0035l_{ey}/b) \quad \text{--------(2.28.)}$$

2.2.4.1 APPROXIMATE COLUMN DIMENSIONS.

At initial stage column cross-sectional dimensions are approximated from the equation for stability criterion. Approximate column end moment is estimated as 40 per cent of adjacent beam span moment, when the beam is considered simply supported. The actual column dimensions are found by referring to crushing and buckling criterion (Ref. 13).

To satisfy stability criterion

i. $L_{eff}/20 \leq h_{col} \leq L_{eff}/10$ ----------(2.29.)

ii. $b_{col} \leq L_{ff}/25.$ ----------(2.30.)

Where column effective height, L_{ff}, is approximate value found from the product of the column actual height and the recommended factor, considering the condition of fixity at column ends.

By calculating N/bh and M/bh^2 and making reference to design chart of CP 100, part 3 a check is made to ensure that the assumed column dimension are adequate.

2.2.4.2. ACTUAL COLUMN EFFECTIVE HEIGHT

Equation 22 and 23 of section 3.5.1.4 of CP 110 give actual effective height of unbraced column. The lesser of the two values is used. Thus:

$$L_{eff}=L\ [1.0+0.15(C_1+C_2)] \quad \text{------- eq. 22 ---------} \quad (2.31.)$$

$$L_{eff}=L\ (2.0+0.3C_{min}) \quad \text{------- eq. 23 ---------} \quad (2.32.)$$

where L is the actual column height

C_1 is the ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at one end of the column in the same plane of bending.

C_2 is the ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at the other end of the column, in the same plane of bending.

C_{min} is the minimum of C_1 and C_2.

The code recommends that C_{min} could be taken as unity when column base is designed to carry column moment.

CHAPTER THREE

D E S I G N O F C O M P O S I T E S T R U C T U R E

This chapter is concerned with design of steel-concrete composite consturction in building. The building consists of three blocks. Two identical, parallel blocks and one block in between them, as shown in the layout. The building is a framed building and its slabs span in two directions.

3.1. DESING OF TWO-WAY CONCRETE SLABS.

In the following design the slabs are designed as two-way slabs. Elastic analysis of the slabs is by use of coefficients given in table 13, CP 110: Division of panels is as shown in page 246 . These are grouped into three, each group satisfying a particular case of table 13 of the code. Panels 1,5,6,7,11,12 and 15 satisfy case 2. Panels 2,9,10,13 and 14 satisfy case 1, while panels 4 and 8 satisfy case 4.

Complete design process of panel 1 is shown. For the other panels only the final results of the calculations are shown in the following tables.

3.1.1. DESIGN DATA.

Concrete	-	grade 30
Reinforcement	-	high yeild, plane bars
Condition of exposure	-	Moderate
Concrete cover	-	30 millimeters
Slab thickness	-	150mm, as provided by Architect
Live Load	-	3.4KN/m^2,NCP 1, part 3, live load for hospital building.

3.1.2. LOAD ESTIMATION.

Slab self weight is $0.15 \times 24 = 3.60 KN/m^2$

Floor finishes, assume $0.60 KN/m^2$

Characteristic dead load, $G_k = 4.20 KN/m^2$

Characteristic live load, $Q_k = 3.43 KN/m^2$

Total ultimate load, $= 1.4G_k + 1.6Q_k$ --------(3.1.)

$= 11.40 KN/m^2$

Design load, $n = 11.40 \times 1.0 = 11.40 KN/m$

3.1.3. DESIGN OF PANEL 1

Long Span, $L_y = 5000$

Short Span, $L_x = 4000$

Spans ration, $L_y : L_x = 1.25$, say 1.3

Design according to case 2.

3.1.3.1. MAIN BARS.

TABLE 3.1. Design of reinforcement for panel 1.

POSITION	MID SPAN		CONTINUOUS EDGE	
DIRECTION	SHORT SPAN, L_x	LONG SPAN, L_y	SHORT SPAN, L_y	LONG SPAN, L_y
COEFFICIENTS, B.	0.039	0.028	0.051	0.037
$M = B \times n \times L_x^2$ (KNM)	7.10	5.10	9.10	6.70
$M/bd \cdot f_{cu} \cdot$	0.018	0.13	0.023	0.016
L_d (from lever arm curve	0.95	0.95	0.95	0.95
$Z = L_a.d$ (mm)	109.25	109.25	109.25	109.25
REINFORCEMENT, A_s (mm^2)	182.20	131	239	162
MINIMUM, (A_s (mm)	172.50	172.50	172.50	172.50
SELECTION.	16Y8	10Y8	19Y8	10Y8

3.1.3.2. EDGE STRIP REINFORCEMENT.

This is an area with a width equal to one-eigth of span. Edge strip is both edges of both spans.

Amount of reinforcement;

min. A_s=0.15bd/100=173mm^2

Provide Y8 at 300 (168mm^2)

Width of edge strip;

Long span - 5000/8 = 625mm

Short span - 4000/8 = 500mm

Provide;

Long span - 3 bars

Short span - 2 bars

3.1.3.3. TORSIONAL REINFORCEMENT.

This is provided at corners of discontinous edges. Its amount is equal to three-eigth of the area required for maximum mid-span moment.

A_s=(3/8) x 201 = 75.38mm^2

Provide Y6 at 300 (94mm^2)

Width where required is;

L_x/5 = 800mm in both directions.

3.1.3.4. DEFLECTION.

In order to contain excessive deflection limits are set on span-effective depth rations.

$100A_s/bd$ = 201x100/ 1000x115 = 0.175

From table 10, CP 110

Modification factor = 1.60

Limiting span-effective depth ratio;

26 x 1.6 = 41.80

Actual span-effective depth ratio;

4000/115 = 34.78

The slab depth of 150mm is adequate.

3.1.3.5. SHEAR STRESS.

Load on supporting beams is given by clause 3.4.3.3.

These are:

Shorter beam = $nL_x^2/4$ = 45.6KN

Longer beam = $(nL_x/2)-(nL_x^2/4)$ = 68.4KN

Existing shear stress,

$V = F/bd = 0.34\ N/mm^2$

Ultimate shear stress,

$100A_s/bd = 0.175$

ultimate shear stree in beam is

$0.35\ N/mm^2$ (table 5, CP 110) and

slab factor is 1.30 (table 14, CP 110).

$\therefore V_c = 1.30 \times 0.35 = 0.46\ N/mm^2$

The slab is adequate in shear.

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3.3.3.6. LOCAL BOND.

Total bar perimeter

U_{ST}=1000/250 x 3.142 x 8

=100.54mm

Shear force

F=45.6KN

Local bond stress,

f_{bs}=45.6x10^3/100.54x115

= 1.94 N/mm^2

Ultimate local bond stress is 2.2 N/mm^2

The section is therefore adequate in local bond.

The following tables show the result of calculations for the rest of panels 2 to 14. Similar calculations as for panel 1 were carried out for these panels.

TABLE 3.2 Final results for the design of suspended slabs.

DIRECTION	SHORT SPAN			LONG SPAN			TORSIONAL REINFORCEMENT
LOCATION	MID - SPAN	CONT. EDGE	EDGE-STRIP	MID - SPAN	CONT. EDGE	EDGE STRIP	
PANEL 1.	16Y8 at 250	19Y8 at 200	2Y8 at 250	10Y8 at 300	10Y8 at 300	3Y8 at 250	3Y6 at 300
PANEL 2.	Y8 at 300	Y8 at 300	2Y8 at 250	11Y8 at 300	Y8 at 300	3Y8 at 250	NIL
PANEL 3.	Y8 at 300	Y8 at 250	2Y8 at 300	Y8 at 300	3Y8 at 300	3Y8 at 300	NIL
PANEL 4.	Y8 at 200	Y8 at 150	3Y8 at 250	Y8 at 300	Y8 at 200	3Y8 at 250	4Y6 at 200
PANEL 5.	Y8 at 300	Y8 at 300	2Y8 at 300	Y8 at 300	Y8 at 300	2Y8 at 300	2Y6 at 300
PANEL 6.	Y8 at 300	Y8 at 300	2Y8 at 250	Y8 at 300	Y8 at 300	3Y8 at 250	3Y6 at 300
PANEL 7.	Y8 at 300	Y8 at 300	2Y8 at 250	Y8 at 300	Y8 at 300	3Y8 at 250	3Y6 at 300

Maximum reinforcement spacing is 3d = 3 x 115 = 3 x 115 = 345mm

TABLE 3.2. (Cont'd)

DIRECTION	SHORT SPAN			LONG SPAN			TORSIONAL REINFORCEMENT
LOCATION	MID - SPAN	CONT. EDGE	EDGE STRIP	MID - SPAN	CONT. EDGE	EDGE STRIP	
PANEL 8.	Y8 at 300	Y8 at 200	2Y8 at 250	Y8 at 300	Y8 at 300	3Y8 at 250	3Y6 at 200
PANEL 9.	Y8 at 300	Y8 at 300	2Y8 at 250	Y8 at 300	Y8 at 300	3Y8 at 250	NIL
PANEL 10.	Y8 at 300	Y8 at 300	2Y8 at 250	Y8 at 300	Y8 at 300	2Y8 at 250	NIL
PANEL 11.	Y8 at 300	Y8 at 300	2Y8 at 250	Y8 at 300	Y8 at 300	2Y8 at 250	3Y6 at 300
PANEL 12.	Y8 at 300	Y8 at 300	2Y8 at 250	Y8 at 300	Y8 at 300	3Y8 at 250	3Y6 at 30
PANEL 13.	Y8 at 300	Y8 at 300	3Y8 at 250	Y8 at 300	Y8 at 300	3Y8 at 250	NIL
PANEL 14.	Y8 at 250	Y8 at 200	3Y8 at 250	Y8 at 300	Y8 at 300	4Y8 at 250	NIL

Maximum reinforcement spacing is 3d = 345.

3.1.4 CANTILEVER SLABS.

As indicated on the architectural drawings certain parts of the slab project for approximately one meter (1.0M). This section is considered as a cantilever slab. This is designed as follows:

Load estimation:

Ultimate distributed load = 11.40 KN/M^2

Point load due to wall = 10.40 KN/M

Bending Moment:

$M = wl^2/2+pl = 11.40x1.0^2/2+10.40x1.0$

$= 16.10$ KNM

Bending Reinforcement:

$M/bd^2f_{cu}=0.041$

From lever arm curve, $l_a=0.94$

Lever arm, Z=108.1mm

$A_s = 16.10x10^6/0.87x410x108.1$

$= 418mm^2$

Provide Y8mm at 300mm ($168mm^2$)

Y10m at 300mm ($262mm^2$)

Checking shear stress:

Shear force,

F = 11.40+10.40 = 21.80KN

Shear stress

$V = F/bd = 0.19\ N/mm^2$

Slab thickness factor = 1.30

Ultimate shear stress in beam (a factor of $100A_s/bd$) = $0.43\ N/mm^2$

Ultimate shear stress = 1.30 x 0.43

= $0.55\ N/mm^2$

The section is adequate in shear.

Span-Effective Depth Ratio:

$100A_s/bd$ = 402x100/1000x115

= 0.35

Modification factor = 1.46

Limiting span-effective depth = 7x1.46 = 10.22

Actual span-effective depth = 1000/155= 8.70

The overall contilever depth is adequate.

Local Bond:

Bars perimeter

U_{ST}=1000x3.142x8/125 = 201mm

Local bond stress,

f_{bs}= $21.8x10^3/201x115=0.94N/mm^2$

This is below the ultimate local bond stress value of $2.2\ N/mm^2$. The section is again adequate in local bond.

3.2. DESIGN OF COMPOSITE BEAMS.

All composite beams in X-plane of the building are considered continous. Beam-column connection are, therefore, rigid. In the Y-plane the beams are assumed simply supported. For this the connections are simple. Beam column connection of BS 449 is employed in the design. Any contribution that the column concrete may provide for stiffning the simple joing is neglected. The beams are uncased composite beams except the parapet wall beams which are designed as encased beams. Design of continous composite beams is by considering individual spans one at a time.

Beams in frame 1 are designated B1. Similar designation is made for all the other beams.

3.2.1. DESIGN DATA.

Slab thickness	150mm
structural steel	grade 50
Beam depth	L/15 (BS 449)
Shear connectors:	
height	100mm
diameter	19mm
design load	69.6KN
Load factor	1.7

3.2.2. DESIGN OF BEAM B_1

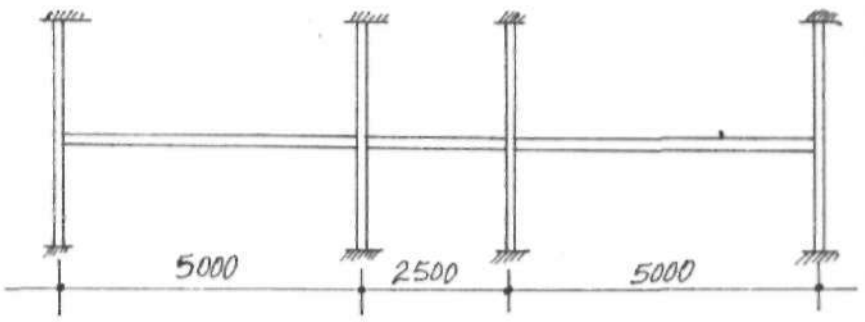
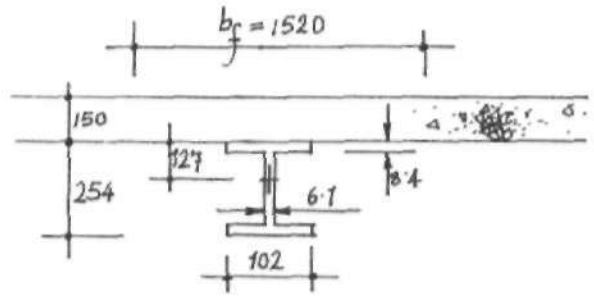


FIG. 3.1. Sub-frame showing beam B_1.

3.2.2.1. DESIGN OF 5 METER SPAN.

CALCULATIONS	OUTPUT
PRELIMINARY SIZING. Depth of column - assume 450mm Clear span of beam - 4550mm Effective width of beam flange, b_f is the lesser of: i. 4550/3 - 1520mm ii. beam spacing - 4000mm iii. 12 times slab thickness - 1800mm Beam depth, $h_1+h_2=L/15$, say 400mm Depth of slab, h_1=150mm Depth of steel section h_2=250mm Lever arm, $(h_1+h_2)/2$=200mm	b_f=1520mm h_2=250 Lever arm 15 200mm
LOADING. Weight of slab - 3.6 Weight of floor finishes - 0.6 Live load - 3.43 Total slab working load = 7.63KN/M² Equivalent loading per beam is $(2nl_x/6)[3-(L_x/L_Y)^2]$ = 24KN/M cl. 3.4.3.3, CP 110 Weight of steel beam, assume 2.5KN/M Weight of partition wall = 10.40KN/M Total load = 36.9KN/M	Load per meter length of beam is 36.9KN/M
BENDING MOMENTS. Ultimate bending moment, $M = 1.7x36.9x4.55^2/8$ = 162.33 KNM	

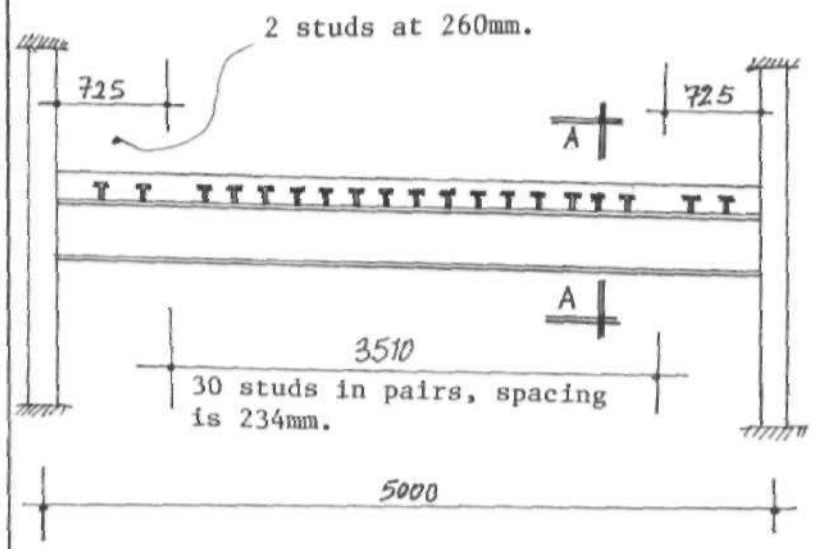
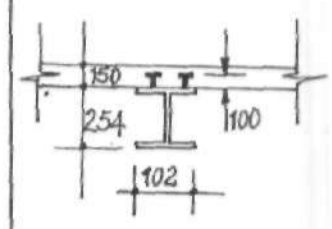
CALCULATIONS	OUT PUT
Negative bending moment, $M_n=1.7x36.9x4.55^2/_{12}$ =108.22 KNM	M=162.33KNM M_n=108.22KNM
STEEL SECTION $A_s=M/0.87x350x200$ $=2666mm^2$ $0.87A_sfy=811.65KN$ $0.4bh_1.f_{cu}=2736KN$ Force is concrete is greater than in steel. Neutral axis lies in steel. Steel selected is UB254x102x25Kg/M  	N.A is in steel. UB 254x102x25. $A_s=3230mm^2$ I = $3046mm^4$ $Z_p=305.6cm^3$
SHEAR CAPACITY. Limiting value; $F_c=0.87A_w.fy/1.7321$ ----------(3.2.) = 0.87x254x6.1x350/1.7321 = 272.4KN Shear force, $F = 1.7w^1/2+M_n/1$ ------(3.3.) = 164.34 KN Existing shear force is less than the limiting value	F_c>F Section is adequate in shear.

CALCULATIONS.	OUTPUT.
PLASTIC MOMENTS OF RESISTANCE. Depth of plastic nuetral axis, Y_1; $0.4by_1f_{cu} = 0.87A_sf_y$. $Y_1 = 54mm$. Depth of steel centroid; $d = 150 + 127 = 277mm$ Composite Section; $M_p = 0.87A_sf_y(d - 0.5Y_1)$ $= 0.87x3230x350(277 - 54)$ $= 246$ KNM. Steel section; $M_{ps} = 0.87Z_pf_y$ $= 0.87x305600x350$ $= 93$ KNM. Neglecting slab reinforcement, at this stage, in resisting negative moment. $M_{pn} = M_{ps} = 93$ KNM. Moment ratios; $k = M_{pn}/M_p = 0.38$ The value of k is below the lower limit of 0.5 (section 2.1.5.1). Therefore assume the lower limit (k = 0.5). There is continuity at both ends of beam, so; $(1 + k)M_p = 1.7wl^2/8$ $1.5M_p = 1.7x36.9x4.55^2/8$ $M_p = 108.22$ KNM $M_{psn} = 0.5x108.22$ $= 54.11$ KNM.	$M_p = 246$ KNM. $M_{psn} = 54.11$ KNM.

CALCULATIONS	OUTPUT
ADDITIONAL SLAB TOP REINFORCEMENT Distance of steel top flange from, steel centroid; $d_{wc}=d-h_1-t_f$ $=277-150-8.4$ $=118.3mm$ Depth of steel centroid in compression $d_{sc}=d-0.5d_{wc}$ $=217.9mm$ Reinforcement cover $d_r=30+5=35mm$ so $d_{sc}-d_r=183.9mm$ In order to ensure that stresses at working load are not excessive, let us assume an increment of 20 percent of negative plastic moment of resistance in order to provide a margin over M_{pn}. (Ref. 5). Let this increment be contributed by slab reinforcement Increment = 54.11x20%= 10.82 Force in reinforcement; $F_r=10.82x10^6/183.9=58.85KN$ Area of reinforcing bars; $A_r=58.85x10^3/0.87x357$ $=165mm^2$ Provide 6Y6 ($179mm^2$) Actual force in the bars; $F_r=170x0.87x410$ = 60.69KN	

CALCULATIONS	OUTPUT
From $2d_w.t_w.f_{sd}=A_r.f_{rd}$ ---------(3.5.) d_w=170x0.87x410/2x6.1x0.87x350 = 16.4mm $d-h_1-t_f$=118.6mm So $d_w<d-h_1-t_f$. Therefore the value of d_w is correctly given above, and $d_{sc}=-0.5d_{wc}$ =277-0.5x118.3 =217.85mm	Provide bars 6Y6 ($170mm^2$) d_w=16.4mm
ACTUAL NEGATIVE PLASTIC MOMENT OF RESISTANCE. $M_{pn}=M_{psn}+F_r(d_{sc}-d_r)$ --------(3.6.) =54.11+11.10 =65.21 KNM	M_{pn}=65.21
M_p+M_{pn}=246.0+65.21=311.21KNM $M+M_n$ =162.33+108.22=261.55KNM	The section is adequate in flexure
APPLICABILITY OF PLASTIC THEORY. Steel grade 50 b_f=102mm t_f=8.4mm d_w=237.2mm t_w=6.7mm b_f/t_f=12.14 d_w/t_w=38.9	

CALCULATIONS	OUTPUT
C=170x410/3230x350 =0.0617 58(1-4.4C)=53 Flange and web slendernesses are within the required limit.	The steel section is suitable for plastic theory.
SHEAR CONNECTORS i. IN THE MID-SPAN REGION. Stud design load; P_d=0.8x100/1.15=69.6KN Force in steel at yield; $F_y=0.87 \cdot A_s.f_y$ =983.54 KN Number of connectors; $N=F_y/P_d=15$ This is the number required in half of the positive moment region. Therefore total number required is 30. Spacing of connectors; Length of positive moment region is 3.51 M. Connector pitch, P=3.51/30=0.117M When in pairs spacing is 0.234M. ii. IN THE HOGGING MOMENT REGION Force in steel at yield; $F_r=0.87.A_r.f_y$ = 60.64KN.	30 studs in pairs, at spacing of 234mm

CALCULATIONS	OUTPUT
Number of connectors; $N=1.2.5F_r/P_d$ =2 Spacing; P=0.52/2 =0.26M.  	 SECTION A-A 2 Studs at spacing of 260mm

3.2.2.2. DESIGN OF 2.5 METER SPAN

CALCULATION	OUTPUT
PRELIMINARY SIZING. Depth of column, assume 450mm Clear span of beam = 2050mm Effective beam flange = 2050/3=683 Beam depth, assume as previous = 400mm Depth of slab, h_1=150mm Depth of steel section, h_2=250mm Lever arm, $(h_1+h_2)/2$=200mm	

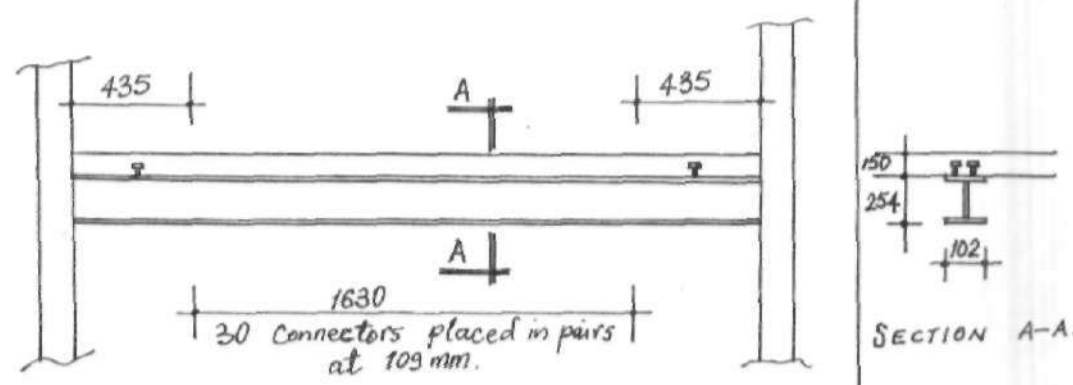
CALCULATIONS	OUTPUT
LOADING. Total slab load, n=7.63KN/M² Equivalent loading per beam is $2nL_x/3$ _____ CP 110 =2x7.63x2.5/3 =12.72KN Weight of steel beam = 2.5KN Weight of partition wall = 10.4KN Total working load = 25.72KN/M	25.72KN/M.
BENDING MOMENTS. Ultimate bending moment, $M=1.7x25.72x22.05^2/8$ =22.97KNM Negative bending moment, $M_n=1.7x25.72x2.05^2/12$ =15.31KNM	M=22.97KNM M_n=15.31KNM
STRUCTURAL STEEL SECTION. $A_s=M/0.87xf_yx0.5\ (h_1+h_2)$ =337mm² $0.87A_s.f_y$=114.85KN $0.4bh_1.f_{cu}$=1229KN Force in concrete is greater than in steel. Neutral axis is in steel. Try steel section as previously. A smaller section could be adequate, but no smaller section satisfies condition for plastic theory application.	N.A. is in steel. UB 254x102x25kg shear capacity of the section was checked previously.

CALCULATIONS	OUTPUT
PLASTIC MOMENTS OF RESISTANCE. Depth of neutral axis; $0.4bY_1.f_{cu}=0.87A_s.f_y$ $Y_1=0.87x3230x350/0.4x683x30$ $=120mm$ Depth of steel centroid; $d=150+127=277mm$ Composite section; $M_p=0.87A_sf_y(d-0.5Y_1)$ $=213.4KNM$ Steel section alone; $M_{ps}=0.87Z_pf_y$ $= 93KNM$ Neglecting the contribution of slab reinforcement, at this stage, in resisting the negative bending moment. $M_{pn}=M_{ps}=93KNM$ Moments ratio; $K=M_p/M_{pn}=0.436$ assume $k=0.5$ So, $(1+k)M_p=1.7wl^2/8$ $M_p=15.31$ KN $M_{psn}=0.5x15.37$ $=7.66KNM$	$Y_1=120mm$ $d=277mm$ $M_p213.4KNM$ $M_{psn}=7.66KNM$

CALCULATIONS	OUTPUT
ADDITIONAL SLAB TOP REINFORCEMENT. Distance of steel top flange from steel centroid; $d_{wc}=d-h_1-t_f$ =118.3mm Depth of centroid of compression steel; $d_{sc}=d-0.5d_{wc}=218.9mm$ Reinforcement cover; $d_r=cc+0.5dia=35mm$ Distance of reinforcement from steel centroid; $d_{sc}-d_r=183.9mm$ Assume an increment of 20 percent of plastic moment of resistance. Let this increment be contributed by reinforcing bars. 20x7.66/100=1.53KNM Force in reinforcement; $F_r=1.53x10^3/183.90=8.33KN$ Area of reinforcement required; $A_r=8.33x10^3/0.87x410=23.4mm^2$ Provide 1Y6 ($28.3mm^2$) Force in the provided bar; $F_r=28.3x0.87x410=10.09KN$	

CALCULATIONS	OUTPUT
Distance of neutral axis from steel centroid; $2d_w.t_w.f_{sd}=A_r.f_{rd}$ $d_w=28.3x0.87x410/2x6/1x0.87x350$ = 16.4mm The value of d_w is correctly given since d_{wc} is greater than d_w. $d_{sc}=d-0.5d_{wc}$ =217.85mm	
ACTUAL NEGATIVE PLASTIC MOMENT OF RESISTANCE. $M_{pn}=M_{psn}+F_r(d_{sc}-d_r)$ =7.66+1.84 =9.50KNM	M_{pn}=9.50KNM
M_p+M_{pn}=213.4+9.50 = 222.90 KNM $M+M_n$=22.97+15.31=38.28KNM The steel section was previously found suitable for plastic design	The section is adequate in flexure
SHEAR CONNECTORS. i. IN THE MID-SPAN REGION. Stud design load; P_d=69.6KN Force in steel at yield; $F_y=0.87A_{sfy}$=983.54KN Number of connectors required; $N=F_y/P_d$=15	

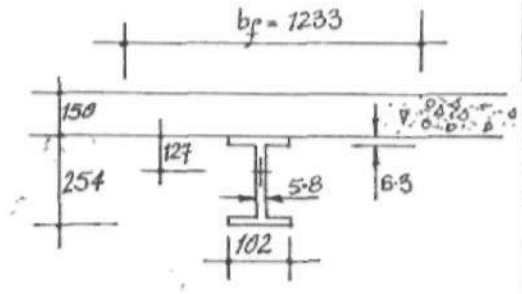
CALCULATIONS	OUTPUT
Number required in the positive moment region is 30. Length of positive moment region, L_n=1.63. Spacing of connectors (in pairs). $P=L_n/2N=0.109$ M. ii. IN THE HOGGING MOMENT REGION. Force in reinforcement at yield; $F_r=0.87A_rf_y$ =10.09KN Number of studs required; $N=1.25F_r/P_d=1$	



3.2.3. DESIGN OF BEAM B_5

This beam is in the Y-axis of the building and has ten equal spans of 4.0 meters. Each span is considered simply supported at both ends. The design of this beam differs slightly from the design of beams in the X-axis. The following design represents each of the ten spans.

CALCULATIONS	OUTPUT
PRELIMINARY SIZING. Length of beam 4000mm Breadth of column 300mm Beam clear span 3700mm Beam effective flange 1233mm Depth of composite section, assume 400mm Depth of concrete slab 150mm	
LOADING. Slab load 7.63KN/M^2 Equivalent loading per unit length of beam; From the smaller slab; $w_1=(nl_x/6)[3-(l_x/l_y)^2]$ -------(3.7.) =(7.63x2.5/6)(3-0.39) =8.3 KN per M. From the longer beam; $w^2=nl_x/3$ --------(3.8.) =7.63x4/3 =10.17KN per M	Total working load is 30.87KN/M

CALCULATIONS	OUTPUT
From the longer beam; $w_2=nl_x/3$ $=7.63xx4/3$ $=10.17$KN per M. Weight of steel beam assume 2.0 KN/M Weight of partition wall = 10.4 KN/M Total working load = 30.87 KN/M	Total working load is 30.87KN/M
BENDING MOMENT. $M = 1.7wl^2/8$ $= 1.7x30.87x3.7^2/8$ $= 89.80$ KNM	M=89.80KNM
STRUCTURAL STEEL SECTION. Area of steel required; $A_s=M/0.87f_y(h_1+h_2)0.5$ $=1475mm^2$ Position of neutral axis; $0.87A_s.f_y=449$ KN $0.4bh_1.f_{cu}=2219$ KN The nuetral axis lies in concrete. The selected steel section is as follows: 	UB254x102x22kg A=2840mm² I=2867cm⁴ Z_p=26I.9cm³ E =225.7cm³

CALCULATIONS	OUTPUT
SHEAR CAPACITY. Limiting value; $F_c=0.87A_w.f_y/1.732$ =258.99KN. Existing value; F=1.7w1/2 =103.38 KN.	The section is adequate in shear.
PLASTIC MOMENT OF RESISTANCE. Depth of neutral axis; $Y_1=0.87A_s.f_y/0.4b:fcu$ =58.45mm Depth of neutral axis; d=277mm Plastic moment of resistance; $M_p=0.87A_s.fy(d-0.5Y_1)$ =214.27 KNM	M_p=214.27KNM
M_p is greater than M. The section is adequate in flexure.	
SHEAR CONNECTORS. Stud design load; P_d=69.6 KN Force in steel at yield; $F_y=0.87A_s.f_y$=869.78KN	

CALCULATIONS	OUTPUT
Number of stud connectors; $N=F_y/p_d=13$ The total number required is 26. Stud spacing, when placed in singles; $P=L_n/2N$ $=3700/26=142mm$	26 connectors in singles at a spacing of 142mm

3.2.4. DESIGN OF PARAPET WALL BEAMS.

These are designed as encased beams. The design is by elastic method of clause 16; CP 117; part 2. In this design the the longitudinal bending stresses must not exceed the permissible stresses for steel and concrete given in BS 449 and CP 114.

3.2.4.1. DESIGN DATA.

Permissible stress for steel - $230N/mm^2$

Permissible stress for concrete - $10N/mm^2$

Modular ratio, m, 15

Universal beam UB254x102x25kg.

Unpropped construction method.

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3.2.4.2. DESIGN OF BEAM B_8.

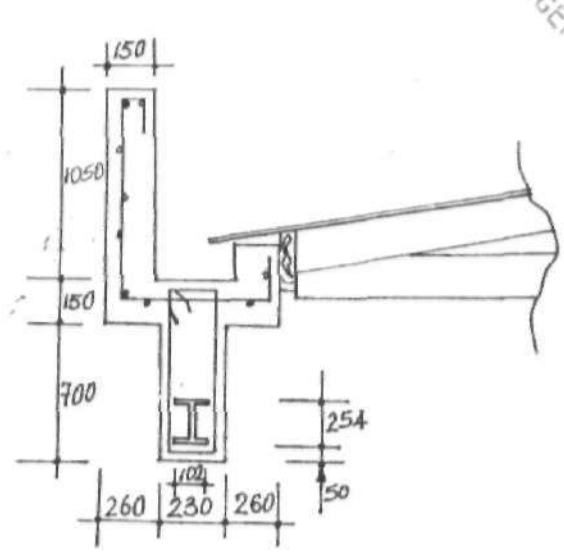


FIG. 3.2. Encased beam B_8.

CALCULATIONS	OUTPUT
LOADING. Clear span of beam = 3848mm Weight from roof = 4.35KN Live Load from roof = 2.25KN Weight from ceiling = 0.238KN Weight of gutter = 8.244KN Self wight of beam = 2.52KN Weight of steel beam = 2.5KN Weight of formwork is neglected.	LOAD=20.102KN/M
BENDING MOMENT. $M=wl^2/8$ $= 20.102 \times 3.848^2/8$ = 37.21 KNM.	
STEEL BEAM PROPERTIES UB 254x102x25kg/m $A_s=3230mm^2$ $I_s=3046cm^4$ $t_f=8.4mm$ $t_w=6.7mm$	
POSITION OF NEUTRAL AXIS. Moment of area below the surface of top flange; $mA_s(d-h_1)+(A_1-A_s)(d_1-h_1)$ $=15x3230x127+(304x230-3230)152$ $=16290cm^3$	

CALCULATIONS	OUTPUT
Moment of area above the top flange; $=.5bh_1^2=0.5x230x396^2$ $=18034cm^3$ The neutral axis lies above the top flange of steel, that is in concrete. Its position is given by; $mA_s(d-Y_1)+(A_1-A_s)=0.5bY_1^2$ ------(3.9.) Where d=523mm $A_1=230x304mm^2$ $d_1=548$ $Y_1=387.5mm$	N.A. 387.5mm
MOMENTS OF INERTIA. $I_{comp}=bx^3/3m+I_s+A_s(d-Y_1)^2+I_1/m$ --------(3.10.) I_1=moment of inertia of concrete below the surface of top flange $=50386.68cm^4$ $I/m=3359.11cm^4$ $bx^3/3m=2973.93cm^4$ $I_s=3046cm^4$ $A_s(d-Y_1)^2=5930.36cm^4$ $I_{comp}=15309.4cm^4$	$I_{com}=15309.4$
SECTIONAL MODULAS For concrete; $Z_c=mI/Y_1=5926.22cm^3$ For steel; $Z_s=I/(D+h_1-Y_1)=474.71cm^3$	

CALCULATIONS	OUTPUT
where D is the overall depth below the surface of top flange of steel.	
STRESSES. In concrete in compression; $f_c = M/Z_c = 6.26$ N/mm^2 In steel in tension; $f_s = M/Z_s = 78.40$ N/mm^2 Both stresses in concrete and steel are below the permissible values given, respectively, in table 6, CP 114 and table 2, BS 449. The values are 10 and 230 N per square millimeters, respectively.	Section is adequate in flexure

The following tables show the result of calculations for the other beams.

TABLE 3.3. Final results of design of composite beams.

BEAM	SPAN(M)	STEEL SECTION	EXTRA SLAB REINFORC.	CONNECTORS IN MID-SPAN (PAIRS)	SPACING OF OF CONNEC-TORS (mm)	CONNECTORS AT ENDS OF SPAN	LENGTH OF NEGATIVE MOMENTS REGION (M)
B_1 THREE SPANS 5.0,2.5,5.0	5.0	UB 254 X 102 X 25KG	6Y6	30	234	2	0.745
	2.5		1Y6	30	109	1	0.435
B_2 THREE SPANS 5.0,2.5,5.0	5.0		4Y6	30	106	1	1.705
	2.5		1Y6	30	98	1	1.732
B_3 THREE SPANS 4.7,3.0,4.3	4.7		4Y6	30	285	1	0.565
	3.0		1Y6	30	97	1	0.724
	4.3		3Y6	30	128	1	0.654
B_4 THREE SPANS 4.7,3.0,4.3	4.7		5Y6	30	157	1	1.099
	3.0		2Y6	30	133	1	0.436
	4.3		5Y6	30	143	1	1.006

TABLE 3.3. (cont'd).

BEAM	SPAN	STEEL SECTION	EXTRA SLAB REINFORCEMENT	CONNECTORS AT MID-SPAN (IN PAIRS)	SPACING OF CONNECTORS (M)	CONNECTORS AT ENDS OF SPAN	LENTGH OF NEG. MOMENT REGION (M)
B_5 TEN SPANS	ALL SPANS	UB 254x102x22	N.A.	26	0.142	N.A.	N.A.
B_6 TEN SPANS	ALL SPANS	UB254x102x22	N.A.	26	0.142	N.A.	N.A.
B_7 FOUR SPANS 6.0,4.7,3.0 & 4.3	6.0 SPAN	UB305x102x33	5Y8	38	0.196	2	1.704
	4.7 SPAN	UB254x102x25	5Y6	30	0.157	1	1.099
	3.0 SPAN	DITTO	2Y6	30	0.133	1	0.436
	4.3 SPAN	DITTO	2Y6	30	0.143	1	1.006
B_8 TEN SPANS	ALL SPANS	UB254x102x25	N.A.	N.A.	N.A.	N.A.	N.A.
B_9 THREE SPANS	ALL SPANS	DITTO	N.A.	N.A.	N.A.	N.A.	N.A.

3.3. DESIGN OF COMPOSITE COLUMNS.

The columns are considered continous passed joints, at floor level. Floor beams are considered attached to column faces in both axis. Column design loads and bending moments are got by the method of limited frame. Alternate beams on both ends of columns are partially loaded (with dead loads only) while the other beams are fully loaded.

Calculations in a tabular form, for frame F_1, are given in the following pages. For the rest of the frames only the final results are tabulated.

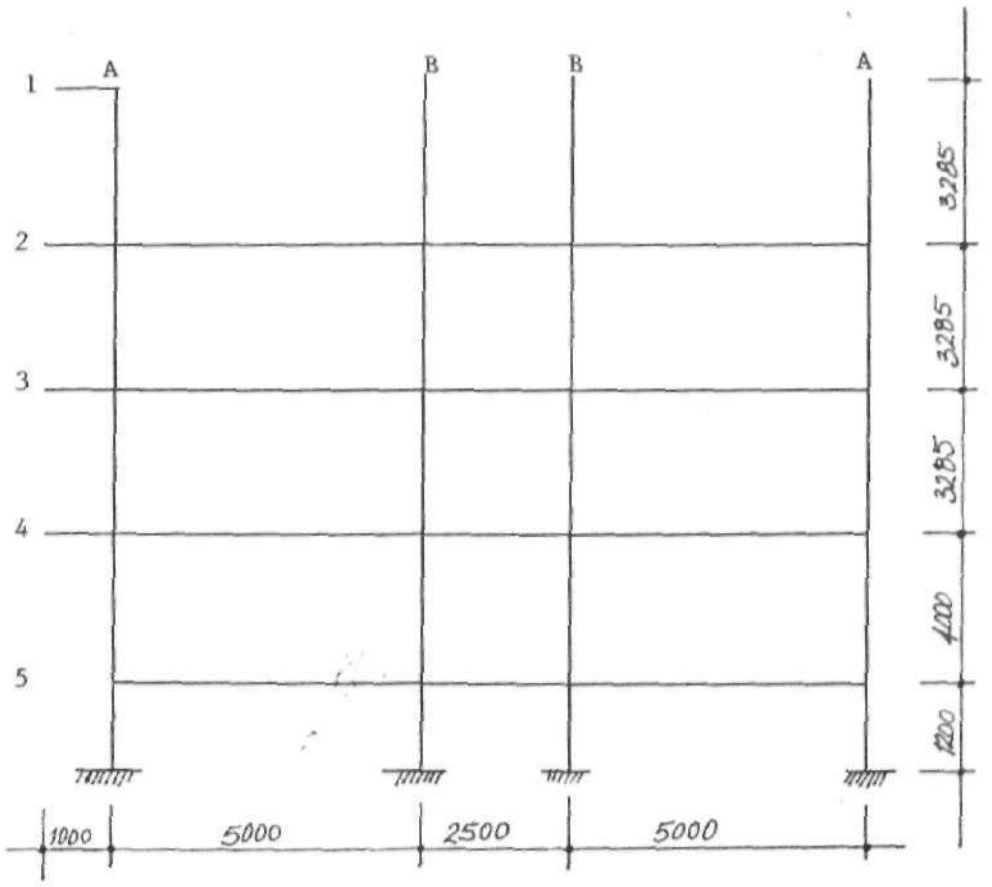


FIG. 3.4. Frame F_1 showing columns.

3.3.1. DESIGN DATA.

Structural steel grade 50

Concrete grade 30

Mild steel bar dia. 10mm

3.3.2. DESIGN OF COLUMNS IN FRAME 1.

There are two column types in this frame. These are columns type A and type B. Columns type A have assumed composite section of 300mm by 252mm while column type B have a cross-section of 252mm by 252mm

3.3.2.1. DESIGN OF COLUMN TYPE A.

Materials cross-section

Concrete - 252x300 $=75600mm^2$

Steel UC152x152x37 $=4740mm^2$

Dia. 10mm bar $=314.2mm^2$

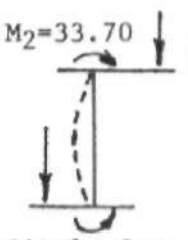
COLUMN LENGTH A_1-A_2.	OUTPUT
LOADING. **Beams in the Y-axis (simply jointed)** F_2=2x0.5x3.99x20.102x1.7 =80.45KN **Columns self weight;** Concrete = 0.252x0.30x24x3.825x1.7 = 11.80KN Steel = 3.7 x 3.825x1.7 =24.06KN **Total dead load** =122.21KN **Live load** =2x0.5x3.99x2.25x1.7 =15.27KN	Design load 131.60KN

CALCULATIONS	OUTPUT
MOMENTS. At the upper end of column load is applied at an accentricity of 1.0; M_1=95.72x1.0 = 95.72KNM At the lower end of column; From cantilever slab to the left, M_{2L}=4.0x7.63x1.0^{2}x0.5x1.7 =25.94KNM From beam to the right M_{2R}=(1.7x26.12x4.848^2)/12 =86.97KNM Out of balance M_2=61.03KNM	M_1=95.72KNM M_2=61.03KNM  Bent in single curvature.
PROPERTIES OF CROSS-SECTION Area of concrete; A_c=75600-4740-314 =70546mm^2 Ultimate load; N_u=4740x304+314.2x356+70546x13.5 =2505.19KN Concrete contribution factor; q=952.37/2502.19 =0.38	Moment ratio $K=M_2/M_1$ =0.643 Positive

CALCULATIONS	OUTPUT
Moments of inertia; $I_r=314.2(106.30)^2=181.48cm^4$ $I_s=2218+181.48=2399.48cm^4$ $I_c=(252x300^3/12)-2399.48$ $=54300.52cm^4$ Radius of gyration $r^2=(250I_s+0.24I_c.f_{cu})/N_u$ $r^2=3955.12mm^2$ $r=62.89mm$	
PLASTIC MOMENT OF RESISTANCE Cross section other dimensions(mm); b = 252 d_c=50 h = 300 d_r=30 t_w = 81 A_f=1748 t_f = 11.5 Depth of neutral axis; $Y_1=(300x8.1)/[2x8.1+4x252(30/304)]$ $=96.62mm$ $d_c+t_f=50+11.5=61.5mm$ $d_{wc}=Y_1-(d_c+t_f)=35.12$ $M_p=[0.5x4770(300-96.62)-1748(50-35.12)$ $-35.12x81(96.62-35.12]304$ $+0.87x314.2410(0.5x300-30).$ $=132.87KNM$	Y_1=96.62 d_c+t_y=61.2 $Y_1>d_c+t_f$ d_{wc}=35.12 M_p=132.87KNM

CALCULATIONS	OUTPUT
LOAD RATIOS. The load ratio of axial load, N to ultimate load N_u is designated S_1. This is finally found from load-slenderness ratio curve. Slenderness ratio; $L/r=3285/62.89=52.23$ From load-slenderness ratio curve; $S_1=0.88$ For a concrete contribution factor of q=0.38 $S_{20}=0.9q^2+0.2(\leq 0.75)$ $=0.33$ For a moment ratio of K=0.643 $S_2/S_{20}=[90-25(2k-1)(1.8-q)-L/r] \div [30(2.5-q)]$ $=0.634$ But S_2/S_{20} should be between zero and one. So, $S_2=0.643x0.32=0.21$ For major axis bending S_3 is approximately equal to zero. So, $S_3=0$	$S_1=0.88$ $S_{20}=0.33$ $S_2=0.21$ $S_3=0$

CALCULATION	OUTPUT
COLUMN STRENGTH. M=95.71KNM M_p=132.89KNM N =131.60KN N_u=2505.19KN N/N_u=0.053 $i_p=S_1-(S_1-4s_3)(M/M_p)-45_3(M/M_p)^2$ =0.22 M is less than M_p N/N_u is less than i_p	The section is adequate in flexure.
COLUMN LENGTH A_2-A_3	
Maintain column cross-section as length as length A_1-A_2 LOADING. From beam A.2-B.2; F_1=1.7x36.9x0.5x4.828 =152.48KN From beams in the minor axis; F_2=22.57x3.99x1.7 =90.05KN From column length A_1-A_2 L_c=131.60KN	Design load is 409.94KN

CALCULATIONS	OUTPUT
Factored self weight; N_c=35.86 KN Design load; N=409.94KN	
MOMENTS. Hogging moment of beam A.2-B.2; M_{pn}=62.52KNM From cantilever slab to the left; M_{1L}=(4.2x4+10.4)0.924 =25.13KNM Out of balance moment at the upper end of column; M_1=62.52-25.13 =37.39KNM At the lower end of column; $M_{2R}=0.5gl^2$=4.11KNM, From cantilever to the left, M_{2L}=(7.63x4+10.4)0.924 =37.81KNM Out of balance moment at the lower end of column $M_2=M_{2L}-M_{2R}$=33.70KNM	M_1=37.39 M_2=33.70  Single Curvature.

CALCULATIONS	OUTPUT
But S_2/S_{20} should be between zero and one, so S_2=0.049 *For major axis bending;* S_3=0 (approx.)	S_2=0.049 S_3=0
COLUMN STRENGTH. M=37.39 KNM M_p=132.89 KNM N=409.94 KN N_u=2505.19 KN N/N_u=0.164 i_p=0.67 M is less than M_p N/N_u is less than i_p	The section is adequate in flexure.
COLUMN LENGTH A_3-A_4	
Maintain column section as for A_2-A_3	
LOADINGS. From beam A.3-A.4 to the right; F_{1R}=36.9x0.5x4.848x1.7 =152.43 KN From cantilever slab to the left; F_{1L}=1.7(10.40+4x4.2) =46.24 KN From beams in the Y - axis; F_2=1.7x22.57x3.99x2x0.5 =90.05 KN	

CALCULATIONS	OUTPUT
Factored self weight; N_c=35.86 KN Load from column above; L_c=409.94 KN Desing axial load; N = 734.52 KN	N=734.52KN
MOMENTS. At the upper end of column. From beam to the right; $M_{1R}=M_{pn}$=62.52 KNM From cantilever slab to the left; M_{1L}=(4.2x4+10.4)0.924 =25.13 KNM Out of balance moment M_1=18.7 KNM At the lower end of column $M_2=0.5gl^2/12$ =5.48 KNM	M_1=18.7 M_2=5.48  double curvature.
MOMENTS RATIO. Condition of curvature is double So moment ratio is negative $k=(M_2/M_1)$= - 0.29	Negative
PROPERTIES OF CROSS-SECTION These are the same as in the previous calculations.	

CALCULATIONS.	OUTPUT
PLASTIC MOMENT OF RESISTANCE. Same as in the previous calculations.	Y_1=96.62mm M_p=132.87KNM
LOAD RATIOS. S_1=0.88 S_{20}=0.33 From similar calculations as previously and for a moment ratio of k=-0.29 S_2/S_{20}=1.476 But S_2/S_{20} should be between zero and one. therefore, S_2/S_{20}=1 For major axis bending S_3=0(approx.)	S_2=0.88 S_{20}=0.33 S_2=0.33 S_3=0
COLUMN STRENGTH M = 18.7 KNM M_p = 132.89 KNM N = 734.52 KNM N_u = 2505.19 KNM N/N_u=0.293 $i_p = S_1-(S_1-S_2-45_3)(M/M_p)-4S_3(M/M_p)^2$ $= 0.808$ M is less than M_p N/N_u is less than i_p	The Section is adequate in flexure

CALCULATIONS	OUTPUT
COLUMN LENGTH A_4-A_5	
Maintain column section as previously.	
LOADING. From beam A.4-B.4, to the right; F_1=152.43 KN From beams in the Y-axis; F_2=90.05 KN Column self weight; N_c=35.86 KN From column above; L_c=733.52 KN Design load; N=1012.86	N=1012.86
MOMENTS. At the upper end of column. Hogging moment of beam A.4-B.4 M_{pn}=62.52 KNM From cantilever slab to the left; M_{1L}=25.13 KNM Out of balance moment M_1=18.70 KNM At the the foot of column, assume; M_2=1.15M_1=21.51 KNM.	M_1=18.70 M_2=21.51  Double Curvature.

CALCULATIONS	OUTPUT
MOMENTS RATIO. Condition of curvature is double. Moment ratio is therefore negative. $k = M_2/M_1 = -0.87$	Negative
PROPERTIES OF CROSS-SECTION These are the same as those of the previous column lengths.	
PLASTIC MOMENT OF RESISTANCE. Depth neutral axis; $Y_1 = 96.61$ mm Plastice moment; $M_p = 132.89$ KNM	
LOAD RATIOS. $S_1 = 0.88$ $S_{20} = 0.33$ From similar calculation as before and for a moment ratio of k = - 0.87 $S_2/S_{20} = 1.944$ But s_2/S_{20} should be between zero and one. So, $S_2/S_{20} = 1$ $S_2 = 0.33$ and $S_3 = 0$ (aaprox.)	

CALCULATIONS	OUTPUT
COLUMN LENGTH. M =35.95 KNM M_p =132.89 KNM N =1012.86 KN N_u =2505.19 KN N/N_u=0.49 i_p=0.709 M is less than M_p N/N_u is less than i_p	The section is adequate in flexure.

3.3.2.2. DESIGN OF COLUMN TYPE B.

Material cross-section.

Concrete - 252x252=63504mm^2

Steel - UC152x152x37Kg=4740mm^2

Dia. 10mm bar =314.2mm^2

COLUMN LENGTH B_1-B_3	
LOADING. From beam A.2-B.2; F_{IL} =369x0.5x4.848x1.7 =152.43 KN From beam B.2-B.2; F_{IR}=12.72x0.5x2.348x1.7 =25.38 KN From beamns in the Y-axis; F_2=30.87x3.99x2x05x1.7 =209.39 KN	Design load =451.84 KN

CALCULATIONS	OUTPUT
Self weight of column; F_C=32.32 KN Weight of column B_1-B_2 above; L_C=32.32 KN Design load; N=451.84 KN	
MOMENTS. At the upper end of column $M_{1L}=M_{pn}$=26.52 KNM $M_{IR}=12.72x1.7x2.348^2/12$ =9.93 KNM Out of balance moment; M_1=16.59 KNM At the lower end of column; $M_{2L}=62.73x0.5x4.848^2/12$ =61.43 KNM $M_{2R}=M_{pn}=7.66x0.5$ =3.83 KNM Out of balance moment; M_2=57.6 KNM	M_1=52.59 M_2=57.60 Bent in double curvature. 

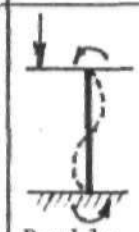
CALCULATIONS	OUTPUT
MOMENTS RATIO. The column is bent in double curvature. The sign of moment ratio is therefore negative. $k=M_1/M_2=-0.913$	Negative
PROPERTIES OF CROSS-SECTION. Area of concrete, $A_c=63504-4740-314.2$ $=58450\ mm^2$ Ultimate load; $N_u=4740x304+3142x356$ $+58450x13.5$ $=2341.87\ KN$ Concrete contribution factor; $q=789.07/2341.89$ $=0.337$ Moments of inertia $I_r=314.2(126-30)^2=289.57cm^4$ $I_s=2218+289.57=2507.57cm^4$ $I_c=(252^4/12)-2507.57$ $=31098.75cm^4$ Radius of gyration; $r^2=(250I_s+0.24I_c x f_{cu})/N_u$ $r^2=2772mm^2$ $r=52.65mm$	

CALCULATIONS	OUTPUT
PLASTICE MOMENTS OF RESISTANCE. Dimension in the cross-section; $b=h=252mm$ $t_w=8.1mm$ $d_r=30mm$ $t_f=11.5mm$ $d_c=50mm$ $A_f=1748mm$ Depth of neutral axis; $Y_1=ht_w/[2t_w+0.46(f_{cu}/0.87f_y)]$ $=78.06mm$ Plastic moment; $M_p=115.70$ KNM	$Y_1=78.06$ $Y_1>d_c+t_f$ $M_p=115.70$
LOAD RATIOS. Slenderness ratio; $L/r = 3285/52.65 \approx 62.39$ From load - slenderness ratio curve; $S_1=0.78$ From a concrete contribution factor of $q = 0.337$ $S_{20}=0.9q^2+0.2=0.30$ For a moment ratio of $k=-0.91$ $S^2/s_{20}=0.44$, so $S_2=0.13$ For major axis bending, $S_3=0$.	

CALCULATIONS	OUTPUT
COLUMN STRENGTH. M = 57.60 KNM M_p =115.70 KNM N =457.84 KN N_u =2341.89 KN N/N_u=0.193 $i_p=s_1-(S_1-S_2-4S_3)(M/M_p)-4S_3(M/M_p)^2$ =0.460 M is less than M_p N/N_u is less than i_p	The Section is adequate
COLUMN LENGTH B_3-B_4	
The column maintain the same cross-section as the column length B_2-B_3 above	252x252
LOADING. From beam A.3-B.3; F_{IL}=36.9x0.5x4.848x1.7 =152.43 KN From beam B.3-B.3; F_{IR}=12.72x0.5x2x2.38x1.7 =25.38 KN From beams in the Y-axis; F_2=30.87x0.5x3.99x2x1.7 =209.39 KN	

CALCULATIONS	OUTPUT
Self weight of column; F_c=32.32 KN Loading from column B_2-B_3; L_c=451.84 KN	Design load N=871.36KN
MOMENTS. At the upper end of column; $M_{IL}=M_{pn}$=62.52KNM $M_{IR}=1.7wl^2/12$=9.93 KNM Out of balance moment M_1=52.59 KNM At the lower end of column; $M_{2L}=1.7wl/12$=122.86 KNM $M_{2R}=M_{pn}$=7.66 KNM Out of balance moment; M_2=115.20 KNM	M_1=52.59 M_2=115.20  Double curvature.
MOMENTS RATION The column is bent in double curvature. $k=M_1/M_2$=-0.46	Negative
PROPERTIES OF CROSS-SECTION They are the same as those of column length B_2-B_3 A_c=58450 mm^2 N_u=2341.89 mm^2 q=0.337 I_r=289.57cm^4 I_s=2507.57 cm^4	

CALCULATIONS	OUTPUT
I_c=31098.75 cm^4 r=52.65 mm	
PLASTIC MOMENT RESISTANCE. Same as in the previous calculations.	Y_1=78.06 M_p=115.70
LOAD RATIOS. From similar calculations as previously; S_1=0.78 S_{20}=0.30 S_2/S_{20}=0.42, but this should be between zero and one. So, S_2/S_{20}=0 S_2=0.13 S_3=0	
COLUMN STRENGTH. N = 871.36 KN N_u=2841.89 KN M =115.20 KNM M_p=115.70 KNM N/N_u=0.372 i_p=0.395 M is less than M_p N/N_u is less than i_p	The section is adequate.
COLUMN LENGTH B_4-B_5	

CALCULATIONS	OUTPUT
The column maintains the same cross-section as that of the other columns above it.	
LOADING. From beams at the upper end of the column; F_1=387.20 KN Self weight of column; F_c=32.32 KN From the column above; L_c=871.36 KN Design load; N = 1290.88 KN	N=1290.88
MOMENTS. At the upper end of column; M_1=26.30 KNM At the foot of column M_2=1.15 M_1=30.25 KNM	 Double curvature bending.
MOMENTS RATIO. Double curvature bending k=-0.87	Negative
PROPERTIES OF CROSS-SECTION. All are also in the previous calculations.	
PLASTIC MOMENT OF RESISTANCE. Y_1=78.06 M_p=115.70	

CALCULATIONS	OUTPUT
LOAD RATIO From similar calculations as previously; $L/r = 400/52.65 = 75.97$ $S_1=0.65$ $S_{20}=0.30$ $S_2 =0.12$ $S_3 =0$	
COLUMN STRENGTH $N = 1290.88$ KN $N_u = 2341.89$ KN $M = 30.25$ KNM $M_p =115.70$ KNM $N/N_u=0.0.48$ $i_p=0.51$ M is less than M_p N/N_u is less than i_p	The section is adequate in flexure.

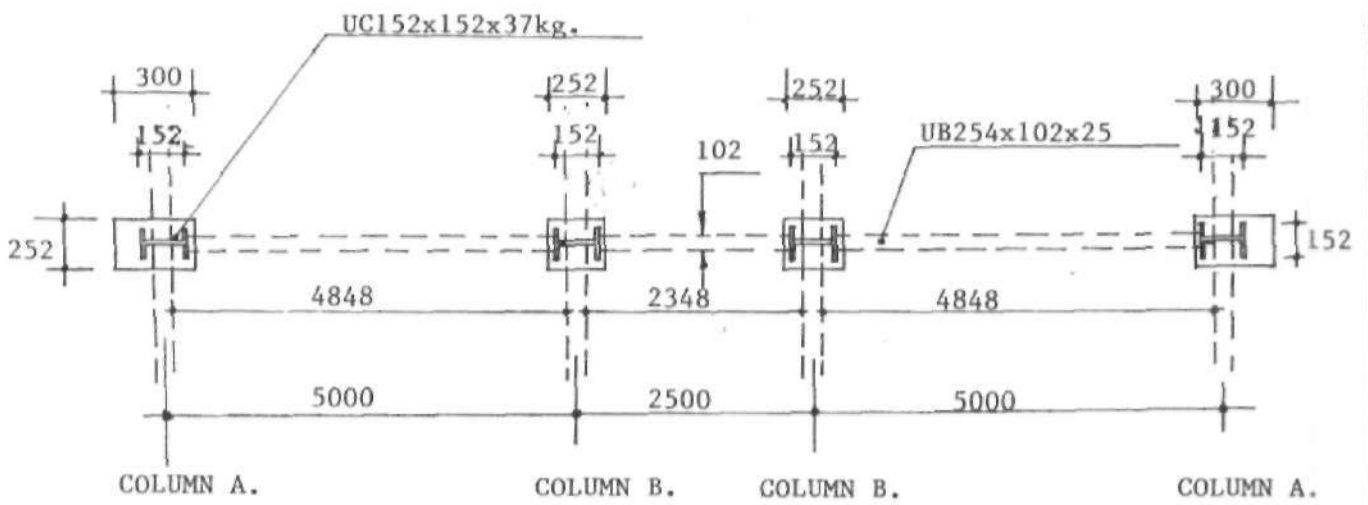


FIG. 3.5. Plan of frame F_1 showing columns' cross-section.

TABLE 3.4. Final results of design of composite columns.

COLUMN	LENGTH	HEIGHT mm	ASSUMED SECTION STEEL	ASSUMED SECTION AVERALL	M (KNM)	M_p (KNM)	N (KN)	N_u (KN)	N_u	i_p
A & H	A_1-A_2	3285	UC152x152x37	300x252	131.60	132.89	131.60	2505.19	0.053	0.22
	A_2-A_3	3285	ditto	ditto	37.39	132.89	409.94	2505.19	0.164	0.67
	A_3-A_4	3285	ditto	ditto	18.70	132.89	734.52	2505.19	0.293	0.803
	A_4-A_5	4000	ditto	ditto	35.95	132.89	1012.86	2502.19	0.409	0.709
B & G	B_1-B_2	3285	ditto	252x252	-	115.70	32.32	2341.89	0.041	0.39
	B_2-B_3	3285	ditto	ditto	57.60	115.70	451.84	2341.89	0.193	0.46
	B_3-B_4	3285	ditto	ditto	57.60	115.70	871.36	2341.89	0.372	0.46
	B_4-B_5	4000	ditto	ditto	30.25	115.70	1290.88	2341.89	0.48	0.51
C & L	C_1-C_2	3285	UC152x152x37	300x252	41.45	132.89	117.84	2505.19	0.046	0.800
	C_2-C_3	3285	ditto	ditto	31.26	132.89	310.01	2505.19	0.099	0.830
	C_3-C_4	3285	ditto	ditto	31.26	132.89	484.49	2505.19	0.190	0.830
	C_4-C_5	4000	ditto	ditto	35.95	132.89	669.89	2505.19	0.180	0.400

$i_p = S_1 - (S_1 - S_2 - 4S_3)(M/M_p) - 4S_3(M/M_p)^2$

REMARK: In all cases the Section is adequate

TABLE 3.4. (cont'd).

COLUMN	LENGTH	HEIGHT mm	ASSUMED SECTION STEEL	AVERALL	M (KNM)	M_p (KNM)	N (KN)	N_u (KN)	N/N_u	i_p
D & K	D_1-D_2	3285	ditto	252x252	26.87	115.70	88.18	2341.89	0.038	0.590
	D_2-D_3	3285	ditto	ditto	48.40	115.70	425.35	2341.89	0.180	0.510
	D_3-D_4	3285	ditto	ditto	48.40	115.70	762.52	2341.89	0.210	0.510
	D_4-D_5	4000	ditto	ditto	29.71	115.70	1101.16	2341.89	0.470	0.720
E & J	E_1-E_2	3285	UC152x152x37	252x252	4.85	115.70	53.8	2341.89	0.02	0.66
	E_2-E_3	3285	ditto	ditto	7.66	115.70	190.60	2341.89	0.08	0.75
	E_3-E_4	3285	ditto	ditto	7.66	115.70	327.40	2341.89	0.14	0.75
	E_4-E_5	4000	ditto	ditto	8.81	115.70	465.62	2341.89	0.20	0.74
F	F_1-F_2	3285	ditto	300x252	131.60	132.89	131.60	2505.19	0.053	0.22
	F_2-F_3	3285	ditto	ditto	37.39	132.89	409.94	2505.19	0.164	0.67
	F_3-F_4	3285	ditto	ditto	18.70	132.89	734.52	2505.19	0.293	0.803
	F_4-F_5	4000	ditto	ditto	63.94	204.65	1336.10	3015.49	0.44	0.75
M	M_4-M_5	4000	ditto	252x252	58.93	115.70	334.59	2341.59	0.140	0.350

REMARK: In all cases the Section is adequate.

3.4. DESIGN OF BEAM-COLUMN CONNECTIONS.

Two types of beam-column connections are design in this work. For beams that span in the X-axis of the building rigid connections are designed, while simple connections are designed for beams in the Y-axis. Rigid connection is by means of welding end plates to the beam ends and bolting same to the column steel flanges. Simple connection is by providing seating and web cleats to the beam ends and bolting these to the web of steel in column.

3.4.1. DESIGN DATA.

1. Structural steel grade 50

   UB254x102x25kg per M.

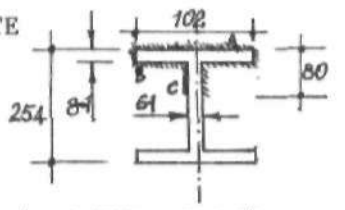
   UC152x152x37kg per M.

2. Shop fillet weld to clause 54,BS 449.
3. Black bolts of strength grade designation 4.6 to clause 50, BS449 and BS4190.
4. Spacing of bolt holes in columns and beams to comply with table 21, BS 449.
5. Weld, to resist end-moment, provided round top flange of beam down to a distance of 80mm from the surface of top flange.
6. Web and seating cleats 75x750x8mm.

3.4.2. DESIGN OF RIGID CONNECTIONS.

This is provided to beams, B_1, B_2, B_3, B_4, and B_7. Complete calculation for designing connections for beam B_1 is shown. For the rest of the beams the result of calculations are shown in the following tables.

3.4.2.1 CONNECTIONS FOR BEAM B_1

CALCULATIONS	OUTPUT
CONNECTION FOR 5.0M. SPAN End moments = 72.53 KNM Shear force = 152.06 KN	
WELD TO BEAM-END PLATE  Assume 6mm weld, to carry moment. To find top flange weld centre of gravity take moment of weld about the surface of top flange; Weld A = 102x1x0 =0 Weld B = 2x48x8.4x1 =806.4 Weld C =2x65.8(32.9+14.4)x1=6224.68 Total =7031.08 Length of weld group; L_w=102+95.9+2x65.8 =329.5mm Centre of gravity from to flange; C_g=7031.08/329.5 =21.3mm Lever arm; Z=254-21.3 =232.7mm Force applied to centre of gravity of weld group; F = M/Z =72.53/0.2327 =311.69 KN	L_w=329.5mm C_g=21.3mm Z=232.7mm F=311.69

CALCULATIONS	OUTPUT
Load per mm run of weld; = 311.69/329.5 = 0.95 KN/mm From permissible load table for butt weld to grade 50 steel, 10mm weld has a load capacity of 1.12 KN/mm --- Weld round remainder of web and bottom flange, to carry shear. Length of weld group; L_w=102+2x172+95.9 =541.9mm Load per mm run of weld; = 152.06/541.9 =0.28 KN/mm Use 5mm fillet weld(0.56KN/mm)	Use 10mm fillet weld to resist tensile force created by end moment. Provide 5mm F.W. to resist vertical shear.
BOLTS TO COLUMN-END PLATE Bolts to carry moment. Assume 4 bolts Load per bolt =311.69/4 =77.9 KN From table 20, B. 449, part 2, use 4 dia., 33mm black bolts grade 4.6 (90.2KN). These are in the upper group.	4dia.33mm

CALCULATIONS	OUTPUT
Bolts to carry vertical shear; Assume dia 24mm black bolt grade 4.6 bolt value = 36.2 KN. Single shear is critical for flange thickness of 11.5mm. A number of bolts N = 72.53/36.2 =2 bolts	2 dia 24mm bolts.
THICKNESS OF END-PLATE. Let 2 bolts be placed at a distance of 30mm (from edge of hole) on both sides of top flange. Flange Force = 72.53/(0.254-0.0084) = 295.3 KN Elastic moment induced in plate: m = (30/2)x(295.3/2) = 2214.75KN mm Plastic modulus required; $Z_p=2214.75x10^3x1.70/245x10^3$ $=15.37cm^3$ Let breadth of plate, b, be 120mm. Thickness of plate; $t^2=4Z_p/b$ ----------(3.11.) $=4x15.37x10^3/120$ $=512.33mm^2$ t=22.6mm say, t=25mm	End plate 400x120x25mm

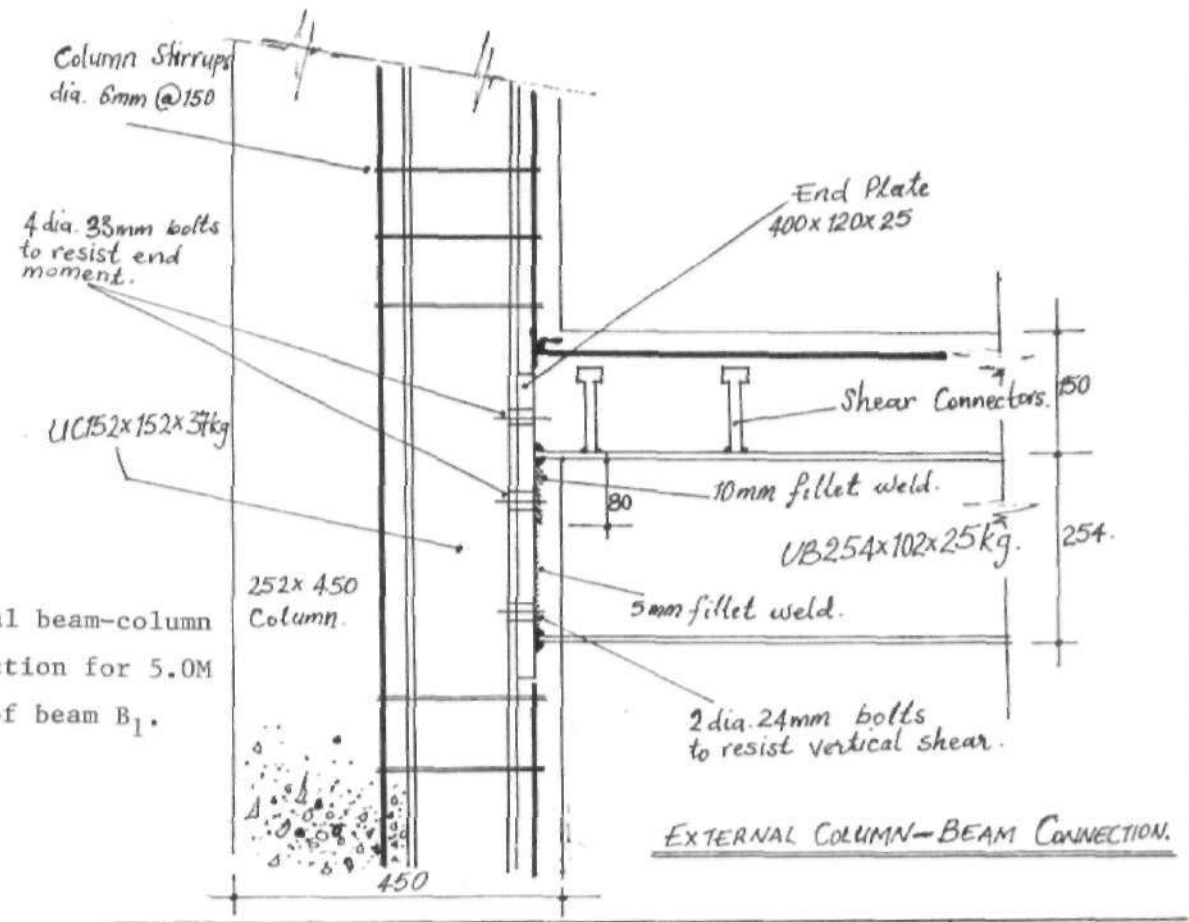


FIG. 3.6.
Typical beam-column connection for 5.0M span of beam B_1.

CONNECTION FOR 2.5M. SPAN

End moment = 10.04 KNM

Shear force = 43.72 KN

WELD TO BEAM-END PLATE.

Weld to carry moment;

Moment of weld about surface of

top flange;

M = 102x1x0+2x48x4.8x1+2x65.6x45.3

= 7031.08

Length of weld group;

L_w=329.5mm

Centre of gravity from top flange;

C_g=7031.08/329.50

=21.3

CALCULATIONS	OUTPUT
Lever arm Z = 254 - 21.30 = 232.7mm Force applied to C.G. of weld group; F = M/Z =10.04/0.2327 =43.15 KN Load per millimeter run of weld; =43.15/329.5 =0.13 KN/mm From permissible load table for fillet weld to steel grade 50, 5mm weld has a load of 0.56KN per mm run.	Use 5 mm fillet weld
Weld to carry shear force; This is provided round reminder of web and bottom flange. Shear force; F= 43.72 KN Length of weld group; L_w=102+2x172+95.9 =541.9mm. Load per mm. run of weld; =43.72/541.9 =.081KN/mm Use 3mm fillet weld (0.34KN/mm)	Use 3mm fillet weld

CALCULATIONS	OUTPUT
BOLTS TO COLUMN-END PLATE Bolts to carry moment: Assume 4 bolts Load per bolt. =43.15/4 =10.79 KN Use 4 dia. 14mm black bolts (12.3KN) Bolt to carry vertical shear; Diameter 20mm black bolts in single shear has a value of 25.1KN (single shear is critical for flange thickness of 11.5mm) Shear force; F=43.72KN Number of bolts; N = 43.72/25.1 =1.74 Use 2 dia. 20mm grade 4.6	Use 4 dia 14mm Use 2 dia 20mm black bolts.
THICKNESS OF END PLATE Assume a clear distance of 30mm from bolt hole to top flange on both sides of the flange. Flange force; =10.04/(0.254-0.0084) =40.88 KN Elastic moment induced in plate; M = (30/2)(40.88/2) = 036.6KN mm	

CALCULATIONS	OUTPUT
Plastic modulus required; $Z_p = 306 \times 10^3 \times 1.7 / 245 \times 10^3$ $= 2.13 cm^3$ Thickness of plate; $t^2 = 4Z_p / b$ $= 4 \times 6.38 \times 10^3 / 120$ $= 70.9 mm^2$ $t = 8.4$ mm say, $t = 10$mm	Use plate 400x120x10mm

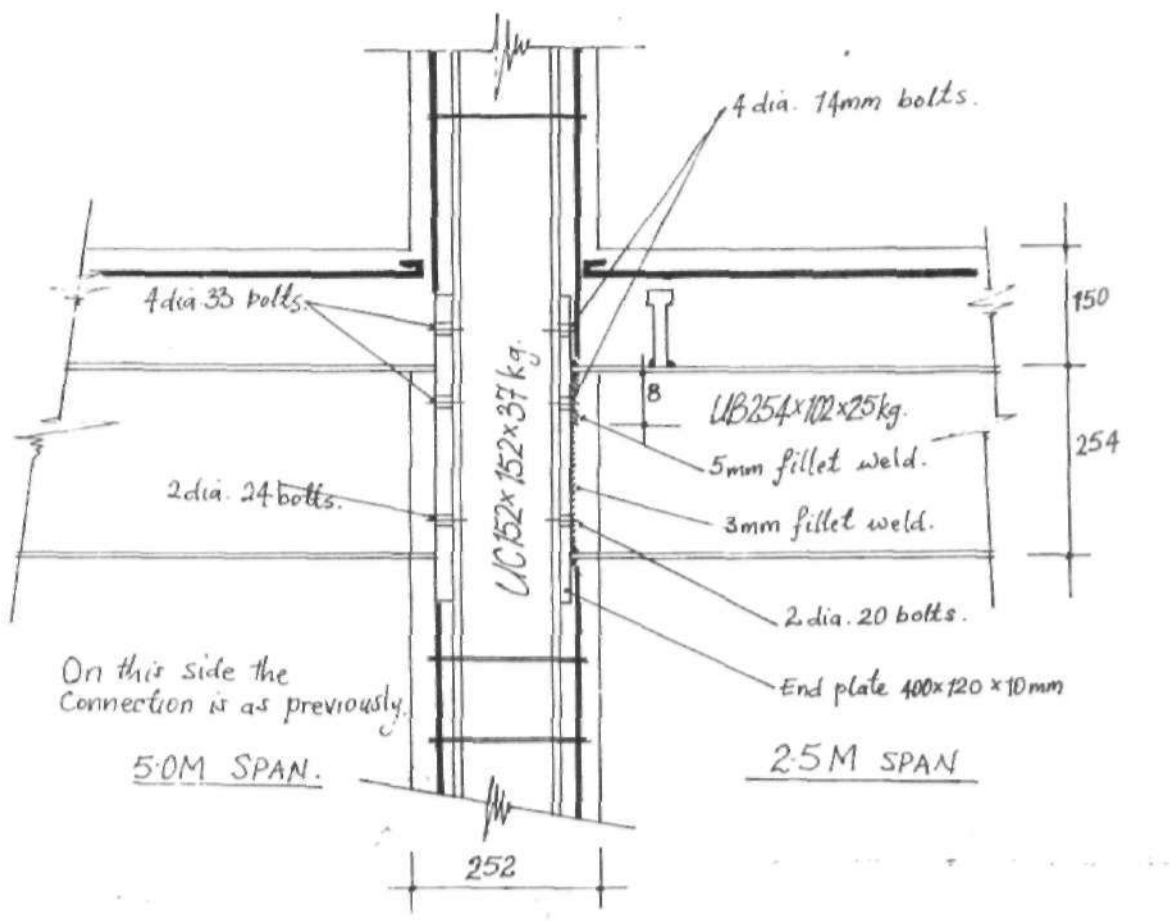


FIG. 3.7. Beam-column connection for column B and beam B_1.

3.4.3. DESIGN OF SIMPLE CONNECTIONS.

The assumption in simple connection is that columns are continuous while beams are attached to the web of steel in column. Web and seating cleats are provided to the ends of beams. Black bolts of steel of strength designation 4.6 are also used. Bolts connecting beams, from both sides of columns are in double shear. Web and seating cleat are of unequal angle 75x50x8 mm. A clear space of 5mm is allowed between column web and end of beam.

Four beam types are connected by simple connection. These are beams B_5, B_6 and B_9. The beams are in Y-axis of the building. Complete design procedure for beam B_5 connection is shown below. The results of the design for the other beams are shown in the following tables.

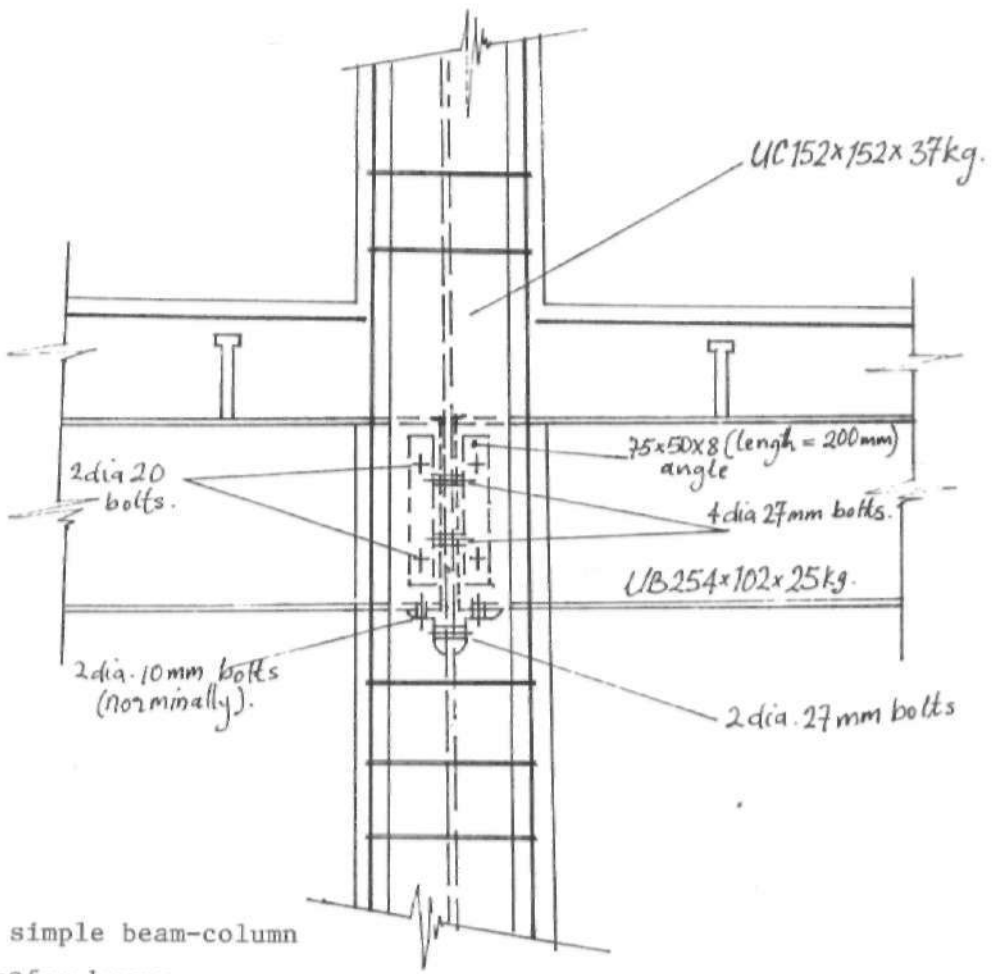
3.4.3.1. CONNECTION FOR BEAM B_5

THE BEAM HAS TEN EQUAL SPANS OF 4.0M EACH. Shear force; At External columns - 113.05 KN At Internal columns - 226.10 KN	
CONNECTION TO INTERNAL COLUMNS	
SEATING CLEAT BOLTS. Assume 2 bolts of dia 27mm double shear value = 91.60 KN enclosed bearing value for a thickness of 8mm (web) = 43.20 KN ∴ bearing value is critical	

CALCULATIONS	OUTPUT
Load carried by seating cleat bolts = 86.4 KN Reminder of load to be carried by web cleat; =139.7 KN	2 dia 27mm
WEB CLEAT BOLTS. Cleat-column connection; Assume bolts of dia 27mm bearing value is critical Number of bolts required N = 139.70/43.2 = 3.23 Provide 4 bolts	4 dia 27mm
Cleat-beam connection; Assume 2 bolts. Vertical load per bolt, = 139.7/2 = 69.85 KN modulus of bolt group; $=(2x50^2)/50$ =100 units Bolt hole distance from agle root, for leg length of 50mm =30mm	

CALCULATIONS	OUTPUT
Horizontal load per bolt =(26.65x30)/100 =20.96 KN Vector sum $V^2=69.85^2+20.96^2$ $V^2=616.91$ V =24.84KN Provide 2 dia 20mm bolts	2 dia 20mm
CONNECTIONS TO EXTERNAL COLUMNS. SHEAR FORCE = 113.05 KN	
SEATING CLEAT BOLTS. Assume 2 bolts of dia 27mm Single bearing value is critical = 34.6 KN Load carried by seating cleat blots = 69.2 KN Reminder of load to be carried by web cleat; =113.05-69.2 =43.85 KN	2 dia 27mm
WEB CLEAT BOLTS. Cleat-column connection; Assume dia 20mm bolts. single bearing value is cir- tical = 25.1KN	

CALCULATIONS	OUPUT
Number of bolts required =43.85/25.1 =1.75 Provide 2 dia 20mm bolts	2 dia 20mm
Cleat-beam connection; Assume 2 bolts, Vertical load per load =43.85/2 =21.98 KN Modulus of bolt group, $=(2x35^2)/35$ = 70 units Bolt hole distance from angle root, for leg length of 50mm =30mm Horizontal load per bolt; =(43.85x30)/70 =18.79 KN Vector sum, $V^2=18.79^2+98^2$ $V^2=836.18$ V =28.92 Provide 2 dia 22mm bolts(30.4KN)	2 dia 22mm



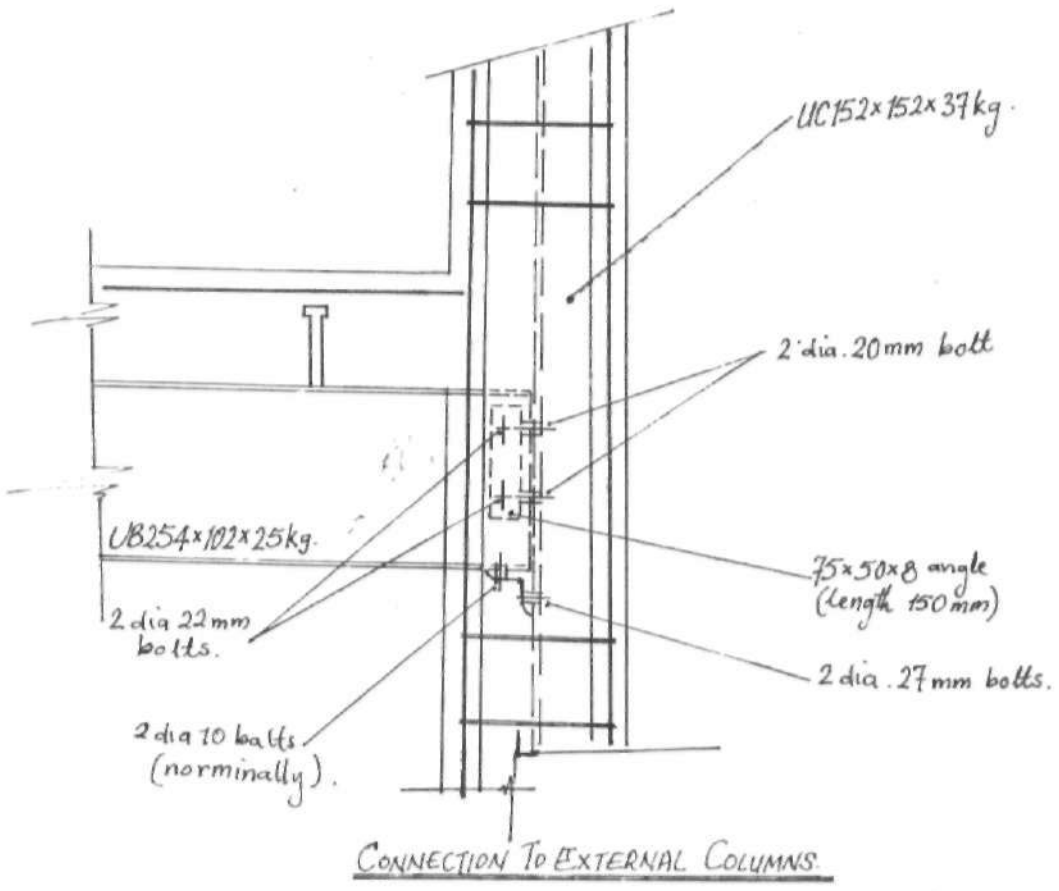


FIG. 3.8.

Typical simple beam-column connections for beams the Y-axis.

TABLE 3.6 Final results for beam-column connection design.

BEAM COLUMN CONNECTION FOR.		SHEAR FORCE (KN)	SEATING CLEAT BOLTS	WEB-CLEAT BOLTS	
BEAM	COLUMN			CLEAT-COL. CONNECTION	CLEAT BEAM CONNECTION
B_5	INTERNAL	226.10	2 dia 27mm	4 dia 27	2 dia 20
	EXTERNAL	113.05	2 dia 27	2 dia 20	2 dia 22
B_6	INTERNAL	113.60	2 dia 20	2 dia 20	2 dia 16
	EXTERNAL	56.80	2 dia 20	2 dia 10	2 dia 10
B_7	OTHERS	79.04	2 dia 27	2 dia 10	2 dia 10
	CORNER	39.52	2 dia 20	2 dia 10	2 dia 10
	CORNER	41.66	2 dia 20	2 dia 20	2 dia 10

3.5. DESIGN OF FOUNDATION.

Independant column footing is considered in the design. Where this is found inappropriate combined footing is designed. Bottom plate is welded to the column steel. This is then fixed to the concrete foundation by hold-down anchor bolts. The design of foundation is by the method of CP 110.

The design of foundation is followed by the design of column-foundation connection. This follows the method of BS.449 part 2, 1969.

3.5.1. DESIGN DATA.

Soil capacity, P_{gr}=175KN/M^2

Depth of bearing stratum = 1.2M.

Concrete grade 30

High yield, plain bars

Hold down anchor bolts with strength designation grade 4.6

Bottom plate to clauses 67 and 76A BS 449.

Weld diameter 10mm

3.5.2. DESIGN OF FOUNDATION FOR COLUMN A.

LOCAL DESIGN DATA.

Total design load, N=1078.65 KN

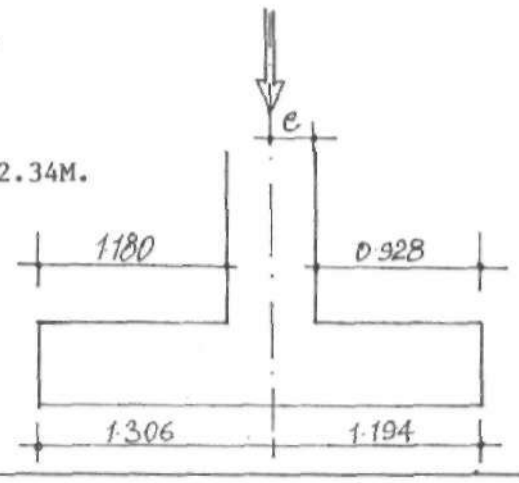
Unfactored load, N_a=1078.65/1.7
=634.50 KN

Bending moment, M =35.95 KNM

Load eccentricity, e =M/N_a=56.66mm

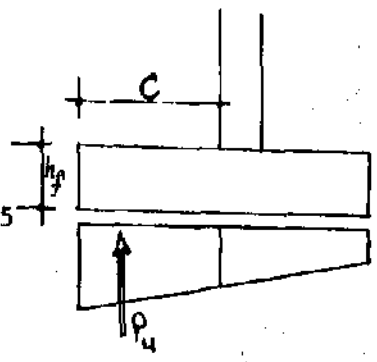
Column dimensions, h_{col}=450mm

b_{col}=252mm

CALCULATIONS	OUTPUT
BASE AREA. $A = N_a/(P_{-gr} - 20H_f) = 4.20m^2$ ----------(3.12.) for a rectangular base; $B^2 = A/1.5$ $B = 1.68m$ $L = 1.5B = 2.34M.$  	B=1.70M L=2.50M
UPWARD GROUND PRESSURE The eccentricity is within the third rule $P_u = (N/LB) \pm (6M/LB^2)$ ----------(3.13.) $P_{u1} = 325.99\ KN/M^2$ $P_{u2} = 249.29\ KN/M^2$	
THICKNESS OF FOUNDATION. a. $h_f \geq x(B - b_{col})$ ----------(3.14.) where the value of x is got from table. It depends on soil capacity. $h_f \geq 0.37(1.50 - 0.252) = 0.462M$ b. $0.3(B - b_{col}) \geq h_f \leq 0.5(B - b_{col})$ -----(3.15.) $0.374 \leq h_f \leq 0.624$ c. $h_f \geq L + 70$ to 100mm ----------(3.16.) where L is the minimum anchorage length, depending on grade of concrete and type of and diameter of reinforcement.	

CALCULATIONS	OUTPUT
$h_f \geq 0.630$ to 0.660 M For economic consideration; Let, h_f=0.60M Effective depth d =600-60-10 =530mm	h_f=0.60 d=530mm
CHECKING PUNCHING SHEAR. Critical perimeter; $p=2(h_{col}+b_{col})+291(1.5h_f)$ ----------(3.17.) =6.114 M. Shear force; $F = P_u [BL-\{2(h_{col}+b_{col})(1.5h_f) + (h_{col}.b_{col})+91(1.5h_f)^2\}]$ ----------(3.18.) =306.82[4.25-(1.264+0.113+2.545)] =100.64KN Shear stress; V=F/pd $=100.64x10^3/6114x530$ $=0.031N/mm^2$ Shear ultimate value; V_c=0.38	The section is adequate in punching shear.

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CALCULATIONS	OUTPUT
DESIGN OF REINFORCEMENT. Bending moment; C=1.180M $M=0.5C^2BP_u$ =119.21 KNM  $M/0.4f_{cu}.bd^2$ =0.035 l_a=0.95 $Z=d.l_a$=504mm Are of steel required $A_s=293.75x10^6/0.87x410x504$ =663mm² Provide Y 16 at 300 (670mm²)	Y10 at 100
LOCAL BOND Shear force; $F=BC_u$ =575.29 KN Total bars perimeter; $U_{ST}=3.14x16x18$ =402.18mm Local bond stress; $f_{b_s}=F/U_{ST}.d$ =1.69 N/mm² Ultimate local bond stress; f_{bu}=2.2 N/mm²	The section is adequate in local bond.

CALCULATIONS	OUTPUT

STEEL BOTTOM PLATE.

Let the length of the base plate be 3 times the dimension of the structural steel, i.e 456mm. And let the width of the plate be 450mm.

The eccentricity is again, within the middle third. So, there is no tension in the base plate. The pressure diagram is therefore trapezoidal.

Base pressure;

$$f_p=(N/0.456x0.5)\pm(6M/0.456x0.45^2)$$

$$=5256.58\pm2335.93$$

$$f_{p1}=7592.51\ KN/m^2$$

$$f_{p2}=2920.65KN/m^2$$

Average base pressure;

$$f_{pa}=5.26\ N/mm^2$$

Plate greater projection;

$$A=(0.456-0.152)/2$$

$$=0.152mm$$

Plate lesser projection;

$$B=(0.450-0.152)/2$$

$$=0,149\ M.$$

Permissible bending stress in plate;

$$f_b=185N/mm^2$$

CALCULATIONS	OUTPUT
Bottom plate thickness (clause 386 BS. 449) $t^2=\{3_{fpa}(A^2-B^2/4)\}/f_b$ $=2060.91mm^2$ $t=45.39mm$ say, $t=46mm$	t=46mm
HOLDING DOWN BOLTS. Since there is no tension in the base plate plate holding down bolts are only to be provided norminally. Therefore provide 4 dia 22mm bolts of grade designation 4.6 to BS 4190, 1967.	

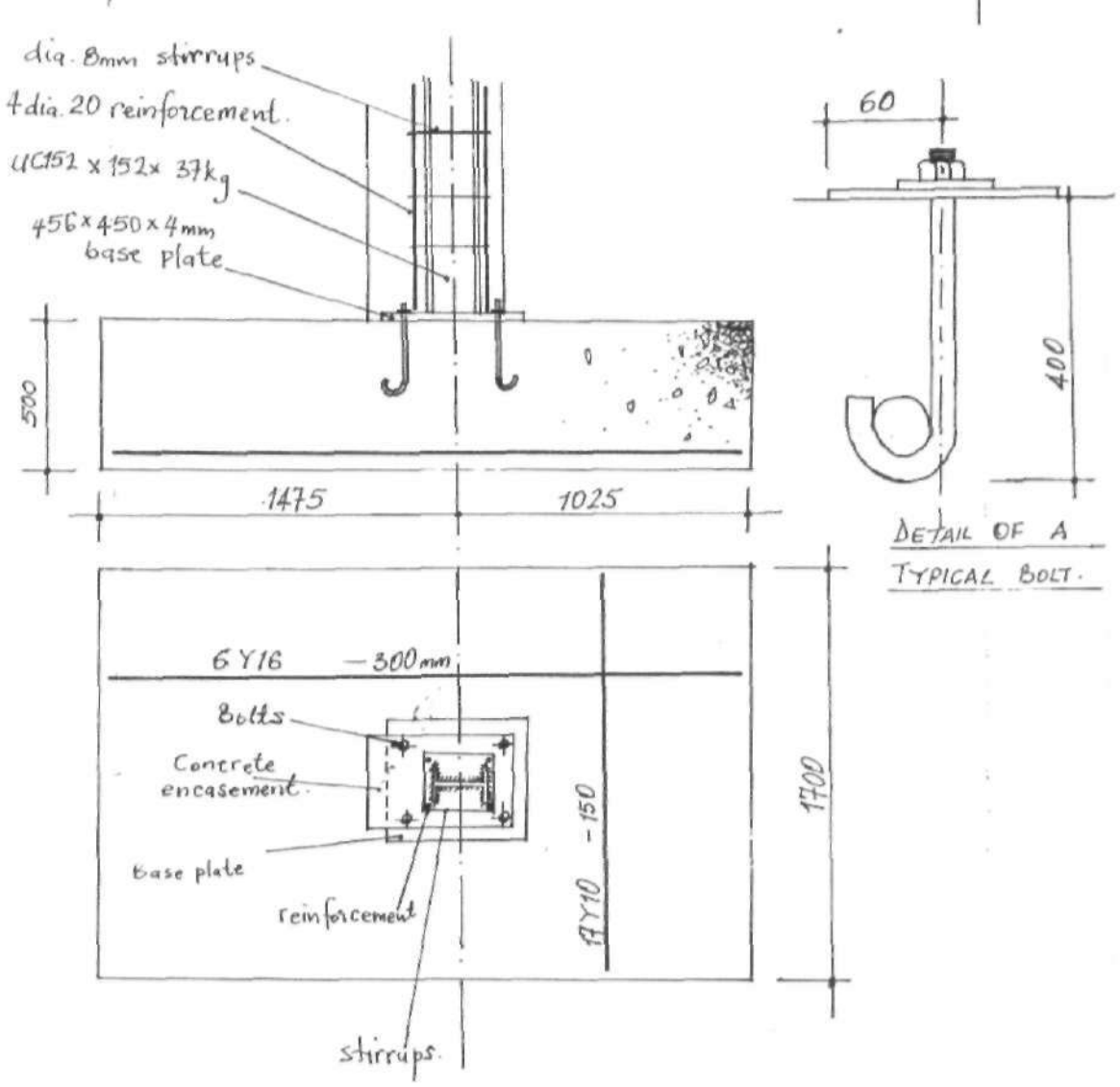


FIG. 3.9. A typical column-foundation connection for column A.

3.5.3. DESIGN OF FOUNDATION FOR COLUMN B.

LOCAL DESING DATA.

Total design load, N=1290.88KN

Unfactored load, N_a=1290.88/1.7

=759.34 KN

Bending moment; M=30.25 KNM

Load eccentricity, e=M/N_a=39.84mm

Column dimensions, h_{col}=b_{col}=252mm

This is designed as combined pad footing for the two B columnS.

Distance between column centres;

L_o=2.5M

Distance between eccentric loads,

=2.5+2x39.84

=2.58M

COLUMN BASE

Length;

L=(1.6 to 1.7) L_o ----------(3.19.)

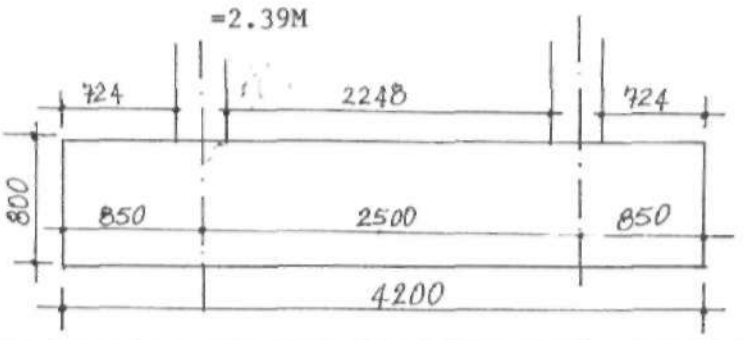
=4.0 to 4.25M

Say L=4.20M

Bredth; L=4.20M

$B=2N_a/(P_{gr}-24_4)L$ ----------(3.20) B=2.40M

=2.39M



CALCULATIONS	OUTPUT
UPWARD GROUND PRESSURE. $P_u=2N/BL$ $=2x1290.88/4.2x2.4$ $=256.13\ KN/M^2$	$P_u=256.13$
THICKNESS OF FOUNDATION. a. $h_f \geq x(B-b_{col})$. $\geq 0.37(2.40-0.252)$ $\geq 0.795\ M$ b. $0.3(B-b_{col}) \leq h_f \leq 0.5(B-b_{col})$ $0.644 \leq h_f \leq 1.074$ Say, $h_f=0.8\ M$ $d=0.73M$	$h_f=0.8M$ $d=0.73$
CHECKING PUNCHING SHEAR. Critical perimeter; $P=2(h_{col}+b)+2.91(1.5h_f)$ $=8.549M.$ Shear force; $F=256.66[3.545-(0.756+0.064+1.767)]$ $=245.88\ KN$ Shear stress; $V=F/pd$ $=245.88x10^3/8549x430$ $=0.11\ N/mm^2$ Ultimate shear stress; $V_c=0.38\ N/mm$	The Sectio is adequate in punchin shear.

CALCULATIONS	OUTPUT
MOMENTS. Overhang moment; $M_1=BC^2P_u/2$ $=2.40\times0.724^2\times256.66/2$ $=161.44$ KNM Span moment; Maximum will occur between the two columns; $M_2=-1290.88\times1.25+2.4\times2.1\times256.66/2$ $=-1613.6+1358.24$ $=-255.36$ KNM Transverse moment; $M_3=256.66\times1\times(1.20-0.0.126)^2/2$ $=148.03$ KNM	
REINFORCEMENT. Longitudinal; $M_1/0.4bd^2f_{cu}=0.078$ $l_a=0.95$ $Z=l_a.d=693.5$mm $A_S=M_1/0.87f_y.z$ $=652.62mm^2/M$ min $A_S=0.15bd/100$ $=1089mm^2$ Provide Y20 at 300($1050mm^2$) Transverse; $M_3/0.4bd^2f_{cu}=0.032$ $l_a=0.95$ $z=l_a.d=693.5$	Y20at300

CALCULATIONS	OUTPUT
$A_S=M_3/0.87f_y.z$ $=598.41mm^2/M$ min. $A_S=0.15bh/100=600mm^2$ Provide Y16 at 300 ($607mm^2$)	Y16 at 300
Top main bars; $M_2/f_{cu}bd^2=0.084$ $l_a=0.95$ $z=693.5mm$ $A_S=M_2/0.87f_y.z=1032mm^2$ min. $A_S=0.15bd/100=1095mm^2$ Provide Y20 at 300 ($1050mm^2$)	
LOCAL BOND. Shear force; $F = BCP_u$ $=2.40x0.724x256.66$ $=445.97$ KN Total bars perimeter; $U_{ST}=3.142x20x10$ $=628.40mm$ Local bond stress; $f_{bs}=F/d.U_{ST}$ $=0.97$ N/mm^2 Ultimate local bond stress; $f_{bu}=2.2$ N/mm^2	$U_{ST}=628.40$ The Section is adequate in local bond.

CALCULATIONS	OUPUT
STEEL BOTTOM PLATE. Let the dimensions of the plate for each of the two columns be 456x450. Eccentricity is within the third rule. Base plate pressure; $f_p=(N/0.456x0.45)\pm(6M/0.456x0.45^2)$ $=6290.84\pm1965.56$ $f_{pa}=4325.28$ Average base pressure; $f_{pa}=6.29N/mm^2$ Plate greater projection; $A=(0.456-0.152)/2$ $=0.152mm$ Plate lesser projecter; $B=(0.450-0.152)/2$ $=0.149mm$ Permissible bending stress in plate; $f_b=185\ N/mm^2$ Plate thickness; $t^2=[3f_{pa}(A^2-B/^2/4)]/f_b$ $=1790.48mm^2$ $t=42.31$ say t=46mm	t=46mm

CALCULATIONS	OUTPUT
HOLDING DOWN BOLT. Bolts with strength designation 4.6 to BS 4190, 1967 and BS 449; 1969. Since there is no tension in the base plate bolts are to be provided norminally. Therefore, provide 4 dia 22mm to each of the two base plates.	4 dia 22

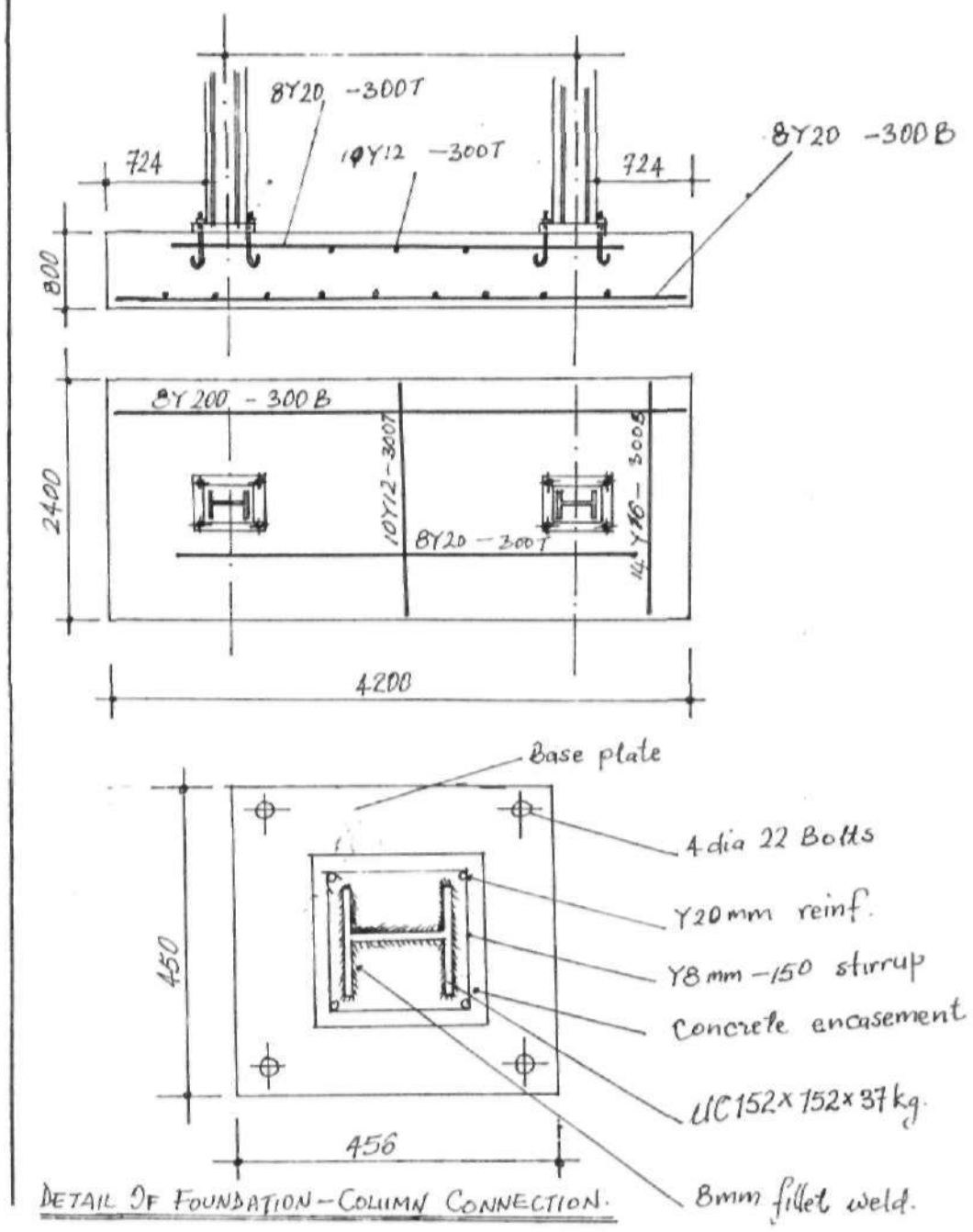


FIG. 3.10. Column-foundation connection for columns B.

TABLE 3.7 Final results for column-foundation connection design.

FOUNDATION TO COLUMN	LOAD	MOMENT	ECCEN.	COL. SECTION	FOUNDATION DIMENSION			REINFOR-CEMENT	BOT. PLATE (mm)	H.D. BOLT
	N(KN)	M(KNM)	e(mm)		L(M)	B(M)	h_f(M)			
A.	1078.65	35.95	56.66	300 x 252	2.50	1.70	0.60	Y16at300	456x450x46	4ddia 22
Combined B.	1290.88	30.25	39.84	252x252	4.200	2.40	0.80	Y20at300	456x450x46	4 dia 22
C.	671.05	35.95	91.00	300x252	2.00	1.30	0.42	Y12at200	456x450x35	4 dia 22
D.	1101.16	29.77	46.00	252x252	2.50	11.70	0.60	Y12at250	456x450x40	4 dia 22
E.	465.62	8.81	32.00	252x252	1.80	1.00	0.37	Y10at175	456x450x35	4 dia 20 in Tension Zone
F.	1336.10	58.93	75.00	252x252	2.80	1.90	0.60	Y12at250	456x450x46	4 dia 22
Combined G.	1290.00	73.53	97.00	252x252	4.20	2.40	0.80	Y20at300	456x450x46	4 dia 22
H.	1012	35.95	57.86	300x252	2.50	1.70	0.60	Y10at300	456x450x46	4 dia 22
J.	465.62	8.81	32.00	252x252	1.80	1.00	0.37	Y10at175	456x450x35	4 dia 20 in Tension Zone.

CHAPTER FOUR

4.0. REINFORCED CONCRETE CONSTRUCTION

The reinforced concrete frames are analysed elastically by moment distribution. All internal beams are continuous T-beams, while outer beams are continuous L-beams. The beams are considered as rectangular beams since it is generally considered that T and L beams have rectangular cross-section provided that the depth of compression zone lies within the slab. The slab is designed as a two-way spanning slab. Column footing are designed as independant footings. Combined footing are also considered where independant footing is found unsuitable.

4.1. DESIGN OF SLAB.

This is a two-way slab. Its design is shown in Section 3.1. The design carried out in the mentioned section also serves this section.

4.2. PRELIMINARY ANALYSIS.

4.2.1. DETERMINATION OF BEAM SIZES.

For the T-Beams Spanning in the X-plane of the building;

Approximate beam depth;

$h = L/15$ _____CP 114.

$= 5000/15$

$=333.33mm$; say, $h=460$

Beam breadth must not be greater than half of beam depth. So,

$b = 230mm$

Let, concrete cover be, $d_c=30mm$

half bar diameter, $t = 5mm$

So, effective depth, d = 460 -35

=425mm

Beam flange is the lesser of;

i. beam spacing = 4000mm

ii. beam rib plus E.A (section 2.2.3.)

for a T-beam = 230+0.2x0.7x5000

= 930mm

= 930mm

Therefore, b_f=930mm

Depth of beam flange

h_f=150mm

Approximate beam rib weight;

=(0.46-0.15)x0.23x24

=1.71KN per M.

Approximate weight of partition wall;

=10.4KN per M

Slab weight;

dead load = 4.2KN Per m^2

Live load = 3.42 KN per M^2

Approximate beam load

dead load = $W.l_x.l_y-W.l_x^2/2$

= 4.2x4x5- -4.2x4^2/2

=50.4KN

=10.08 KN per M.

live load =41.04 KN

=8.21 KN per M.

Approximate total dead load;

= 22.19 KN per M

Approximate total live load

= 8.21 KN per M.

Approximate beam ultimate load;

F =1.4x22.19+1.6x8.21

= 44.10 KN per M.

Approximate beam ultimate moment;

M_u=0.1xFxL^2

=110.25 KNM

Approximate beam shear force;

V =0.5FL

= 110.25 KN

Actual beam depth are calculated in relation to bending, shear and serviceability requirements. Thus;

In relation to bending;

$h \geq (5.2 \text{ to } 7)(M_u/f_{cu}.b_f)^{\frac{1}{2}}$

$\geq$ 327 to 440mm

In relation to shear;

$h \geq (2.4 \text{ to } 2.8)(V/f_{cu\ shear})^{\frac{1}{2}}$

$\geq$393 to 459 mm

In relation to serviceability requirements;

d$\geq$ L/26 x 0.77-- for 3% reinforcement

$\geq$ 250mm

$\therefore$ h$\geq$ 285 mm

The assumed beam depth is within the specified ranges. Therefore, all the approximate dimensions of the beam are the actual values used in the design, that is,

beam depth, h = 460mm

beam breadth, b = 230mm

4.2.2. DETERMINATION OF COLUMN SIZE.

Approximate column effective lengthes are as follows:

i. ground floor columns;

l_{eff}=1.5x4.0 =6.0 M

ii. first and second floor columns;

l_{eff}=1.5x3.285 = 4.928 M

iii. third floor columns;

l_{eff}=0.85x3.285=2.792M

To satisfy stability criteria the column dimension at ground floor are as follows:

i. $l_{eff}/20 \leq h_{col} \leq l_{eff}/10$

$300 \leq h_{col} \leq 600mm$

ii. $b_{col} \leq l_{eff}/25$

$b_{col} < 240mm$

or $b_{col}=b_{beam}=230mm$

Therefore assum;

Internal columns =300x250mm

External columns =400x250mm

4.2.2.1. COLUMN LOADS.

Weight of roof = 1.45 KN per M^2

Weight of ceiling = 8.95 KN per M^2

Weight of gutter = 8.244 KN per M

Roof live load = 0.75 KN per M^2

Two-way slab:

dead load = 4.2 KN per M^2

Live load = 3.42 KN per M^2

Partition wall = 10.4 KN per M

Parapet wall beams = 0.54 KN per M

Roof level internal beams = 2.54 KN per M

From a combination of the appropriate loads above and self weight of columns the loads are found as follows, from frame 1.

i. COLUMN LENGTH B_1-B_2

Total G_k=48.75 KN

Total Q_k=11.25 KN

Column total load, F=1.4x48.75+1.6x11.25

=86.25 KN

ii. COLUMN LENGTH B_2-B_3

Slab loads on beams:

In the X-plane; dead load = 10.08 KN/M

live load = 8.21 KN/M

In the Y-plane; dead load =7.81 KN/M

live load =6.36 KN/M

Weights on column;

X-plane beams;

dead load from slab

=10.08(2.5+1.25)=37.08 KN

live load from slab;

=8.21x3.75=30.79 KN

Beam weight

=(0.46-0.15)0.23x24=4.8 KN

weight of partition wall

=10.4(5+2.5)/2 = 39 KN

Y-plane beams;

dead load from slab:

=7.1x4=31.24 KN

live load from slab;

=6.36x4 = 25.44 KN

beam weight

=24(0.46-0.15)0.23x3.85=6.58KN

weight of partition wall;

=10.4x4 = 41.6 KN

Self weight of column = 5.91 KN

Total G_k=152.2 KN

Total Q_k=56.23 KN

Column total load,

$F = 1.4\ G_k+1.6Q_k$=303.05 KN

Loading from column length B_1-B_2

F_c=82.72 KN

Column design load;

$F + F_c$=385.77 KN

iii. COLUMN LENGTH B_3-B_4

Characteristic load is as for column B_2-B_3

F =303.05 KN

Loading of column lenght B_2-B_3

F_c=385.77KN

Column design load

$F+F_c$=688.82 KN

iv. COLUMN LENGTH B_4-B_5

Slab and beam loads are as for B_2-B_3

Column self weight,

=0.3x0.25x4x24=7.2 KN

Total G_k=159. 4 KN

Total Q_k= 56.23 KN

Column total load;

$F = 1.4\ G_k + 1.6\ Q_k$=313.13 KN

Loading from column length B_3-B_4

F_c= 688.82 KN

Column design load,

$F+F_c$=1001.95 KN

TABLE 4.1. Columns' design loads.

COLUMN	LENGTH	LOAD (KN)
A	A_1 - A_2	108.83
	A_2 - A_3	375.67
	A_3 - A_4	642.51
	A_4 - A_5	922.79
B	B_1 - B_2	86.25
	B_2 - B_3	385.77
	B_3 - B_4	688.82
	B_4 - B_5	1001.95
C	C_1 - C_2	72.45
	C_2 - C_3	205.87
	C_3 - C_4	339.29
	C_4 - C_5	479.43
D	D_1 - D_2	68.40
	D_2 - D_3	219.93
	D_3 - D_4	371.45
	D_4 - D_5	528.02
E	E_1 - E_2	47.72
	E_2 - E_3	123.48
	E_3 - E_4	199.25
	E_4 - E_5	277.53
F	F_1 - F_2	106.09
	F_2 - F_3	366.38
	F_3 - F_4	626.67
	F_4 - F_5	900.40 IN FRAME 7 1088.66
G	G_1 - G_2	84.12
	G_2 - G_3	409.76
	G_3 - G_4	735.40
	G_4 - G_5	1062.53

TABLE 4.1. (cont'd).

COLUMN	LENGTH	LOAD (KN)
H	$H_1 - H_2$	106.09
	$H_2 - H_3$	366.38
	$H_3 - H_4$	626.67
	$H_4 - H_5$	900.40
J	$J_1 - J_2$	48.79
	$J_2 - J_3$	130.20
	$J_3 - J_4$	211.61
	$J_4 - J_5$	293.39
K	$K_1 - K_2$	69.82
	$K_2 - K_3$	232.64
	$K_3 - K_4$	395.46
	$K_4 - K_5$	559.03
L	$L_1 - L_2$	69.99
	$L_2 - L_3$	200.13
	$L_3 - L_4$	213.34
	$L_4 - L_5$	467.14
M	$M_4 - M_5$	361.88

The above column loads are computed following the same procedure as for Column B. Complete calculation are shown for computing loads for column B. above.

4.2.3. CHECKING THE ADEQUACY OF COLUMN SECTIONS

Ground floor column lengths are checked since these carry the largest amount of axial load.

Approximate column moment

Beam length = 5.0 M

Approx. beam ultimate load = 43.56 KN/M

Simply supported beam moment

$= W_1^2/8 = 136.125$ KNM

Approx. column moment

= 40 per cent of $W1^2/8$

= 54.45 KNM

Column B.

Design load = 1001.95 KN

Column size = 300x250mm

$\therefore N/bh = 13.36$

$M/bh^2 = 2.42$

Column concrete cover, $d_c = 40$mm

Reinforcement, assume dia 12 mm

Effective depth, $d = h-(d_c+dia/2) = 254$mm

So, $d/h = 254/300 = 0.85$

Refering to design chart for

$f_{cu}=30$, $f_y=410$ and $d/h=0.85$

$\therefore 100A_{sc}/bh = 2.2$

This is within the required limit of 1 to 6 per cent. Therefore the column section is adequate. The same procedure (not shown) have been used and found that all the columns cross-sectional dimensions are adequate.

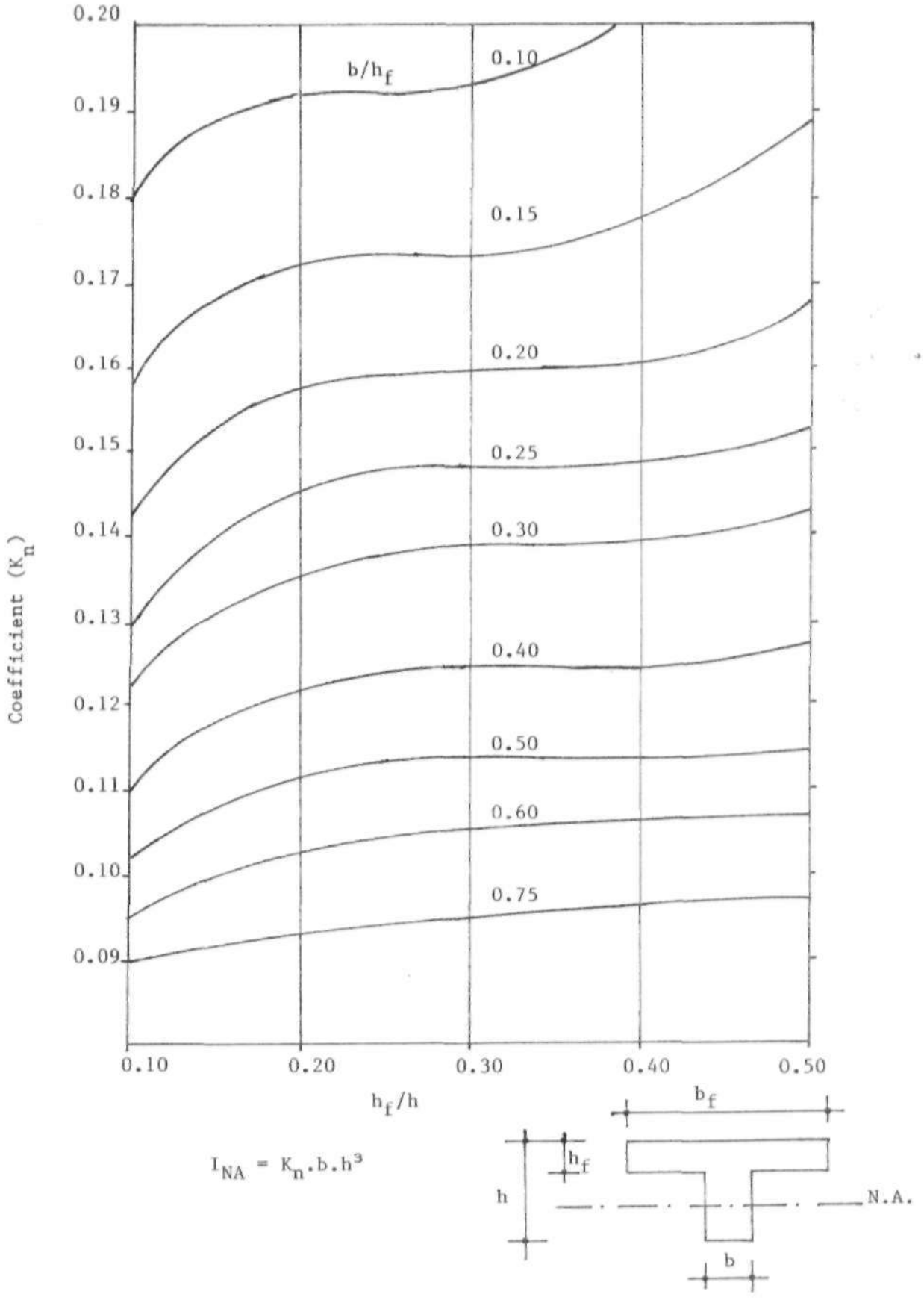


FIG. 3.11. Moments of inertia of a flanged beam.

4.2.4. BEAM AND COLUMN STIFFNESSES.

The stiffness, k, of a structural member plays a very important role in frame elastic analysis. Member stiffness is defined as EI/L. The modulus of elasticity, E, has a constant value for the concrete grade and is therefor given a unity value. The second moment of area, I, depends on member cross-section. L, is the length of the member between centres of joints.

4.2.4.1. BEAM STIFFNESS.

For a flanged beam the second moment of area about the neutral axis, I, is equal to $k_n.b_h^3$, where k_n is a constant whose value is found from moment of inertia chart. In the chart k_n is plotted against h_f/h for each value of b/b_f between 0.1 to 0.75 (Reference 12, P. 44; also see appendix),

i. BEAMS A - B (FRAME 1).

This is a T-beam whose b_f=930mm

b/b_f=230/930 = 0.25

h_f/h=150/460 = 0.33

From chart, k_n=0.153

Moment of inertia, $I_{NA}=K_n.b.h^3$ --------(4.1.)

$=0.153x230x460^3$

$=34.25x10^3$ mm^4

Stiffness, K_{AB}=EI/L ----------(4.2.)

$=1x34.25x10^3/5000$

$=6.85x10^5$mm^3

The following table shows the beams stiffnesses.

TABLE 4.2. Beam stiffnesses (k).

BEAM	b_f (mm)	K_n	$I_{NA}=K_n.b.h^3$	EI/L
A - B	930	0.153	34.25×10^8	6.85×10^5
C - D	580	0.125	27.98×10^8	5.60×10^5
B - B	580	0.125	27.98×10^8	11.19×10^5
D - E	375	0.104	25.52×10^8	10.2×10^5
F - G	802	0.136	30.45×10^8	7.08×10^5
G- H	858	0.148	33.13×10^8	7.05×10^5
G - G	650mm	0.138	$30\times89\times10^8$	10.30×10^5
J - K	440	0.114	$25\times52\times10^8$	8.15×10^5
K - L	559	0.125	27.98×10^8	5.95×10^5
M - F	1070	0.158	35.37×10^8	5.89×10^5
M - F L -BEAM	650	0.138	30.89×10^8	5.15×10^5
IN THE X- AXIS				
T-beam	790	0.148	33.13×10^8	8.28×10^5
L-beam	510	0.125	33.13×10^8	7.0×10^5

The above table shows the computed beam stiffnesses, K. Method of calculations is the same as for beam A - B in frame 1. Complete calculations for this beam's stiffness is shown above.

4.2.4.2. COLUMN STIFFNESS.

The stiffnesses of each column are determined for each axis, since the columns are biaxially bent.

A. i. INTERNAL COLUMNS' MOMENT OF INERTIA.

X - PLANE:

$I = bh^3/12 = 5.63\times10^8$ mm^4

Y - PLANE:

$I = hb^3/12 = 3.91 \times 10^8 \text{ mm}^4$

ii. EXTERNAL COLUMN'S MOMENT OF INERTIA

X - PLANE:

$I = bh^3/12 = 13.33 \times 10^8 \text{ mm}^4$

Y - PLANE:

$I = bh^3/12 = 15.21 \times 10^8 \text{ mm}^4$

B. i. GROUND FLOOR INTERNAL COLUMNS' STIFFNESS

X - PLANE:

$EI/L = 1.41 \times 10^5$

Y - PLANE:

$EI/L = 0.98 \times 10^5$

ii. GROUND FLOOR EXTERNAL COLUMNS' STIFFNESS.

X - PLANE:

$EI/L = 3.33 \times 10^5$

Y - PLANE:

$EI/L = 1.30 \times 10^5$

iii. UPPER FLOOR INTERNAL COLUMN'S STIFFNESS.

X- PLANE:

$EI/L = 1.71 \times 10^5$

Y- PLANE:

$EI/L = 1.19 \times 10^5$

iv. UPPER FLOORS EXTERNAL COLUMNS' STIFFNESS

X - PLANE:

$EI/L = 4.06 \times 10^5$

Y - PLANE:

$EI/L = 1.59 \times 10^5$

4.2.4.3. ACTUAL COLUMNS' EFFECTIVE HEIGHT

Column effective height is considered as height between points of contraflexure. This is controlled by the relative stiffnesses of the column and its adjoining beams. The lesser of equations 22 and 23 in clause 3.5.1.4 of CP 110 gives effective height of unbraced columns.

$L_{eff}=L[1.0+0.15(C_1+C_2)]$ ------- eq. 22 --------(4.3.)

$L_{eff}=L[2.0+0.3C_{min}]$ ------- eq. 23 --------(4.4.)

where L. is the actual column height

C_1. is the ratio of the sum of column stiffnesses to the sum of beams stiffnesses at one end of the column in the same plane of bending.

C_2. is the ratio of the sum of column stiffnesses at the other end of the column in the same plane of bending

C_{min}. is the minimum of C_1 and C_2.

1. COLUMN A_1 - A_2

X - axis: $C_1 = 0$

$C_2 = 4.06x10^5x2/4.36x10^5 = 1.86$

Y - axis: $C_1 = 2x1.59x10^5/2x4.31x10^5 = 0.37$

$C_2 = 0.37$

X - axis:

$L_{eff}=3.285[1.0+0.15(0+1.86)] = 4.2$

$L_{eff}=3.285[2.0+0.3x0] = 6.57$

$\therefore L_{eff} = 4.2.M.$

Y - axis:

$L_{eff}=3.285[1.0+0.15(0.37+00.37)]=3.65$

L_{eff}=3.285[2.0+0.3x0.37] = 6.90

$\therefore L_{eff}$ = 3.65 M

TABLE 4.3. Computed columns' effective lengths.

COLUMN	LENGTH	STIFFNESS RATIO				EFFECTIVE LENGTH	
		X-AXIS		Y-AXIS		X-AXIS	Y-AXIS
		C_1	C_2	C_1	C_2	(M)	(M)
A	A_1 - A_2	0	1.86	0.37	0.37	4.20	3.65
	A_2 - A_3	1.86	1.86	0.37	0.37	5.12	3.65
	A_3 - A_4	1.86	1.69	0.37	0.34	5.03	3.65
	A_4 - A_5	1.69	1.00	0.34	1.0	5.61	4.80
B	B_1 - B_2	0	0.30	0.23	0.23	3.43	3.51
	B_2 - B_3	0.30	0.30	0.23	0.23	3.58	3.51
	B_3 - B_4	0.30	0.78	0.23	0.27	3.82	3.50
	B_4 - B_5	0.78	1.0	0.21	1.0	5.07	4.73
C	C_1 - C_2	0	2.35	0.37	0.74	4.44	3.83
	C_2 - C_3	2.35	2.35	0.74	0.74	5.60	4.01
	C_3 - C_4	2.35	2.14	0.74	0.68	5.50	4.00
	C_4 - C_5	2.14	1.0	0.68	1.0	4.83	5.01
D	D_1 - D_2	0.18	0.35	0.23	0.46	3.55	3.62
	D_2 - D_3	0.35	0.35	0.46	0.46	3.63	3.74
	D_3 - D_4	0.35	0.34	0.46	0.22	3.62	3.62
	D_4 - D_5	0.34	1.0	0.22	1.0	4.80	4.73
E	E_1 - E_2	0.27	0.54	0.22	0.55	3.68	3.69
	E_2 - E_3	0.54	0.54	0.55	0.55	3.82	3.83
	E_3 - E_4	0.54	0.54	0.55	0.55	3.80	3.80
	E_4 - E_5	0.50	1.0	0.50	1.0	4.90	4.90
F	F_1 - F_2	0	1.71	0.37	0.37	4.13	3.45
	F_2 - F_3	1.71	1.71	0.37	0.37	4.97	3.65
	F_3 - F_4	0.71	1.56	0.37	0.34	4.40	3.63
	F_4 - F_5	0.88	1.0	0.34	1.0	5.13	4.80
G	G_1 - G_2	0	0.38	0.12	0.23	3.47	3.46
	G_2 - G_3	0.38	0.38	0.23	0.23	3.66	3.51
	G_3 - G_4	0.38	0.34	0.23	0.24	3.64	3.52
	G_4 - G_5	0.34	1.0	0.24	1.0	4.80	4.74
H	H_1 - H_2	0	1.87	0.16	0.32	4.21	3.52
	H_2 - H_3	1.87	1.87	0.32	0.32	5.13	3.60
	H_3 - H_4	1.87	1.70	0.32	0.28	5.04	2.58
	H_4 - H_5	1.70	1.0	0.28	1.0	5.62	4.77

TABLE 4.3. (CONT'D)

COLUMN	LENGTH	STIFFNESS RATIO				EFFECTIVE LENGTH	
		Y-AXIS		Y-AXIS		X-AXIS	Y-AXIS
		C_1	C_2	C_1	C_2	(M)	(M)
J	$J_1 - J_2$	0.33	0.66	0.28	0.56	3.77	3.70
	$J_2 - J_3$	0.66	0.66	0.56	0.56	3.94	3.84
	$J_3 - J_4$	0.66	0.60	0.56	0.50	3.91	3.81
	$J_4 - J_5$	0.6	1.0	0.5	1.0	4.96	4.90
K	$K_1 - K_2$	0.19	0.38	0.23	0.37	3.57	3.52
	$K_2 - K_3$	0.38	0.38	0.47	0.47	3.66	3.64
	$K_3 - K_4$	0.38	0.35	0.47	0.24	3.64	3.63
	$K_4 - K_5$	0.38	0.35	0.47	0.24	3.64	3.63
L	$L_1 - L_2$	1.11	2.22	0.37	0.74	4.93	3.83
	$L_2 - L_3$	2.22	2.22	0.74	0.74	5.47	4.01
	$L_3 - L_4$	2.22	2.22	0.74	0.67	5.37	3.98
	$L_4 - L_5$	2.01	1.0	0.67	1.0	5.81	5.00
M	$M_4 - M_5$	0.39	1.0	0.11	1.0	4.83	4.67

Complete calculation of effective length for column-length $A_1 - A_2$ is shown above. The results of calculations for the rest of the columns are shown in the above table

4.3. FRAME ELASTIC ANALYSIS.

As a whole elastic analysis is to determine forces and moments present in a structural member. Hardly cross' method of of moment distribution is used in the analysis in this thesis. Beams loading recommended by section 3.4.3.3 of CP 110 is used.

Moment distribution for each floor level is carried out, but complete calculation is shown only for beam B_2 of frame 1. For the rest of the beams only the results of moment distribution is shown in the following tables.

Complete calculations for shear force for beam B_1 is shown, while for the other beams only the final results are

4.3.1. BENDING MOMENTS (X - AXES)

4.3.1.1. INTERNAL BEAMS (FRAME 1)

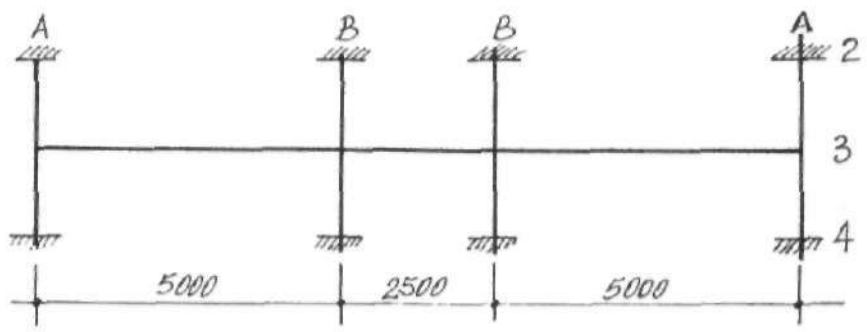


TABLE 4.4. DISTRIBUTION FACTOR:

JOINT	MEMBER	STIFFNESS(K)	∑K	DISTR. FACTOR
A	A2	4.06×10^5	14.97×10^5	0.271
	AB	6.85×10^5		0.458
	A4	4.06×10^5		0.271
B	BA	6.85×10^5	21.46×10^5	0.319
	B2	1.71×10^5		0.080
	BB	11.19×10^5		0.521
	B4	1.71×10^5		0.080

LOADING:

According to clause 3,4,3,3, CP 110 total load per unit length of beam, from a two-way slab is $(n1_x.1_y/2)-(n.1_x^2/4)$ for a longer beam and $n.1_x^2/4$ for a shorter beam, where n, is the loading per square length of slab.

dead load from slab = 4.2 KN/M

live load form slab = 3.43KN/M

pertition wall weight = 10.4 KN/M

beam self weight = 1.1968KN/M

Beam A - B;

dead load = $2[14.2x4x5/2)-(4.2x4^2/4)]+1.968+10.4$

= 62.77 KN per M

live load = $2[3.43x4x5/2)-(3.43x4^2/4)]$

= 41.16 KN per M.

Beam B - B;

dead load = $2(4.2x2.5^2/4)+1.968+10.4$

= 25.49 KN per M

live load = $2(3.43x2.5^2/4)$

= 10.72 KN per M

Ultimate loads:

Beam A - B

Maximum load = $1.4G_k+1.6Q_k$

= 153.73 KN per M

Minimum load = $1.4\ G_k$

= 87.88 KN per M

Beam B - B

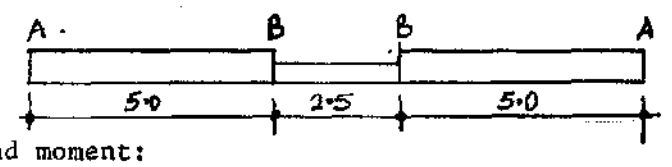
Maximum load = $1.4G_k+1.6Q_k$

= 52.84 KN per M

Minimum load = $1.4G_k$

= 35.69 KN per M

Load Case 1.



Fixed end moment:

A-B = B-A = $153.73 \times 5^2/12$ = 320.27 KNM

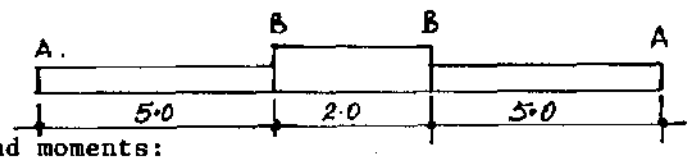
B-B = $35.69 \times 25^2/12$ = 18.59 KNM

Span moments:

A-B = $153.73 \times 5^2/8$ = 480.41 KNM

B-B = $35.69 \times 2.5^2/8$=27.88 KNM

Load Case 2.



Fixed end moments:

A-B= B-A = $87.88 \times 5^2/12$ = 183.08 KNM

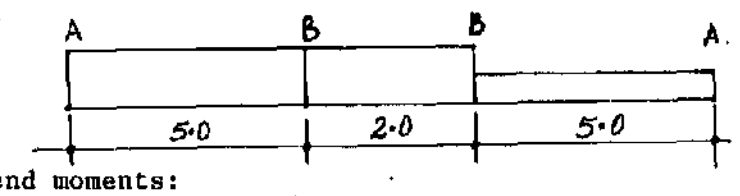
B-B = $52.84 2.5^2/12$=27.52 KNM

Span moments:

A-B = 274.63 KNM

B-B = 41.28 KNM

Load Case 3.



Fixed end moments:

A-B = B-A = 320.27 KNM

B-B_1=27.52 KNM

B_1-A=A-B_1=183.08 KNM

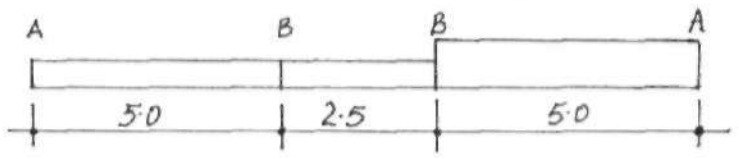
Span moments:

A-B = 480.41 KNM

B-B_1=41.28 KNM

B_1-A=274.63 KNM

Load Case 4.



Fixed end moments:

A-B = B-A = 183.08 KNM

$B-B_1=B_1-B$ = 18.59 KNM

$B_1-A=A-B_1$ = 320.27 KNM

Span moments:

A-B = 274.63 KNM

B-B = 27.88 KNM

B-A = 480.41 KNM

Complete bending moment calculations for beam B_1 is shown in the following tables, while the result of calculation for the rest of the beams is tabulated.

4.3.2. SHEAR FORCES (X-AXIS).

4.3.2.1. INTERNAL BEAMS (FRAME 1)

Load Case 1.

SPANS A-B

Ultimate load = 153.73 KN/M

Moments M_{AB}=254.03 KNM

M_{BA}= 247.13 KNM

So, F_{BA}=153.73x5/2+(250.03-247.13)/5

=385.71KN

F_{BA}=153.73x5-385.71

=382.94 KN

TABLE 4.5
Moment Distribution for Load Case 1.



JOINT	A			B				B				A		
Member	A-2	A-D	A-4	B-A	B-2	B-B	B-4	B-4	B-B	B-2	B-A	A-4	A-B	A-2
Dist.Factor	0.271	0.458	0.271	0.319	0.080	0.521	0.080	0.080	0.521	0.080	0.319	0.271	0.458	0.271
F.E.M.		-320.27		320.27		-18.59			18.59		-320.27		320.27	
Balancing	86.79	146.68	86.79	-96.24	-24.13	157.18	-24.13	24.13	157.18	24.13	96.24	-86.79	-146.68	-86.79
Distribution		-48.12		73.34		78.59			-78.59		-73.34		48.12	
Balancing	13.04	22.04	13.04	-48.47	-12.15	-73.16	-12.15	12.15	73.16	12.15	48.47	-13.04	-22.04	-13.04
Distribution		-24.24		11.02		36.58			-36.58		-11.02		24.48	
Balancing	6.57	11.10	6.57	-5.23	-3.81	-24.80	-3.81	3.81	24.80	3.81	15.18	-6.57	-11.10	-6.57
Distribution		-7.59		5.55		12.40			-12.40		-5.55		7.59	
Balancing	2.06	3.47	2.06	-5.23	-1.44	-9.84	-1.44	1.44	9.84	1.44	5.23	-2.06	3.47	-2.06
Distribution		-2.62		1.74		4.92			-4.92		-1.74		2.62	
Balancing	0.71	1.20	0.71	-2.12	-0.53	-3.47	0.53	0.53	3.47	0.53	2.12	-0.71	-1.20	-0.71
Distribution		-1.06		0.60		1.73			-1.73		-0.60		1.06	
Balancing	0.28	0.49	0.28	-0.74	-0.19	-1.21	-0.19	0.19	1.21	0.19	0.74	-0.28	-0.49	-0.28
T O T A L	109.45	-218.92	109.45	244.54	-42.25	-154.03	-42.25	42.25	154.03	42.25	-244.54	-109.45	218.92	-109.45

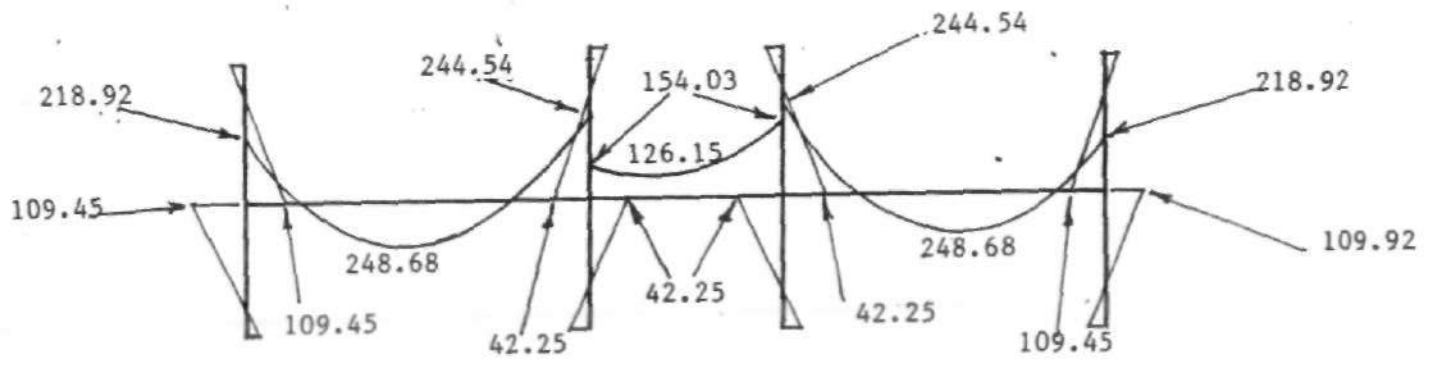
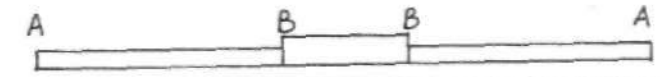
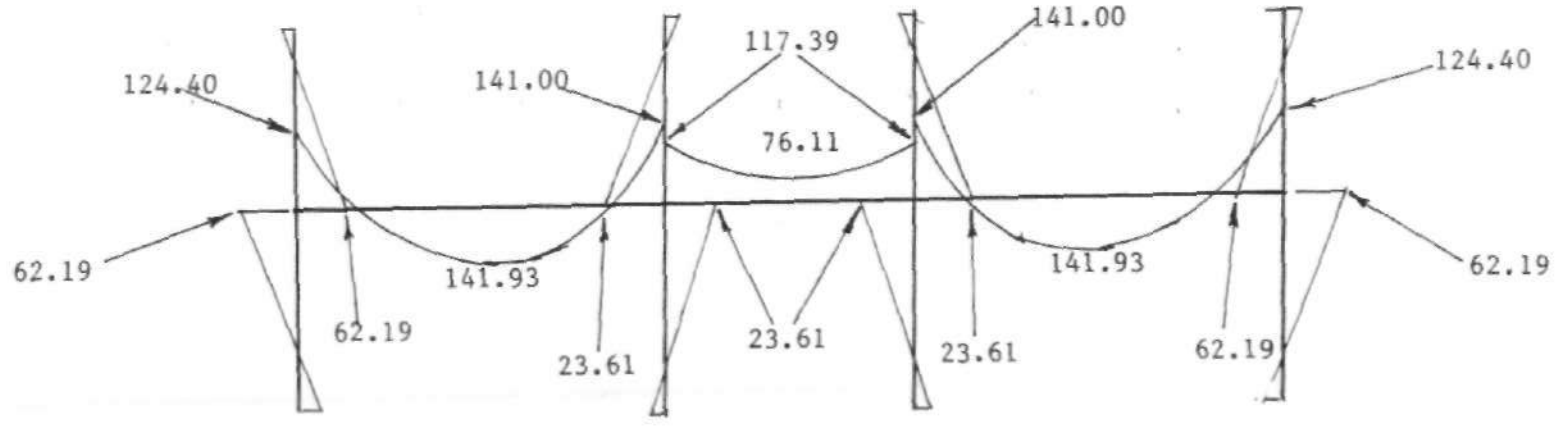


TABLE 4.6

Moment Distribution for Load Case 2.



JOINT	A			B				B				A		
Member	A-2	A-B	A-4	B-A	B-2	B-B	B-4	B-4	B-B	B-2	B-A	A-4	A-B	A-2
Dist.Factor	0.271	0.458	0.271	0.319	0.080	0.521	0.080	0.080	0.521	0.080	0.319	0.271	0.458	0.271
F.E.M.		-183.08		183.08		-27.52			27.52		-183.08		183.08	
Balancing	49.61	83.85	49.61	-49.62	-12.44	-95.50	-12.44	12.44	95.50	12.44	49.62	-49.61	-83.85	-49.61
Distribution		-24.81		41.93		47.75			-47.75		-41.93		24.81	
Balancing	6.72	11.36	6.72	-28.61	-7.17	-55.90	7.17	7.17	55.90	7.17	28.61	-6.72	11.36	-6.72
Distribution		-14.30		5.68		-27.95			-27.95		-5.68		14.30	
Balancing	3.88	6.54	3.88	-10.73	2.69	17.52	-2.69	2.69	17.52	2.69	10.73	-3.88	-6.54	-3.88
Distribution		-5.37		3.27		8.76			-8.76		-3.27		5.37	
Balancing	1.46	2.45	1.46	-3.84	-0.96	-6.27	-0.96	0.96	6.27	0.96	3.84	-1.46	2.45	-1.46
Distribution		-1.92		1.23		3.14			3.14		-1.23		1.92	
Balancing	0.52	0.88	0.52	-1.39	-0.35	-2.28	-0.35	0.35	2.28	0.35	1.39	-0.52	-0.88	-0.52
TOTAL	62.19	-124.40	62.19	141.00	-23.61	-117.39	-23.61	23.61	117.39	23.61	-141.00	-62.19	124.40	-62.19



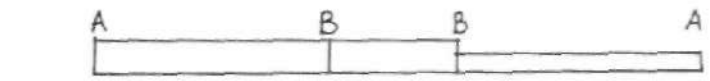


TABLE 4.7
Moment Distribution for Load Case 3.

JOINT	A			B				B				A		
Member	A-2	A-B	A-4	B-A	B-2	B-B	B-4	B-4	B-B	B-2	B-A	A-4	A-B	A-2
Dist.Factor	0.27	0.458	0.271	0.319	0.080	0.521	0.080	0.080	0.521	0.080	0.319	0.271	0.458	0.271
F.E.M.		-320.27		320.27		-27.52			27.52		-183.08		183.08	
Balancing	86.79	146.68	86.79	-93.39	-23.42	-152.52	-23.42	12.44	95.50	12.44	49.62	-49.61	-83.85	-49.61
Distribution		-46.69		73.34		47.75			-76.26		-41.93		24.81	
Balancing	12.65	12.65	12.65	-38.63	-9.68	-63.09	-9.68	9.46	61.57	9.46	37.70	-6.72	-11.36	-6.72
Distribution		-19.31		10.69		30.79			31.54		-5 .68		18.85	
Balancing	5.23	8.84	5.23	-13.23	-3.32	-21.61	-3.32	2.38	19.39	2.38	11.87	-5.11	-8.63	-5.11
Distribution		-6.61		4.42		9.69			-10.8		-4.31		5.93	
Balancing	1.79	3.03	1.79	-4.450	-1.13	-7.35	-1.13	1.21	7.87	1.21	4.82	-1.61	-2.72	-1.61
Distribution		-2.25		1.52		3.94			-3.18		-3.18		2.4	
Balancing	0.61	1.03	0.61	-1.74	-0.44	-2.8	-0.44	0.36	2.37	0.36	1.45	-0.65	-1.10	-0.65
Distribution		-0.87		0.52		1.19		1.19	-1.43		-0.55		0.73	
Balancing	0.24	0.39	0.24	-0.55	-0.14	-0.89	-0.14	-0.89	1.03	0.16	0.63	-0.20	-0.33	-0.20
T O T A L	107.31	-214.65	107.31	158.72	-38.13	-182.47	-38.13	26.01	92.04	26.01	-130.82	-63.90	127.82	-63.90

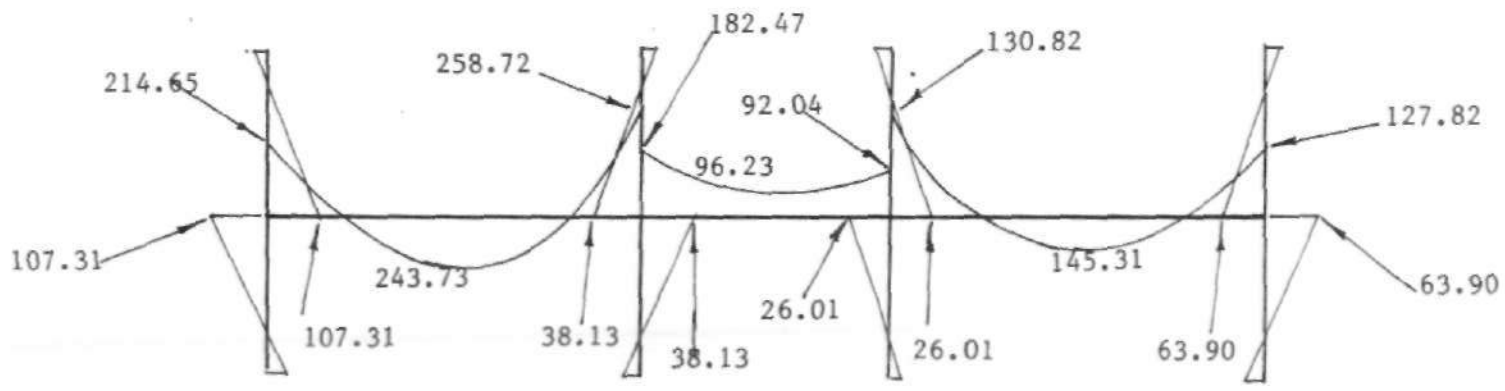
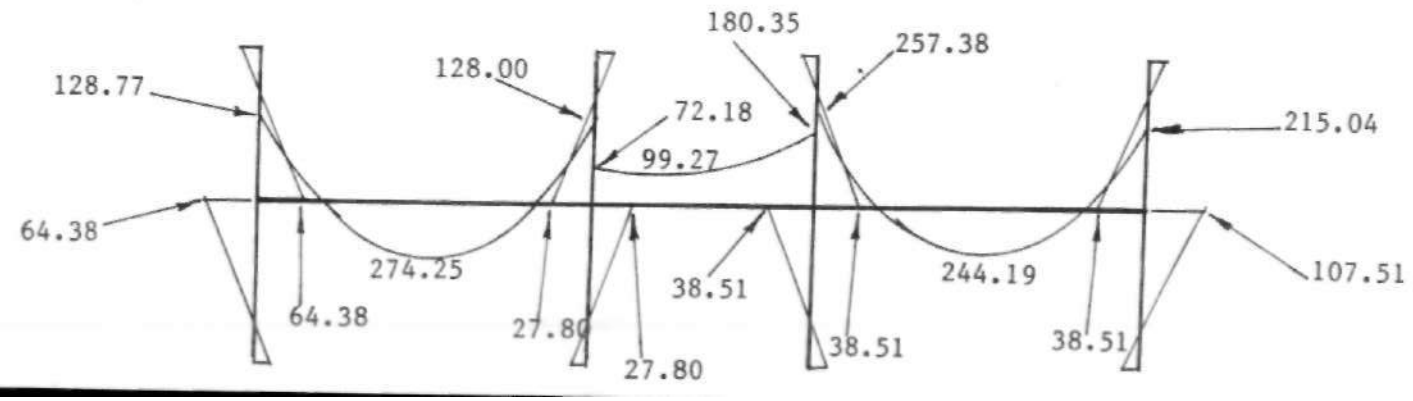


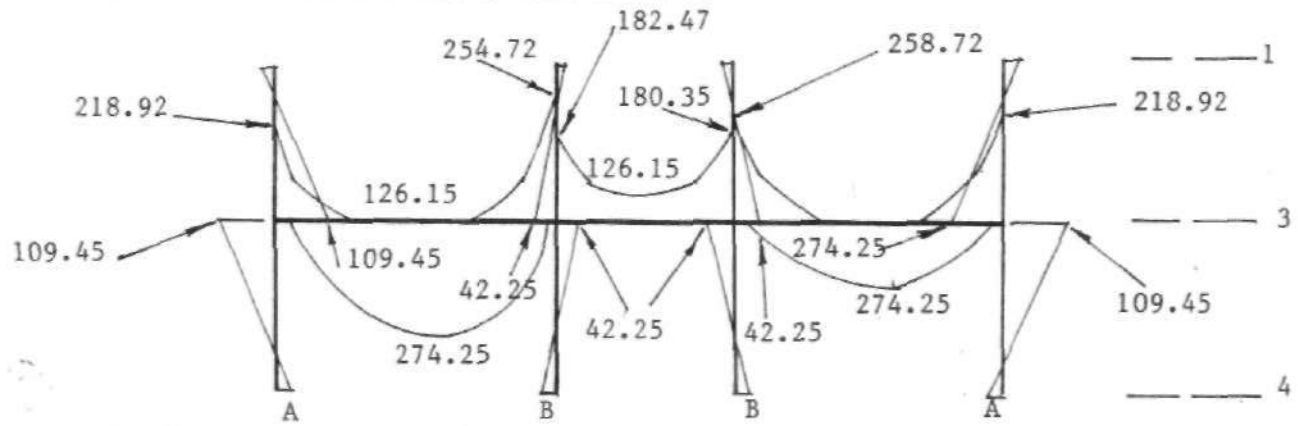
TABLE 4.8
Moment of Distribution for Load Case 4.



JOINT	A			B				B				A		
Member	A-2	A-B	A-4	B-A	B-2	B-B	B-4	B-4	B-B	B-2	B-A	A-4	A-B	A-2
Dist.Factor	0.271	0.458	0.271	0.319	0.080	0.521	0.080	0.080	0.521	0.080	0.319	0.271	0.458	
F.E.M.		-183.08		183.08		-18.59			18.59		-320.27		320.27	
Balancing	49.61	83.85	49.61	-52.47	-13.16	-85.70	-13.16	24.13	157.18	24.13	96.24	-86.79	-146.68	-86.79
Distribution		-26.24		41.93		78.59			-42.85		-73.34		48.12	
Balancing	7.11	12.02	7.11	-38.45	-9.64	-62.79	-9.64	9.30	60.53	9.30	37.06	-13		
Distribution		-19.22		6.01		30.26			-31.39		-11.02		18.53	
Balancing	5.21	8.80	5.21	-11.57	-2.90	-18.89	-2.90	3.39	22.10	3.39	13.53	-5.02	-8.49	-5.02
Distribution		-5.79		4.40		11.05			-9.49		-4.24		6.77	
Balancing	1.57	2.65	1.57	-4.93	-1.63	-8.05	-1.63	1.10	7.15	1.10	4.38	-1.83	-3.10	-1.83
Distribution		-2.47		1.33		3.58			-4.03		-1.55		-2.19	
Balancing	0.67	1.13	0.67	-1.57	-0.39	-2.56	-0.39	0.45	2.19	0.45	1.78	-0.59	-1.01	-0.59
Distribution		-0.78		0.56		1.45			-1.28		-0.50		0.89	
Balancing	0.21	0.36	0.21	-0.32	-0.08	-0.53	0.08	0.14	0.57	-0.24	0.57	0.24	-0.41	-0.24
T O T A L	64.38		64.38		-27.80		-27.80		180.35		-257.36		215.04	
		-128.7		128.00		-72.18		38.51		38.51		-107.51		-107.51



3. Second Floor Beams (Floor Level 3)



Bending Moment Envelope.

TABLE 4.9. Results of moment distribution for second floor (fig. 2.5.).

JOINT	A			B			
MEMBER	A-2	A-B	A-4	B-2	B-B	B-4	B-A
MOMENT	109.45	218.92	109.45	42.25	182.47	42.25	258.72
SPAN MOMENT		274.25			-126.15		

Above is the bending moment result for moment distribution for second floor (floor level 3). Frame 1.

TABLE 4.10 Results of moment distr. for all other beams as shown.

THIRD FLOOR INTERNAL BEAMS (FRAME 1)

JOINT	A		B		
Member	A-B	A-2	B-A	B-2	B-B
End Moments	199.07	199.07	261.91	73.26	190.15
Span Moments	260.70				-139.20

FIRST FLOOR EXTERNAL BEAMS

Joint	A			B			
Member	A-3	A-B	A-5	B-A	B-3	B-B	B-5
End Moments	148.11	269.25	121.14	268.75	57.68	163.24	47.65
Span Moments		211.41				-121.96	

PARAPET WALL BEAMS

Joint	C		D			E		
Member	C-2	C-D	D-C	D-2	D-E	E-D	E-2	E-A
End Moment	12.01	12.01	19.32	7.5	11.82	11.82	7.5	19.32
Span Moment		15.87						

TABLE 4.10 (Cont'd)

SECOND AND THIRD FLOOR BEAMS.

JOINT	C			D				E		
Member	C-2	C-D	C-4	D-C	D-2	D-E	D-4	E-D	E-2	E-4
End Moment	69.31	138.62	69.31	151.91	27.93	123.37	27.93	29.52	5.99	5.99
Span Moment		126.35				-26.73		-26.73		

FIRST FLOOR, EXTERNAL BEAMS

JOINT	C			C				E		
Member	C-3	C-D	C-5	D-C	D-3	D-E	D-5	E-D	E-3	E-5
End Moment	77.85	141.53	63.68	152.82	15.48	124.60	12.75	30.11	16.50	13.61
Span Moment		120.11				-18.28 12.92		-18.28 12.92		

THIRD FLOOR, INTERNAL BEAMS

JOINT	F		G			G_1			H	
Member	F-G	F-3	G-F	G-3	G-G_1	G_1-G	G_1-3	G_1-H	H-G_1	H-3
End Moment	107.72	107.72	157.29	45.41	123.94	177.22	54.01	222.98	152.31	152.31
Span Moment	167.91				-76.19			222.45		

SECOND AND THIRD FLOOR, INTERNAL BEAMS.

JOINT	F			G				G_1				H		
Member	F-2	F-G	F-4	G-F	G-2	G-G_1	G-4	G_1-G	G_1-2	G_1-4	G_1-4	H-G_1	H-2	H-4
End Moment	73.27	146.52	73.27	158.23	31.85	105.50	31.85	150.80	40.74	220.90	40.74	201.72	100.87	100.87
Span Moment		137.98				-51.22				182.06				

TABLE 4.10 (Cont'd)
PRAPET WALL BEAM (FRAME 3.)

JOINT	J			K			L	
Member	J-F	J-2	J-K	K-J	K-2	K-L	L-K	L-2
End Moment	23.93	11.20	12.73	15.15	14.32	29.47	15.09	15.09
Span Moment			-5.42			19.98		

SECOND AND THIRD FLOOR EXTERNAL BEAMS

JOINT	J			K				L		
Member	J-2	J-K	J-4	K-J	K-2	K-L	K-4	L-2	L-K	L-4
End Moment	14.63	29.30	14.63	114.88	27.04	162.14	27.04	73.84	141.93	73.84
Span Moment		-21.63				122.34				

FIRST FLOOR EXTERNAL BEAMS

JOINT	J			K				L		
Member	J-3	J-K	J-5	K-J	K-3	K-L	K-5	L-3	L-K	L-5
End Moment	16.07	29.31	13.24	101.77	34.89	164.41	28.75	81.14	147.68	66.54
Span Moment		-20.91				121.01				

TABLE 4.10 (Cont'd)

FIRST FLOOR INTERNAL BEAM

JOINT										
Member	M-F	M-5	F-3	F-M	F-5	F-G	G-F	G-3	$G-G_1$	G-5
End Moment	190.97	190.97	189.93	471.43	146.80	235.15	190.74	2.75	137.10	
Span Moment	290.95					-7.85		29.00	-58.32	
						85.80			14.67	

(First Floor Internal Beam (Cont'd)

Joint							
Member	G_1-G	G_1-3	G_1-H	G_1-5	$H-G_1$	H-3	H-5
End Moment	222.71	46.46	323.21	38.72	195.29	106.16	87.13
Span Moment			190.12				

SPAN B_1-B.

Ultimate load = 35.69 KN/M

Moments, $M_{BB_1} = M_{B_1B}$=132.12 KNM

So, $F_{BB_1} = F_{B_1B}$=35.69x2.5/2+(132.12-132.12)/2.5

=44.61 KN

LOAD CASE 2

SPANS A-B

Ultimate load = 87.88 KN/M

Moments, M_{AB}=143.30 KNM

M_{BA}=146.12 KNm

So, F_{AB}=87.88x5/2+(143.30-146.13)/5

=219.13 KN

F_{BA}=87.88x5 - 219.13

=220.27 KN

SPAN B_1 - B

Ultimate load = 52.84 KN/M

Moments, $M_{BB_1} = M_{B_1B}$=87.60 KNM

So, F_{BB}=66.05 KN

LOAD CASE 3.

SPANS A-B

Ultimate load = 153.75 KN/M

Moments, M_{AB}=249.20 KNM

M_{BA}=260.00 KNM

So, F_{BA}=382.22 KN

F_{AB}=386.54 KN

SPAN B_1 - B

Ultimate load = 52.84 KN/M

Moments, M_{BB_1}=159.83 KNM

M_{B_1B}=66.03 KNM

F_{BB_1}=103.57 KN

F_{B_1B}=94.58 KN

SPAN B_1-A

Ultimate load = 87.88 KN/M

Moments M_{B_1A} =134 .98 KNM

M_{AB_1}=147.17 KNM

So, F_{B_1A}=217.26

F_{AB_1}=222.14 KN

LOAD CASE 4

SPAN A-B

Ultimate load = 87.88 KN/M

Moments, M_{AB}=148.34 KNM

M_{BA}=131.67 KNM

F_{AB}=223.03 KN

F_{BA}=216.37 KN

SPAN B-B_1.

Ultimate load = 35.69 KN/M

Moment, M_{BB_1}=59.93 KNM

M_{B_1B}=150.61 KNM

F_{BB_1}=8.34 KN

F_{B_1B}=80.88 KN

SPAN B_1-A

Ultimate load = 153.75 KN/M

Moments, M_{B_1A}=256.59 KNM

M_{AB_1}=249.38 KNM

F_{B_1A}=385.82 KN

F_{AB_1}=382.94 KN

Find results for shear force computation for all the other beams shown in the following tables.

TABLE 4.11 Results for shear force computed for internal beams in frame1.

JOINT	A	B		B_1		A
Member	A - B	B - A	B - B_1	B_1-B	B_1 - A	A - B_1
Shear Force for 3rd Floor Beams	384.43	384.23	104.21	104.21	384.27	384.43
Shear Force for 2nd Floor Beams	385.71	386.54	103.57	103.57	385.82	385.71
Shear Force for 1st Floor Beams	384.43	384.43	104.31	104.31	384.43	384.83

i. INTERNAL BEAM (FRAME 1)

TABLE 4.12. Computed shear force for external beams in frame 2.

JOINT	C	D		E		A
Member	C-D	D-C	D-E	E-D	E-A	A-E
Shear Force for Parapet Wall Beams	23.79	26.69	12.61	12.61	26.69	23.79
S.F. For 2nd & 3rd Floor Beams	212.90	216.75	90.91	26.74		
Shear Force for 1st Floor Beams	129.24	133.76	77.29	15.41		

ii.EXTERNAL BEAM (FRAME 2)

TABLE 4.13. Computed shear force for external beams in frames 4 & 7.

JOINT	M	F		G		G_1		H
Member	M-F	F-M	F-G	G-F	G-G_1	G_1-G	G_1-H	H-G_1
S.F. for 3rd Floor Beams			250.58	277.20	112.20	142.76	350.20	314.17
S.F. For 2nd Floor Beams			263.28	266.15	110.04	136.28	334.27	327.92
S.F. For 1st Floor Beams	517.23	678.03	343.43	229.41	96.70	144.35	358.05	324.62

iii. INTERNAL BEAMS (FRAMES 4 & 7).

TABLE 4.14. Computed shear force for external beams in frame 3.

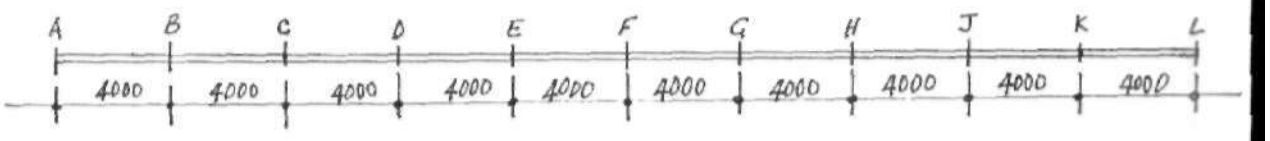
JOINT	F	J		K		L
Member	F-J	J-F	J-K	K-J	J-L	L-K
S.F. for Parapet Wall Beams	23.79	26.69	14.19	14.51	26.69	23.79
S.F. for 2nd 3rd Floor Beams			44.71	94.57	188.39	182.13
S.F. for 1st Floor Beams			46.52	97.79	193.21	183.91

iv. EXTERNAL BEAM (FRAME 3).

4.3.3. BENDING MOMENT (Y-AXIS)

The structure consists of ten span beams of 4.0 meters each. The beams are therefore analysed using co-efficients provided in table 4,CP 110. The columns are then analysed using the method of limited frame analysis. In the column analysis the beams stiffnesses are considered to have half their actual values.

4.3.3.1. INTERNAL BEAMS.



LOADING.

Dead load

$=(4.2\times4^2/4)+[(4.2\times4\times3/2)-(4.2\times3^2/4]+10.4+1.968$

$=44.92$ KN/M

Live load

$=(3.43\times4^2/4)+[(3.43\times4\times3/2)-(3.43\times3^2/4)]$

$=26.58$ KN/M

Ultimate load;

$W = 1.4x44.92+1.6x26.58$

$= 105.42$ KN/M

Moments;

$M_{AB}=M_{KL}=0.091x105.42x4x4 = 153.49$ KNM

$M_B=M_K=-0.111x105.42x4^2=-187.23$ KNM

$M_{BC}=M_{CD}=M_{EF}=M_{FG}=M_{GH}=M_{HJ}=M_{JK}$

$=0.071x105.42x4^2$

$=119.76$ KNM

$M_C=M_D=M_E=M_F=M_G=M_H=M_J$

$=-0.1x105.42x4^2$

$=-168.67$ KNM

4.3.3.2 INTERNAL COLUMNS.

Beam stiffness (T-Beam)= $8.28x10^5$

Column stiffness: Ground floor = $0.98x10^5$

Other floors = $1.19x10^5$

TABLE 4.15. DISTRIBUTION FACTOR.

At Floors 2 and 3 level

Member	Stiffness	Sum of Stiff.	D.F.
Beam (L)	$0.5x8.28x10^5$	$10.66x10^5$	0.341
Col. (U)	$1.19x10^5$		0.159
Beam (R)	$0.5x8.28x10^5$		0.341
Col. (L)	$1.19x10^5$		0.159

At floor 4 level

Member	Stiffness	Sum of Stiff.	D.F
Beam (L)	$0.5x8.2810^5$	$10.45x10^5$	0.351
Col. (U)	$1.19x10^5$		0.164
Beam (R)	$0.5x8.28x10^5$		0.351
Col. (L)	$0.98x10^5$		0.135

TABLE 4.15

Joint out of balance moment at floor level

$M_P=105.42x4^2/12-62.89x4^2/2$

$=140.54-83.85$

$= 56.69$ KNM

Column moments:

At joints 2 and 3:

$M_{c(L)}=M_{c(u)}=0.159x56.69$

$=9.01$ KNM

At joint 4;

$M_{c(u)}=0.164x56.69$

$=9.30$ KNM

$M_{c(L)}=-.135x56.69$

$=7.65$ KNM

At base level (joints 5);

$M_{c(base)}=0.5x7.65$

$=3.83$ KNM

Following the same procedure the Y-axis bending moments for the rest of the columns were calculated as showing in the following table.

4.3.3.3. EXTERNAL BEAMS

$M_{AB}=M_{KL}=91.25$ KNM

$M_B=M_K= -111.30$

$M_{BC}=M_{CD}=M_{EF}=M_{FG}=M_{GH}=23.45$ KNM

$M_C=M_D=M_E=M_F=M_GM_H=M_J= - 100.29$

TABLE 4.16 Column bending moments in the Y-axis.

COLUMNS	LENGTH	END	MOMENT (KNM)
INTERNAL	1 - 2	UPPER	0
		LOWER	9.01
	2 - 3	UPPER	9.01
		LOWER	9.01
	3 - 4	UPPER	9.01
		LOWER	9.30
	4 - 5	UPPER	7.65
		LOWER	3.83
EXTERNAL	1 - 2	UPPER	2.88
		LOWER	33.87
	2 - 3	UPPER	33.87
		LOWER	33.87
	3 - 4	UPPER	33.87
		LOWER	35.42
	4 - 5	UPPER	29.23
		LOWER	14.62
CORNER	1 - 2	UPPER	8.72
		LOWER	18.68
	2 - 3	UPPER	18.68
		LOWER	18.68
	3 - 4	UPPER	18.68
		LOWER	19.80
	4 - 5	UPPER	16.11
		LOWER	8.06

4.3.4. SHEAR FORCE.

Table 4, CP 110 also provides coefficients for shear force in beams. Simplified rules for bar curtailment in beams is given in clause 3.11.7.2, CP 110.

4.3.4.1 INTERNAL BEAMS.

Ultimate load = 105.42 KN

Shear at outer support,

$F_{AB}=F_{KL}=0.45\times105.42\times4$

$=187.76$ KN

Shear at first interior support

$F_{BA}=F_{KL}=4\times105.42-187.76$

$=231.92$ KN

$F_{BC}=F_{KJ}=0.6\times105.42\times4$

$=253.01$ KN

Shear at interior supports,

$F_{CB}=F_{JK}=4\times105.42-253.01$

$=168.67$ KN

$F_{CD}=F_{DE}=F_{EF}=F_{JH}=F_{HG}=F_{GF}$

$=0.55\times105.42\times4$

$=231.92$ KN

$F_{DC}=F_{ED}=F_{HJ}=F_{GH}=F_{FG}=F_{FE}$

$=4\times105.42-231.92$

$=189.76$ KN

4.3.4.2 EXTERNAL BEAMS

Ultimate load = 62.67

$F_{AB}=F_{LK}$= 0.45x62.67x4 =112.81 KN

$F_{BA}=F_{KL}$=4x62.67-112.81 = 137.87 KN

$F_{BC}=F_{KJ}$=0.6x62.67x4 = 150.41 KN

$F_{CB}=F_{JK}$=4x62.67-150.41 = 100.27 KN

$F_{CD}=F_{DE}=F_{EF}=F_{JH}=F_{GH}=F_{FG}$=137.87 KN

$F_{DC}=F_{ED}=F_{HJ}=F_{HG}=F_{GF}$=112.81 KN

4.4. DESIGN OF REINFORCED CONCRETE BEAMS.

In Section 4.2.1 beam depth of 460mm and rib of 230mm were found adequate. These were found adequate. These dimensions are used for all the beams in the structure. Below are the calculations for detailed design and serviceability requirements for the beams in the first floor. For the rest of the beams the results of the design calcualtions are shown in tables.

4.4.1. DESIGN DATA.

High yield, plain bars -----fy = 410 N/mm^2

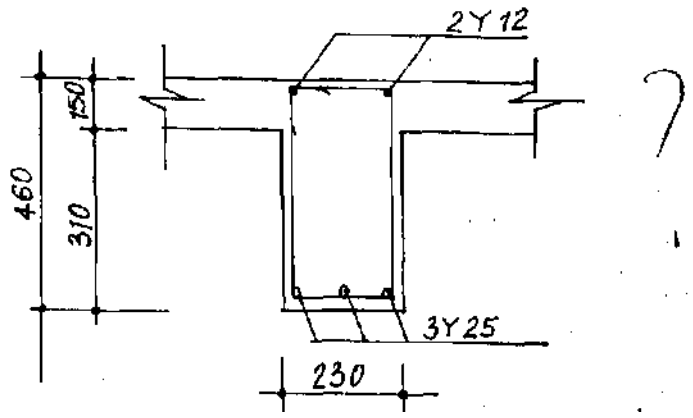
Concrete grade 30

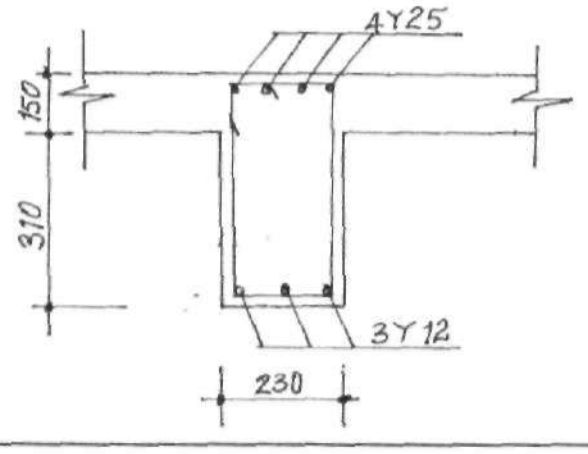
Concrete cover for moderate exposure = 30mm

4.4.2. DESIGN OF BEAMS IN THE FIRST FLOOR.

4.4.2.1. DESIGN OF BEAM B_1 (FRAME 1)

BEAM SPANS A-B AND B_1 - A
AT MID - SPAN $M/b_f.f_{cu}.d^2=274.25x10^6/930x30x425^2$

CALCULATIONS	OUTPUT
= 0.054 From lever arm curve, l_a=0.93 Lever arm, Z = l_a.d =0.93x425 =395.35mm Depth of Neutral Axis, X = (d-z)/0.45 =66.44 Depth of beam flange, h_1=150mm	Neutral Axis is in slab.
REINFORCEMENT: A_s=M/0.87x$f_y$xz. =274.25x10_6/0.87x395 =1500mm^2	
Minimum area of steel required: A_s=0.0015xbxd =0.0015 x 200 x 395 =118.5mm^2 Maximum area of steel required; A_s=0.04xbxh =0.04x200x460 = 3440 mm^2 Selection: Provide 3Y 25 bars (1470mm^2)  	Area of reinforcement is within the required limit.

CALCULAITONS	OUTPUT
AT EXTERIOR SUPPORT, A. The Section is to be designed as rectangular beam. $M/b_c.f_{cu}.d^2=218.92x10^6/230x30425^2$ $=0.176$ This is greater than 0.15. Therefore comperession reinforcement is required. Compression Reinforcement: $A_{sc}=(M-0.15f_{cu}.bd^2)/0.72fy(d-d_c)$ $=(218.92x10^6-0.15x30x230x425^2)/0.72x410x395$ $=274\ mm^2$ Selection: Provide 3Y12 bars ($339mm^2$ Tension Reinforcement: $A_s=(0.2f_{cu}.b.d+0.72fyA_{sc})/0.87fy.$ $=(0.2x30x425+410x339)0.78x410$ $= 1925\ mm^2$ Selection: Provide 4 Y 25 bars ($1960mm^2$)  	

CALCULATIONS	OUTPUT
AT INTERIOR SUPPPORT B. Bending moment is 258.72 KNM Provide the same reinforcement a for beams section at mid-span	
BEAM SPAN B - B_1	
AT MID-SPAN The section is designed as rectangualr seciton due to hogging moment. $M/bd^2f_{cu}=126.19x10^6/230x425^2x30$ $=0.101$ From lever arm curve, $l_a=0.860$ Lever arm, $Z=0.860x425$ $=366.00mm$	
Reinforcement: $A_s=126.19x10^6/0.87x410x366$ $=967mm^2$ Selection: Provide 2Y25mm bars ($982mm^2$)	
AT SUPPORTS, B, B_1 $M/bd^2f_{cu}=182.47x10^6/230x425^2x30$ $=0.146mm^2$ From lever arm curve, $l_a=0.76$ Reinforcement: $A_s=182.47x10^6/0.87x323x410$ $= 1583\ mm^2$	

CALCULATIONS	OUTPUT
Selection: Provide 3Y25 bars (1470mm^2)	
SHEAR STRESSES	
BEAM SPAN A-B	
AT MID-SPAN The code recommends that nominal links should be provided such that A_{sv}/S_v is equal to, or greater than $0.00126b_t$ for high yield links. Where A_{sv} is the area of links, S_v is links spacing and b_t is thickness of beam rib. Nominal links: $0.0012b_t$=0.0012x230=0.276 Selection: Assume Y8 links at 300mm A_{sv}/S_v=168/300 the nominal links are quite adequate	Y8 at 300mm links
AT EXTERIOR SUPPPORT, A. Shear force, F_s=385.71 KN Shear stress, V=F_s/bd =385.7110^3/230x425 =3.95 N/mm^2 Maximum shear stress allowed =4.10N/mm^2	Nominal links are adequate
AT INTERIOR SUPPORT B. Shear force, F_s=386.54 KN Provide same as for exterior support	Nominal Links

TABLE 6, CP110

CALCULATIONS	OUTPUT
BEAM SPAN B-B	
AT MID-SPAN Nominal links: $0.0012b_t=0.0012x230=0.276$ *Selection:* $A_{sv}/S_v=0.287$	Nominal links are adequate
AT SUPPORTS B AND B_1 Shear force, F_s=103.57 KN Shear stress, V = F_s/bd =1.06 N/mm² Maximum allowed value = 4.10N/mm² Provide nominal links of Y8 at 350	Nominal links are adequate
LOCAL BOND STRESSES	
Since the beam is continous local bond stresses check is only at points of contra-flexure. No check at supports since there is no simple support.	
BEAM SPAN A-B	
Point of contra flexure is at a distance of 0.15 of span from support. Shear force, F_s=386.54-0.15x153.73 =271.24 KN Total bar perimeter for 3Y25	

TABLE 6,
CP 110

TABLE 21
CP 110

CALCULATIONS	OUTPUT
$U_{ST}=4x3.142x25$ $=236mm$ Local bond stress: $f_{bs}=271.24x10^6/236x425$ $=2.17\ N/mm^2$ Ultimate value: $f_{ul}=2.2\ N/mm^2$	Section is adequate in local bond.
BEAM SPAN B-B_1	
It is not necessary to check local bond stress, since there is no point of contraflexure.	
SPAN-EFFECTIVE DEPTH RATIO	
BEAM SPAN A-B.	
$100A_s/b_f.d=100x1470/930x425$ $=0.37$ Modification factro = 1.41 For a flanged beam: $b/b_f=230/930$ $=0.247$ From span-effective depth modification factor curve for a flanged beam: further modification factor =0.80 Maximum span-effective depth: 26x1.41x0.80 = 29.33 Actual span-effective depth: L/d = 500/425 =11.76	Beam depth is adequate

CALCULATIONS	OUTPUT
BEAM SPAN B-B	
$100A_s/bd = 100x982/230x425$ $=1.005$: Modification factor =0.99 Maximus span-effective depth ratio: $26x0.99 = 25.74$ Actual span-effective depth ratio: $2500/425=5.88$ The beam depth is excessively adequate but this is allowed in order to maintain uniformity in beam depth.	

TABLE 10, CP, 100.

4.4.2.2. DESIGN OF BEAM B_2 (FRAME 2)

BEAM SPAN C-D	
AT MID-SPAN. $M/b_f.f_{cu}.d^2 = 120.11x10^6/580x30x425^2$ $=0.038$ From lever arm curve; $l_a=0.95$ Lever arm; $Z=0.95x425$ $=404mm$ Depth of neutral axis; $X=(d-z)/0.45$ $=46.67mm$ Depth of beam flange; $h_1=150mm$	N.A. is in slab

CALCULATIONS	OUTPUT
Reinforcement: $A_s = M/0.87f_y.z$ $= 120.11 \times 10^6 / 0.87 \times 410 \times 404$ $= 833\ mm^2$ Minimum area of steel required: $A_s = 0.0015bd$ $= 0.0015 \times 230 \times 425$ $= 146.6mm$ Maximum area of steel required: $A_s = 0.04bh$ $= 0.04 \times 230 \times 460$ $= 4232\ mm^2$ Selection: Provide 2Y20 bars ($628mm^2$) 1Y16 bar ($201mm^2$)	Area of steel is within the specified limit.
AT EXTERIOR SUPPORT, C $M/b \cdot f_{cu} \cdot d^2 = 141.53 \times 10^6 / 230 \times 30 \times 425^2$ $= 0.114$ From lever arm curve, $l_a = 0.84$ Lever arm, $z = 0.84 \times 425$ $= 357mm$ Reinforcement: $A_s = 141.53 \times 10^6 / 0.87 \times 410 \times 357$ $= 1111mm^2$ This is within the required limit. Selection: Provide 3Y20 bars ($943mm^2$) 2Y16 bars ($402mm^2$)	

CALCULATIONS	OUTPUT
AT INTERIOR SUPPORT, D. $M/b \cdot f_{cu} \cdot d^2 = 152.82 \times 10^6 / 230 \times 30 \times 425^2$ $= 0.123$ From lever arm curve, $l_a = 0.82$ Lever arm, $Z = 0.82 \times 425$ $= 349mm$ Reinforcement: $A_s = 152.82 \times 10^6 / 0.87 \times 410 \times 349$ $= 1228\ mm^2$ This is within the required limit Selection: Provide 3Y20 bars ($943\ mm^2$) 2Y16 bars ($402\ mm^2$)	
BEAM SPAN D-E	
AT MID-SPAN Hogging moment cotrols the section. So, design as rectangular beam. $M/bd^2 f_{cu} = 18.28 \times 10^6 / 230 \times 30 \times 425^2$ $= 0.015$ From lever arm curve, $l_a = 0.95$ Lever arm, $Z = 0.95 \times 425$ $= 404mm$	N.A. is in Slab

CALCULATIONS	OUTPUT
Reinforcement $A_s = M/0.87f_y.Z$ $= 18,20x10^6/410x404$ $= 127mm^2$ Selection: Provide 2Y10 bars (157 mm^2)	
AT INTERIOR SUPPORT, D. $M/bf_{cu}d^2 = 124.60x10^6/230x30x425^2$ $= 0.100$ From lever arm curve, $l_a = 0.855$ Lever arm, $Z = l_a.d$ $= 363mm$ Reinforcement: $A_s = 124.60x10^6/0.87x410x363$ $= 962mm^2$ Selection: Provide 3Y20 bars ($943mm^2$).	
AT EXTERNAL SUPPORT, E. $M/bd^2f_{cu} = 30.11x10^6/230x30x425^2$ $= 0.024$ From lever arm curve, $l_a = 0.95$ Lever arm, $Z = l_a.d$ $= 404mm$	

CALCULATIONS	OUTPUT
Reinforcement A_s=M/0.87fy·Z =208mm² Selection: Provide 2Y12 bars (226mm²)	2Y12 bars
SPAN-EFFECTIVE DEPTH RATIO	
BEAM SPAN C-D	
$100A_s/b_f.d$=100x829/580x425 =0.34 Modification factor = 1.39 For a flanged beam b/b_f=230/580 =0.40 From span-effective depth modification factor curve for a flanged beam Further modification factor =0.82 Maximum span-effective depth ratio: 26x1.39x0.82=30.0 Actual span-effective depth ratio: 5000/425 = 11.74	Beam depth is adequate
BEAM SPAN D-E	
$100A_s/bd$ = 100x157/230x425 =0.161 Modification factor = 1.86 Maximum span-effective depth ratio: 26x1.86=48.36 Actual span-effective depth ratio; 2500/425 =5.88	Beam depth is adequate

TABLE 10
CP 110

TABLE 10
CP 110

TABLE 6
CP 110

CALCULATIONS	OUTPUT
SHEAR STRESSES.	
BEAM SPAN C-D	
AT MID-SPAN Nominal links: $0.0012b_t=0.0012x230$ $=0.28$ Assume Y8 links at 300mm $A_{sv}/s_v=100.54/300$ $=0.34$	Nominal links are adequate
AT EXTERIOR SUPPORT, C. Shear force, $F_c=129.24$ KN Shear stress, $V=F_s/bd$ $=129.24x10^6/230x425$ $= 1.32$ N/mm^2 Allowable value = 4.10 N/mm^2	Provide nominal links of Y8 at 300
AT INTERIOR SUPPORT, D. Shear force, $F_s=133.76$ KN Shear stress, $V=F_s/bd$ $=133.76x10^3/230x425$ $= 1.37$ N/mm^2 Allowable value= 4.10 N/mm^2	Provide nominal links of Y8 at 300
BEAM SPAN D-E	
Shear force is relatively low. Therefore, provide nominal links as for span C-D	Y8 at 300
LOCAL BOND STRESSES	
BEAM SPAN C-D	
Point of contraflexure is approximately 0.15 of span from support.	

TABLE 21

CALCULATIONS	OUTPUT
At point of contraflexure: Shear force, F_s=133.76-0.15x5x153.73 =18.46 KN Total bar perimeter for 2Y20 and 1Y16 U_{ST}=125.68+50.27 =175.95mm^2 Local bond stress: $f_{bs}=F_s/d\cdot U_{ST}$ =0.25 N/mm^2 Allowable value =2.2 N/mm^2	Section is adequate
BEAM SPAN D-E	
No check for local bond stress is required since there is no point of contraflexure.	

4.4.2.3. DESIGN OF BEAM B_4/B_7 (FRAME 4 AND 7)

BEAM SPAN M-F	
AT MID-SPAN. $M/b_f.f_{cu}.d^2$ =290.95x10^6/1070x30x465^2 =0.41 From lever arm curve, L_a=0.94 Lever arm, $Z=d\cdot l_a$ =0.94x465 =437 mm Depth of neutral axis; X = (d-z)/0.45 =65.82mm	

CALCULATIONS	OUTPUT
Depth of beam flange: h_1=150mm Reinforcement: A_s=M/0.87f_y.Z. =290.95x10^6/0.87x410x437 =1867mm^2 Selection: Provide 4Y25 bars (1960mm^2)	4Y25 bar
AT EXTERIOR SUPPORT, M. M/b·f_{cu}.d^2 =190.92x10^6/230x30x465^2 =0.127 From lever arm curve, l_a=0.80 Lever arm, Z=0.80x465 =372mm Reinforcement: A_s=M/0.87f_y.Z = 1439mm^2 Selection: Provide 3Y25 bars (1470mm^2)	
AT INTERIOR SUPPORT, F. M/bf_{cu}.d^2 =471.43x10^6/230x30x465^2 =0.316 This is greater than 0.15 Compression reinforcement is required Compresiion reinforcement: A_{SC}=(471.43x10^6-0.15x30x130x465^2)/0.72 X 410x465 =1804mm^2	

CALCULATIONS	OUTPUT
Selection: Provide 2Y32 bars) } (1881$1mm^2$) 1Y16 bar) Tension reinforcement A_s=(0.2x30x230x465+0.72x410 X 1811)/0.87x410 = 3298 mm^2 Selection: Provide 4Y32 bars (3220mm^2)	
BEAM SPAN F-G	
AT MID-SPAN $M/b_f d^2 f_{cu}$=85.80x10^6/632x30x465^2 =0.083. From lever arm curve, l_a=0.88 Lever arm, Z=0.88x465 =365.2mm Depth of neutral axis: X =(d-z) /0.45 =110.67 mm Reinforcement: A_s=M/0.87fy·Z =86.8x10^6/0.87x410x365.2 = 666mm^2 Selection: Provide 2Y20 bars (628.mm^2)	

CALCULATIONS	OUTPUT
AT FIRST INTERIOR SUPPORT, F. $M/bf_{cu}d^2$ $=235.15x10^6/230x30x465^2$ $=0.158$ Provide compression reinforcement Compression reinforcement: $A_{sc}=(M-0.15f_{cu}.bd^2)/0.72fy(d-d_c)$ --------(4.5) $=82mm^2$ Selection: Provide 2Y10 bars ($157mm^2$) Tension reinforcement: $A_s=(0.f_{cu}.bd+0.72fyA_{sc})/0.87fy$ --------(4.6) $=1929\ mm^2$ Selection; Provide 2Y32 bars } ($1811mm^2$) 1Y16 bar }	
AT MIDDLE SUPPORT, G. $M/bd^2f_{cu}=190.74x10^6/230x30x465^2$ $=0.128$ From lever arm curve, $l_a=0.80$ Lever arm, $Z=0.80x465$ $=372mm$ Reinforcement: $A_s=M/0.87fy.Z$ $=190.74x10^6/0.87x410x372$ $=1437mm^2$ Selection: Provide 3Y25 bars ($1470mm^2$)	

CALCULATIONS	OUTPUT
BEAM SPAN G-G_1	
AT MID-SPAN Hogging moment controls the section. $M/bd^2f_{cu}=58.32x10^6/230x30x465^2$ $=0.309$ From lever arm curve, $l_a=0.94$ Lever arm, $Z=0.94x465$ $=437.1mm$ Reinforcement: $A_s=58.32x10^6/0.87x410x437$ $=374mm^2$ Selection: Provide 2Y16 bars ($402mm^2$	
AT MIDDLE SUPPORT G. $M/bd^2f_{cu}=137.10x10^6/230x30x465^2$ $=0.09$ From lever arm curve, $l_a=0.87$ Lever arm, $Z=0.87x465$ $=405mm$ Reinforcement $A_s=137.10x10^6/0.87x410x405$ $=718\ mm^2$ Selection: Provide 2Y20 bars ($628mm^2$) 1Y12 bar ($113mm^2$)	

CALCULATIONS	OUTPUT
AT INTERIOR SUPPORT FROM THE OTHER END, G_1 $M/bd^2f_{cu}=222.71x10^6/230x30x465^2$ $=0.149$ From lever arm curve, $l_a=0.76$ Lever arm, $Z=0.76x765$ $=358mm$ Reinforcement: $A_s=M/0.87f_y.Z$ $=222.71x10^6/0.87x410x353$ $=1769mm^2$ Selection: Provide 3Y25 bars ($1470mm^2$) 1Y20 bar ($314\ mm^2$)	
BEAM SPAN G_1-H	
AT MID-SPAN $M/b_fd^2f_{cu}=190.12x10^6/688x465^2x30$ $=0.043$ From lever arm curve, $l_a=0.935$ Lever arm, $Z=0.935x465$ $=435$ Reinforcement: $A_s=190.12x10^6/0.87x410x435$ $=1225mm^2$ Selection: Provide 2Y25 bars ($982mm^2$) 1Y16 bar ($314mm^2$)	

CALCULATIONS	OUTPUT
AT INTERIOR SUPPORT FROM THE OTHER END G_1 $M/bd^2f_{cu}=323.21x10^6/230x465^2x30$ $=0.22$ Provide compression reinforcement. Compression reinforcement: $A_{sc}=(323.21x10^6-0.15x30x230x465^2)/0.72$ x 410.430 $= 783mm^2$ Selection: Provide 2Y20 bars ($628mm^2$) 1Y12 bar ($113mm^2$) Tension reinforcement: $A_s=(0.2x30x230x465+0.72x410x741)/0.87x410$ $=2412\ mm^2$ Selection; Provide 5Y25 bars ($2450mm^2$	
AT EXTERIOR SUPPORT, H. $M/bd^2f_{cu}=195.29x10^6/230x465^2x30$ $=0.130$ mm From lever arm curve, $1_a=0.79$ Lever arm, $Z=0.79x465$ $=367mm$ Reinforcement: $A_s=195.29x10^6/0.87x410x367mm$ $=1491mm^2$ Selection: Provide 3Y25 bars ($1470mm^2$)	

TABLE 10
CP 110

CALCULATIONS	OUTPUT
SPAN-EFFECTIVE DEPTH RATIO	
BEAM SPAN M-F.	
$100A_s/b_f d = 100x1960/1070x465$ $= 0.39$ Modification factor = 1.39 For a flanged beam $b/b_f = 230/1070$ $= 0.21$ From span-effective depth modification factor curve for a flanged beam. Further modification factor = 0.80 Maximum span-effective depth ratio: $26x1.39x0.80 = 28.91$ Actual span-effective depth ratio: $6000/465 = 12.90$	The beam depth is adequate
BEAM SPAN F-G	
$100A_s/b_{f.d} = 628x100/632x465$ $= 0.21$ Modification factor = 1.60 For a flanged beam; $b/b_f = 230/632 = 0.36$ Further modification factor = 0.82 Limiting span-effective depth ratio: $26x1.60x0.36$ Actual span-effective depth ratio: $4300/465 = 9.25$	The beam depth is adequate

	CALCULATIONS	OUTPUT
	SPAN SPAN G-G_1	
TABLE 10 CP 100	$100A_s/b \cdot d$ = 402x100/230x465 =0.38 Modification factor = 1.42 Limiting span-effective depth ratio: 26x1.42=36.92 Actual value: 3000/465 = 6.45	beam sectio is adequate
	BEAM SPAN G_1-H	
TABLE 10 CP 110	$100.A_s/b_f d$=1296x100/688x465 =0.41 Modification factor = 1.35 b/b_f=230/688 = 0.334 further modification factor = 0.81 Limiting span-effective depth ratio: 26x1.35x0.81 = 28.43 Actual value: 4700/465 = 10.11	Beam depth is adequate
	BEAM SPAN G_1-H	
	$100 \cdot A_s/b_f d$=129x100/688x465 =0.41 Modification factor = 1.35 b/b_f=230/688 =0.334 Further modification factor = 0.81	

CALCULATIONS	OUTPUT
Limiting span-effective depth ratio: 26x1.35x0.81 = 28.43 Actual value: 4700/465 = 10.11	 Beam depth is adeqaute
SHEAR STRESSES.	
BEAM SPAN M-F	
AT MID-SPAN Nominal links: $0.0012b_t$=0.0012x230 =0.276 Assume Y8 links at 300mm A_{SV}/S_V=0.34	 No shear bars are required
AT EXTERIOR SUPPORT, M. Shear force, F_s=517.23KN Shear stress, V = F_s/bd =4.84N/mm^2 Limiting shear stress V_s=4.10 N/mm^2 For the provided tension bars $100A_s/bd$ = 1470x100/230x465 = 1.37 Ultimate shear stress V_c=0.77 For shear links: A_{SV}/S_V= b(V-VC)/0.87fyv =230(4.(4.84-0.77)/0.87x410 =2.62 Assume Y8 links at 35mm A_{SV}/S_V= 2.87	 <

CALCULATIONS	OUTPUT
Shear resistance of nominal links and concrete $V_n=[(A_{SV}/S_V)0.87fyv+bv_c]d$ ----------(4.7.) $=(0.34x0.87x410+230x0.77)465$ $=138.75$ KN Shear links of Y8 at 35mm is required over a distance of: $S=(F_s-V_n)\div F/L$ ----------(4.8.) $=(517.23-138.75)/153.73$ = 2.46m from exterrior support M	2.46M
AT INTERIOR SUPPORT, F. Shear force, F_S=687.03 KN Shear stress, $V=F_S=/bd$ $=6.34$ Nmm^2 Allowable shear stress: $V_S=4.10$ N/mm^2 For the provided tension bar: $100A_s/bd = 1960x100/230x465$ $=1.83$ Ultimate shear stress: $V_c=0.85$ For shear links: $A_{SV}/S_V=b(V-V_c)/0.87fyv$ ----------(4.9.) $=30(6.34-0.85)/0.87x410$ $=3.54$ Assume Y10 links at 40mm $A_{SV}/S_V=3.39$ Shear resistance of nominal links and concrete:	

CALCULATIONS	OUPUT
$V_n=[(A_{SV}/S_V)0.87fyv+bv_c]d$ -------(4.9.) =(0.34x0.87x410+230x0.85)465 =147.30 KN Distance over which the links are required; $S = (F_S V_n) \div F/L$ =(678.03-147.30)÷ 153.73 =3.43 M. from interior support, F.	
BEAM SPAN F - G	
AT MID-SPAN Nominal links: $0.0012b_t=230x0.0012$ =0.276 Assume Y8 links at 300mm $A_{SV}/S_V=0.34$	Nominal links are adequate
AT FIRST INTERIOR SUPPORT, F. Shear force, F_S=343.43 KN Shear stress, $V=F_S/bd$ = 3.21 N/mm² Allowable shear stress V_S=4.10 N/mm²	Provide nominal links Y8 at 300
AT MIDDLE FUPPORT, G. Shear force, F_S=229.41 KN Shear stress, V =F_S/bd =2.15 N/mm² Allowable value: V_S=4.10 N/mm²	Provide nominal links

TABLE 6, CP 110.

TABLE 6, CP 110

CALCULATIONS	OUTPUT
BEAM SPAN G-G_1	
Provide nominal links since the shear force is relatively low.	Y8 links at 300
BEAM SPAN G_1-H	
AT MID-SPAN Provide nominal links of Y8 at 300	
AT INTERIOR SUPPORT FROM THE OTHER END, G_1 Shear force, F_S=358.05 KN Shear stress $V=F_S/bd$ = 3.35 N/mm² Allowable value: V_S=4.10 N/mm²	Provide Nominal Links
AT EXTERIOR SUPPORT, H. Provide nominal links	
LOCAL BOND STRESSES.	
BEAM SPAN M-F	
At the point of contraflexure, which is approx. 0.15 of span from support, Shear force; F_S=687.03-0.15x6x153.73 =339.67 KN Total bar perimeter for 4Y25 U_{ST}=242.2mm Local bond stress: $f_{bs}= F_S/d \cdot U_{ST}$ =2.02 N/mm² Ultimate local bond stress f_{ul}=2.20 N/mm²	Adequate

TABLE 6, CP 110

TABLE 21, CP 110

CALCULATIONS	OUTPUT
BEAM SPAN F - G	
At the point of centraflexure: F_S=343.43 -0.15x4.3x153.73 =224.27 KN Total bar perimeter for 2Y20 U_{ST}=226.27mm Local bond stress $F_{bs}=F_s/d\cdot U_{ST}$ Ultimate local bond stress f_{ul}=2.20 N/mm^2	The Section is adequate in local bond.
BEAM SPAN G_1-H	
At the point of contraflexure ; Shear force; F_S=358.05-0.15x4.7x153.73 =249.67 KN Total perimeter of IY16 and 2Y25 U_{ST}=326.77mm Local bond stress $f_{bs}=F_s/dU_{ST}$ =1.84 N/mm^2 Ultimate local bond stress f_{ul}=2.20 N/mm^2	The section is adequate in local bond

TABLE 21, CP 110

TABLE 4.17 RESULT OF DESIGN FOR BEAMS IN THE FIRST FLOOR

BEAM	SPAN (M)	SELECTION			SHEAR LINKS		
		AT SUPPORT	MID SPAN	AT SUPPORT	SUPPORT	MID_SPAN	SUPPORT
B_1 THREE SPANS	5.0 A-B	3Y12 (BOT.) 4Y25 (TOP.)	3Y25(BOT.)	3Y12 (BOT) 4Y25 (TOP.)	NOMINAL	NOMINAL	NOMINAL
	2.5 B-B_1	3Y25 (TOP)	2Y25(BOT.)	3Y25 (TOP)	DITTO	DITTO	DITTO
	5.0 B_1-A_1	AS FOR A-B	AS FOR A-B	AS FOR A - B	DITTO	DITTO	DITTO
B_2 THREE SPANS	5.0 C ~ D	2Y20 2Y16 (TOP)	2Y20 1Y16 (BOT.)	3Y20 2Y16(TOP)	NOMINAL	NOMINAL	NOMINAL
	2.5 D ~ E	3Y20 (TOP)	2Y10(TOP)	2Y12 (TOP)	DITTO	DITTO	DITTO
	5.0 E ~G	AS FOR C-D	AS FOR C-D	AS FOR C-D	DITTO	DITTO	DITTO
B_3 THREE SPANS	4.3 J ~ K	2Y12(TOP)	2Y12(BOTT)	2Y12 2Y20(TOP)	ditto	DITTO	DITTO
	3.0 K ~ L	4Y20 (TOP)	2Y20 1Y16(BOTT)	4Y20(TOP)	DITTO	DITTO	DITTO
	4.0 L-J_1	3Y20 (TOP)	2Y12(BOTT)	3Y12 (TOP)	DITTO	DITTO	DITTO
B_4 & B_2 FOUR SPANS	6.0 M-F	3Y25 (TOP)	4Y25 (BOT)	2Y32 (BOT) 4Y32 (TOP)	Y8 at 35	DITTO	Y10 at 40
	4.3 F-G	2Y10 (BOT.) 3Y32 (TOP)	2Y20(BOT)	3Y25(TOP)	NOMINAL	DITTO	NOMINAL
	3.0 G-G_1	2Y20 1Y12 (TOP)	2Y16(BOT	3Y25 1Y20 (TOP)	DITTO	DITTO	DITTO
	4.7 G_1-H	2Y20 (BOTT) 5Y25 (TOP)	2Y25 1Y16 (BOT)	3Y25 (TOP)	DITTO	DITTO	DITTO

NOMINAL LINKS IS Y8 at 300mm (DOUBLE LEGS)

TABLE 4.18 RESULT OF DESIGN FOR BEAMS IN THE SECOND FLOOR

BEAM	SPAN	SELECTION			SHEAR LINKS		
		AT SUPPORT	MID - SPAN	AT SUPPORT	SUPPORT	MID-SPAN	SUPPORT
B_1 THREE SPANS	A - B	2Y20 B 3Y32 T	2Y25 B 1Y20	2Y20B 1Y32T	Y8 at 45	NOMINAL	Y8 at 45
	B - B_1	2Y25T	2Y20 B 2Y12	2 Y 25T	NOMINAL	DITTO	NOMINAL
	B_1-A_1	2Y20B 3Y32T	2Y25 B 1Y20	2Y20 B 3Y32 T	Y8 at 45	DITTO	Y8 at 45
B_2 THREE SPANS	C - D	2Y25 2Y12	3Y20 B	2Y25 T 1Y16	NOMINAL	NOMINAL	NOMINAL
	D -E	2Y25 T	2Y12 B	2Y12 T	DITTO	DITTO	DITTO
	E -C_1	2Y25 T 1Y16	3Y20 B	2Y25 T 1Y12	DITTO	DITTO	DITTO
B_3 THREE SPANS	J - K	3Y10 T	2Y10 B	2Y20 T 2Y12	DITTO	DITTO	DITTO
	K- L	3 Y 25 T	3 Y 20 B	3Y25 T	DITTO	DITTO	DITTO
	L - J_1	4Y20 T	2 Y 12 B	3 Y 12 T	DITTO	DITTO	DITTO
B_4 THREE SPANS	F - G	2Y25 T 1Y16	2Y25.B	2Y25 T 1Y16	DITTO	DITTO	DITTO
	G - G_1	2Y20 T 1Y16	3 Y 12 B	2 Y 25 1 Y 20	DITTO	DITTO	DITTO
	G_1 - H	3Y12 B 4Y25 T	2Y25 B 1Y20	2Y10 B 3Y25 T	Y8 at 45	DITTO	Y8 at 45

NOMINAL LINKS IS Y8 at 300mm (DOUBLE LEGS)

TABLE 4.19

RESULT OF DESIGN FOR BEAMS IN THE THIRD FLOOR

BEAM	SPAN	SELECTION			SHEAR LINKS		
		AT SUPPORT	MID - SPAN	AT SUPPORT	SUPPORT	MID-SPAN	SUPPORT
B_1	A-B	3Y12 B 3Y25 T	4Y25 B	3Y25 B 3Y32 T	Y8 at 45	NOMINAL	Y8 at 45
	B ~ B_1	3Y25 B 3Y32 B	4Y25 B	3Y12 B 3Y25 T	NOMINAL	DITTO	NOMINAL
	B_1-A_1	3Y25B 3Y32B	4Y25 B	3Y12 B 3Y25 T	Y8 at 45	DITTO	Y8 at 45
B_2	C-D	2Y25 1Y12T	3 Y 20 B	2 Y 25 1 Y 16 T	NOMINAL	DITTO	NOMINAL
	D - E	2Y25 T	2 Y 12 B	2 Y 12 T	DITTO	DITTO	DITTO
	E - C_1	2Y25 1Y16 T	3Y20 B	2Y25 1Y12 T	DITTO	DITTO	DITTO
B_3	J - K	3Y10 T	2 Y10 B	2Y20T 2Y12T	DITTO	DITTO	DITTO
	K - L	3Y25T	3Y20 B	3Y25 T	DITTO	DITTO	DITTO
	L - J_1	4Y20 T	2Y12 B	3Y10 T	DITTO	DITTO	DITTO
B_4	F - G	2Y20 1Y16 T	2Y16 1Y12 B	2Y25 1Y16 T	DITTO	DITTO	DITTO
	G - G_1	2Y25. T	3 Y 25 1 Y 12 B	3Y25. T	DITTO	DITTO	DITTO
	G_1 - H	2 Y 20 B. 4 Y 25.T.	3Y25 1Y16 B	2Y25 1Y16 T	Y8 at 50	DITTO	Y8 at 50

RESULTS OF DESIGN FOR BEAMS IN THE Y-AXIS

TABLE 4.20

BEAM	S E L E C T I O N			
	END-SPAN	INTERIOR SPANS	FIRST INT. SUPPORT	OTHER INT. SUPPORT
INTERNAL	2Y25, 1Y10 } B	2Y20, 1Y16 } B	2Y16 B	2Y25, 1Y16 } T
			2Y32 T	
EXTERIOR	2Y20 B	2Y20 B	2Y20 } T	2Y20 } 12
			1Y12 }	1Y12 }

1. Nominal shear links of Y8 at 300 is to be provided.
2. Checks for span-effective depth ratio and local bond stress were carried out and found, in all cases, that the beams are adequate.

4.5. DESIGN OF REINFORCED CONCRETE COLUMNS.

In section 4.2.2. cross-sectional dimensions of 300 by 250mm and 400 by 250mm for internal and external columns, respectively, are found adequate. Column loads are found in section 4.2.2.1., effective length in section 4.2.4.3. and bending moment in section 4.3.

All the columns are slender columns and are biaxially bent. Section 3.5.73 of CP 110 is used in the design. This gives:

$$M_{tx}=M_{ix}+(Nh/1750)(l_{ex}/h)^2(1-0.0035l_{ex}/h) \text{ --------(4.11.)}$$

$$M_{ty}=M_{iy}+(Nb/1750)(l_{ey}/b)^2(1-0.0035l_{ey}/b) \text{ --------(4.12.)}$$

Where M_t is total moment while M_i is initial moment calculated in the analysis.

4.5.1. DESIGN DATA.

High yield, plain bars ----- fy = 410 N/mm^2

Concrete grade 30

Concrete cover = 40mm

Design charts for rectangular columns

4.5.2. DESIGN OF COLUMN A (FRAME 1).

COLUMN . LENGTH A_5-A_4	
Design load, N = 922.79 KN X-axis maximum moment., M_{ix}=121.14KNM Y-axis maximum moment., M_{iy}=5.30 KNM	

Effective lengthes, l_{ex}=5.61 M l_{ey}=4.80 M	
Total Moments M_{tx}=121.14+(922.79xx0.4/1750) $(5.61/0.4)^2(1-0.0035x5.62/0.4)$ =121.14+(0.21)(196.70)(0.95) =160.42 KNM M_{ty}=5.30+(922.79x0.25/1750) $(4.8/0.25)^2(1-0.0035x4.8/0.25)$ = 5.30+(0.13)(368.64)(0.29) =19.21 KNM	
For a first estimate, let us sum the two moments and treat as a bending moment applied about the x-axis. $N/bh = 922.79x10^3/250x400$ =9.23 $M/bh^2=(160.42+19.21)/250x400^2$ =4.49 $d = h-(d_c-dia/2) = 354$ $\therefore$ d/h = 354/400 = 0.90	
Referring to design chart. $100A_s/bh = 2.80$ $A_s=2.8x250x400/100$	

CALCULATIONS	OUTPUT
$=2800mm^2$ Selection: Provide 6Y25 bars ($2950mm^2$) Actual $100A_s/bh = 2.95$	6Y25
Checking the maximum moment capacity for each axis. Assuming full axial load is applied. $N/bh = 9.23$ $100A_s/bh = 2.95$ From design chart $M/bh^2 = 4.61$ Ultimate moments: $M_{UX}= 4.61x250x400^2=184.4$ KNM $M_{UY}= 4.61x400x250 =115.25$ KNM For each axis the applied moment is less than the ultimate moment.	6Y25 bars are adequate
COLUMN LENGTH A_4 - A_3	
N = 642.51 KN M_{ix} =148.11 KNM M_{iy} = 6.47 KNM L_{ex} = 5.03 M L_{ey} = 3.65 M	
TOTAL MOMENTS: $M_{tx}=148.11+(642.51x0.4/1750)(5.03/0.4)^2$ $x(1-0.0035x5.03/0.4)$ = 170.06 KNM $M_{ty}=6.47+(64.51x0.25/1750)$	

CALCULATIONS	OUTPUT
$x(3.65/0.25)^2(1-0.0035x3.65/0.25)$ $=25.08$ KNM	
Let the sum of the two moments be applied to the x-axis: $M/bh^2=(149.36+25.08)x10^6/250x400^2$ $=4.36$ $N/bh =642.51x10^3/250x400$ $=6.43$ $d/h =0.90$	
Referring to design chart. $100A_s/bh = 2.32$ $A_s=2.32x250x400/100$ $=2320mm^2$ Selection: Provide 6Y25 bars ($2950mm^2$) Actual $100A_s/bh = 2.95$	6Y25 bars
Checking the capacity of each axis. $N/bh = 6.45$ $100A_s/bh = 2.95$ From chart: $M/bh^2 = 5.22$ Ultimate moment of each axis: $M_{UX}=5.22x250x400^2$ $=208.80$ KNM $M_{UY}=5.22x400x250^2$ $=130.50$ KNM	

CALCULATIONS	OUTPUT
For each axis the applied bending moment is lower than the ultimate moment.	The Section with 6Y25 bars is adequate.
COLUMN LENGTH A_3 - A_2	
N = 018.83 KN M_{ix}=0 M_{iy}=9.01 KNM L_{ex}=4.20 M L_{ey}= 3.65 M	
TOTAL MOMENTS. M_{tx}=0 M_{ty}=9.01+(108.83x0.25/1750) x(3.65/0.25)^2(1-0.0035x3.65/0.25) =12.15 KNM	
For Y-axis bending. N/bh =1.09 M/hb^2 =0.49 d/b =0.80	
Referring to design chart. $100A_s/bh$ = 0 Provide nominal reinforcement. $100A_s/bh$ = 0 Provide nominal reinforcement	4Y16 bars

4.5.2. DESIGN OF COLUMN B (FRAME 1)

COLUMN LENGTH B_5- B_4	
Design load, N = 1001.95 KN. X-axis max. moment M_{ix}=47.65 KNM Y-axis max. moment M_{iy}=7.65 KNM Effective lengthes L_{ex}=5.07 M L_{ex}=4.73 M	
Total moments. M_{tx}=47.65+(1001.95x0.30/1750) x(5.07/0.30)^2(1-0.0035x5.07/0.30) =96.46 KNM M_{ty}=7.65+(1001.95x.0.25/1750) x(4.73/0.25)^2(1-0.0035x4.73/0.25) =55.50 KNM	
Let the sum of the two moments be applied to the x-axis. M/bh^2 = 151.96x10^6/250x300^2 =6.75 N/bh =1001.95x10^3/250x300 =13.36 d/h = 0.85	
Referring to design chart. $100A_s/bh$ = 6.50 Assume $100A_s/bh$ = 5.0 A_s=5.0x250x300/100 =3750 Selection:	

COLUMN LENGTH B_5-B_4	
Provide 8Y25 bars ($3900mm^2$) Actual $100A_s/bh = 5.24$	8Y25 bars OUTPUT
Checking the column capacity. $N/bh = 13.36$ $100A_s/bh = 5.24$ From design chart $M/bh^2 = 5.40$ Ultimate moments. $M_{ux} = 5.4x250x300^2$ $=121.50$ KNM $M_{uy}=5.40x300x250^2$ $=101.25$ KNM	The column is adequate
COLUMN LENGTH B_4-B_3	
$N=688.82$ KN $M_{ix}=57.86$ KNM $M_{iy}=9.30$ KNM $L_{ex}=3.82$ M $L_{ey}=3.50$ M	
Total moments: $M_{tx}=M_{ix}+M_{add}$ $=76.15$ KNM $M_{ty}=27.64$ KNM	
Let the sum of the moments be applied to the X-axis. $M/bh^2 = (76.15+27.64)x10^6/250x300^2$	

CALCULATIONS	OUTPUT
=4.61 N/bh =688.82x10^3/250x300 =9.81 d/h =0.85	
Referring to design chart. 100A_s/bh = 3.40 A_s=3.40x250x300/100 =2550mm^2 Selection. Provide 6Y25 bars (2950mm^2) Actual 100A_s/bh = 2.61	6Y25bars
Checking the column capacity N/bh = 9.18 100A_s/bh = 2.61 From design chart M/bh^2 = 4.05 Ultimate moment M_{ux}=4.05x250x300^2 =91.13 KNM M_{uy}=4.05x300x250^2 =75.94 KNM	The Column is adequate
COLUMN LENGTH B_3- B_2	
N = 385.77 KN M_{ix}=73.26 KNM L_{ex}=3.58 M L_{ey}=3.51 M	

CALCULATIONS	OUTPUT
Total moments. M_{tx}=73.26+9.02 = 82.28 KNM M_{ty}= 9.01+10.33= 19.34 KNM	
Let the sum of the two moments be applied to the X-axis. M/bh^2 = $101.62x10^6/250x300^2$ =4.52 N/bd =$385.77x10^3/250x300$ =5.14 d/h =0.85	
Referring to design chart. $100A_s/bh$ =3.60 A_s=3.60x250x300/100 =2700mm^2 Selection: Provide 6Y25 bars (2950mm^2) Actual $100A_s/bh$ = 3.93	6Y25 bars
Checking the column capacity N/bh = 5.14 $100A_s/bh$ = 3.93 From design chart. M/bh^2 = 5.45 Ultimate moments: M_{UX}=5.45x250x300^2 =122.63 KNM	

CALCULATIONS	OUTPUT
$M_{UY}=5.45x300x250^2$ $=102.12$ KNM	The Column is adequate
COLUMN LENGTH B_2-B_1	
N=82.72 KN M_{ix}= 0 M_{iy}=9.01 KNM L_{ex}=3.43 M L_{ey}=3.51 M	
Total moments. M_{tx}= 0 M_{ty}=9.01+2.21=11.22 KNM	
For Y-axis bending N/bh = 1.10 M/bh^2= 0.60 d/b = 0.82	
Referring to design chart $100A_s/bh$ = 0.20 This is less that one per cent. Therefore, provide nominal reinforcement.	4Y16 bars

For the rest of the columns the design procedure is the same as for the above two (2) columns. Therefore only the result of calculations is shown in the following table.

TABLE 4.21. Final results of columns design.

COLUMN	LENGTH	TOTAL MOMENT		$A_s(mm^2)$	SELECTION
		M_{tx} (KNM)	M_{ty} (KNM)		
A	A_5-A_4	160.42	19.12	2800	6Y25
	A_4-A_3	170.06	25.08	2320	6Y25
	A_3-A_2	212.53	20.15	3220	6Y25
	A_2-A_1	0	12.15	0	4Y16
B	B_5-B_4	96.46	55.50	3750	8Y25
	B_4-B_3	76.15	27.64	2550	6Y25
	B_3-B_2	82.28	19.34	2700	6Y25
	B_2-B_1	0	11.22	150	4Y16
C	C_5-C_4	83.78	41.69	2873	6Y25
	C_4-C_3	86.15	31.51	2753	4Y25
	C_3-C_2	80.80	25.82	2250	4Y25
	C_2-C_1	14.59	20.98	0	4Y16
D	D_5-D_4	34.62	32.86	975	4Y16
	D_4-D_3	36.81	44.43	1350	4Y16
	D_3-D_2	33.22	40.53	1279	4Y16
	D_2-D_1	29.50	35.81	1500	4Y16
E	E_5-E_4	25.57	43.42	1088	4Y16
	E_4-E_3	21.80	41.65	1088	4Y16
	E_3-E_2	9.27	37.79	750	4Y16
	E_2-E_1	8.12	35.28	833	4Y16
F	F_5-F_4	185.81	61.01	4600	6Y25 2Y16
	F_4-F_3	206.59	24.39	3300	6Y25
	F_3-F_2	120.09	16.80	1500	4Y25
	F_2-F_1	0	8.96	0	4Y16

TABLE 4.21 (Cont'd)

COLUMN	LENGTH	TOTAL MOMENT		$A_s(MM^2)$	SELECTION
		M_{tx} (KNM)	M_{ty}(KNM)		
G	G_5-G_4	53.90	23.78	2663	6Y25
	G_4-G_3	64.23	29.10	2250	4Y25
	G_3-G_2	64.02	19.98	1478	4Y25
	G_2-G_1	0	11.20	0	4Y16
H	H_5-H_4	125.76	49.0	2720	4Y25
	H_4-H_3	127.90	23.91	1730	4Y25
	H_3-H_2	164.47	16.52	2400	4Y25
	H_2-H_1	0	9.07	0	4Y16
J	J_5-J_4	26.19	44.23	1050	4Y16
	J_4-J_3	21.95	42.07	990	4Y16
	J_3-J_2	18.30	38.02	1073	4Y16
	J_2-J_1	15.89	34.32	1125	4Y16
K	K_5-K_4	52.02	56.02	2250	6Y16
	K_4-K_3	44.45	46.73	1650	4Y16
	K_3-K_2	32.72	40.56	1350	4Y16
	K_2-K_1	28.66	35.75	1500	4Y16
L	L_5-L_4	87.92	26.08	1000	6Y16
	L_4-L_3	93.44	30.51	1400	6Y16
	L_3-L_2	81.89	23.12	1320	6Y16
	L_2-L_1	76.17	20.90	1400	6Y16
M	M_5-M_4	201.55	32.97	3850	6Y25

In every case the ultimate moment of the columns is greater than the applied moment.

4.6. DESIGN OF FOUNDATION FOR REINFORCED CONCRETE

Independant column footings are designed for each column. Where footings are found too close combined foundation is designed. It is also assumed that footings resist the moments at column bases. Column moments at foundation level is assumed as half of that at upper ends, for columns in the ground floor[11].

4.6.1. DESIGN DATA

Depth of bearing stratum -- 1.20 M.

Soil capacity ----- 175 KN/M^2

Concrete grade 30

High yield plain bars ----- f_y=410 N/mm^2

Size of starter bar = 25mm dia

Cross-section of columns:

Internal = 250x300mm

External = 250x400mm

4.6.2. FOUNDATION DESIGN.

4.6.2.1. DESIGN OF FOOTING FOR COLUMN A (FRAME 1)

CALCULATIONS			OUTPUT
LOADS.			
	Dead load	Live load	
Column A_1-A_2	69.17KN	7.50KN	
Column A_2-A_3	138.06KN	45.97KN	
Column A_3-A_4	138.06KN	45.97KN	
Column A_4-A_5	147.66KN	45.97KN	
Total	492.95KN	145.41KN	

CALCULATIONS	OUTPUT
Total service load, N_a=638.36 KN Ultimate load, N = 922.79 KN Column base moment. M = 121.14/12 = 60.57 KNM Load eccentricity e = M/N = 0.066 M. BASE Area, A= $n_a/(p_g-20H_f)$ =638.36/(175-20x1.20) =4.228 M^2 Bredth, B=$(4.228/1.5)^{\frac{1}{2}}$ =1.68 M Length, L=1.5x1.68 =2.52M	e=0.066M M=60.57KNM N_a=638.36KN N=922.79KN B=1.70M L=2.50M
UPWARD SOIL PRESSURE, P_u. L=2.50 M e=0.066M eccentricity is within the middle third. $P_{u1}=N/LB+6M/LB^2$ =922.79/(2.5x1.7)+6x60.57/(2.5x1.7^2) =267.43 KN/M^2 $P_{u2}=N/LB - 6M/LB^2$ =217.127 - 46.57 =170.55 KN/M^2	P_{u1}=267.43 P_{u2}=170.55

	CALCULATIONS	OUTPUT
	FOUNDATION THICKNESS.	
	a. $h_f \geq 0.25(B-b) = 0.25(1.70-0.25)$	
	$= 0.363M$	
	b. $0.3(b-6) < h_f < 0.5(B-b)$	
	$0.435 < h_f < 0.725$	
	c. $h_f \geq x(B-b)$	
Table CP 110	where x is o.37 for $P_g = 175\ KN/M^2$	
	$h_f \geq 0.37(1.7-0.25)$	
	$\geq 0.537\ M$	
	d. $h_f \geq k.dia.+(70\ to\ 100mm)$	
	K is 28x25 for $f_y = 410N/mm^2$ and	
From Table	$f_{cu} = 30N/mm^2$	
	$h_f \geq 28x25+(70\ to\ 100mm)$	
	$\geq 0.77M$ to $0.80M$	$h_f = 0.60M$
For economic design let $h_f = 0.60M$		$d = 0.53M$
So, $d = h_f - 0.06 - 0.01 = 0.53M$		
PUNCHING SHEAR		
	Critical perimeter,	
	$C_p = 2(h+b)+9.426h_f$	
	$= 2(0.4+0.25)+9.426x0.6$	
	$= 6.956M$	
	Punching shear force;	
	$F = (P_{u1}+P_{u2}/2([BL-(3(h+b)h_f+bh$	
	$+7.07h_f^2)]$	
	$= 218.99[4.25-(1.17+0.10+2.545)]$	
	$= 95.26\ KN$	

CALCULATIONS	OUTPUT
Punching shear stress; $V = F/C_p xd$ $= 95.26x10^3/6956x530$ $= 0.026\ N/mm^2$ This value is not excessive	
BENDING Average soil pressure at one side of the column; $P_{u3} = (267.43+257.55)/2$ $= 262.47\ KN/M^2$ Moment at column face, $M = 262.49x1.05^2x1.70/2$ $= 245.99\ KNM$ $M/bd^2F_{cu} = 0.029$ From lever arm curve, $l_a = 0.95$ Lever arm, $Z = d \cdot l_a = 503.5mm$ Longitudinal reinforcement, $A_s = 245.99x10^6/0.87x410x404.2$ $= 1706mm^2$ $min.A_s = 0.15bd/100 = 645mm^2$ Selection Provide 11Y20 bars at 200mm(1728) Transverse reinforcement, $A_s = 0.12bh_f/100$ $= 720mm^2$	

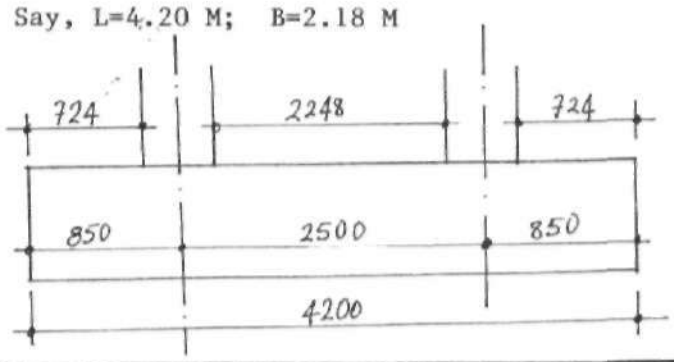
Table 5
CP 110

CALCULATIONS	OUTPUT
Selection, Provide 17Y12 bars at 150mm(754)	
SHEAR STRESS Shear force, $F=B[0.5(B-h)-1.5d]P_{u3}$ $=1.70(1.05-1.5x0.53)262.49$ $=113.79$ KN Shear stress, $V=F/bd$ $=113.79x10^3/1000x530$ $=0.22N/mm^2$ Shear ultimate value $100A_s/bd = 0.33$ $V_C=0.42$ N/mm^2	Section is adequate in shear
LOCAL BOND Shear force, $F=1.7x1.04x247.09$ $=441.06$ KN Total bars perimeter, $U_{ST}=11x3.142x20$ $=691.24mm$ Local bond stress, $f_{bs}=F/U_{ST}.d$ $=441.06x10^3/691.24x530$ $=1.20Nmm^2$ Ultimate local bond stress = 2.2N/mm^2	Section is adequate

TABLE 21

4.6.2.2. DESIGN OF FOOTING FOR COLUMN B. (FRAME 1)

This consists of two internal columns. It is found that independant footing for each is unsuitable because they overlap. Therefore, a combined footing is designed.

CALCULATIONS	OUTPUT
LOADS. Total service load, N_a=689.97 KN Ultimate load, N=1001.95 KN MOMENT AT COLUMN BASE. M =23.83 KNM ECCENTRICITY e=0.024M DISTANCE BETWEEN COLUMN CENTRES L_o=2.5M DISTANCE BETWEEN ECCENTRI LOADS L_e=2.58 M	
BASE DIMENSIONS. Length, L=(1.6 to 1.7) L_o = 4 to 4.25 M Bredth, $B=2N_a/(P_{gr} - 20H_f)L$ =2.176 M. Say, L=4.20 M; B=2.18 M  	

CALCULATIONS	OUTPUT
UPWARD GROUND PRESSURE. P_u=2N/BL =2x1001.95/4.2x2.18 =218.86 KN/M^2	
FOUNDATION THICKNESS. a. $0.3(B-b)<h_f<0.5(B-b)$ $0.579<h_f<0.965$ b. $h_f \geq x(B-b)$ where x=0.37 for P_{gr}=175KN/M^2 For economic design, let h_f=0.70M $\therefore$ d_f=0.630M	h_f=0.70 d=0.630
PUNCHING SHEAR. Critical perimeter, $C_p=2(h+b)+9.426h_f$ =2(0.3+0.25)+9.426x0.70 =7.698 M. Punching shear, $F=P_u[BL-(3(h+b)h_f+bh+7.07h_f^2)]$ =218.86[4.25-3.61] =139.71 KN Punching shear stress, $V=F/C_p.d$ =0.029 N/mm^2 This value is not excessive.	Foundation depth is adequate

Table 5

CALCULATIONS	OUTPUT
BENDING MOMENTS. $M_1=BC^2P_u/2$ $=2.18x0.724^2x218.86/2$ $=125.05$ KNM Span Moment, $M_2=-1001.95x1.25+2.18x2.1^2x218.86/12$ $=-1252.44+1052.04$ $=-200.40$ KNM Transverse moment, $M_3=218.86x1x(1.09-0.125)^2/2$ $=117.64$ KNM	
REINFORCEMENT Longitudinal, $M_1/0.4bd^2f_{cu}=125.05x10^6/0.4x30/10^3x430^2$ $=0.056$ From lever arm, $l_a=0.95$ Lever arm, $Z=0.95x630=599mm$ Area of steel, $A_s=M/0.87f_yZ$ $=585mm^2$ min. $A_s=0.15bd/100=945mm^2$ Selection, Provide Y20 bars at 300mm($1050mm^2$)	

CALCULATIONS	OUTPUT
Transverse, $M/0.4bd^2f_{cu}=0.053$ $l_a=0.95$ Z=580mm Area of Steel, $A_s=569mm^2$ $min.A_s=0.12bh/100=600mm^2$ Selection, Provide Y16 bars at 300mm ——— Top Reinforcement, $M/f_{cu}bd^2 = 0.017$ $l_a=0.95$ Z =599mm Area of steel $A_s=M/0.87f_yZ.$ $=937mm^2$ $min.A_s=945mm^2$ Selection, Provide Y20 bars at 300mm ——— Top transverse reinforcement, Provide nominally Y12 bars at 300	

TABLE 4.22 Final results of foundation design.

FOUNDATION TO COLUMN	SIZE (M)			REINFORCEMENT	
	L	B	h_f	LOCATION	SELECTION
A	2.50	1.70	0.60	Main.	11Y20 at 300
				Transv.	17Y12 at 150
B Combined	4.20	2.18	0.70	Main	8Y20 at 300
				Transv.	14Y16 at 300
				Top	8Y20 at 300
C	2.2	1.40	0.60	Main.	6Y16 at 250
				Transv.	11Y16 at 250
D	2.0	1.20	0.60	Main.	5Y16 at 250
				Transv.	8Y16 at 250
E	1.40	0.92	0.60	Main.	4Y16 at 250
				Transv.	6Y16 at 250
F	2.60	1.60	0.60	Main.	7Y20 at 250
				Transv.	11Y16 at 250
G	2.70	1.80	0.60	Main.	9Y20 at 200
				Transv.	11Y16 at 250
H	2.60	1.60	0.60	Main.	7Y20 at 250
				Transv.	11Y16 at 250
J	1.40	1.0	0.60	Main.	4Y16 at 250
				Transv.	6Y16 at 250
K	2.00	1.40	0.60	Main.	6Y16 at 250
				Transv.	8Y16 at 250
L	1.80	1.20	0.60	Main.	5Y16 at 250
				Transv.	8Y16 at 250
M	1.70	1.10	0.60	Main.	5Y16 at 250
				Transv.	7Y16 at 250
				TOP.	5Y12 at 250

IN ALL CASES THE SECTIONS ARE ADEQUATE IN PUNCHING SHEAR, NORMAL SHEAR AND LOCAL BOND STRESSES.

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CALCULATIONS	OUTPUT
LOCAL BOND. Shear force, $F=BCP_u$. $=2.18 \times 0.724 \times 218.86$ $=345.43$ KN Total bars perimeter, $U_{ST}=3.142 \times 20 \times 8$ $=503$mm Local bond stress, $f_{bs}=F/d \cdot U_{ST}$ $=345.43 \times x10^3/630 \times 503$ $=1.09$ N/mm^2 Ultimate value is 2.2 N/mm^2	Section is adequate in local bond

4.7. DESIGN OF STAIRS AND RAMP

The stairs and ramp design in this section is meant for both the composite structure and the reinforced concrete structure. They have not been previously designed in chapter three.

4.7.1. STAIRS.

The type of stair designed below is for a public building. The stair is a half turn type. Stringer beams are provided at each end of flight, and they frame into columns. The landings at half turn cantilever from the stringer beams.

4.7.1.1. DESIGN DATA.

Flight travel = 4.30M

Flight lift = 2.0M (in the ground floor)

= 2.913M (in the other floors)

Riser height = 159mm

Step going = 342mm

Stair Slope = 25 degree

Waist thickness, assume 140mm

Live load = 3.43 KN/M^2

4.7.1.2. DESIGN OF STAIRS

SLOPE LENGTH OF STAIRS.

$L_S=(4.30^2+1.913^2)^{\frac{1}{2}}$

=4.706 M

WEIGHT OF WAIST AND STEPS.

$G_K=(0.14x4.706+0.159x0.342/2)x24$

=17.12 KN

LIVE LOAD ON STAIRS

$Q_K=3.43x4.30$

=14.75 KN

DESIGN LOAD

F=1.4x17.12+1.6x14.75

=47.57 KN

MOMENT (considering partial end restraint)

M=47.57x4.30/8

=25.57 KNM

CALCULATIONS	OUTPUT
BENDING $M/bd^2f_{cu}=0.068$ From lever arm curve, $l_a=0.907$ Lever arm, $Z=d\cdot l_a=101.58mm$ LONGTIDINAL BARS $A_s=M/0.87f_y\cdot A$ $=706mm^2$ per M $min.A_s=0.15bd/100=168mm^2/M$ Provide 11Y12 bars at 150mm($754mm^2$) TRANSVERSE BARS. $A_s=0.12bh_f/100$ $=168mm^2$ per M Provide 15Y8 bars at 300 ($168mm^2$) CONTINUITY BARS. This is provided at the top and bottom of each span. This should be 50 per cent of the mid span bars. $A_s=0.5x706 = 353mm^2$ Provide 6Y12 bars at $300mm^2$	
LANDING	
WEIGHT. $G_K=1.50x3.23x0.15x24$ $=17.44$ KN $Q_K=1.50x3.23x3.43$ $=16.62$ KN $F=1.4x17.44+1.6x16.62$ $=51.01$ KN	

CALCULATIONS	OUTPUT
MOMENT $M=51.01x1.5^2/2$ $=57.38$ KNM BENDING $M/bd^2f_{cu}=0.145$ From lever arm curve, $l_a=0.760$ Lever arm, $Z=d \cdot l_a=87.40$mm REINFORCEMENT $A_s=M/0.87f_y.Z$ $=1841$ mm^2 Provide 19Y20 bars at 175 mm(1800) . DISTRIBUTION BARS. Provide nominal, i.e. Y10 at 450mm	
SHEAR. Shear stress, $V=F/bd$ $=51.01x10^3/1000x112$ $=0.44$ N/mm^2 Ultimate shear stress in beam $= 0.82$N/mm^2 Slab thickness factro = 1.30 Ultimate shear stress in slab $V_c=1.30x0.82=1.07$ N/mm^2	Land is adequate in shear.

CALCULATIONS	OUTPUT
STRINGER BEAM	
Half stairs from each of top and bottom = 47.57 KN. Landing = 51.01 KN Self weight=3.23x0.23x0.46x24x1.6 =10.67 KN Total = 109.25 KN	
ENDS Moment, M = 109.25x3.23/12 =29.59 KNM Bending, $M/bd^2f_{cu}=0.006$ $l_a=0.95$ Z=404mm Reinforcement $A_s=M/0.87f_y.Z=205mm^2$ Provide 2Y12 bars ($226mm^2$) SPAN Moment, M=109.25x3.23/24 =19.80 KNM Provide 2Y10 bars	
BEAM SHEAR Loading = 109.25/2 = 54.63 KN	

	CALCULATIONS	OUTPUT
TABLE 6	Shear stress, $V=54.63x10^3/230x425$ $=0.55\ N/mm^2$ Ultimate value, $V_c=4.10\ N/mm^2$	Stringer beam is adequate shear
TABLE 21	LOCAL BOND Shear force, F=54.63 Total bars perimeter $U_{ST}=3.142x2x10$ $=62.84$ mm Local bond stress, $F_{bs}=F/d \cdot U_{ST}$ $=2.05\ N/mm^2$ Ultimate value = $2.2\ N/mm^2$	The section is adequate in local bond

4.7.2. RAMP

This is provided at the front ends of the parallel blocks of the building. The ramp is designed to span between floor levels. Landing is also provided at both ends of stringer beams at both ends, while the stringer beams frame into columns.

4.7.2.1. DESIGN DATA

Flight travel = 12.5M

Flight lift = 4.0M (in the ground floor)

=3.825 (in the other floors).

Slab slope =250

Slab thickness=200mm

Slab width =1.50M

Live load =3.43 KN/M^2

Ramp slope =17°.

4.7.2.2. DESIGN OF RAMP.

CALCULATIONS	OUTPUT
SLOPE LENGTH OF RAMP $L_s=(12.5^2+4^2)^{\frac{1}{2}}$ = 13.124M WEIGHT OF RAMP G_k= (0.20x1.0x13.124)x24 =63.00 KN LIVE LOAD ON RAMP Q_K= 13.125x1.0x3.43 =45.02 DESIGN LOAD F = 1.4x63.00+1.6x45.02 =160,22 KN MOMENT (considering partial end restraint) M=160.22x12.5/8 =250.34 KNM	

CALCULATIONS	OUTPUT
BENDING. $M/bd^2/f_{cu}=250x34x10^6/1000x165^2x30$ $=0.31$ Provide compression reinforcement COMPRESSION BARS. $A_{sc}=(M-0.15f_{cu}.bd^2)/0.72f_y(d-d_c)$ $=3331mm^2$ Provide Y25 bars at 150mm(3270) TENSION BARS. $A_s=(0.2f_{cu}bd+0.72f_yA_{sc})/0.87f_y$ $=5482\ mm^2$ Provide Y 32 bars at 150mm(5360) TRANSVERSE BARS $A_s=0.12bh_f/100$ $=300mm^2$ Provide Y 10 bars at 250mm(314) CONTINUITY BARS $A_s=0.55360$ $=2680mm^2$ Provide Y16 bars at 75mm	
LANDING	
Provide same bars as for stairs landing. I.e., Main = 19Y20 bars at 175mm Distr. bars = Y10 at 450mm	

CALCULATIONS	OUTPUT
STRINGER BEAM	
LOADING. Half ramp flight from each of top and bottom flights. =160.22 KN Landing =51.01 KN Self weight =3.23x0.2.23x0.46x24x1.6 =10.67KN Total =221.90 KN ENDS Moment = 221.90x3.23/12 =59.73 KNM Bending $M/bd^2f_{cu}=0.048$ $l_a=0.93$ Z=395mm $A_s=M/0.87f_y.Z$ =424 mm^2 Provide 2Y16 bars (402mm^2) SPAN Moment = 221.90x3.23/24 29.86 KNM Bending $M/bd^2f_{cu}=0.024$ $l_a=0.95$	

CALCULATION	OUTPUT
Z=0.95x425=404 Reinforcement. $A_s=M/0.87\cdot Z\cdot f_y$ $=207mm^2$ Provide 3Y12 bars ($226mm^2$)	
BEAM SHEAR Loading = 221.90/2 = 110.95 Shear Stress, $V=110.95x10^3/230x425$ $=1.14\ N/mm^2$ Ultimate value = 4.10 N/mm^2	Section is adequate in shear
LOCAL BOND Shear force, Total bars perimeter, $U_{ST}=3.142x3x12$ $=113.11mm$ Local bond stress, $F_{bs}=110.95x10^3/425x113.11$ Ultimate value = 2.2 N/mm^2	

Table 6

Table 21

CHAPTER FIVE

D E S I G N C O M P A R I S O N A N D D I S C U S S I O N S

5.1. DESING COMPARISON

In the project beams, columns and foundation bases are designed in both methods, while slabs, stair cases and ramps are designed in reinforced concrete only, because they can not be designed compositely. Therefore the former group of elements is considered for the comparison.

Comparison between the two methods is made in terms of ease of design, member sizes and in building weight. However, since savings in weight may not necessarily mean reduction in overall cost, for other factors such as material price and labour cost in a particular region may also influence cost, a comparative cost analysis is also carried out. This is in order to make a comparison in term of construction economy.

5.1.1. EASE OF DESIGN

Though both methods are based on the ultimate strength of members, they differ in their consideration for design load. While CP 110 considers different partial factors of safety for different load combinations, the composite method considers a single load factor for the sum of dead and imposed loads. In the design process composite method suggests that individual beam spans in a continous beam are considered one at a time. These are analysed and designed individully without considering whether adjacent spans are loaded or not. On the other hand reinforced concrete method recommends that

the structure be divided into sub-frames. The commonest being the one floor level sub - frame. This includes the continuous beam at one level together with all adjoining columns both above and below[10]. The sub-frame is analysed as a whole, thus, involving lengthy computations. Plastic method is only concerned with strength, so no serviceability checks are necessary. However, serviceability considerations have very significant role in CP 110 method.

It is deduced from the foregoing that design by composite method is simpler and takes smaller space and little time than reinforced concrete method. This ease at design stage is very significant in project planning.

5.1.2. MEMBERS SIZES.

In both methods effort is made to ensure that economic reasons govern the choice of member sizes. Slab which has a thickness of 150 mm forms the flanges of beams. All beams in each of the two design methods have the same cross-sectional dimensions. Table 5.1. shows the beam sizes for a typical beam B_1.

TABLE 5.1. Comparison of rib dimensions for beam B_1.

BEAM B_1	RIB DIMENSIONS (mm)		SLAB THICK-NESS(mm)	TOTAL BEAM DEPTH (mm)	RIB CROSS SECTION (mm^2)	RIB WEIGHT PER M.
	DEPTH	WIDTH				
COMPOSITE	254	102	150	404	3230	2.5KN
R.CONCRETE	310	230	150	460	71300	1.71KN

It is noticeable from the table that the composite method led to smaller beam depth than the reinforced concrete one.

Smaller beam depth associated with composite structure means greater headroom allowable space in the building, thus, leading to reduction in floor height. On the other hand, composite beams are heavier than concrete one. The effect of this is more noticeable when the load of the whole building is considered, in section 5.1.3.

Cross-sectional dimensions of structural columns are fixed during preliminary member sizing. Table 5.2. shows the relative dimensions of the columns. It is found that reinforced concrete columns have greater cross-sectional dimensions than the composite ones, while composite columns are heavier. The effect of weight is more important in section 5.1.3.

TABLE 5.2. Comparison of column dimensions and weight.

DESIGN METHOD	INT. COLUMNS		WEIGHT PER M. (KN)	EXT. COLUMNS		WEIGHT PER M. (KN)
	DEPTH (mm)	BREATH (mm)		DEPTH (mm)	BREDTH (mm)	
COMPOSITE	252	252	8.45	300	252	9.38
R.CONCRETE	300	250	1.80	400	252	2.40

As a result of higher dead load in composite beams and columns than those of reinforced concrete, load at foundation level is also higher in the composite structure.

In both sections foundation bases are designed by the method of CP 110. However, results of foundation design in sections 3.5 and 4.6 do notgive a clear picture to which of the two methods leads to savings in materials.

This is because corresponding bases have almost the same volume of concrete and amount of reinforcement, except in one or two case. This ambiguity may result from the differences in consideration of column moments at foundation level. While column moment to be resisted by foundation is calculated during elastic analysis in reinforced concrete method, this is only approximated in the composite structural design. Comparison of results of foundation design is more comprehended when deduced in term of cost savings, presented in section 5.1.4.

5.1.3. BUILDING WEIGHT.

For comparison building weight per floor area is calculated for each of the two designs. Self weight of the structure, only, is considered since live load is the same in both designs.

5.1.3.1. COMPOSITE CONSTRUCTION

In this section dead load for slabs, beams, and columns is calculated.

CALCULATIONS	OUTPUT
SUSPENDED SLAB.	
PLAN AREA:	
Two parallel blocks;	
$=2x(12.5+0.23)x(43.5+0.23)$	
$=1113.37\ M^2$	
the adjoining block;	
$= 28.0x(12.0+0.23)$	

CALCULAITONS	OUTPUT
=342.44M^2 Cantilever slabs; =8.0x1.0x3+4.0+1.0 =28.0M^2 Front projection; =7.0x9.0 =63.0M^2	
Total area = 1546.81M^2 VOLUME OF CONCRETE: =1546.81x0.150 =232.02 M^3	1546.81M^2 232.02M^3
WEIGHT OF SLAB: =232.02x24 =5568.48 KN	 5568.48KN
BEAM RIB. TOTAL BEAM LENGTH: Beam B_1=12.50 M These are eighteen in number; =18x12.50 =225.0M Beam B_2=12.50 M These are four in number; =4x12.5=50.0 M Beam B_3=12.0 M. These are two in number; =2x12.0=24.0M	

CALCULATIONS	OUTPUT
Beam B_4=12.0M These are foun in number =4x12=48.0M Beam B_5=40.0M These are four in number; =4x40.0=160.0M Beam B_6=43.5 M These are four in number; =4x43.5 = 174.0M Beam B_7=18.0 M These are three in number; = 3x18.0 = 54.0M Total = 735.0M	735.0M
WEIGHT PER UNIT LENGTH OF RIB: From table 5.1. = 2.5KN per M.	2.5 KN/M
TOTAL BEAM RIB WEIGHT: =735x2.5 =1837.5 KN	1837.50KN
EXTERNAL COLUMNS. TOTAL COLUMNS' LENGTH: Length per column = 4.0M The columns are forty-two in number; ∴ 42x4.0=168.0M	168.0M
WEIGHT PER UNIT LENGTH: From table 5.2. = 9.38 KN/M	

CALCULATIONS	OUTPUT
TOTAL WEIGHT: =168.0x9.38 =1575.84 KN	1575.84 KN
INTERNAL COLUMNS. TOTAL COLUMNS' LENGTH: Length per column = 4.0M They are forty-two in number; ∴ 42x4.0 =168M	168.0M
WEIGHT PER UNIT LENGTH: From table 5.2. = 8.45 KN/M TOTAL WEIGHT: = 268.8.45 =1419.60 KN	1419.60 KN
TOTAL DEAD LOAD PER FLOOR TOTAL LOAD: =5568.48+1837.50+1575.84 +1419.60 =10401.42 KN TOTAL FLOOR AREA: =1546.81 M^2 LOAD PER FLOOR UNIT AREA =10401.60/1546.81 =6.72KN/M	6.72KN/M^2

5.1.3.2. REINFORCED CONCRETE CONSTRUCTION.

As in the previous section, in this section also the dead load per floor unit area is calculated for the reinforced concrete structure.

CALCULATIONS	OUTPUT
SUSPENDED FLOOR This is the same as for the composite structure. Therefore, the load is equal to 5568.48KN	5568.48KN
BEAM RIB. The same length of beams as for composite structure is obtained. Therefore: TOTAL LENGTH: =735.0M	735.0M
WEIGHT PER UNIT LENGTH OF RIB: From table 5.1. = 1.71 KN TOTAL RIB WEIGHT: =735.0x1.71 =1256.85 KN	1256.85KN
EXTERNAL COLUMNS: TOTAL LENGTH: This is the same as for composite structure. =168.0M	168.0M
WEIGHT PER UNIT LENTH: From table 5.2. = 2.4KN	

CALCULATIONS	OUTPUT
TOTAL WEIGHT: =168.0 x 2.4 =043.2	403.2KN
INTERNAL COLUMNS TOTAL COLUMN LENGTH: This is the same as for the composite structure. =168.0	168.0M
WEIGHT PER UNIT LENGTH: From table 5.2. = 1.8KN	
TOTAL WEIGHT: =168.0x1.8 =302.40 KN	302.40KN
TOTAL DEAD LOAD PER FLOOR TOTAL LOAD: =5568.48+1256.85+403.2+302.40 =7530.93 KN	
TOTAL FLOOR AREA: =1546.81 M^2	
LOAD PER UNIT AREA: =7530.93/1546.81 =4.89KN PER M^2	4.89KN/M^2

From the above two dead load calculations, it can be seen that dead load that is transferred to foundation is greater in the composite structure.

In the reinforced concrete structure the load is only 73 per cent of that of composite structure.

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5.1.4. CONSTRUCTION ECONOMY,

Cost is one of the major factors that influence decision in the choice of one method of construction or another. This section is intended to show cost difference that results in the same structure designed compositely and in reinforced concrete.

5.1.4.1. COST ANALYSIS FOR COMPOSITE STRUCTURE,

Three elements, viz; beam ribs, columns and foundation bases are considered in this cost analysis. Floor slab is not included since it is the same in both cases.

CALCULATIONS	OUTPUT
BEAM RIBS.	
UNIVERSAL BEAM:	
Standard UB254x102x25Kg=9.0M	
Cost per length = ₦780.00	
Cost per M. =780/9 = 86.67	₦86.67
CONNECTIONS:	
Allow 25% for end plate, rivet bolts and shear connectors.	
∴ 0.25x86.67 =21.67	₦21.67
TREATMENT.	
Surface area to be treated per meter = 0.79M^2	

CALCULATIONS	OUTPUT
Red-oxide paint = ₦6.22 Per M^2	
∴ =.79x6.22=4.91	₦4.91
TOTAL COST PER METER:	
=113.25	₦113.25
TOTAL LENGTH OF BEAM RIBS:	
From section 5.1.4.1. total	
length is 735 M.	
TOTAL COST OF BEAM RIBS:	
=735x113.25	
=83238.75	₦83238.75
EXTERNAL COLUMNS.	
UNIVERSAL COLUMN:	
Standard UC152x152x37Kg = 9.0M	
Cost per length = ₦1005.00	
Cost per meter = ₦111.67	₦111.67
CONCRETE:	
Volume per meter is 0.25x0.300x1.0	
= 0.071 M^3	
Cost of plain concrete = 308.28	
Cost of concrete prer meter is	
0.071x308.28 = ₦21.89	₦21.89
REINFORCEMENT:	
Four longitudinal 10mm bars:	
Cost per meter = ₦1.89	
Cost of four bars = 4x1.89	
=7.56	₦7.56

CALCULATIONS	OUTPUT
Three 6mm links of 0.884 length each.	
Cost per meter = ₦0.89	
Cost of links = 3x0.884x0.89	
=₦2.36	₦2.36
TOTAL COST PER METER:	
= 143.48	₦143.48
TOTAL LENGTH OF EXTERNAL COLUMNS	
From section 5.1.4.1. total	
length is 168.0meters	168.0M
TOTAL COST OF COLUMNS:	
=268x143.48	
=24,104.64	₦24,104.64
=24,104.64	₦24,104.64
INTERNAL COLUMNS.	
UNIVERSAL COLUMN:	
As for external column	₦111.67
CONCRETE:	
Volume per meter is 0.252x0.252x1.0	
less Vol of universal column =0.059m^3	
Cost per meter length of column is	
0.059x308.28 = 18.19	₦18.19
REINFORCEMENT:	
Four longitudinal 10mm bars.	
Cost as for external columns	₦7.56

CALCULATIONS	OUTPUT
Three 6mm links of 0.788 meter length each.	
Cost per meter = ₦0.89	
Cost of links = 3x0.788x0.89	
=2.10	₦2.10
TOTAL COST PER METER:	
= 139.52	₦139.52
TOTAL LENGTH OF COLUMNS:	
From section 5.1.4.1. total length is 168.0 meters	168.00M
TOTAL COST OF INTERNAL COLUMNS:	
=168.0x139x52	
=23,439.36	₦23,439.36
FOUNDATIONS CONCRETE: The following volume of concrete is calculated calculated from column footing dimensions given in table 3.7. Column A. footing = 2.55 M^3 Column B. footing = 8.06M^3 Column C. footing = 1.09M^3 Column D. footing = 2.55M^3 Column E. footing = 0.67M^3 Column F. footing = 3.19^3 Column G. footing = 8.06M^3	

CALCULATIONS	OUTPUT
REINFORCEMENT.	
Amount of reinforcement is cal-	
culated from bending schedule:	
20mm bars = 0.51 tons	
Cost per ton = ₦3,080.00	
Total cost = 0.51x3080.00	
= ₦1570.80	₦15.70.80
16mm bars = 0.15 tons	
Cost per ton = ₦3250.00	
Total cost = 0.15x3250.00	
= ₦487.50	₦487.50
12mm bars = 0.075 tons	
Cost per ton = 2750.00	
Total cost = 0.075x2750.00	
= ₦206.25	₦206.25
10mm bars = 0.15 ton	
Cost per ton = ₦3100.00	
Total cost = 0.15x3100.00	
= ₦465.00	₦465.00
Total cost of reinforcement	
=₦2729.55	₦2729.55
FOUNDATION-COLUMN CONNECTIONS	
Allow 10% of the cost of concrete	

CALCULATIONS	OUTPUT
Column H. footing = 2.55M³	
Column J. footing = 0.67M³	
Column K. footing = 2.55M³	
Column L. footing = 1.09M³	
Column M. footing = 2.08M³	
TOTAL VOLUME:	
36 Column A = 36x2.55=91.8M³	
18 Column B = 18x8.06=145.08	
4 Column C = 4x1.09 =4.36	
4 Column D = 4x2.55 =10.20	
4 Column E = 4x0.67 =2.68	
7 Column F = 7x3.19 =22.33	
7 Column G = 7x8.06 =56.42	
7 Column H = 7x2.55 =17.85	
2 Column J = 2x0.67 =1.34	
2 Column K = 2x2.55 =5.10	
2 Column L = 2x1.09 =2.18	
3 Column M = 3x2.08 =6.24	
Volume of concrete in foundation	
footings = 361.42M³	361.42M³
Cost of plain concrete per cubic	
meter = ₦308.28	
Total cost of concrete	
=361.42x308.28	
=111,418.55	₦111,418.55-

CALCULATIONS	OUTPUT
for bottom plate, welding and hold-down bolts.	
∴ 0.10x111,418.55	
= ₦11,141.86	₦11,141.86
TOTAL COST OF FOUNDATION	
= 111,418.55+2729.55+11141.86	
= 125,289.95	₦125,289.95
COST OF FOUNDATION PER FLOOR	
The building consists of four floors (including ground floor).	
Therefore cost per floor is	
= 125,289.95/4	
= 31,322.49	₦31,322.49
BEAM RIB, COLUMNS AND FOUNDATION	
TOTAL COST PER FLOOR:	
=₦162,105.23	₦162,105.23
COST PER UNIT AREA	
From section 5.1.4.1. the total area per floor is 1546.81M^2	
∴ 162,105.23/1546.81	
= ₦104.80	₦104.80

5.1.4.2. COST ANALYSIS FOR REINFORCED CONCRETE STRUCTURE

The same three elements, as in the previous section are considered again in this section for the following cost analysis.

CALCULATIONS	OUTPUT
BEAM RIBS.	
CONCRETE:	
All the beams in the building have the same cross-section. Therefore rib dimensions given in table 5.1. will be used.	
Cross-section = 0.0713 M^2	
From 5.1.4.1. total beam lengths is equal to --- 735.0M	
Volume of concrete is = 0.0713x735 = 52.41 M^3	52.41 M^3
Cost of plain concrete =₦308.28	
Cost of concrete =52.41 M^3 =₦16156.95	₦16,156.95
REINFORCEMENT:	
Amount of reinforcement calculated from bending schedule:	
32mm bars = 0.144 ton	
Cost per ton = ₦3010.00	
Total Cost = 0.144x3010.00 = ₦433.44	₦433.44
25mm bars =0.49 ton	
Cost per ton =₦3050.00	
Total Cost =₦1494.50	₦1494.50

CALCULATIONS	OUTPUT
BEAM RIBS.	
CONCRETE:	
All the beams in the building have the same cross-section. Therefore rib dimensions given in table 5.1. will be used.	
Cross-section = 0.0713 M^2	
From 5.1.4.1. total beam lengths is equal to --- 735.0M	
Volume of concrete is	
= 0.0713x735	52.41 M^3
= 52.41 M^3	
Cost of plain concrete =₦308.28	
Cost of concrete =52.41 M^3	
=₦16156.95	₦16,156.95
REINFORCEMENT:	
Amount of reinforcement calculated from bending schedule:	
32mm bars = 0.144 ton	
Cost per ton = ₦3010.00	
Total Cost = 0.144x3010.00	
= ₦433.44	₦433.44
25mm bars =0.49 ton	
Cost per ton =₦3050.00	
Total Cost =₦1494.50	₦1494.50

CALCULATIONS	OUTPUT
20mm bars = 0.94 ton	
Cost per ton = 3080.00	
Total cost = 2895.20	₦2895.20
16mm bars = 0.21 ton	
Cost per ton = ₦3250.00	
Total Cost = ₦682.50	₦682.50
12mm bars = 0.091 tons	
Cost per ton = 2750.00	
total cost = ₦250.25	₦250.25
10mm bars = 0.065 ton	
Cost per ton = ₦3100.00	
Total Cost = ₦201.50	₦201.50
8mm bars = 0.21 ton	
Cost per ton = ₦2480.00	
Total cost = ₦520.80	₦520.80
Total reinforcement cost = 6478.19	₦6478.19
TOTAL COST OF RIB:	
= 16,156.95+6478.19	
= 22,635.14	₦22,635.14
EXTERNAL COLUMNS.	
CONCRETE:	
From table 5.2. cross-section	
of column is 0.40x0.25=0.10M^2	
From section 5.1.4.1.total length	
of columns = 168.0M	

CALCULATIONS	OUTPUT
Volume of concrete = 0.10x168=16.8M^3	16.8M^3
Price of plain conc.=308.28	
Cost of concrete =6214.93	₦5179.10
REINFORCEMENT:	
Calculated from bending schedule:	
25mm bars = 6.04ton	
Price per ton =3050.00	
Total cost =₦18,422.00	₦18,422.00
16mm bars =0.24 ton	
Price per ton =₦3250.00	
Total cost =₦780.00	₦780.00
Total reinforcement cost = 19,202.00	₦19,202.00
TOTAL COST OF COLUMN:	
= 5179.10+19,202.00	
= 24,381.10	₦24,381.10
INTERNAL COLUMNS.	
CONCRETE:	
From table 5.2. cross-section of column is o.30x0.25=0.075M^2	
From section 5.1.4.1. total length of columns is 168.0M	
Volume of concrete =0.075x168=12.6M^3	12.6M^3
Cost of concrete =3884.33	₦3,884.33
REINFORCEMENT:	
Amount is calculated from bending schedule:	

CALCULATIONS	OUTPUT
25mm bars = 6.73 ton	
Total cost = 20526.50	₦20526.50
16mm bars = 0.37 ton	
Total cost = 1202.50	₦1202.50
Total reinforcement cost = 21,729.00	₦21,729.00
TOTAL COST OF COLUMN:	
=25,613.33	₦25,613.33
FOUNDATIONS.	
CONCRETE:	
The following concrete volume is calculated by referring to table 4.22. and section 5.1.4.1. Foundation for:	
36 column type A = 36x2.55 =91.80	
18 column type B = 18x6.41 =115.38	
4 column type C = 4x1.85 = 7.4	
4 column type D = 4x1.44 = 5.76	
4 column type E = 4x0.77 = 3.09	
7 column type F = 7x2.50 = 17.47	
14 column type G = 14x2.92 = 40.82	
7 column type H = 7x2.50 = 17.47	
2 column type J = 2x0.84 = 1.68	
2 column type K = 2x1.68 = 3.36	
2 column type L = 2x1.30 = 2.59	
3 column type M = 3x1.12 = 3.37	
Total volume of concrete = 310.19	
Cost of concrete = 308.28x310.19	
=95625.37	₦95,625.37

CALCULATIONS	OUTPUT
Reinforcement: Amount of reinforcement is calculated from bending schedule: 20mm bars = 3.61 tons Price per ton= 3080.00 total cost = ₦11,118.80	₦11,118.80
16mm bars = 4.90 tOns Price per ton = ₦3250.00 Total cost = ₦15,925.00	₦15,925.00
TOTAL COST OF FOUNDATION =95,625.37+15,925.00+11,118.80 =122,669.17	₦122669.17
COST OF FOUNDATION PER FLOOR: =122669.17/4 =30667.29	₦30667.29
BEAM RIB, COLUMNS AND FOUNDATION TOTAL COST PER FLOOR: =103296.86	₦103296.86
COST PER UNIT AREA From section 5.1.4.1. the total area per floor is 1546.81M^2 ∴ 103296.86/1546.81 =₦66.78	₦66.78

From the above two cost analysis it is found that the building structure which is designed in reinforced concrete costs only ₦66.78 per square meter of floor. On the other hand the same

building structure when designed in composite it costs ₦104.80 per square meter. The reinforced concrete structure costs only 63.72 per cent of that of composite.

5.2. DISCUSSIONS.

Economy, simpilicity, buildability and strength consideration are the main factors that may govern the choice of one construction method or another. Of these some may be more influencial than others depending on the type of structure and the purpose for which it is intended. For those structures for which the main design criterion is that of strength plastic design could be most suited, for simple plastic theory is concerned with the strength of members. Thus, the method is not suited to bridge composite members, because in bridge structures other criteria, apart from strength are also equally important.

For those problems for which it issuited, plastic theory is far easier than elastic analysis[8]. Further, plastic design method is simple and rational, for the designer can be confident that his calculations for plastic moment of resistance are based on the actual working bending moments.

In composite design, by plastic method no consideration for shear resistance that may be provided by the concrete material is made. This is only made in terms of the steel section alone. Similarly, other considerations such fatigue of the structure are neglected. Certain design criteria in beam and column design may lead to a possible over-design of members which could ultimately result to a relatively too expensive structure.

Applicability of composite design is limited to beams and columns only. Other structural members have to be designed by reinforced concrete method while beam-column connections and column-foundation joints are designed by the method of BS. 449. The designer in composite has to be conversant in the other two methods. Thus, rendering composite design a multi-method design than is ordinarily referred.

It is believed that in steel-concrete composite beam there is increased stiffness due to composite action and continuity of beams.[24] This led to greater savings than in steel beam. Beam stiffness in reinforced concrete beam is dependant on beam cross-sectional dimensions and its span. Improvement of stiffness of a reinforced concrete beam could be made by increasing the magnitude of its span-effective depth ratio.

Failure of composite beam is when ultimate capacity of shear connectors is exceeded, rather than failure of steel or concrete which are the main materials. For this standardised connectors have to be used their strength in lateral shear have to be tested. NOrmally, the load-slip curve of shear connectors is almost the same as stress-strain curve of steel reinforcement. Provision of connections in composite elements has to be given considerable attention for its specific role. Connections at beam-column joints and column-foundation joints are designed in order to ensure that all members are well connected.

In reinforced concrete structures mode of failure is that it begins with yeilding of tension reinforcement followed by crushing of concrete. This is referred to as tension failure. When

crushing of concrete and yeilding of reinforcement occur at the same time, it is referred to as balanced failure.[12] Connections between elements in reinforced concrete structure is not much of a problem. This is because natural bond between concrete and steel comes into play and makes the two to act as a unit.

It is seen in section 5.1. that composite design is ultimately heavier and more costly. This may come about as a result of possible over-design. In the following sub-sections checks are carried with a view to find whether composite members have been over-designed. Economic load and span of beams and economic load of columns were calculated.

5.2.1. ECONOMIC SPAN OF THE BEAMS.

This considers the same beam section with the same design load used in the text. Maximum moment is:

$$M=1.7wl^2/8 \text{ ----- equ. 5.1.}$$

Plastic moment of section is M_p. This is found, in section 3.2.2.1., to be 246.0KNM. At near collapse stage M is equal to M_p. Therefore, equation 5.1. becomes:

$$M_p=1.7wl^2/8 \text{ ---- equ. 5.2.}$$

In a continous beam the ratio of the negative to positive plastic moments of resistance, k, is a minimum of 0.5. Actual plastic moment of resistance from equation 5.2. becomes:

$$(1+k)M_p=1.7wl^2/8\text{---equ. 5.3.}$$

By rearranging equation5.3,

$$l^2 8(1+0.5)M_p/1.7w$$

From section 3.2.2.1. W is 36.9 KN per meter length of beam.

$\therefore 1^2 = 8x1.5x246/1.7x36.9$

$= 87.06$

$1 = 6.86$ M

The limiting value for shear capacity of the same beam section is found to be F_c=272.4 KN. If the beam has a span of 6.86M. the existing shear force is:

$$F=1.7wl/2 + M_n/l \text{------eq. 5.4}$$

The negative bending moment is:

$$M_n=1.7wl^2/12 \text{ -------eq. 5.5}$$

From equation 5.4

$$F=251.02 \text{ KN}$$

The section will still be adequate in shear.

The provided beam span of 5.0 meters is 1.86 meters or 27.11 per cent less than its economic span.

5.2. 2. ECONOMIC LOAD OF THE BEAM.

This section is to find out the collapse load of the beam if 5.0 meter span is maintained. By rearranging equation 5.3

$$W=8(1+0.5)M_p/1.71^2$$

$$=8x1.5x246/1.7x5^2$$

$$=69.46 \text{ KN per M.}$$

If the beam is loaded with this amount of load it is found, by trial calculations using equation 5.4, that the beam will fail in shear. Therefore, the design load that satisfies the requirement of the section is, by rearranging equation 5.4

$$W = 2(F-M_n/1)/1.71\text{-----eq.5.6.}$$

Assuming M_n=208.22 KNM as initially, and that F=F_c

W=2(272.4-108.22/5)/1.7x5

=59.0 KN per meter

With this load the actual M_n from equation5.5 is:

M_n=208.96 KNM

Therefore, actual W is:

W=2(272.4-208.96/5)/1.7x5

=54.26 KN per meter.

The beam design load of 36.9 KN per meter is 17.36 KN (or 32 per cent) less than its economic load.

5.2.3. ECONOMIC LOAD FOR THE COLUMNS.

This is to consider the number of stories that can load the ground floor columns to their maximum capacity. Floor loading remains the same and therefore column end moments remain constant while axial load increases.

Column A. is to be used for the following calculation. The same column cross-section is maintain. From section 2.1.7.

$N/N_u \leq i_p$ --------eq. 5.7

where $i_p = S_1-(S_1-S_2-4S_3)(M/M_p)-4S_3(M/M_p)^2$

=0.709

N_u =2505.19 KN

Rearranging equation 5.7

N=0.709x2505.19

=1776.18 KN.

This column section is provided for a loading which is 763.61 KN or 43 per cent lower than the columns' economic load.

This ineconomy increases directly with the number of floors, because the column maintains a constant cross-section throughout its length.

For obvious reasons the provided column composite section is the smallest possible. This is found to have a capacity which is much more than necessarily required, for a four story building. The four stories in question have a total load, at foundation level, for column A as 1012.86KN. This is approximately 253.22KN per floor. Therefore, the column can only be loaded economically, if the building consists of seven stories.

TABLE 5.3. Comparison of beam and column loads and beam spans.

COMPOSITE ELEMENT	ACTUAL LOAD (KN)	ECONOMIC LOAD (KN)	DIFFE-RENCE (%)	ACTUAL SPAN (M)	ECONOMIC SPAN (M)	DIFFE-RENCE (%)
BEAM	39.90	54.26	32.0	5.0	6.86	27.11
COLUMN A	1012.86	1776.18	43.00	-	-	-

From the foregoing it is found that in all cases the composite elements are more than 27 per cent over-designed. But the amount by which the composite members are over-designed cannot make it more economical than the reinforced concrete ones. It is therefore, reaffirmed that reinforced concrete construction is more economical.

5.3. SUMMARY.

Comparisons and discussions carried out in sections 5.1. and 5.2. are summarised below:

1. At design stage composite construction is easier, less complex and takes shorter time. This helps in project planning.

2. Beam rib in composite beams have smaller dimentions. Shallower beam depths may result in savings in floor height. This savings is more important and increases directly with increasing number of stories. Composite columns have smaller cross-sectional dimentions, but on the other hand are heavier than reinforced concrete ones.

3. Dead load per unit area is heavier in the composite construction. Dead load in the reinforced concrete structure is 73 per cent of that composite structure.

4. Elements designed by composite method cost more per floor unit area than when designed by reinforced concrete method. Reinforced concrete structures cost 63.72 per cent of composite structures.

5. Over design in composite beams, which resulted from effort to satisfy condition for plastic design, led to provision of larger steel section than required.

6. Larger composite column section, than necessary, has been provided in the ground floor. While the section is the smallest possible, its larger section than required has resulted due to effort to satisfy a design criterion. It is therefore concluded that composite construction is ineconomical for a building of three stories, unless the building consists of heavily loaded floors.

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6.1. CONCLUSION.

"Over the past sixty years the very large amount of theoritical, experimental and constructional works carried out on composite structural system have amply proved the efficiency and economy of the method".[2]

Similar statements to the above are what is obtained in text books. Various authorities on composite bridge and building structures have discussed about such advantages gained by application of composite constructional system.

The comperative design study carried out in this thesis has rather come up with an opinion which is slightly contrary to the above statement. Four points have been noted two of which are advantages of composite construction over reinforced concrete construction. The first is its easeness in design. This resulted from the following factors:

1. Compostie design is less complex;
2. Is associated with shorter design time;
3. It involves a single load factor;
4. In a continous beam one span is considered at a time;
5. No serviceability checks are necessary, because strength is the main criteria.

The second advantage of composite construction is its shallower beam depth. The factors that contribute to this include:

1. Adequately stiffened steel girder flanges;
2. Presence of shear connectors, upon which beam failure depends;
3. Rigidity of connections at beam-column joints.

The other two noted points conclude that composite construction is more disadvantageous when compared with reinforced concrete. The first of these is that it is heavier in total building dead load. This called for more costly foundations. Heaviness of composite structure comes about from the following factors:

1. Presence of steel sections in concrete;
2. Weight of steel girders which make up the beam ribs;
3. Provision of bigger section than necessary.

The second disadvantage of composite construction is that it is more costly. Factors that contribute to this are:

1. Relatively higher cost of steel stanchions and girders;
2. Cost of fabricating steel members;
3. Cost of shear connectors;
4. Cost of beam-column and column-foundation jointing materials;
5. Cost of false-ceiling that may be required to camouflage the encased steel girders.

If the two advantages of composite construction are deduced in terms of financial benefit the gains that can be realised cannot off-set its disadvantages. It is therefore concluded that reinforced concrete construction is more advantageous. However, this conclusion is with reference to lightly loaded buildings and to buildings involving not more than few stories.

This thesis is not outrightly disputing the statement present at the opening of this chapter, but is rather shaping or qualifying it. To put it exactly, for a lightly loaded building with a number of stories not more / than three reinforced concrete construction is more advantegeous. Composite construction can only compete with the reinforced concrete one in high rise buildings involving many stories, and in heavily loaded floor buildings. The greater the number of stories the building is made of the more advantageous composite construction becomes.

Another factor that may inhibit the application of composite construction is its longer construction time[25]. This is because fabrication of steel structures in position and casting of concrete have to be done at different times. These usually involve different set of people.

In the final, if the chief consideration in the design of a framed building is that of strength definately composite method is the most suitable.

6.2. RECOMMENDATIONS.

In general terms, the economic benefits that may be achieved in the choice of one construction method over another depends on many factors. among these are the type of the structure and its intended use, location and relative material prices and labour cost in a particular location. But, in our particular case, from the study in the previous chapter the following can be recommended:

1. For a building with lightly imposed loads or with stories not more than three / it should be constructed in reinforced concrete.
2. Composite construction should only be advocated in heavily loaded buildings. The greater the number of stories in the building the more composite construction finds its application.
3. Where economic advantage is foreseable through quicker construction and where delay at design stage is tolerable reinforced concrete method is more recommended.
4. In building structure where strength requirement outweigh economic consideration composite method is most recommended. This will find its application in structures subjected to abnormal external loading.
5. There is a need to carry more research work about beam-column joint in a composite frame. This should be with a view to incorporate the effect of concrete, towards further stiffining of the joint.

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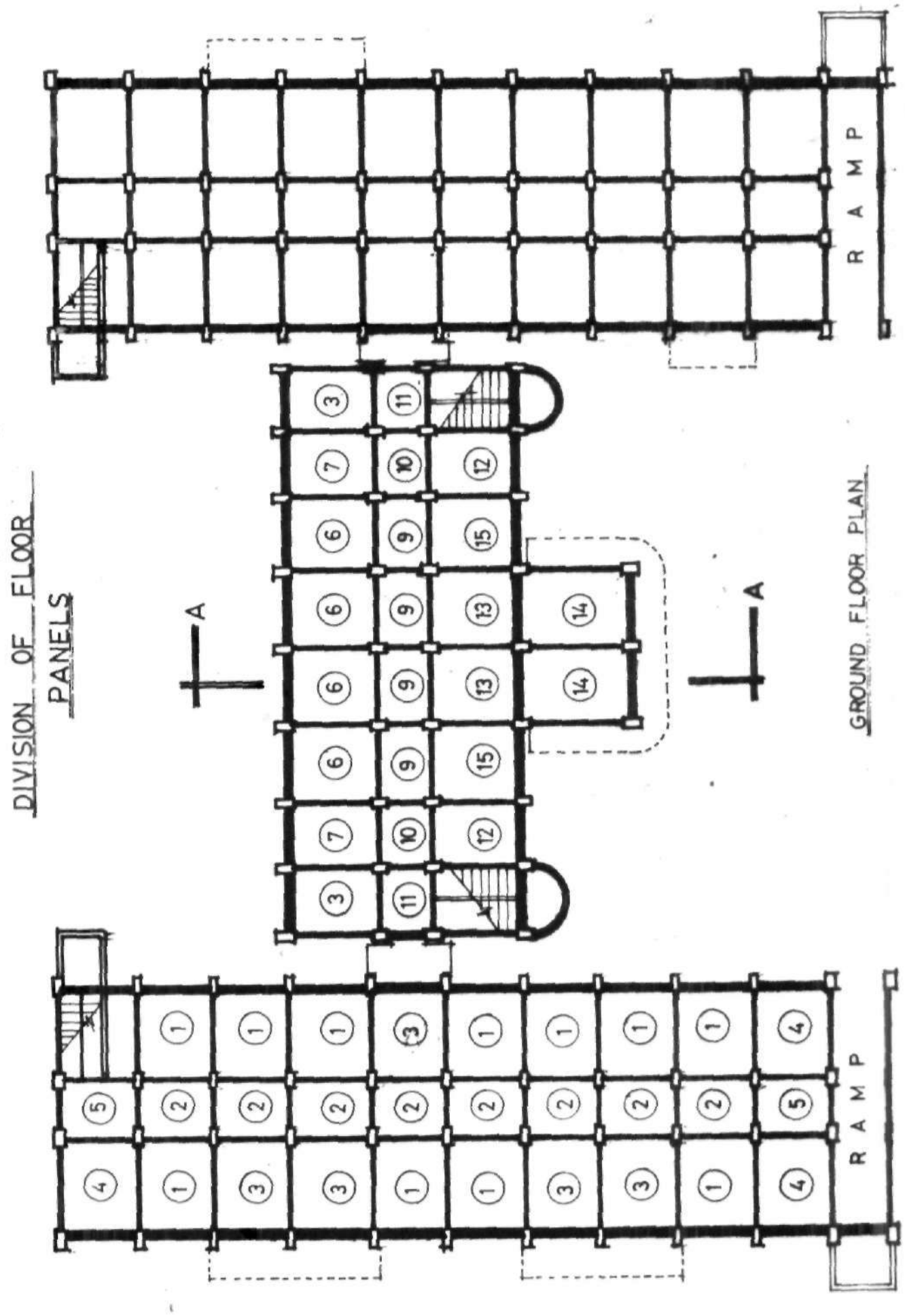
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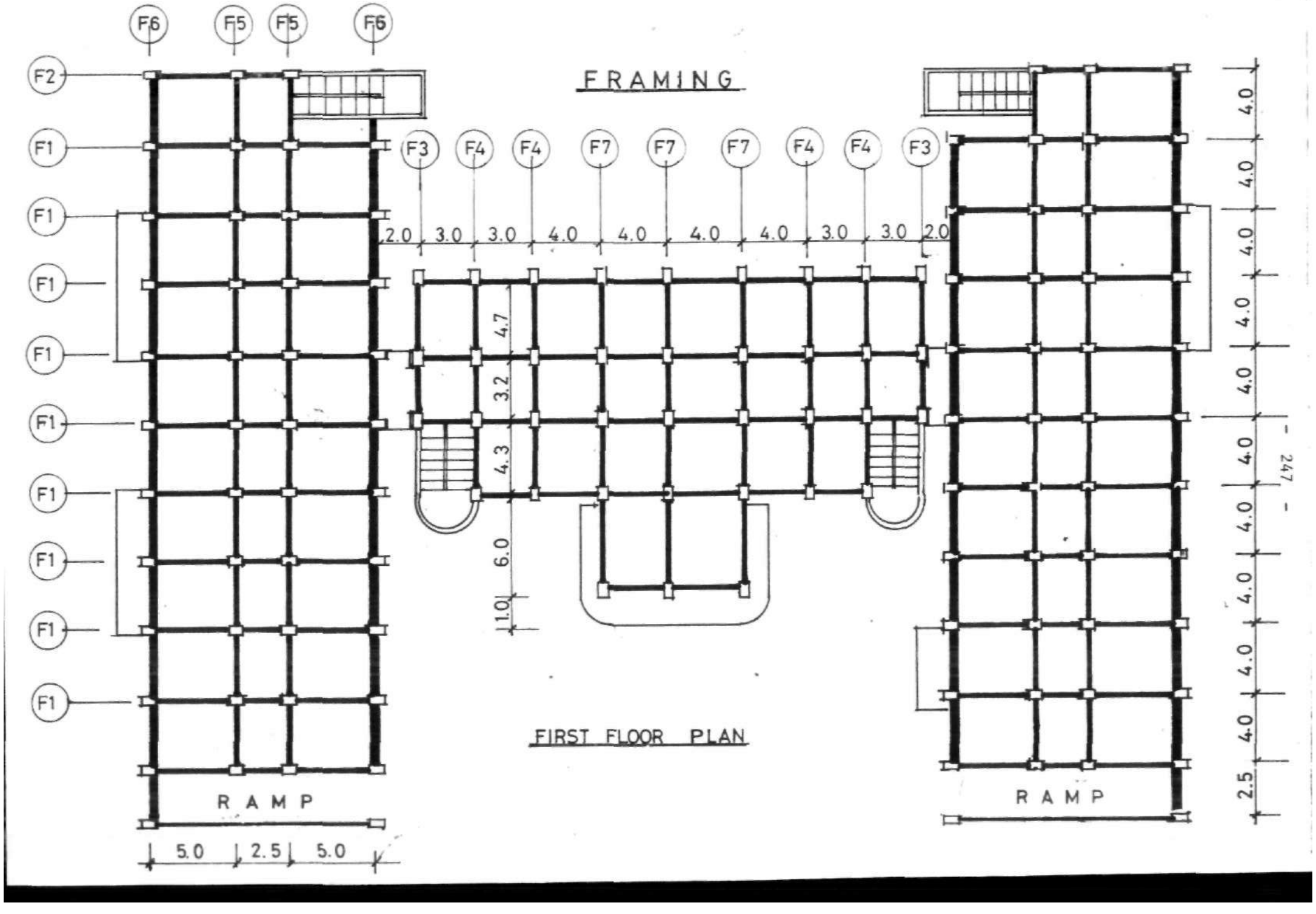
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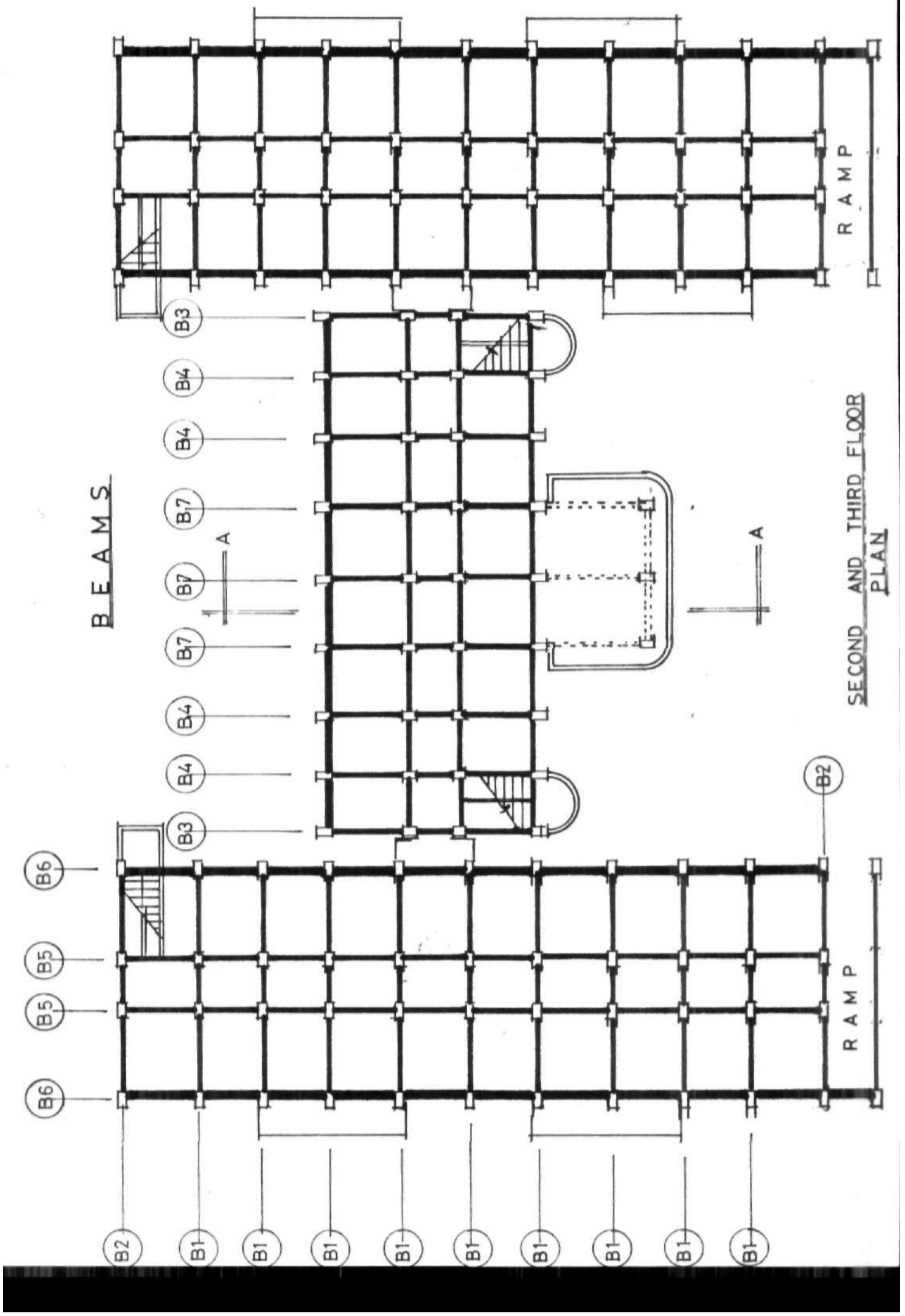
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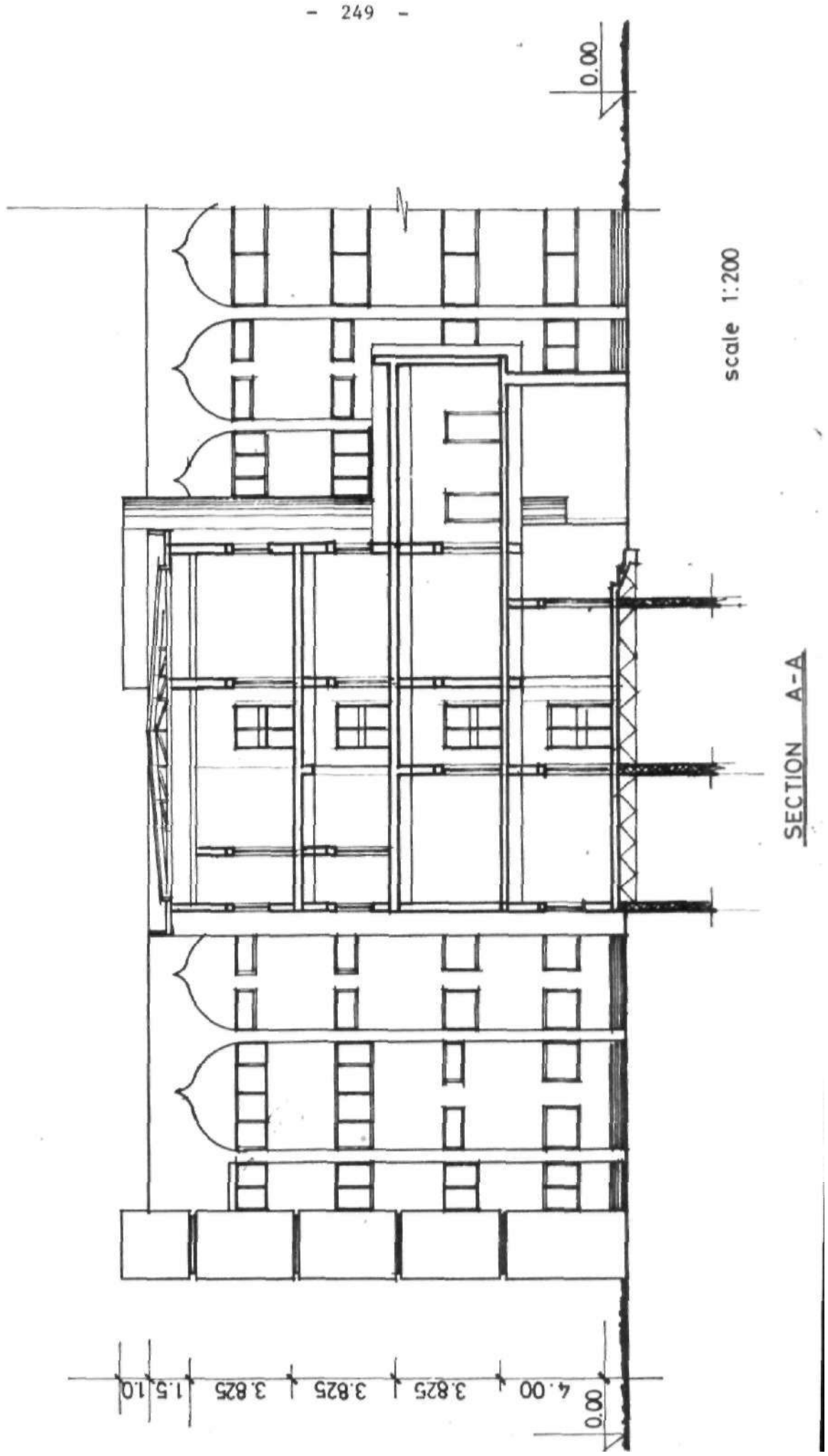


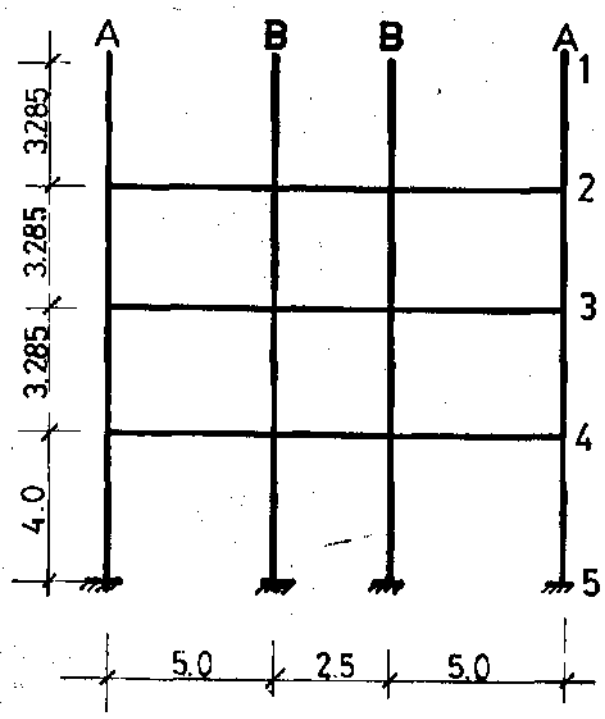
DIVISION OF FLOOR PANELS

GROUND FLOOR PLAN

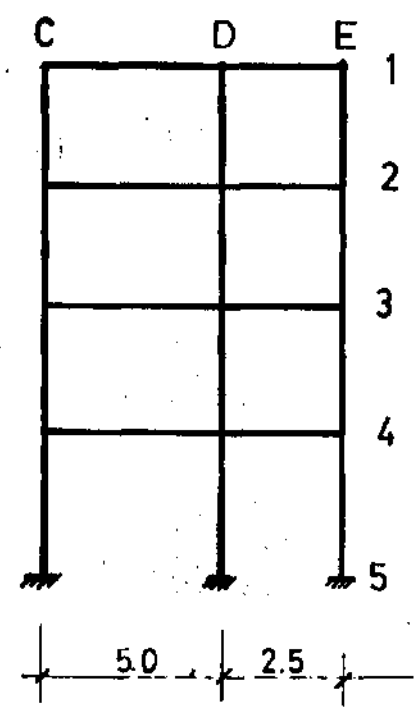

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FIRST FLOOR PLAN


BEAMS
SECOND AND THIRD FLOOR PLAN
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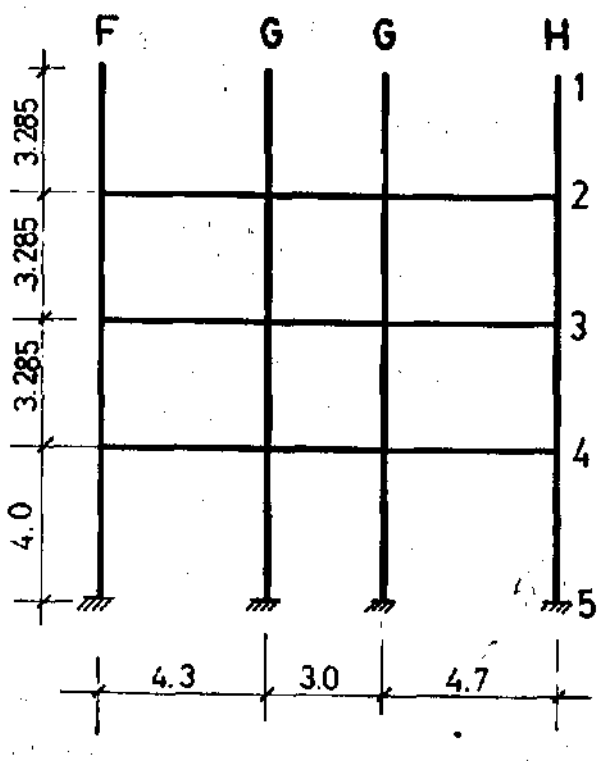

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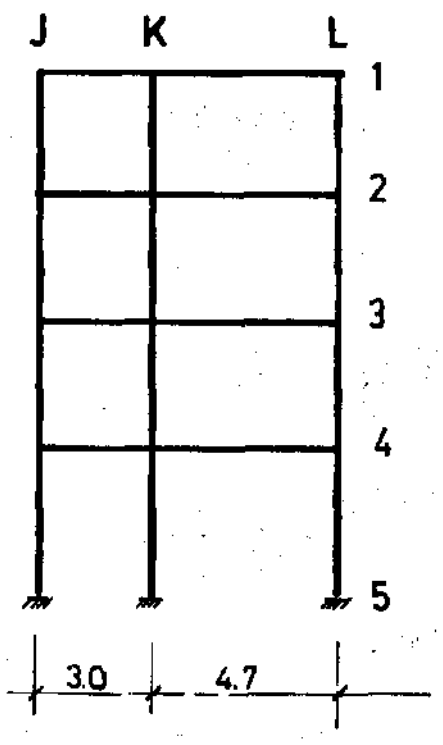
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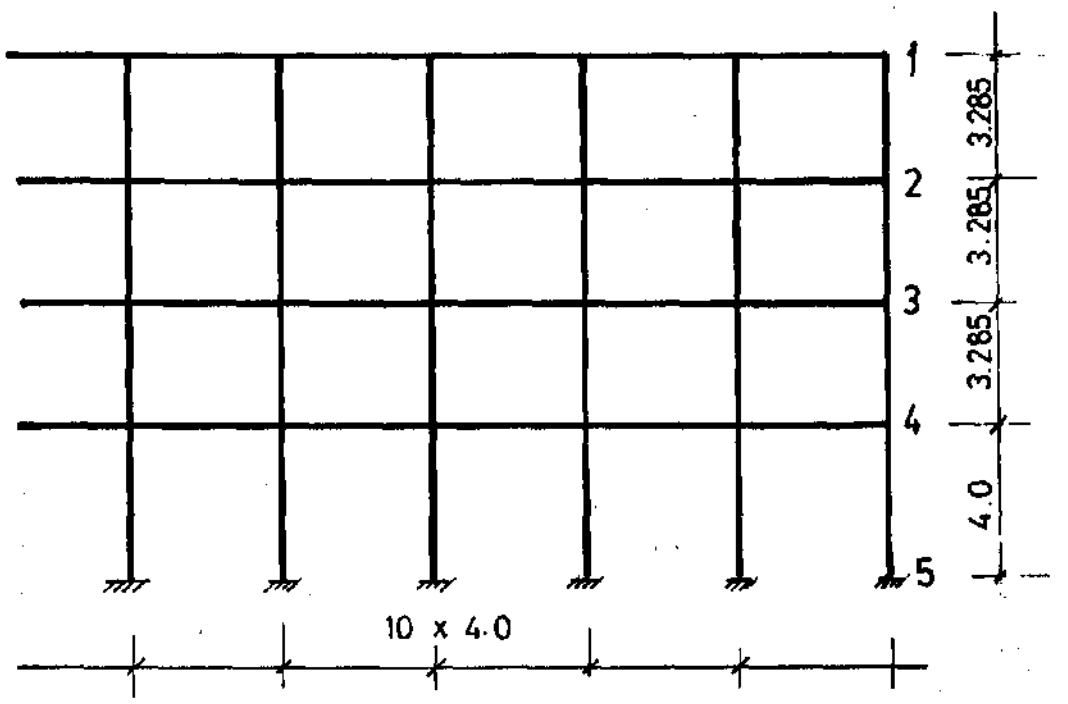
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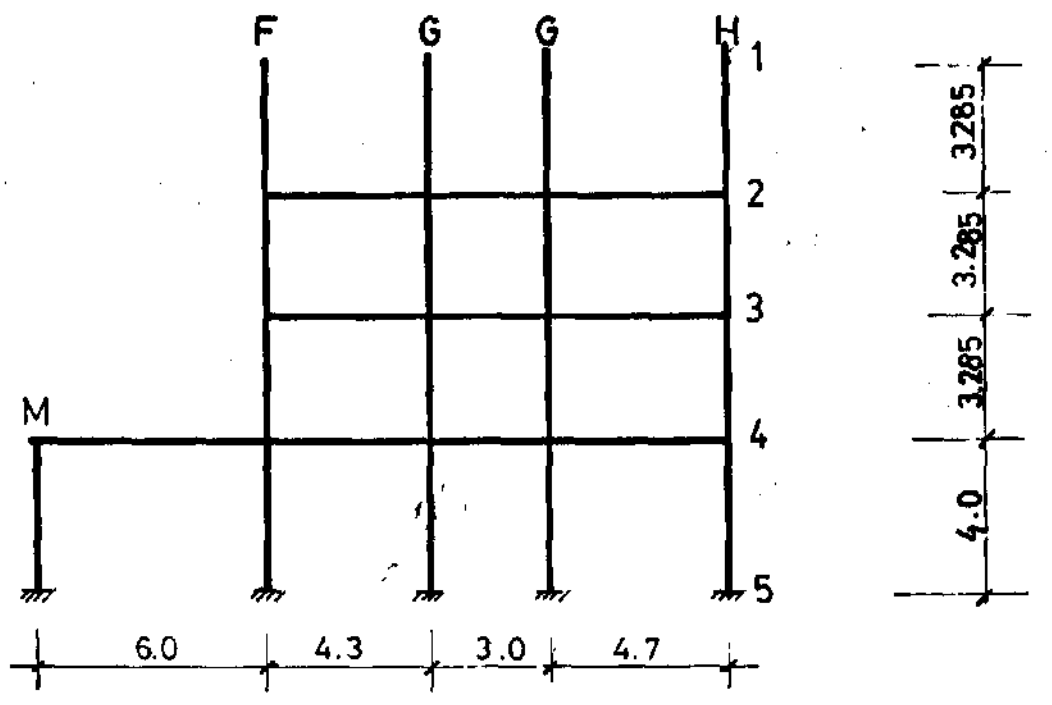
FRAME 4



FRAME 3



FRAME 5 & 6

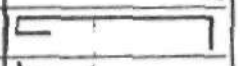
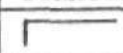
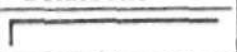
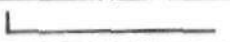
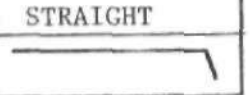


FRAME 7

BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL NUMBER	LENGTH (mm)	SHAPE
PANEL FIVE	028	Y8	6	6	36	3450	STRAIGHT
	029	Y8	11	6	66	2100	STRAIGHT
	030	Y8	10	6	60	2700	STRAIGHT
	031	Y8	4	6	24	4200	STRAIGHT
	032	Y8	4	6	24	2700	STRAIGHT
	033	Y6	24	6	144	850	STRAIGHT
	034	Y6	16	6	96	850	STRAIGHT
PANEL SIX	035	Y8	12	15	180	2750	STRAIGHT
	036	Y8	11	15	165	4950	
	037	Y8	6	15	90	4200	STRAIGHT
	038	Y8	24	15	360	2600	STRAIGHT
	039	Y8	4	15	60	5100	STRAIGHT
	040	Y8	11	15	165	2250	STRAIGHT
	041	Y6	24	15	360	850	
PANEL SEVEN	042	Y8	12	6	72	1850	STRAIGHT
	043	Y8	9	6	54	4950	STRAIGHT
	044	Y8	4	6	24	5100	STRAIGHT
	045	Y8	9	6	54	2250	STRAIGHT
	046	Y8	24	6	144	2050	STRAIGHT
	047	Y8	6	6	36	3400	STRAIGHT
	048	Y6	24	6	144	850	STRAIGHT
PANEL EIGHT	049	Y8	9	6	54	4950	STRAIGHT
	050	Y8	9	6	54	2250	STRAIGHT
	051	Y8	4	6	24	5100	STRAIGHT
	052	Y8	12	6	72	2600	STRAIGHT
	053	Y8	4	6	24	3400	STRAIGHT
	054	Y8	12	6	72	850	STRAIGHT
	055	Y6	24	6	144	850	STRAIGHT
PANEL NINE	056	Y8	8	12	96	2750	STRAIGHT
	057	Y8	16	12	192	2600	STRAIGHT
	058	Y8	11	12	132	2150	STRAIGHT
	059	Y8	6	12	72	4400	STRAIGHT
	060	Y8	4	12	24	3400	STRAIGHT

BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL NUMBER	LENGTH (mm)	SHAPE
STAIRCASE	01	Y12	6	6	36	1670	
	02	Y20	19	6	114	2500	
	03	Y12	11	6	66	4740	
	04	Y12	6	6	36	1380	
	05	Y8	15	6	90	1500	STRAIGHT
RAMPS	01	Y25	7	6	42	15614	STRAIGHT
	02	Y10	106	6	636	2000	STRAIGHT
	03	Y32	14	6	84	13528	STRAIGHT
	04	Y8	12	6	72	2500	STRAIGHT
	05	Y16	17	6	102	1880	STRAIGHT
	06	Y25	7	6	42	2500	STRAIGHT
	07	Y8	7	6	42	1500	STRAIGHT
	08	Y16	17	2	34	1880	STRAIGHT
	09	Y25	7	6	42	2150	STRAIGHT
	10	Y16	17	6	102	1880	STRAIGHT
	11	Y16	2	12	24	2900	STRAIGHT
	12	Y12	3	12	36	2900	STRAIGHT
	13	Y10	4	12	48	4100	STRAIGHT
BEAMS — BEAM B_1	01	Y25	8	54	432	1540	
	02	Y12	4	54	216	5180	STRAIGHT
	03	Y25	6	54	324	1880	STRAIGHT
	04	Y25	2	54	108	1360	STRAIGHT
	05	Y25	2	54	108	1900	STRAIGHT
	06	Y25	2	54	108	3980	STRAIGHT
	07	Y25	4	54	216	5440	
	08	Y12	2	54	108	2690	STRAIGHT
BEAM B_2	09	Y20	4	12	48	1540	
	10	Y16	4	12	48	5180	
	11	Y20	3	12	36	1910	ASTRAIGHT
	12	Y20	2	12	24	3600	STRAIGHT
	13	Y20	2	12	48	5440	
	14	Y16	2	12	24	3980	STRAIGHT
	15	Y10	2	12	24	2690	STRAIGHT
	16	Y20	1	12	12	5180	

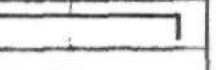
BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL NUMBER	LENGTH (mm)	SHAPE
BEAM B7	51	Y25	3	9	27	1890	
	52	Y12	2	9	18	3540	
	53	Y32	2	9	18	1690	
	54	Y32	2	9	18	2550	
	55	Y16	1	9	9	1230	
	56	Y12	2	9	18	2680	
	57	Y32	1	9	9	1230	
	58	Y32	2	9	18	1820	
	59	Y12	2	9	18	2000	
	60	Y25	1	9	9	1300	
	61	Y25	4	9	32	1940	
	62	Y12	2	9	18	2900	
	63	Y25	3	9	27	1540	
	64	Y25	2	9	18	5300	
	65	Y25	2	9	18	6360	
	66	Y20	2	9	18	2600	
	67	Y12	2	9	18	3260	
	68	Y16	1	9	9	3300	
	69	Y25	2	9	18	5300	
	70	Y16	1	9	9	2850	
COL. A & FOUN-DATION	01	Y16	4	18	72	3420	STRAIGHT
	02	Y25	12	18	216	4100	"
	03	Y8	141	18	2538	760	
	04	Y25	6	18	108	4670	
	05	Y25	6	18	108	2100	
	06	Y20	11	18	198	2400	
	07	Y12	17	18	306	1600	"
	08	Y16	22	28	396	1140	"
COL. B & FOUN-DATION	09	Y8	88	18	1584	1140	"
	10	Y16	8	18	144	3420	"
	11	Y25	24	18	432	4100	"
	12	Y25	16	81	288	4670	"
	13	Y25	16	18	288	2100	
	14	Y20	8	18	144	4100	

BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL NUMBER	LENGTH (mm)	SHAPE
BEAM B_3	17	Y20	4	6	24	1370	
	18	Y12	2	6	12	2910	
	19	Y20	2	6	12	1310	
	20	Y20	2	6	12	1920	
	21	Y20	2	6	12	3250	
	22	Y12	2	6	12	4820	
	23	Y16	1	6	6	3040	
	24	Y20	2	6	12	5300	
	25	Y12	2	6	12	3230	
	26	Y12	2	6	12	4870	
BEAM B_4	27	Y20	2	12	24	4450	
	28	Y12	2	12	24	1740	
	29	Y25	3	12	36	1700	
	30	Y32	1	12	12	1090	
	31	Y32	2	12	24	2075	
	32	Y12	2	12	24	3450	
	33	Y16	2	12	24	3450	
	34	Y25	2	12	24	1750	
	35	Y16	1	12	12	3290	
	36	Y25	2	12	24	1750	
	37	Y25	2	12	24	1175	
BEAM B_5	38	Y25	4	12	48	4450	
	39	Y10	2	12	24	3300	
	40	Y20	16	12	192	4450	
	41	Y16	8	12	96	3300	
	42	Y16	4	12	48	1980	
	43	Y25	12	12	144	3300	
	44	Y16	6	12	72	1980	
B_6	45	Y20	4	12	48	4450	
	46	Y20	16	12	192	4450	
	47	Y20	4	12	48	1200	
	48	Y12	2	12	24	1200	
	49	Y20	12	12	144	1250	
	50	Y12	6	12	72	1050	

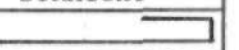
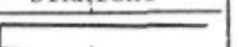
BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL NUMBER	LENGTH (mm)	SHAPE
COL. B (Cont	15	Y16	15	18	270	2100	STRAIGHT
	16	Y12	8	18	144	3000	STRAIGHT
	17	Y12	13	18	234	2100	"
	18	Y8	88	18	1584	760	
	19	Y8	44	18	792	660	ditto
COL. C & FOUND- DATION	20	Y16	4	4	16	3420	———
	21	Y25	8	4	32	4100	———
	22	Y25	6	4	24	4670	———
	23	Y25	6	4	24	2100	
	24	Y16	6	4	24	2160	———
	25	Y16	611	4	44	1320	———
COL. D & FOUN- DATION	26	Y16	4	4	16	3420	———
	27	Y16	4	4	16	4100	———
	28	Y16	4	4	16	4100	———
	29±	Y16	4	4	16	4670	———
	30	Y16	5	4	20	1940	———
	31	Y16	8	4	32	1140	———
	32	Y25	4	4	16	2100	
COL. E & FOUN- DATION	33	Y16	8	4	32	4100	STRAIGHT
	34	Y16	4	4	16	4670	STRAIGHT
	35	Y25	4	4	16	2100	
	36	Y16	4	4	16	1340	STRAIGHT
	37	Y16	6	4	24	860	
	38	Y16	4	4	16	3420	STRAIGHT
COL. F & FOUN- DATION	39	Y16	4	2	8	3420	STRAIGHT
	40	Y25	4	2	8	4100	STRAIGHT
	41	Y25	6	2	12	4100	STRAIGHT
	42	Y25	6	2	12	4670	STRAIGHT
	43	Y16	2	2	4	4670	"
	44	Y25	4	2	8	2100	
	45	Y16	21	2	42	1750	STRAIGHT
	46	Y12	11	2	22	1200	STRAIGHT
	47	Y8	86	2	179	1140	
	48	Y8	38	2	76	760	STRAIGHT

BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL NUMBER	LENGTH (mm)	SHAPE
COL. G & FOUNDA-TION	49	Y16	4	2	8	3400	STRAIGHT
	50	Y25	8	2	16	4100	STRAIGHT
	51	Y25	6	2	12	4670	"
	52	Y25	6	2	12	2100	⌐
	53	Y16	15	2	30	2180	STRAIGHT
	54	Y16	12	2	24	300	STRAIGHT
	55	Y8	68	2	136	7140	STRAIGHT
COL. H & FOUNDA-TION	56	Y16	4	2	8	3420	STRAIGHT
	57	Y25	8	2	16	4100	STRAIGHT
	58	Y25	6	2	12	4670	"
	59	Y25	6	2	12	2100	⌐
	60	Y16	20	2	40	1550	STRAIGHT
	61	Y12	12	2	24	1200	STRAIGHT
	62	Y8	88	2	176	1140	STRAIGHT
COL. J & FOUNDA-TION	63	Y16	8	2	16	4100	STRAIGHT
	64	Y16	4	2	8	4670	STRAIGHT
	65	Y16	4	2	8	1340	STRAIGHT
	66	Y16	4	2	8	2100	⌐
	67	Y16	6	2	12	860	STRAIGHT
	68	Y16	4	2	8	3420	STRAIGHT
	69	Y8	78	2	156	1140	⌐
COL. K & FOUNDA-TION	70	Y16	4	2	8	3420	STRAIGHT
	71	Y16	8	2	16	4100	STRAIGHT
	72	Y16	4	2	8	4670	STRAIGHT
	73	Y25	4	2	8	2100	⌐
	74	Y16	5	2	10	1940	STRAIGHT
	75	Y16	8	2	16	1140	STRAIGHT
	76	Y8	88	2	176	1140	⌐
COL. L & FOUNDA-TION	77	Y16	4	2	8	3420	STRAIGHT
	78	Y25	8	2	16	4100	STRAIGHT
	79.	Y25	6	2	12	4670	STRAIGHT
	80	Y20	6	2	12	2100	⌐
	81.	Y16	6	2	12	2160	STRAIGHT
	82	Y16	10	2	20	1320	STRAIGHT
	83	Y8	88	2	176	1140	⌐

BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL (mm) NUMBER	LENGTH (mm)	SHAPE
PANEL TEN	061	Y8	9	6	54	2750	STRAIGHT
	062	Y8	8	6	48	2150	STRAIGHT
	063	Y8	8	6	48	2150	STRAIGHT
	064	Y8	4	6	24	3400	STRAIGHT
	065	Y8	4	6	24	4400	STRAIGHT
PANEL ELEVEN	066	Y8	9	6	54	2600	STRAIGHT
	067	Y8	8	6	48	2600	STRAIGHT
	068	Y8	4	6	24	3400	STRAIGHT
	069	Y8	4	6	24	3400	STRAIGHT
	070	Y6	24	6	144	850	STRAIGHT
PANEL TWEL-VE	071	Y8	9	6	54	4900	
	072	Y8	9	6	54	2300	STRAIGHT
	073	Y8	12	6	72	2050	
	074	Y8	6	6	36	3400	STRAIGHT
	075	Y8	4	6	24	4500	STRAIGHT
	076	Y8	12	6	72	850	STRAIGHT
	077	Y6	12	6	72	850	STRAIGHT
	078	Y8	12	6	72	2500	STRAIGHT
PANEL THIR-TEEN	079	Y8	12	12	144	2700	STRAIGHT
	080	Y8	11	12	132	4900	STRAIGHT
	081	Y8	11	12	132	2250	STRAIGHT
	082	Y8	24	12	288	2750	STRAIGHT
	083	Y8	4	12	48	4700	STRAIGHT
	084	Y8	6	12	72	4400	STRAIGHT
PANEL FOUR-TEEN	085	Y8	15	4	60	2700	STRAIGHT
	086	Y8	11	4	44	5260	STRAIGHT
	087	Y8	15	4	60	2480	
	088	Y8	15	4	60	2750	STRAIGHT
	089	Y8	6	4	24	4400	STRAIGHT
	090	Y8	6	4	24	6400	STRAIGHT
	091	Y8	11	4	44	2490	
	092	Y8	34	4	136	1860	
	093	Y8	22	4	88	3080	STRAIGHT

BENDING SCHEDULE

MEMBER	BAR MARK	TYPE & SIZE	NO. IN ELEMENT	NO. OF ELEMENT	TOTAL NUMBER	LENGTH (mm)	SHAPE
COL. M '& FOUNDATION	84	Y25	6	3	18	4670	STRAIGHT
	85	Y25	6	3	18	2100	
	86	Y16	8	3	18	1850	STRAIGHT
	87	Y16	10	3	30	1260	STRAIGHT
	88	Y8	12	3	36	1140	

KAS... ARM... ZARI... ...RARY ...VERSITY