STRUCTURAL DESIGN OF A PROPOSAL 3- STOREY OFFICE COMPLEX.

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CERTIFICATION

I, hereby testify that this work has been supervised, read and has met part of the requirement for the award of post graduate diploma(PDG) in the Department of Civil Engineering, Federal University Of Technology, Minna, Niger State. Nigeria.

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Date

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Date

DEDICATION

I, gladly dedicate this project to ALLAH THE ALMIGHTY for HIS GRACE, MERCY and STRENGTH given me in writing it.

ACKNOWLEDGEMENTS

I acknowledge the grace of ALLAH in my life for given the opportunity to be one of the students of the Federal University of Technology (FUT) Minna and for His guidance in the course of writing this project. I appreciate the support of my Head of Department of Civil Engineering; Prof. Sadiku who has his doors open at all time to anybody for advice.

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Appreciation equally goes to my wives, Hajiya Kafeelat, Hajiya Saratu, my children and brothers for their support throughout my stay in the University. ~ay Allah bless you all. (Amin).

ABSTRACT

This project covers the analysis, design and detailing of a proposed office complex for the State Security Services In Abuja. The project was prepared based on the standard and principle set out by the structural use of concrete B5811 0 parts 1,2 and 3 to achieve the desired objectives. The roof members, beams slabs, stair-case, columns and the foundations were analysed and designed in accordance to BS8110. The results were used to produce simple and neat structural detailed drawing to ease estimation and construction of the proposed project.

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NOTATIONS

As.pro.	area of tension reinforcement provided
As.req	area of tension reinforcement required
Asv.	cross - sectional areas of the two legs of a link
b	width of section
bw	breadth of web or rib of a member
d	effective depth of tension reinforcement
dı	dept to compression reinforcement
tbs	bond stress
feu	characteristic concrete cube strength
fy	characteristic strength of reinforcement
fyl	characteristic strength of longitudinal
fyf	characteristic strength of link reinforcement
Ok	characteristic dead load
g	distributed dead load
gk	characteristic dead load per unit area
hf	thickness of flange
hmax	larger dimension of section
I	second moment of inertia
Ie	effective height of column or wall
lex	effective height for bending about major axis
ley	effective height of bending about the major axis
10	clear height of column between end restraint
M	bending moment
Madd	maximum additional moment

M_i	Maximum initial moment in a column due to ultimate load (but not less than O.ONI)
M_{iy}	Initial moment about minor axis of slender column
M_t	Total moment in a column due to ultimate loads
M_{tx}	Total moment about the minor axis of a slender column
M_{ty}	Total moment about the minor axis of a slender column
M_u	Ultimate moment of resistance
N	Ultimate axial l'0ad at a section
N_{uz}	Axial load capacity of a column ignoring all bending
n	Total ultimate load per unit area
Q_k	Characteristics imposed load
q	Distributed line load
q_k	Characteristic imposed load
S_v	Spacing of link along the member
T	Torsional moment due to ultimate load
U	Perimeter
V	Shear force due' to ultimate load
V_c	Ultimate shear stress in concrete
X	Neutral axis depth
X_i	Smaller dimension of a link
Y_i	Larger dimension of a link
Z	Lever arm
Z_o	Lever arm factor Z/d
red	Ratio of reduction in resistance
U_s	Sum of effective perimeter of the tension reinforcement
O	Bar size

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CHAPTER ONE

1.0 INTRODUCTION:

The security office is an office where security is the watch-word. This project of the office complex is to house the state security services (SSS) for their movement from Lagos to Abuja. The design recognizes this fact of security and as a result, basement is introduce as a detained room for any support for a period of 48-hours before prosecution.

1.1 AIM

To design a structurally and functional office complex for state security service (SSS) in Abuja.

1.2 OBJECTIVES

(a) To determine the appropriate quality of reinforcement in a member, so as to make member serviceable for the intended period of life.

(b) To ensure that, the structure is safe under the worst condition of load application.

(c) To ensure that the deformation of the structure is not impairing the appearance, durability and performance of the structure, under the working load (serviceability).

(d) To ensure that, the structure is economical"

(e) To ensure that the structure can comply with future functional and structural requirements.

(f) To achieve an aesthetically pleasing and structurally gratifying structure.

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REFERENCE

British Standard (BS 8110) Part I and II (1989) structural use of concrete.

British Standard (BS) 5268 Part (1988).

Musley W.H, J.H Bungey 4th Edition Reynolds Reinforced combats Designers Hand Book by Charles .e. Reynolds, James C. Steadman and Anthony

Simplified Reinforced concrete design by Victor .O. Oyenuga

DESIGN PARAMETERS

Slabs : Ribbed slabs

: Clay pots of size 2000 x 200 x 400mm are used with a weight of 0.0077 kN

: Cover is 20mm

: Concrete topping is reinforced with R8 @ 300 x 300 mesh for anticracks

Beams : Cover of 20mm is used

Concrete : Concrete grade of C- 30 (i.e fcu = *30N/mm2)* is recommended for all the members having a characteristics strength of

7N/mm2 at 3 days

14N/mm2 at 7 days

30N/mm2 at 28 days

Reinforcement: High yield steel of strength are used for 41*ON/mm2* at the tensile and compressive members

- Mild steel are used as stirrups except some exceptional cases as indicated in the design

Foundations: 500mm cover is used

50mm blinding of strength 7N/mm2 (grade C - 7)

Allowable bearing pressure of 300 kN/m2

Fire Resistance ------- 1$_{2}$~~rs.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 INTRODUCTION

(History of the use of Reinforced Concrete),

Reinforced concrete was invented in France 1850, and now one of the most important materials used in building construction. It is a combination of concrete and steel, Concrete (which can resist very high compressive forces, but has little resistance to the tensile, Shearing, twisting and other forces to which the parts of a structure are subjected) and steel, which has a high resistance to tensile forces and other force imposed on the structures by their own weight, the loads they carry, and by other forces, such as the wind. Reinforced concrete is designed so that, generally, the compressive forces are resisted by the concrete and the tensile forces by the steel reinforcement. The reinforcement is generally in the form of mild steel round bars up to 40inm in diameter, but square, Indented, and twisted bars are used. The development of high tensile steel has increased further the Load-bearing capacities of reinforced concrete. An advantage of the steel reinforcement is that it also resists the tensile stresses induced in the concrete -when the alter shrinks during the setting and hardening process. Another is that concrete at steel expand and contract at the same rate with changes of temperature. A further advantage of reinforced concrete is its resistance to fire. The use of reinforcement has made possible the erection of large cantilevers, thin domes long spans, and shapes that would be impossible or uneconomical in any other material. It has also made possible reinforced concrete beams, transmission line poles. Lamp and fence .posts and many other products which can be made in a factory away from the Site of erection with a consequent reduction of the amount of labour required on the site.

2.2 COMPOSITION OF REINFORCED CONCRETE

Reinforced concrete is a man made composite of the major constituent of which is cement, aggregates water, and steel.

2.2.1 Cement: can be describe as a material with adhesive and cohesive properties, which makes it capable of binding mineral fragment into a compact whole. It can also be explained as a materials which is added iin an appropriate form to a non-coherent assemblage of particles, will subsequently hardened by physical or chemical means and bind the particles into a coherent mass. Thus therefore, allows such diverse material like bitumen, lime to be grouped under the umbrella of cement. For construction purposes the term cement is restricted to binding materials used with stones, sand, bricks, building blocks etc.

The cement of interest is that for making concrete and similar building materials which have the property of setting and hardening in the presence of water (mixture) by virtue of chemical reactions called hydration. Cement which is the most expensive ingredient in concrete making and the most reliable as it provide greater percentage of the concrete strength. The principal requirement is tht the cement should be able to produce strong dense and durable concrete with defmite setting and harden characteristic.

Cement is manufactured from basic raw materials of calcium carbonate found in calcareous rocks such as Limestone or chalk, silica, alumina, and Iron oxide found in argillaceous rocks such clay or shale. It is prepared by first intimately grinding and mixing the raw constituents in certain proportion (wet or dry process) and subsequently burning this mixture at very high temperature (1450$_0$C) in a rotary or shaft kiln to produce clinker. The cooled clinker is finally ground with addition of 1-50/0gypsum to the required fineness. The gypsum retards the hydration of the aluminates Component of the cement to avoid what is called flash set. The most common cement used is the Portland cement which was developed in 1824 AD and derives it name from Portland limestone in Doreset UK.

Gengeral, the Oxide composition of a typical Portland cement is presented in TABLE 2.1

TABLE 2.1 COMPOSITION OF PORTLAND CEMENT

OXIDE	010 COMPOSITION
Lime, CaO-	64.7
Silica, S_1O2-	21.20
Alumina, Ah 03-	5.22
Iron Oxide, fe-O	3.08
Magnesia MgO	1.04
Sulphur trioxide, S03	2.01
Soda, Na20-	0.42
Loss on ignition, LOI	1.45
Insoluble residue, IR	0.66
	100.00
Free line, CaO-	1.60

The insoluble residue which is determined by treating the cement with hydrochloric acid is a measure of cement adulteration arising horn impurities in gypsum. The BSI2, (1958) and NIS 11. (1974).

Fineness:- The ordinary Portland cement should have an average specific Surface of not less than 2.500cm^2/g ..

Settings:- (a) Initital setting time should not be less than 45 minutes (b) Final Setting time should not be less or equal to 10 hours.

Soundness: Should not have an expansion more than 10mm and of aerated less or equal to 5mm.

TYPES OF CEMENT

The deliberate variation on the proportions of the four main compounds Ordinary Portland cement (OPC) with their hydrates added with chemicals/admixtures enable cements with different properties to be produce, to suit different circumstances of construction, such cement types includes:

(1) Ordinary Portland cement (OPC). This cement is the most common and has a medium rate of hardening, making it suitable for most concrete work. It has a low resistance to chemicals and has a final setting of 10 hours.

(2) Rapid hardening Portland cement: is in many ways similar to ordinary Portland cement but produce. a much higher early strength. The increased rate of hydration is accompanied by a high rate of heat development which makes it unsuitable for large massess concrete.

(3) Low heat Portland cement: has a limited use but is suitable for very large structures, such as concrete dams, where the use of ordinary

cement would result in unacceptably large temperature gradients within the concrete.

(4) Portland blast furnace cement is,produced by mixing up to 65 percent granulated blast furnace slag with ordinary Portland cement.

(5) Hydrophobic 'cement: Made by grinding the cement clinker with Small amount of film forming water repellent material sprayed into the mill.

(6) Other types of cement are:

Sulphate resisting Portland cement

Extra-rapid hardening Portland cement

Ultra-high early-strength Portland cement

Water proof and water repellent Portland cement

Air-entrianing Portland Cement

Super sulphated cement

Pozzolanic cement

2.2.2 Aggregate: It is much cheaper than cement and maximum economy is obtained by using as much aggregate as possible in concrete. Its use also considerable improves both the volume, stability and the durability of the resulting concrete. The commonly held view that aggregate is a completely inert filler in concrete is not true, its' physical characteristics and in some cases its chemical composition affecting to a vary degree the properties of concrete in both plastic and hardened states.'

~ .Types of aggregate

In the previous sections, discussion has been mainly confined to rock aggregates. Although other types of aggregate are use for making concrete their contribution IS very small m companson with rock aggregates.

- Heavyweight aggregate: Provide an effective and economical use of concrete for radiation shielding' by giving the necessary protection against x-rays gammarays and neutrons. The effectiveness of heavyweight concrete, with a density from 400 to 5500kgm^{-3}, depends 'on the aggregate type, the dimensions, and the degree of compaction. It is frequently difficult with heavyweight aggregate to obtain a mix which is both workable and not prone to segregation.
- Normal aggregate: These aggregates are suitable for most purposes and produce concrete with a density in the range 2300 to 2500kgm^{-3} Rock aggregates are obtained by crushing quarried rock to the required particles size or by extracting the sand and gravel deposits formed by alluvial or glacial action. Some sands and gravels are also obtained by dredging from sea and river bed. Aggregates, in particular sands and 'gravels, should be washed to remove impurities such as clay and site. In the case of river and marine aggregates the chloride content should generally be less than 1 percent if these are to be used for structural concrete.
- Lightweight aggregates: find application in a wide variety of concrete products ranging from insulating screeds to reinforced or pre-stressed concrete although their greatest use has been in the manufacture of pre-cast concrete blocks.

'Concretes made with lightweight aggregates have good fire resistance properties. The most commonly used lightweight aggregates in the UK are expanded slate (solite), expanded clay (aglite and leca), clinker,

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foamed sky and sintered pulverized fuel ash (Lytag). They are highly porous and absorb considerabley greater qualntities of water than do normal aggregates.

2.2.3 Water: Water used ııı concrete, in addition to reaction to reacting with cement and thus causing it to set and harden, also facilitates mixing, placing and compacting of the fresh concrete. It is also used (or washing aggregates and for curing purposes. In general wter fit for drinking such as tap water is acceptable for making concrete. The impurities that are likely to have an adverse effect when present in appreciable quantities include silt, clay, acids alkalis and other salts, organic matter and sewage. "The use of seawater does not appear to have any adverse effect on the strength and durability of Portland cement concrete but it is known to cause surface dampness efflorescence and staining and should be avoided where concrete where concrete with good appearance is required. Seawater also increases the risk of corrosion of steel and its use in reinforced concrete is not recommended; When suitability of mixing water is in question, it is desirable to test for both the nature and extent of contamination as prescribed in BS3148.

The quality of water may also be assessed by comparing the setting time and soundness of cement pastes made with water of known quantity and the water whose quality is suspect.

The use of impure water for washing aggregates can adversely affect strength and durability, if it deposits harmful substances on the surface of the water. In general the presence of impuriites in the curing water does not have any harmful effect, although it may spoil the appearance of concrete.

, Water containing appreciable mounts of acid or organic materials should be avoided.

(Jackson 1977)

2.2.4 Steel: This can be describe as the most efficent and certainly one of the most used structural material it 'can be formed in various structur~l shapes, such as wide flange beams, sheet by rolling and plates. It can be cast into complex shapes like those of bridge bearing; it can be bolted riveted or welded. It can be alloyed with other metals such as chromium, nickel and copper to obtain an iricrease resistance to corrosion.

Steel: is one of the few structural materials, which demonstrate a well-defined field (i.e strips above which yield or flows with almost no increase in stress).

The module of elasticity of steel is measured by the scope of the elastic portion of its stress - strain curve, the change from elastic, to plastic appears linear initially but changes abruptly for mild steel and gradual for high yield steel. Because of this variation in the shape of the curves idealized curves which give safe result must be used.

Steel is a dense structural material and its weight is $7850Kg/m^3$, its coefficient of thermal expansion is approximately 1×10^{-5} °C/C but its l~sses it strength rapidly above 400^0C and become brittle at 34^0 em.

Concrete reinforcment: Concrete has low tensile and bending strengths and a high compressive streghth. Steel reinforcement overcomes the deficiencies in the tensile and bending strengths.

The reinforcing steel must have adequate tensile properties and form a stong bond with the concrete since the concrete transmits load to the steel by shearing stresses. The bond is purely mechnicial and arises from surface

TABLE 2.2

	CONCRETE	STEEL
Stregth in tension	Poor	Good
Stregth compression	Good	Good, but slender bars will buck
Strength in shear	Fair	Good
Durability	Good	Corrodes if unprotected
Fire resistance	Good	Poor - suffers "rapid loss of strength at high temperature

It can be seen from this list that the materials are more or less complementary. Thus, when they are combined, the steel is able to provide the tensile strength and probably some of the shear strength while the concrete, strong in compression, proetects the steel to give durability and fire resistance. (Bungey, 1990).

2.4 DESIGN METHODS OF REINFORCED CONCRETE

The design of an on engineerring structure must ensure that:

1. Under the worst loading the structure is safe.
2. During normal working condidtions the deformation of the members does not detract form the appearance, durability, or performance of the structure. Despite the difficulty in assessing the precise loading and variations in the strength of the concrete and steel, these requirements have to be met. Three basic methods using factors of safety to achieve safe, workable structures have been developed. They are as follows:-

2.4.1. the permissible stress method in which ultimate strengths of the materials are divided by a factor of safety to provide design stresses which are usually within the elastic range. This method has proved to be a simple and useful method but it does have some serious inconsistencies. Because it is based on an elastic stress distribution, it is not really applicable to a semi-plastic material such as concrete, nor is it suitable when the deformations are not proportional to the load, as in slender columns. It has also been found to be unsafe when dealing with stability of structures subject to overturning forces.

2.4.2 The load factor method in which the working loads are multiplied by a factor of safety. As this method does not apply factors of safety to the material stresses. It cannot directly take account of the variability of the materials and also it cannot be used to calculate the deflections or cracking at working loads.

2.4.3 The limite state method which multiplies the working loads by partial factor of safety and also divides the materials, ultimate strenghts by further partial factors of safety. This method of design overcomes many of the disadvantages of the previous methods. This is done by applying partial factors of safety, both to the loads and to the material strengths the magnitude of the factors may be varied so that they maybe used either with the plastic conditions in the ultimate state or with the more elastic strength range at working laods. This flexibility is particularly important if full benefits are to be obtained from development of improved concrete and steel properties.

The purpose of design is to achieve acceptable probabilities that a structure will not become unfit for its intended use, that is, that it will not reach a limit state. Thus, any way in which a structure may cease to be fit for use will constitute a limit state and the design aim is to avoid any such condition being reached during the expected life of the structure.

(a) Ultimate limi) stater- This requires that the structure must be able to withstand with an adequate factor of safety against collapse, the loads for which it is designed. The possibility of bucking or overturning must also be taken into account, as must the possibility of accidental damage as caused, for example by an internal expolsion.

(b) Serviceability limit states:- Generally the most important serviceability limit states are:-

1. Deflection: The appearance or efficiency of any part of the structure must not be adversely affected by deflections.
2. Cracking.- Local damage due to cracking and spalling must not affect the appearance, efficiency or durability of the structure.
3. Durability: This must be considered in terms of the proposed life of structure and its conditions of exposure. Other limit states that may be reached include:-
4. Excessive vibration: This may cause discomfort or alarm as well as damage.
5. Fatigue: Must be considered if cyclic loading is likely.
6. Fire resistance: This must be considered in terms of resistance to collapse, flame penetration and heat transfer.
7. Special circumstacnes:- Any special requirements of the structure which are not covered by any of the more common limit states, such as earthquake resistance, must be taken into account.

roughness and friction. Mild steel with a maximum carbon content of 0.25 percent is suitable and it's supplied in three conditions. There are hot rolled (BS 4449), cold rolled (BS 4461) and hard drawn (BS 4482) which give tensile tensile strength between 250 and 485MNm-2~

Reinforcing steels are supplied as plain, indented or twisted round or square bar in a variety of sectional shapes in straight lengths or bent shapes and woven or electrically welded mesh. Protection against corrosion is provided by the highly alkaline environment of the Portland cement hudrates within the concrete. Carbonation that is the reaction of the hydrates with carbon dioxide can, however, break down this protection if it penetrates as far as the steel. (Jackson, 1977).

2.3 PROPERTIES OF REINFORCED CONCRETE

Reinforced concrete is a strong durable building material that can be formed into many varied shapes and sizes ranging from a simple rectangle column to a slender curved dome or shell its utility and versatility is achieved by combining the best features of concrete and steel. Consider some of the widely differing properties of these two materials that are listed below:

2.5 Characteristics Strength and Load

2.5.1 Characteristics Strength:-

This is referred to as strength of concrete cube (feu) at 28 days, or the yeild or proof stress of reinforcement (fy), below which not more than 5% of the test result fall. It is found that the difference in strength in the actual structure may be greater than the strength derived from tests.

This is due to local variation and deterioration in transit. These efforts are allowed for design' by dividing the characteristic strength by a partial safety factor for strengthfYg) Design strength = characteristics strength (fk)/partial factor of safely (Ym). In general the partial safety factor adopted fro concrete is 1.50 while that for steel is 1.15. The characteristics strength of concrete is given in BS 5328 and that of reinforcement is given in BS 8110, table 3.1.

2.5.2 Characteristics Load

This is defined as the load above which not more than 5% of loads on the structure fall within its working life.

1. Dead Loads: The weight of the structural elements, permanent partitions and finishes.
2. Impose load: Due to furniture, occupants machinery, vehicles impact, snow etc, the characteristics imposed·load for all types of building are in BS 6399 (1984) and other hand books.
3. Wind Loads: The loads on structure based on statistical analysis of the meteorological data usually obtained from the gust speed that it is estimated would be exceeded only once in 50 years.

   Ideally these loads should be considered statistically but because complete statistical information on loads is not available. The characteristic' load selected for design should be that which produces the worst effect. (i.e. loads that produce most severe stress). BS 8110 cl 2.4.31 gives a guide on the load combinations for ultimate limit state for the design of the whole or any part of a structure.

2.6 REINFORCED CONCRETE DESIGN TO BS8 110

In the analysis of a cross section to determine its ultimate moment of resistance, the following assumptions set out in clause 3.4.1. 1 should he made.

a. The strain distribution in the concrete in compression and the strain in the reinforcement whether in tension or compression are derived assuming that plane section remain plane.

b. The stresses in the concrete in compression may he derived from the stress curve.

c. The tensile strength of the concrete is ignored.

d. The stresses in the reinforcement are derived from the stress strain curve.

2.7 DESIGN OF THE STRUCTUAL ELEMENTS

A reinforced concrete structure is a combination of beams, columns, slabs and walls, rigidly connected together to form a monolithic frame. Each individual member must be capable of resisting the forces <1:cting on it; so that the determiantion of these' forces is an essential part of the design ~ocess. The full analysis of a rigid frame is rarely simple, but simple, but simplified calculations of adequate precision can often be made if the action of the structure is understood.

2.7.1 Reinforced Concrete Slabs: are plate elements forming floor, roofs and walls of building arid as the decks of bridges, the flour system of a structure can take various forms such as in-situ: solid slabs, ribbed slabs', flat slabs or pre-cast units.

The generally carry distributed loads;

Slabs may simply supported or continous over one or more supports and are classified according to the method of support.

a. Spanning one way between beams

b. Spanning two ways between beams

c. Flat slabs carried on columns with no beams

Stairs with various support conditions' form a special case of sloping slabs. The following analysis-idealization into strips or beams, elastic plate theory and finite element analysis.

a. Elastic analysis - idealization into strips or beams, elastic plate theory and finite element analysis.

b. Semi-empirical design using moment coefficients base on yeild line analysis given in clause 3.5.2.4 and 3.5.3. - 3 BS 8110

c. Yield Line and. Hillerberg strip. method, concrete slabs, behave primarily as flexual members and the design is similar to that of Beams although it is some what simpler because:

   1. The breadth of the slab is already fixed and a unit breadth of 1m is used in the calculation.

   2. The shear stresses are usually low in a slab except when there is hwavy concentrated load.

   3. compression reinforcement is seldom required.

2.7.2 Simplification of load arrangement:

In principle a slab should be designed to withstand the most unfavourable arrangement of design loads but there are greater opprunities by the use of a single load case of maximum design load on all spen or panels provided the following condition are met as stated in clause 3.5.2.3 B S8 110. In one way spanning slab the area of each bay exceeds 30mm2

The ratio of the characteristic imposed load to the characteristic dead load does not exceed 1.25.

The characteristics imposed load does not exceed 5 *KN/m2* excluding partitions.

One way spanning slabs: The slabs are design as if they consist of a series of beams of 1m wide spanning between s:upports.It can he simply supported or continous slab. The effective span of simply supported slab is the clear span L plus effective depth d for the continuous slab L is the distance between centers of supports.

The code stipulated that conditions of clause 3.5.2.3 are met the moments and shear in continuous. One way spanning stabs may be calculated using the coefficient given in table 3.13 (BS 8110).

The main reinforcement spans between the supports or over the interior support of the continuous slab.

Simply supported slabs: The design of simply supported slabs without adequate provision to resist tension at cOlpers and to prevent the comers from lifting, the code gives the following equations for the maximum moment per unit width at mid span for 1x and 1y respectively.

Msx	Bsn1x 1	eqn	2.1
Msy	By NIx 2	eqn	2.2

These equations corresponds to equation 11 ofBS 8 110

Where Bsx and Bsy are given in table 3.14 BS 8110

(1.4gk+ 1.6qk)

Lx = length of shorter span

Ly= Length of longer span

TWO WAY SPANNING SLAB AT RIGHT ANGLES

When floor slab are supported on four sides. Two ways spanning action occurs. In a square slab the action is equal in each direction. In long narrow slab where the length is greater than twice the breadth, the action is effectively one way, though the end beams carry some slab loads. The edge conditions must be defined for slabs, these are:

a. Simply supported slabs where the comers can lift away from the support.
b. One - panel slab that is held down on four sides edge beams. The stiffness of the beam affect the result.
c. Slabs with all edges continuous over support.
d. Slabs with one, two or three edges continuous edge(s) may be simply supported or held down to the edge beam.

Under the two ways spanning slabs are the restrained slabs. Restrained slabs can be classified as:-

1. Restrained slabs where the comers are prevented from lifting and adequate provision is for torsion.
2. Restrained slabs with unequal conditions at adjacent panels.

ultimate limit state and serviceability limit state of deflection may be checked. In most buildings beam sizes are dictated by the architectural drawings but must be confirmed by the structural Engineer. Since wall width is generally 225mm, most beams have webs 225mm wide and usual depths include 450,600,750 and 900mm.

Generally, for guidance purpose only, beams not exceeding 6.0m can be designed for a depth of 450mm; while between 6.00m and 7.50m, a depth would be appropriate.

Beams are either rectangular or flanged beams, flanged beams on the other hand are either L or T _ beams. Effective width of L or T _ beams must be determined and the standard recommends as follows:

a. For T _ beam: web width plus one ': fifth of span or the actual flange width smaller holds.
b. For L - beam: web width plus one'- tenth of the effective span or actual flange with smaller holders.

Beam effective span is actual span that is' from centre of bearing or clear span plus one _ half of the effective depth. In general, effective span can be taken as centre of the bearings for continuous beam and for simple supported beams as clear span plus 225mm. in addtion, beams can be assumed to have satify serviceability limit state of deflection if the span/effective depth ratio enunciated in clause 24.6 of the standard is followed. This is presented in table 2.3 basic span/effective depth ratios fro rectangular or flanged beam.

TABLE 2.3

Support conditions	Rectangular sections	Flange beam with bw/b<O.3
Cantilever	7	5.6
Simply supported	20	16.0
Continuous	26	20.6

The values in th'e table are for beams of 10m span or less. Linear interpolation is allowed for flanged beams with bw/b greater than 0.3 for beam span in excess of 10m, the value in table 2.2 should be multiplied by 10/span.

The above notwithstanding, there may be the need to carryout simple check for deflection since deflection is influenced by the quantity of steel at both the tensions and compression 'zone, values in table 2.2 may be modified as follows:

a. Modification of tension reinforcement:

modification factor = 0.55+ (477-Fs)::; 2.0

120 (0.9 + Mlbd2)

Where M is the design ultimate moment

Andfs 2fy Asreq x 1

3 Asprov B

Asreq = Calculated As required

Asprv = Actual As Provided and

Bb = Re - distribution ratio which should be assumed as I ifnone.

b. Modification for compression reinforcement:

modification factor = 1+ (100 Aspro/bd)

3+ (100 AS prov/bd) ::;i.5

Table 3.10 and 3.11 of BS 8110 part 1:1997, give values of modification factors for tension and compression reinforcement respecitively.

Simply supported beams

a. Continuous beams

b. Beams Subjected to torsion and

c. Arcate beams

The following steps are carried out in the design of beams

1. ·Choice of section
2. Analysis of the beam
3. Design of the beam for tensile and comprhensive reinforcments, if any and
4. Design of shear, local bond and torsion if any.

Beam procedures:

a. Choose or estimate member width (generally, 225mm), and depth (usually, 450mm, 600mm or 750mm).·

b. Estimate the flange width of non-rectangular beam.

c. Analyze the b,eam to obtain imposed moments, shearing forces and tensional moments (if any).

d. Design for einforcements as follows:

Mement of resistance $Mu = 0.156fubd_2$

When applied moment M is less or equal to Mu, design for tension reinforcement, As only.

$K = \frac{M}{Fcubd_2}$ and $Z = d\,(0.5 + V\,(0.25 - klO.9)$

When M exceeds Mu design for As.and As as follows

$'As_1 = \frac{M - Mu}{0.95fy\,(d-d)}$ d^1 can be taken as 50mm

$$As = \frac{Mu}{0.95fy0.78d} + ASl$$

$$= \frac{Mu+Asl}{0.74fyd}$$

e. Design for stirrups (shear) as follows

from V shearing force, calculate

$$V = \frac{V}{bd} N/mm^2$$

to obtain Vc and design for stirrups viz.-

1. When < v ::; 0.5vc provide minimum stirrups e.g 10mm bars at 0.75d maximum

11. 0..5vc :S v :s(vc + 004)

$$Sv = \frac{0.95fyv\ Asv}{0Ab} <: 0.75d$$

III. When (vc + 004) ::;v ::; 0.8 vfcu or 5 N/mm^2 provide limits with the spacing calculated from:

calculated from: "

$$Sv = \frac{0.95fy\ Asv}{b(v-vc)} mm < 0.95d$$

when Sv is less than 125mm, it is better to double the spacing and double the legs of the stirrups. In example, 2 legs R 10mm @ 1OOmmcentres is better replaced by 4 legs R 10mm @ 200mm centres. Note also that Asv is the total area of the stirrups. For example, 2 legs RIOmm has Asv as 157mm^2 and fyv is the stirrups charactristic strength which may be the same with that of the main reinforcement or different. To reduce cost, mild steel round bars

Restrained slabs:

In this slab, the comers are prevented from lifting and provision is made for torsion. The maixmum moments of Msx and Msy at mid span on strips of unit width for spans 1 x and ly are given by:

Msx BsxnIx2

Msy BsynIx2

Where Bsx and Bsy are coefficient from table 3.15 ofBS 8110

Wquation 2.1 and 2.2 represent equation 14 and 15 ofBS 8110

With unequal conditions at adjacent panels the support moments calculated from table 3.15 may differ significantly to adjust then, the following procedures set our in BS8110 clause 3.5.3.5 may be use. They include>

a. Calculate the sum of the moments at mid - span supports (neglecting signs).
b. treat the values from table 3.15 ofBS8110 as fixed end moments (ferns).
c. Distribute of FEM across the supports according to the relative stiffness *of* adjacent spans, given new support moments.
d. Adjust mid span moments: (this should be such that when it is added to the support moments from neglecting signs) the total should be equally to that from (a).

2.7.3 REINFORCED CONCRETE BEAMS

Beams are horizontal members of a building frame receiving loads from the slab and transmitting same through the columns to the foundations. Beams are mainly used in frames or in large openings. Doors and window lintels are beams not exceeding 2. 1m. Any lintel more than 2.1m in length should be regarded as a beam and designed as such. Beams are designed mainly at may be used. However, when heavy shear is experienced, high yield high tensile bars may be used to advantage.

f. When beams are subjected to torsional moments such as roof gutter beams or balcony beams, the following additional design is required.

1. Calculate the imposed torsional moment, T and the torsional stress vt from $Vt = \frac{2T}{b^2 (h-b/3)}$ *N/mm2*

11. Check if (v + vt) exceeds 0.8v feu or S Nzmm", if so, increase beam dimensions most probably the d~pth, h.

111.. Calculate additional links from

$Asv = \frac{T}{0.8xlm (0.95fyv)}$ mm use closed links typ~

IV. Calculate additional longitudinal reinforcements from

$ASl = \frac{Asv}{Sv} (X_1 + Y_1) \frac{fyv}{Fy}$ mm2

Where:

T = Torional

Sv - Spacinf of links along the .member

V· Shear force

V - Shear stress

Vc - Ultimate shear stress in concrete

Asv Cross sectional area' of shear reinforcement in the form of links

Fy - Characteristic strength of reinforcement

b - Width of section

d effective depth of tension reinforcement

(OYENUQA, 2000)

2.7.4 REINFORCED CONCRETE COLUMNS

A column is a vertical load-bearing member with the ratio of its internal dimensions less or equal to 4: 1, that is, the greatest lateral dimension not more than four times its least lateral dimension. When this is violated, the column is said to be wall. The primary function of a column or wall is to act as a vertical support to suspended members and to transit loads from t fundation below.

COLUMN CLASSIFICATIONS:

a. Short or slender: A column is said to be short when the effctive length is not more than 15 times its least internal dimensions for braced columns or 10 times for un-braced columns otherwise the column is said to be slender. Slender columns, in addition any axial load and moments, are subjected to moments due to slenderness. These are usually added to the imposed moments on the column and slenderness should be checked on both axis and vice versa.

   Effective length of a column is defined as B 10, where 10 is the actual length of the column and B is a function of the end restraints of the column and whether or not the column is braced. Values of B as advised in tables 3.19 and 3.20 of the standard.

   Clause 3.8.1.5 of the standard defmes braced columns as those laterally supported by wall buttressing etc. designed to resist all internal forces in that plane. It should otherwise be considered as un-braced.

Clauses 2.5 of BS 8110: part 2 (1985), discussed the analytical method of calculating the effective height of columns as follows:

1. Framed structures and braced columns. Effective height is caluclated from the lesser of: ,

   ie 10 (0.7+0.05 (%11+%12) :S 10 and

   10 (0.85 + 0.05%min) < 10

11. Un-braced columns, the lesser of:

    10 (1.0+0.15 (%11 +%12) and 10 (2.0 + 0.32%min)

    where:

    %11 = 'ratio of the .sum of the column stiffness to the beam at the stiffness at the lower end of a column.

    %12 = ratio of the sum, of the column stiffness to the beam at the stiffness at the lower end of a column.

    %min= Lesser of%11 and %12

    %min = lesser of%11 and %12:'f

In additonal clause 2.5.4 of part 2 BS8 110, discusses the rigorous analysis method of calculating column relative stiffness.

b. Axial, un-axial mid Bi-axial: In terms of load disposition, a column can be categorized as Axially loaded, uni-axially load and Bi-axially loaded.

An axially loaded is subjected to a concentric axial load. That is moments, in both x and y axes are practically insignificant. The total load is then supported by the comprehensive action of both the concrete and steel counterpart of the column, e.g a truly central column.

A uni-axially loaded column is subjected to an axial load and a moment in one direction (x or y axis). The moment in the other direction is assumed to be pracitically insignificant e.g most side column, but not all.

A bi-axially loaded column is a comer column is fact all coner columns are bi-axially loaded while side columns can be bi-axially or uni-axially loaded.

DESIGN PROCEDURES

a. Axially loaded columns: the axial force in a column at the ultimate limit state may be calculated in the absence of any other rigorous analysis like shear from beam calculation on the assumption that beams and slabs transmitting force into it are simply supported. The design procedures for axially loaded columns are as follows

1. Estamating the total axial load at the ultimate limit state

a. Choosing a trial size using this table as guides.

TABLE 2.4

	Size (Hxb) in mm
N ::;500	225 + 225
500::;N < 700	300 + 225
700 ::;N ::;950	300 + 300
950::; N < 1050	450 + 225
1050::; N < 1400	450 + 225

1400:S N < 2100 450450

:S 2100 Choose appropriately

(Oyenuga, 2000)

111. Checking for slenderness

IV. Calculate area of steel required from

Asc N-0.35fcu bh

0.7fy-0.35fcu

When Asc returns negative value, mnumum steel of 0.4% bh must be provided. This should however, not be less than 4-12cm diameter bars for rectangular columns or 6-12mm diameter bars for round columns.

v. Providing links which should be a minimum of Y4of the size of the largest compression bar at a spacing of not more than 12 times the size of the smallest compression bar. It is unusual to adopt 100mm bars as links at a spacing of 200mm for 225 by 225mm columns. it is also advisable not to use less 4No. 16mm diameter bars for any column except the columns load is purely nominal in which case 4nos. 12mm diameter bars can be considered.

b. Uni-axialloaded columns: The design procedures are as follows:

i. Estimate load on the columns as in axially loaded and chose size.

ii. Estimate the imposed moment on column from

Mcol-M Kcol

3 Kcol + 3K beam.

111. Check whether column is short or slender and calculate Madd if found slender Add Madd to the approriate memont or moments

IV. Calculate Nand M

Fcubh Fcubh2

v. Use the charts in 25 to 29 to pick area of steel required

VII. Provide links as appropriate and detail your design

c. Biaxially loaded column: the column is converted to uniaxially loaded column and design as converted. The design procedures are:

1. Calculate loads and moments on column as before and choose column size

11. Covert Column to uniaxially loaded column of calculating increased moment from:

Mx Mxx + BhbMyy: when Mxx/b> Myy/b

Or

My = Myy + B = 1.0-1.6440 and 0 = N

Fcubh

Subject to a minimum of 0.3 when 0 > 0.6

111. Calculate N/fcubh2 and Ml x Ifcubh2~ M'y/fcubh/ and choose Reinforcements from appropriate chart.

IV. Choose links and detail your design.

(OYENUGA, 2000)

2.7.5 REINFORCED CONCRETED FOUNDATION

All structures must be founded on one form of foundation or the other, depending on the nature of the founding soul and the load to be supported.

Foundations vary from the simple strip footing to complex and more reliable pile foundation. Single storey (bungalow) buildings on average to good soil can be founded on strip foundations.

Three story buildings and more on poor soils require foundations ranging from simple pad to pile foundations. Must multistory buildings (6-storys and above) are founded on pile foundations.

Types of foundations:

a. Strip Foundation: Strip foundation provides a continuous ground bearing under the load bearing walls. This type of foundation is placed centrally under the walls and is generally composed of plain concrete often to a mix of 1:3:6 by volume with the thickness being note less than the projection of the' foundation and in no case less than 150mm. in this country, a mix of 1:2:4 by volume is recommended and for walls of 225mm, the width of the strip should be 3B, where B is the wall width and the thickness should in no case be less than B.:
b. Wide stripe foundations: Where the load bearing capacity of the ground is low for example, in marslay ground, soft clay or made up ground, wide strip foundations may be used to spread the load over a large area of soil. It is usual to provide traverse reinforcement to withstand the tensile stresses that will arise. The depth below the ground level should be the same as for ordinary strip foundation.

All reinforcement should be lapped at comers and functions should be any danger of the foundation failing as a beam in the longitudinal direction, it may be necessary to use a reinforced inverted T beam.

c. Pad foundations: These are isolated foundations to support columns. The area of foundation is determined by dividng the column load plus the weight of the foundation, at serviceability limit state, by the allowable bearing pressure. The thickness of foundation may be less than the projection from the column provided the various criteria governing the design of such foundation are met. Pad foundations include the isolated pad footing combined footing and strip footing supports a row of columns.

   Strap Foundations: Strap footing is similar to combined footing, except that a strap beam is constructed to link the two columns. Straps foundation is used when one of the columns is close to either the property line or on obstruction to the extent that projection of the footing beyond the column face.becomes practically impossible.

d. Raft foundation: these cover the Whole area of the building and usually extend beyond it. They consist primarily of a reinforced concrete slab up to 300illm thick, which is often thickened under load bearing colums or walls. Raft foundations are best suited for use on soft natural ground or fill, or on ground that is liable to subsidence observable in mining area. Design of the raft involved the calculation of the loads to be carried and careful assessment of the disposition and distribution of these loads. The primary advantage over strip foundations is the ability of the raft foundation to act as a single unit, thus eliminating differential settlement. However they are expensive.

e. Piled foundations: These are frequently used with multi-storey buildings and in cases where it is necessary to transmit the building loads through weak and unstable solid conditions to a lower stratum of sufficient bearing capacity. Piles may be classified in several ways with end-bearing piles, the shaft passes through soft deposits and the base or point rests on bedrock or penetrates dense sand or gravel, and the pile acts as a column.

A friction pile embedded in cohesive soil (often firm clay) and obtains its support mainly by'the adhesion or skin friction of the soil on the surface of the' shaft. Another method of pile classification relates to displacement poles where soil is forced out of the wall as the pile is driven, and 'replacement' piles where the hole is bored and excavated in the soil and the pile is formed by casting concrete or cement grout in the hole. Preformed solid piles of timber or reinforced concrete, and concrete or steel tubes or shells with the lower and closed are examples of displacement pile.

The choice of pile depends on ~he site and soil conditions, economic considerations and structural requirements. Sometimes, piles are linked by beams to carry load bearing walls.

The foundation type to be chosen for a particular structure or building as earlier on mentioned depends largely on the loads to be transmitted and the receiving soil strata, and must satisfy the following two fundamental and independent requirements.

~ The factor of safely against shear failure of the supporting soil must be adequate and; the settlement should neither cause any unacceptable damage nor interfere with the function of the structure.

~ Thus, the bearing capacity of the soil must be determined through the process of soil/geotechnical investigations prior to the design of the foundation. However, for a relatively small building (a bungalow or two storey building) to be built on a relatively firm soil, the structural engineer may use his experience to choose foundation type.

TABLE 2.5

Soil type	Bungalow	2-storey	3 to 5 story	Medium rise	High rise
Good soil> 100 *KN1M2*	Strip	Strip	Pad	Pad	Pile
Average soils 75 – *100KN/m2*	Strip	Wide strip	Pad	Pile	Pile
Poor soil 40 - *75 KN/m2*	Wide strip	Wide strip	Raft	Pile	Pile
Bad soil < *40KN/m2*	Slab raft	Beam & slab raft	Beam & slab raft	Pile	Pile

Note:

1. It is assumed that the walls of the bungalows and the two-storey buildings are load bearing walls. Where this is not applicable, the building should be framed and pad foundation or raft foundation used as appropriate.
2. When pad or raft foundations are involved, the building must necessarily be framed so as to provide a rigid structure.
3. The two-storey building is assumed to be for residential purposes, where it is to be used as office complex, it should be framed and pad or raft foundation used depending on the soil bearing capacity.

DESIGN PROCEDURES OF STRIP FOUNDATION

a. Determine maximum width of slab/roof to be carried by the wall. That is study the architectural plan and select a wall supporting the widest area.
b. Determine the loads due to slab and or roof statically.
c. Determine wall loads including the section below the ground floor slab.
d. Allow 1.0 wide as ground floor interactive load widths.
e. Sum up all the loads and convert to service loads. A factor of 1.47 to 1.49 may be used for residential occupancy,' 1.47 is more appropriate.
f. Divide the service load by the soil bearing capacity.

2. WIDE STRIP FOUNDATION

Design procedure

a. Repeat step (a) to f above
b. Should the width obtained to less or equal to 3T, design as simple strip foundation otherwise proceed as follows .

1. Determine the net pressure from:

$$F_{net} = \frac{w \times 1.10 - 24T (1.4)}{P_{prov}} \ KN/m^2$$

11. Calculate the moment from:

$M = 0.5 \times P_{prov} \times F_{net}$ KN.m

111. Design for base like a slab using d = T-60mm

IV. Detail the design

3. PADFOUNDATIONS:

Design procedure:

a. Determine the column load at ultimate limit state KN

b. .Calculate area of base required from:

$$A_{req} = \frac{W \times 1.10}{1.47 \times P_b} m^2$$

(b) , Calculate area of base requiredfrom:

$$A_{req} = \frac{W \times 1.10}{1.47 \times P_b} m^2$$

(Note: 1.47 is the converting factor to SLS, the range is 1.47 to 1.49)

(c) Select base size such Aprov2: Areq,Aprov= Area of Base provided

(d). Calculate net pressure at ultimate limit state from

$$F_{net} = \frac{W}{A_{prov}} = x \ 1.1 - 24h \ (1.4) \ KN/m^2$$

(e) , Cheek for punching shear from:

Perit = (Za, + a2) + 3h

Acrit = (a, + 3h) (a2 + 3h)

Where al and a2 are column dimensions and h is depth of base

V punch = fnet (Base area - Acrit)

$$V_{punch} = \frac{V_{punch}}{P_{erit}} \quad Nzmm'', \ d=h- (1.5h)$$

Obtain Vc from table 3.9 BS8110 should V punch> Vc, increase depth by 50mm.

(f) Design for reinforcements from

Moment = 0.512 x fnet and 0.512y fnet KN.m

Where Lx, Ly are the spans of base

As = M mm, as in beam or slab design

0.95fylad

g. Check for shear stress from

V - LsfnetKN

$V \quad \frac{V \times 10^3}{1000d} N/mm^2$

V should be less than Vc from table 3.9 BS8110 otherwise increase h by 500mm or increase the area of reingorcements provided and calculate for Vc agam.

For continuous or combined footing, there may not be the need to check for punching shear. Moments and direct shear analysis may follow that of continuous beam analysis.

(Oyenuga, 2000)

2.7.6 REINFORCE CONCRETE STAIRCASE

The primary function of a stair is to provide access from one floor to another. It is therefore a set of steps comprising treads (horizontal part) and risers (vertical part.) Going is defined as the distance of the tread paralled to the flight direction while rise is the vertical distance of the riser. A step consists of a riser and a tread. The height difference between the floors divided by the rise gives the numer of steps required between the floors. Height of risers should be the same as much as possible as well as the going of the treads. For comfortable usage the best proportions of step is

When:

Going + 2 times riser = 580 to 600mm.

For public building, thebest step is achieved when the going is 300mm and the rise is 150mm. The riser of a private dwelling stair can be increased to 175mm but value higher than this are not recommended. Going or treads should be equal and should be able to take ~man's foot conveniently, that is not less than 25mm. 'a going of 250mm to 275mm is recommended for private dwellings.

A vertical headroom of at least. 2.0m must be provided above the line of nosing. The pitch line is the imaginary line joining the finishing nosing of the steps. A maximum total of 18 steps is recommended without any itnervening landing, A flight that will involve more than, 18n6 steps should be broken with an intervening landing to provide a "resting place" for the users. Public stair should have a width not less than 1500mm while that of domestic dwelling can be a minimum of 90.omm.stair must be provide with guides known as balustrades. The vertical members are called balustrades while the top-slanting member is called the handrail. The standard heigh of balustrades is 840mm above the line of nosing.

Type of stairs:

Stairs can be constructed of reinforced concrete, steel and timber. Timber stairs are becoming obsolete and can, only be seen in old buildings while steels stairs are some how restricted to spiral stairs. Hence, most stairs in common use today are constructed of reinforced concrete. Irrespective of the materials for construction.

a. Straight flight stair two flights between the two floors to accessed
b. Half tum (180°) stair two flights between the two floors

With an intermdiate landing known as half landing. This stair is also known as dogleg stair.

c. Quarter tum (or open well) stair three flights between the two floors with two intervening landings. This creates a big opening between flights 1 and 3 and hence the name open well stair.

d. Free standing or scissors stair similar to half tum but with suspended half landing. That is the half landing is supported by the two flights. This calls for a more rigorous design for torsion at the two ends of the stair. Such stairs are mainly reinforced top and botton and with heavy reinforcements in the region of Y20@ 150mm c/c depending on the span and loading configuation.

e. Helical stair: This is usually common in building of the affluent, occupies less space than straight flight and the shape looks a helix from floor to floor. It is always a straight flight but turning as it rises. Torsion at the ends of the stair must be adequately taken care of. For dometic ubildings, reinforcements should not be less than Y 16mm @ 200mm c/c top and bottom, and if possible, the top and of the stair should be received by a beam.

f. Cantilever stair - in this type of stair, there is a central reinforced concrete wall and each step cantilevers out of this wall. The various Iandings are designed as double cantilever about a beam that in tum cantilevers out of the central cove wall.

In most cases this type of stair is used as external escape stair. This type of stair is elegant and do no occupy space. It gives perfect finish to the steps are pre-cast bolted to the horizontal portion (tread) of the spine beam. The steps can be constucted of steel. Timber or reinforced concrete.

g. Spiral Stair: this is the most economical in terms of space utilization and cost. It consists of several cantilever steps j otting out like leaves from a 'central column. The' final landing spans between this column and the adjoining walls. Each of the steps is designed as a simple cantilever and it is customary to taper the step, being wider at the free end. The steps are pre-cast with a central hole of the same diameter (with some clearance) as the central column. The steps are arranged to form a spiral around the central column. There are attempts to construct spiral stairs in site but they generally appear rough after finishing.

DESIGN OF STAIRS

Stairs are designed either traverse or longitudianlly. Traverse design involves the design of each individual step and such is common with spiral, cantilever stair and steps spanning between two raked beams or ~·alls. In this case, the loading is for each steps including the live load and the step is designed as a rectangular beam with width, be, equal to the going of the state and depth, h. on the other hand, longitudinally designed stair are design like slab with the waste as the depth, h. these stairs include the straight flight stair, half rum and quarter tum stairs, helical stair and scissors stair etc.

Since the truly horizontal·distance will be used, the load must be converted by multiplying all inclined loads (waist own weight and fmishes)

by: $\frac{\sqrt{R^2+T^2}}{T}$

Where R is the rise of the step and T the going. When such flight pans between two beams, the moment is cualculated as $M = 0.125WL^2$, where, W =the flight uniformly distributed load and L the span.

i" (Oynuga 200)

2.8 ANALYSIS AND DESIGN OF REINFORCED CONCRETE FRAMES

In-situ reinforced concrete structures behave as rigid frames, and should be analyzed as such. They can be analyzed as a complete space frame or be divided into a series of plane frames. Bridge deck types of structures can be analyzed as an equivalent grillage, whilst some form of finite element analysis can be utilized in solving complicated shear-wall buildings. All these methods lend themselves to solution by the computer, but many frames can be simplified for solution by hand calculations.

The general procedure for a building frame is to analyze the slab as a continuous members supported by the beams or structural walls. the slabs can be either one-way spanning or two-way spanning. The columns and main beams are considered as a series of rigid plane frames, which can be divided into two types.

1. Braced frames supporting vertical loads only
2. Frames supporting vertical and lateral loads.

Type 1 frames are in building where none of the lateral loads, including wind, are transmitted to the columns and beams but are carried by shear walls or other forms of bracing.

Type 2 frames are designed to carry the lateral loads, which cause bending, shearing and axial forces .in the beams and columns. for both types of frame the axial forces due to the vertical load in the columns can normally be calculated as if the beams and slabs were simply supply supported. (Bungey 1990).

CHAPTER THREE

STRUCTURAL ANALYSIS AND DESIGN

GROUND FLOOR SLAB

3.1 Ribbed slab OS-1 (Gs - lA =L varied) load

Partitions-a) 230mm thick hollow concrete block stone aggregate. S.OOm ,

2.87 x 21_ = 3.30 kN/m^2

20

BS CP3 Reinforced codesign hand h book Reynolds ta table 2.2

b) 100m thick hollow concrete block

2.87 x lQ_ = 1.5kM $1m^2$

20

Fl

1 3.150 ; 3.150 i

6300

Deadload - per rib of 520mrn q2= 47.43kN/m2

q2= 47.43kN/m 2

Finished floor layer - 0;05 x 20 - 1.00 kNl m2

Slab - 0.1 x 24 i 2.40 "

Ribs - 0.12 x 0.20 x 24 = 1.11 kNl m2

0.52

15mm concrete.plaster=- $0.015 \times 22 =$ $0.33kN/m_L$

Pots - $0.077 =$ 0.74

1.4x $5.58kN/m^2$

$= 7.8kN/m^2$

Live load (Lobby, stores)

$Qk = 1.6 \times 5.0 =$ $8.00kN/m^2$

q, =Gk+ Qk

=7.8+8.00= $15.81kN/m^2$

Q, $15.81 \times 0.52 =$ $8.22kN/m$

F, = $(1.5 + 0.53) \times 2.85 \times 1.4 \times 0.52 =$ 4.21kN

Height of wall.

Moments

$$Ml = \frac{q,.t\_}{8} + \frac{EI}{4}$$

BS8110

I~able 3'.14

Reynulds Hlbook

$$= \frac{8.22 \times 6.3^2}{8} + \frac{4.21 \times 6.3}{8}$$

$$= 40.78 + 6.63$$

$$= 47.41 \text{ kNm}$$

b = 520mm, d = 300 - 20 - 8-8 = 264mm

few = 30, fy = 410 N/mm^2

$$= \frac{M}{bd^2 \, feu} = \frac{47.41 \times 10^6}{520 \times 264^7 \times 30} = 0.04 < 0.156$$

= z/d = 0.946 :.7 = 0.946d

$$As = \frac{m}{0.87fyz} = \frac{47.41 \times 10^6}{0.87 \times 410 \times O.946 \times 246} = 530mm^2$$

Provide 2Y20 ($628mm^2$) bottom

1Y 10 (hanger bar) top

.l.

3.2 Ribbed slab, Gs - 2 L = 6.000

BS CP3 Reynolds RIB table 2.3	Os - 2a Gs - 2b Gs - 2c Gs - 2d Gs - 2e

LOADS

a) Dead load - From above *7.81kN/m2*

b) Live load - PI ==5.0 x 1.6 4.00 kNm2

c) - P2 ==5.0 x 1.6 8.00 kNm2

d) Partitions - (equivalent uniformly distributed load

BSCP3 (Bs. Cpg)

lOr

Unknown position We = 0.33 (1.5 + 0.53) x 2.80 x 11.4 = 2.63 *kNI* m2

Reynolds's ConI. Delyor Hlbook table 2.2

e) partition parallel to rib (1OOmmthick) (1.5+0.52) x 2.8 x 1.4 = *7.96kN/m*

(f) partition perpendicular to rib (100m)

FI ==(1.5 + 0.53) x 2.80 x 1.4 x 0.52 = 4.l4kN

(g) = partition parallel to rib (230mm thick)

f2 ==(3.3 + 0.53) x 2.8 5x1.4 x 0.52

==7.95kN

h, - partition paralled to rib (230mm thick)

(3.3 +0.53) x 2.80 1.4

F3 ==i5.28 KN *1m*

For worst loading conditions, consider the following.

Alt 1 - a + c + d

ql = (7.8 +8.00+2.56) x 0.52 ==*9.6kN/m*

$m_1 = \frac{L.G}{8} = \frac{9.6x62}{8}$ ==43.2kN

Alt 2 - a + b'+ e + f

$q2 = \frac{.921}{848} + Ed = 17.85 \times 62 + \frac{4.1x6}{4}$

= 86.5kwm > 42.3kN

Alt 3 -

q3 ==a+b+g

= (7-85+4.0) x 0.52 +9

3.4 Ribbed Slab (Gs-4)&(Gs-4 a)

Gs-4b

ql = 15.8 x 8.22kNI25.81 x 0.52 = *8.22kN/m* 106lines & corridors

1 J

m = i._£ = 8.22 x 3.82

8

As = 2Y12 (180mm2)

3.5 Ribbed slab (Gs - 5) 1.5m

Provide 2YI0 (157mm2)

GROUND' FLOOR BEAMS

3.6 GB - 1 : L = *8048*+8m, Beam size = 400x600m

ql = 47.74kN/m -

~ ..J:ftgmar IOW q2= 47.43kN/m2

RA Rb

8480

LOADS

From slab Gs - 2b

230mm blockwall- (3.30 + .0.53) x 265 x 14 = 14-21

Beam slab weight - *OAxO.30x24x1.4* = 4.03

ql = *47.74kw/m*

from slab Gs - 2b

q2 = 15.81 x 6 x Y2 = 47.43kNm

b = 400mm d = 600-20-8-10 = 562

$K = \frac{M}{} = \frac{598*10^6}{30*400*562^2} = 0.158 > 0.156$

Provide Compression reinforcement

$A1s = \frac{M - 0.156\ fcubd^2}{0.89fy(d - d')}$ $d' = 20 + 8 + 10 = 38mm$

$= \frac{(598-591) * 10^6}{186911} = 37mm^2$

*Provide minimum reinforcement (0.2%*400*600mm=480mm²) 3Y16Top*

$As = \frac{0.156\ fcubd^2}{0.87fyZ} + A1s$

$= \frac{591*10^6}{0.87*410*0.775*562} + 37$

$= 3841mm^2$

Provide 5Y32 Bottom (4020mm²)and 3Y16 Top

SHEAR

Max V= RA = 335kN

$v = \frac{335*10^3}{400*562} = 1.49N/mm^2$

$\frac{100As}{bd} = \frac{100*4020}{400*562} = 1.79$

$v_c = 0.81N/mm^2$ by interpolation

$v_c + 0.4 \leq v < 0.85fcu$

m, = 6-16 x 36 + 7.95 x 9

0 4

27.72 +11.93.

39.6 < 86.5kNm

Alt 4 a+b+h

q, = (7- 85 +4.00)x 0.52 + 15.28

= 6.16 + 15.28 = 21.44kwl

M4 = g1_f = 21.44 x 36 = 96.48 kNm

8 ,8

96.48 »ı 86.5kNm

Hence Alt 4 is the worst condition

K =_m_ = 96.48 x 10^6 = 0.09

bd^2fcu $520x26x4^2x30$

z = 0.887d

As = 96.48 x 10^6 1155mm^2

0.87x410xO.887x264

Provide 4Y20 (1260mm^2)

B BS 8110
Ta table 3.14
R Reynold BIB

V_4 = 21.44x6 = 64-32

V = y_ = 64.32 x 10^3 = 2.03 < *500N/mm^2*

bd 120 x 264

vc = 1.08 x 1.66 = 1.14

3.3 Ribbed Slab Gs - 3 (L = Varries from 5.00m per rib of 520mm *c/c*

LOADs

From slab - ql = 15.81 x 1 x 0.52

= *8.2'2kN/m*

8.22kN/mm

S.OOm

B BS8110
,_J/T table 3.30
R Reynolds
HIB book

MI = *glX* = 8.22x5^2 = *25.69kN/m*

8 8

As = 2Y 16 (40$_2$$mm^2$)

VI = 8.22 x *5/2* = 20.55kN

RA = 47.74x8.48$_2$ + *(8.48x46.43)x1/3* *(8+8)*

8+8x2 8.48

= 296.88kN

RB = 12ql (+ *2/3.* q2 1 = *Yz47.74* x 8+8

,= 403.5kN

Position of BMmax

RA - 47.43 X 12 - 47.74X

8+8

270 = 2.80X2 - 47.71X

X2 - 17.05X - 96

- 17.05 + 17.05 - 4X1.96

2Xl

= +386

4.48M

4.48X270 - 47.74X4.482 -

Yz(5.59) X4.48X4.48

X 113 4~48

1209.0 - 479 - 84

647

Provide R8 @ 150% *(353mm2/m)*

K = JJl = 647 x 106 = 0.23> 0.]56

Bd2fev 400x5622x30

Compression reinforcement in ratio

AI) = m - 0.156 fevb d2 d1 =20+8+ 10 = 38mm

0.87 fy (d-dl) d = 38mm

= (647 - 595.'48) x 10$_6$ = 51.52 x 10$_6$

0.47x410 x (562-38) 186910.8

276mm2 1446mm2

A) = 0.156 fcubd" + A12

0.87fyz

RA = 47.74x8.48 + *(8.48x46.43)xI/3* *(8+8)*

8+8x2 8.48

296.88kN

RB 12ql (+ *2/3.* q2 1 = 1247.74 x 8+8

= 403.5kN

Position of BMmax

RA - 47.43 X 12 - 47.74X

8+8

270 = 2.80X2 - 47.71X

X2 -17.05X -96

17.05 + 17.05 -4X1.96

2Xl

= +386

4.48M

4.48X270 - 47.74X4.48$_2$ -

12(5.59) X4.48X4.48

X *1/3* 4~48

1209.0 - 479 - 84

647

Provide R8 @ 150% *(353mm2/m)*

K = _ill = 647 x 106 = 0.23> 0.156

Bd'fev fl.00x5622x30

Compression reinforcement in ratio

AI3 = m - 0.156 fevb d2 dl =20+8+ 10 = 38mm

0.87 fy (d-dl) d = 38mm

= (647 - 595:48) x 10$_6$ = 51.52 x 10$_6$

0.47x410 x (562-38) 186910.8

276mm2 1446mm2

A3 = 0.156 fcubd" + A12

0.87fyz

= 595.18xlOb + 276

0.17x'410xO.775d

= 3833 +6276

4109m2 = 100As Ibh:s 40 ~ 0.13

SHEAR

Max v = 404.5kN

V = v/db = 403.5x103 = *1.79N/mm2*
400 x 562

100As = 100x5630 = *2.50N/mm2,* ve = *0.91N/m2*
bd 400 x 562

(Ve +0.4) <v < 0.8 fey

Asv = b(v- V~ = 400x(1. 79-0.9) = 532 = 162
Sv 0.87fyv 0.87x250 217.5

As = 1.62

Provide RB @ 125% in pairs i.e double.

3.7 BEAM GB - 2 = 840m2, Size = 400 x 600mm

L = 3.98 + 0.9+ 1.2x2.4+8.48m

q, *47.43kN/mm* --

91 = *55.88kN/m*

RA Rs

q2 = 28.9.3 8.48m

Loads

From slab 05 - 1 - (15.81 + tl)x 6.3 = *51.85/w/m*
6.3 2

Self weight O.4xO.3x24xl.4 = 4.03"

q, - 55.88"

From slab 05 - 4a (l5.81_I)x 312 = *24.90/w/m*
.8 2

". Self weight = 4.03"

. ~.. .:I?<".

Bm_{max} occurs where shear force is

28.95kN/M

55.88kN/M

R.=3J5kN

R.=276kN

zero N

By similar *triangle47.43/8.48=y/x*

.y=5.59x

SF is zero at section N-N, xm from RB to find x

*276-55.88x-1/2(x*5.59x)*

276-55.88x-2.8r

.x2+19.96x-98.5

..x= [-b± >J(b2-4ac)]/2a

X= *[-19.96±>J* (19.96)2-4*1 **(-98.57)]i2** 1

X=[-19.96±28.15]/2

. *Xl=[-19.96+28.15]/2*

Xl=4.10m

Bm_{max} = 276*4.1 - 55.88 * 4.12 - 5.59*4.1 *4.1 * 1*4.1

2 2 3

= 1132 - 470 - 64 = 598kNm

$\frac{598*10^6}{30*400*562^2} = 0.158 > 0.156$

Provide Compression reinforcement

$A1s = \frac{M - 0.156\ kubd^2}{0.89fy(d - d')}$ d' = 20 + 8 + 10 = 38mm

$= \frac{(598-591) *10^6}{186911} = 37mm2$

*Provide minimum reinforcement (0.2%*400*600mm=480mni) 3Y16Top*

$As = \frac{0.156\ kubd2 + A1s}{0.87fyz}$

$= \frac{591*10^6}{0.87*410*0.775*562} + 37$

$= 3841mm^2$

Provide 5Y32 Bottom (4020mm2) and 3Y16 Top

SHEAR

Max V= RA = 335kN

$v = \frac{335*103}{400*562} = 1.49 N/mm^2$

$\frac{100A_s}{bd} = \frac{100*4020}{400*562} = 1.79$

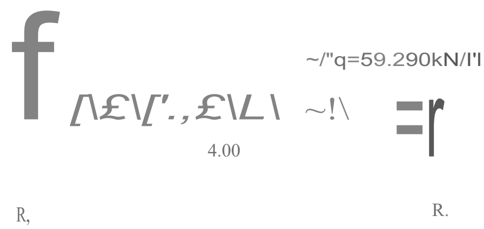
$v_c = 0.81 N/mm2$ by interpolation

$v, + 0.4 \sim U < 0.85fcu$

$$A_{Sv} = \frac{b\,(v - v_c)}{0.87 f_{yv}} = \frac{400\,(1.49 - 0.85)}{0.87 * 250} = \frac{628}{218} = 1.24$$

. *Provide Stirrup R8 double links @J60c/c* *R8@160c/c*

3.8 BEAM GB-9; L= 4.00m, Size = 400x300mm



LOAD

from slab Gb-5 = 0.5*15.81 * 1.5 = *11.86kN/m*

From slab Gb-2 = 0.5*15.81 * 6.0 = *47.43kN/m*

59.29kN/m

REACTIONS

RA = RB = 59.29''' *4/2* = 119Kn

3.9 BEAM Gb-3, L = 6.8 m size (0.4 xO.6m)

x

q. =4 7.43kN/M

q. =27.7SkN/M

x

x

2.120M

4.680M

By similar triangle

47.47/4.68=y/x

v- 10.13x

LOADS

From slab Gs- 4b = 15.81 * ~(3) = 23.72 *kN/m*

Self weight = 0.4 * 0.3 *24 *1.4 = 4.03 *kN/m*

.ql= 27.75 *kN/m*

From Slab Gs-2b; q2= 15.81 * 112(6.00) = *47.43kN/m*

.q2 = *47.43kN/m*

From Slab Gs - 5; q3 = 15.81 * ~(1.5) = *11.86kN/m*

F= 119 =from beam Gb - 9

REACTIONS

I:Ms = 0

6.8 RA= 27.75 * ~(6.62) + 1'2 (2.12 * 11.86) (1.41+4.68) + 1'2 (4.68 * 47.43)

+ *2/3* (4.68) + 119 * 4.68

$= 641.58 + 76.56 + 346.28 + 556.92$

$RA = 238.43kN$

$RA + RB = 27.75 * 6.8 + 119 + 12(2.120) * 11.86 + 12(4.68 * 47.43)$

$= 188.7 + 119 + 12.57 + 11.69 = 432kN$

$RB = 432\ 238.43 = 194kN$

Position of Bmmax= SF = 0

$RB - q.x - (47.43\ x^2)/4.68*2 = 0$

$$194 - 27.75x\ \frac{47.43\sim}{4.68*2} = 0$$

$.x^2 + 5.47x - 38.26$

$$.x = \frac{-5.47 \pm \sqrt{(5.47^2 - 4*1*38.26)}}{2*1}$$

$\therefore\ x = (-5.47 + 13.5)*0.5 = \sim.03m$

Bmmax

$RB*4.03 - 12(4.03) * (10.13*4.03) * 2/3\ (4.03) - ql*(4.03iI2 = 0$

$= 194 * 4.03 - 10.13*(4.03^3)/3 - 27.75 * 6.80^2/2$

$= 781.82 - 221.01 - 94.35$

$= 466kNm$

$$K = \frac{M}{f_{cu}bd^2} = \frac{466*10^6}{30*400*562^2} = 0.13<0.156$$

$z = d\ [0.5 + \sqrt{0.25 - k/O.9}]$

$z = d[0.5 + \sqrt{0.25 - 0.12/d.9}]$

$z = 0.84d=562mm$

$$As=M = \frac{466*10^6}{\quad} = 2767mm^2$$

q2 - *28.93kN/m"*

From slab G5 - 2b

ı 15.81 '+_Q_ - *47.43kN/m*

2

RA = 28.9x8.482+(55.88 -28.95)x(8.48-2.4)2+ 47.43x8.48x2 8.48

8

= 1041 +498+ 1P7=335kN

RB= 47.43x 8.48 +28.95x8.48+(55.8628.95)x6.08

2

= 201 +245.5 + 164 - 335

= 276kN

0.87fy z   0.87*410*0.84*562

Provide 5Y25 + 1Y20 (2769mm2) Bottom   *Provide 5Y25 + 1Y20*

SHEAR

Max (V) = 238.43kN

u= Vlbd= 236.43 *10^3 = *1.06N/mm^2*

400 *562

100As= 100*2769 = 1.23

Bd   400 *562

Mosley Table 5.1 uc = *0.71N/mm^2*

uc + 0.4 = 1.12

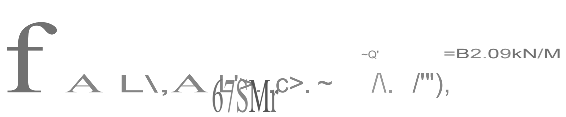
u < Vc + 0.4 < 0.85fcu

Asv = b (V-VC) = 400 (1.06 -0.71) = 0.63

0.87fyv   0.87 *250

Provide R8@275c1c doubled   *Provide R8@275c1c double*

3.10 BEAM GB-4; 1=6.750msize = 400*600mm



LOADS

From Gs-4b;   15.81 * ~(3.80) = *30.00kN/m*

From Gs-2e;   15.81 * ~(6.08) = *48.06kN/m*

= *78.06kN/m*

3.11 BEAM GB - 5; L ::;6.30m, 400x600mm

LOADS

From Slab GB- 4 15.81*3.6/2::;: *30.04kN/m*

Self weight of beam 0.4*0.3*24*1.4= *4.03kN/m*

.Q2 = from 9" wall parallel to beam

::;(3.3+0.53) *2.65 * 1.4 = *14..21kN/m*

F = (3.3+0.53) *2.65 *(0.4/0.52) * 1.4 = 11.8kN

REACTIONS

Rl = *[34.03*(6.32)/2+* 11.8*3.15 + *14.21*3.15*4.725]/6.3*

R_1= [675.53 *+37.17+211.50]/6.3*

R2 = 34.03*6.3 + 14.21*3.15 + 11.6 - 147

Position of the Bm!!ll!l§:

Self weight of Beam =0.4*0.3*24*1.4 = *4.03kN/m*

ql = 82.09kNm

''''''

RA = (82.09 *6.75$_2$)/2= RB= 277kN

Bmmax=ql 12/8 = (82.09 *6.75$_2$)/8= 468kNm

K = M = 468*10$_6$ = 0.12

~ubd2 30*400*5622

Provide same reinforcement as beam Gb-3 *Provide SY2S+1Y20*

SHEAR

V=227kN

v = V/bd= (277*10$_3$) 1(400*562) = *1.23N/mm$_2$*

100As = 100*2769 = 1.23

Bd 400 *562

U = *0.71N/mm$_2$*

Asv = b (v-vc) = 400 (1.23 -0.71) = 0.95

0.87 *250

Provide R8@22ScJc Doubled *Provide R8@22ScJc Doubled*

F:1I.BkN

147

147 - 34.03x - 14.21x = 0

X = *147/48.24* = 3.05m

Bmrnax= 147*3.05 - 34.03 * *(3.05^2)/2* - 14.21 *(3.05^2)/2*

= 448 -158.3 - 66.10

= 224kNm

K= M 224*10^6 =0.06

~ubd2 30*400*5622

z = d [0.5 + "-i(0.25- k/O.9)]

z = 0.93d= 522rnm

As = M = 466*10^6 = 1203mm2

0.87fy z 0.87*410*0.84*522

Provide 4Y20 (1260mnl)

Provide 4Y20 (1260mnl)

SHEAR

Vrnax= 147kN

v = V/bd= *(147*10^3)/(400*562)* = *0.65N/mm^2*

100As = 100*1260 = 0.56

Bd 400 *562

By interpolation

72428

= 2665 + 2929 = 5594mm^2

Provide 8Y25 + 6Y20 (5820mm1) *Provide 8Y25 + 6Y20 (5820mm1)*

SHEAR

V=285kN

Provide R8@225mm double as in Gb- 4

3-13 BEAM GB-7; 1=6.00m Size = 600x300

47.43kN/M

RB

LOADS

G_s-2, *15.81*6/2=47.43kN/m=ql*

G, - 2c, *15.81*612 = 47.43kN/m*

" RA= [47.43*(6^2)/2 + *47.43*6*112*.2/3*6]*116* = 237kN

RB= *47.43*(6/2)* + 47.43*(6/2) - 237 = 94.71kN

Bm_{max} = *[(2qt2)/9"3]* = 0.125qt2

= *(2*47.43*6^2)19"3* + 0.125 * 47.43*6^2

= 219.04 + 213.44 = 432.49kNm

$v_c = 0.55N/mm^2$

Asv = b (v-vc) = 400 (0.65 -0.56) = 0.18

0.87fyv 0.87 *250

Provide R8@300clc

Provide R8@300clc

3.12 BEAM GB-6; L= 6.00m, 600x300mm

$R_A = R_B =$ (qll)/2 = (94.83*6)/2 = 284.5kN

Bm_{max} = (q_1e)/2 = 427kNm

K= M = $427*10^6$ = 0.37>0.156

~ubd2 30*600*2622

A1s = M-0.156fcubd2

= $(427 -193)*10^6$ = $2929mm^2$

79901

Provide 6Y25 (2950mm2) Top

Provide 6Y25 (2950mm2) Top

As= 0.1 56fcubd2 +A1s

0.87 fyz

= $193*10^6$ + 2929

K= M = 432.49*10^6 = 0.35>0.15

~ubd2 30*600*262^2

Compression reinforcement is required; i = 20+ 8 + 10 = 38mm

A1s = M - 0.156fcubd2

0.87 fy (d - d')

= (432.47 - 192.75)*10^6 =3000mm2

79900.8

Provide 6Y25 (2950mnr) Top

Provide 6Y25 (2950mnr) Top

As = 0.156~ubd2 + A1s

0.87 fyz

= 192.75*10^6 + 3000

72428

= 5661.3mm2

Provide 7Y32 Bottom (5630mml)

Provide 7Y32 Bottom (5630mml)

SHEAR

V=237kN

v= 237*103 = *1.51N/rrun2*

600*262

100As = 100*5630 = 3.58

bd 400 *262

v« = 0.97

Asv = b (v-vc) = 600 (1.51 -0.97) = 1.49

Sy ~?87fyv 0.87 *250

Provide R8@120C/c doubled

Provide R8@120C/c doubled

3-14 BEAM GB -7a 600*300mm

From Slab Gs - 2a and Gs - 2b

LOADS

.qi = 1~.81 * (6/2) *2 = 94.86kN/m

RA = 2/3ql = 2/3 (94.86*6) = 379.44kN

Rs = 1/3 ql = 1/3 (94.66*6) = 189.72kN

Bmmax= 2q12/(9-Y3)= 438kNm

Provide same reinforcement on Gb - 7

6Y25 Top, 7Y32 Bottom

Stirrups R8@120C/c doubled

3-14 BEAM GB - 8 and GB - 8b; L= 5.945 - *(1.770/2)* = 5.06m, Size = 500x300mm

=~5.00kN/M

L\.A,,~~~~

t

5.06",1

LOADS

From Slab Gs - 4 and Gs - 4b

.. = 15.81 * ~(3 + 3.8) = *53.8kN/m*

Self weight = 0.5*0.3*24*1.4 = *1.2kN/m*

0.74 + 2.4 + 1.11

.q, = (53.8+ 1.2) *kN/m* = *55.00kN/m*

--*' Rl = *[55*(5.06^2)/2]/5.06* = 139kN= R2

Mmax = *55*(5.06^2)18*

= 176kNm

K= M = 176*10^6 = 0.17>0.156

Compression reinforcement is required

A1s = *[176-0.156*30*500*262^2]/{0.87** 410* (262-38)} d1= 20 + 8 + 10 = 38mln

Provide 3Y12 Top (339mm^2) *Provide 3Y12 Top (339mm^2)*

As = [154.4 *10^6]/ (087*410*0.775*262) +270 z = 0.7750

= 2131.77 + 339 = 2471mm2

.Provide 4Y25 + 2Y20 (2588mnC) Bottom

Provide 4Y25 + 2Y20 (2588mm2) Bottom

SHEAR

v = Vlbd= (139*10^3)/(500*562) = 1.06N/mro2

100As = 100*2588 = 1.98

bd 500 *262

By interpolation U_c= 0.90

v_c + 0.4 = 1.23>1.06

Asv = b (v-vc) = 500 (1.06 -0.90) = 0.367

0.87fyv 0.87 *250

Provide R8@300c/c doubled

Provide R8@300c/c doubled

3.15 GB - 8c L=5.6m 600X300mm

kN/M

1.88M

RA~ 5.6_M ~RB

By similar triangle

y = (32.5/5.6)x= 5.8x

LOADS

From Gs - 4c, 15.81*3.96/2 = 31.30kN/m

From Gs - 4a, 15.81*3.15/2 = 24.94kN/m

From Gs - 1 (15.81 + *2.73)*6.3/2= 58.09kN/m*

Self weight = *1.2kN/m*

.. qJ = 31.30 + 1.2 = *32.5kN/m*

. q2 = 24.94 + 1.2 = 26. *14kN/m*

.q, = 58.09 + 1.2 = *59.29kN/m*

5.6Ra = *12(5.6*32.5)*2/3(5.6) +26.14*1.8*4.7+59.29*3.72*$_2$/2

Ra = (339.7 + *221+410)/5.6* = 173kN

RA =*12(5.6*32.5)*2/3(5.6) +26.14*1.88*4.7+59.29*3.72*$_2$*/2* - 173

RA = 91 + 221 + 49.1 -173 = 188kN

Position ofBmlI!M

RA - 59.29x - 5.80x *1I2x = 0

188- 59.29x-2.9x2

x2 + 20.44x - 64.8 = 0

={ -20.44±" [20.442 - 4*1 *(-64.8i]}/2*1*

= {-20.44±"418 +*259.2}/2*

= {- 20.44 + *26}/2* = 2.79m

Bmmax= 188*2.79 - 59.29*2.792/2 - 5.80 **2.79*$_2$*/2*113 (5.6)*

= 524.52 - 231.42 = = *252kN/m*

K= M = 252*10$_6$ = 0.24>0.156

Compression reinforcement required

ll '

$A1s = (252 - 154.4)10^6 = 1222mm2$

79901

Provide 4Y20 (1260mm1 Top

Provide 4Y20 (1260mm2) Top

$As = \{(154.4*10^6)/ (0.87 *410*224)\}+ 1222$

z=224mm

As = 3l54mm2

Provide 4Y25 +4Y20 (3220mm2) Bottom

Provide 4Y25 +4Y20 (3220mm2) Bottom

SHEAR

V = 188kN

$v = Vrbd= (188*10^3)/(500*562) = 1.44N/mm2$

lOOAs::::100*3220 = 2.46

Bd 500 *262

By interpolation UC = 0.97

Vc+ 0.4 = 1.37

Vc+ 0.4 < U < 0.85fcu

As_v = b (v-vc) :::500 (1.44 -0.97) = 0.86

o.szr; 0.87 *250

Provide R8@230c/c doubled

Provide R8@230c/c doubled

3.16 Beam GB - 9

L= 4.00m, size= 400x300mm

.~.

~0.49kN/"

~A ~~~

R ⊺ 4.00" ~ R B

From Slab Gs - 5, 15.81 *1.512 = 11.86kN/m

" r· Gs - 2, 15.81*6.012 = 47.43kN/m

Selfweight =1.20kN/m

q = 60.49kN/m

REACTION S& MOMENTS

·_ RA= Rs= ql/2 = 60.49*412 =121kN

~mmax=ql2/8 = 60.49* 42/8 !:: 121kN

K= M = 121*10^6 = 0.15<0.156

z = d [0.5 +" (0.25 - 0.15/0.9)]

z=205mm

As=M 121*10^6 = 1653mm^2

0.87fy z 0.87*410*205

Provide 3Y25 + 2Y20 (2098mm1)

Provide 3Y25 + 2Y20 (2098mml)

A

|

SHEAR

V = 121kN

$v = V/bd = (121 \times 10^3)/(400 \times 262) = 1.15 N/mm^2$

$$\frac{100A_s}{bd} = \frac{100 \times 2098}{400 \times 262} = 2$$

. By interpolation $v_c = 0.91 N/mm^2$

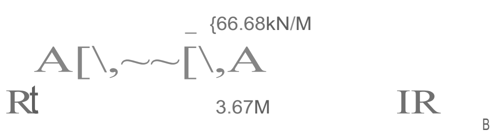
$$A_{sv} = \frac{b(v-v_c)}{0.87f_{yv}} = \frac{400(1.15-0.91)}{0.87 \times 250} = 0.44$$

Provide R8@300dc doubled

Provide R8@300dc doubled

3.17 GB-10

L= 3.67, 450x450mm



LOADS

From Slab Gb -1, *15.81*6.3/2 = 49.80kN/m*

Self weight = 0.45*0.45*2.65*1.4 = *1.60kN/m*

0.74+2.41+1.11

9" wall- *(3.3+0.53)*2.65*1.4 = 15.28kN/m*

q = *66.68kN/m*

Bmmax=qt2/8 = 66.68*3.672/8 =112kN/m

K = M = 112*10^6 = 0.05 d= 450 - 20 - 8 - 10 = 412)

tubd2 30*450*4122

z = d [0.5 + " (0.25 - 0.05/0.9)] = 388mm

As=M = = 809mm^2

0.87fy z 0.87*410*388

Provide 3Y20 (943mm^2) Bottom

Provide 3Y20 (943mm^2) Bottom

REACTION

V = 66.68*3.67^2/2= 122kN

100As = 100*943 = 0.51

Bd 450 *412

v = Vlbd= (122*10^3)/(450*412) = 0.66N/mm2

By interpolation UC = 0.51N/mm2

Asv = b (v-vc) = 400 (0.66 -0.51) = 0.31

S, 0.87fyv 0.87 *250

Provide R8@300cJclinks

Provide R8@300cJclinks

Max S, = 0.75d= 309mm

3.18 GB-ll; L= 2.695+0.225+0.1125 = 3.03m; Size = 250*300mm

. LOAD

From Slab Gb - 4 & Gb -4b

= 15.81 *(3.8+3.0)*1/2 = 53.75kN/m*

Self weight = 0.25*0.3*24* 1.4 = *0.59kN/m*

0.74+2.41+1.11 54.34kN/m

Bm_{max} = *wI2/8 = 54.34*3.032/8=62.4kN/m*

K= M = $62.4*10^6$ = 0.12<0.156

z = d[0.5 + "';(0.25- *0.12/0.9)]* = 262(0.65) = 222mm

As=M $62.4*10^6$

0.87fy z 0.87*410*222

Provide 4Y16 (804mml) Bottom

Provide 4Y16 (804mml) Bottom

SHEAR

V = 0.5*54.39*3.03 = 82.4kN

v = V/bd= $(82.4*10^3)/(250*222)$ = $1.48N/mro^2$

100As = 100*804 = 1.45

bd 250 *222

By interpolation *v; = 0.7SN/mm2*

Asv = b (v-vc) = 250 (1.484 -0.75) = 0.837

0.87fyv 0.87 *250

Provide R8@120cJc *Provide R8@120cJc*

3.18 **Gb-12** L= 4.0 +1.43+ 0.06 - 0.1125 - 1.5 = 3.85m, Size 22Sx300mm

IO.76kN

~--='2k=N7~n---'~

RA RA lRM lRM B

From slab Gb - 2, *15.81 *6/2 = 47.43kN/m*

Self weight = 0.225*0.3*24*1.5 = *0.53kN/m*

0.74+2.4+1.11

Weight of9" wall = (3.3 + 0.53)*2.85*1.4 = 28.07kNm

ql = 47.43 + 0.S3 + 28.07 = 76.03 *kN/m*

..Q2 = IS.81 * *1.5/2* = 11.86 = *12kN/m*

..q3 = 15.81 **1.5/2* = 11.86 k *12kN/m*

F = 9" wall, *15.68*1.5/2*0.9* = 10.76kN

REACTIONS

3.85RA = 7.6 *3.85*1.53+ 10.76 *1.27 + 12* 2.58*2.56 + *!h(1.27)*12*2/3(1.27)*

RA= 142.4kN

RB =7.6 *3.85+ 10.76 + 12* 2.S8+ 'l2(1.27)* 12 - 142.4

RB= 199.54= 200kN

Position of Bm_{max}

$RA - q1X - q2X = 0$

$142.4 (76+12)x = 0$

$x = 1.65m$

$Bm_{max} = RAx - Q1X2/2 - Q2x2/2$

$= 142.4x\ 86 * 1.62/2$

$= 142.4(1.65) - 86(1.28)$

$= 235-110$

$Bmmax = 125kNm$

$$K = \frac{M}{\sim ubd2} = \frac{125*10^6}{30*225*2622} = 0.27$$

Compression of reinforcement

$$A1s = \frac{(125 - 73.9)10^6}{80614.2} = 632mm2$$

Provide 4Y16 Top (804mm2)

$$As = \frac{0.156\ fcubd2}{0.87fyz} + A1s$$

$$= \frac{73.9*10^6}{0.87*410*0.775*262} + 671 == 1691mm2$$

Provide 3Y25 + lY20 (1 764mm1)

Provide 3Y25 + lY20 (J764mm1)

SHEAR

V=200kN

v = V/bd= $(200*10^3)/(225*226) = 3.9N/mm^2$

$$\frac{100As}{Bd} = \frac{100*1764}{225*226} = 3.51$$

By interpolation $v_c = 1.04N/mm^2$

$$Asv = \frac{b'(v-v_c)}{0.87f_{yv}} = \frac{225\,(3.9 - 1.04)}{0.87*410} = 1.87$$

. *Provide Y8@l20c/c*

Provide Y8@120c/c

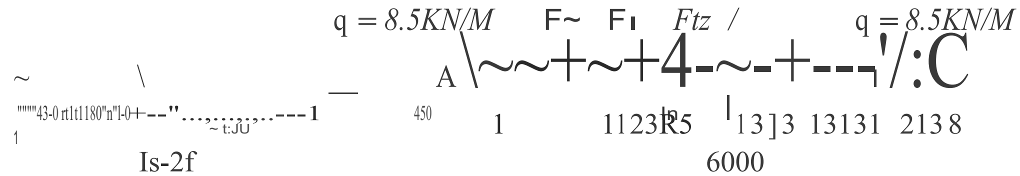
1ST AND 2ND FLOOR SLAB AND BEAMS

3.20 Ribbed slabs Is-1, Is-2 same on Gs-l, Gs-2

3.21 Ribbed slabs Is-3 II= 3.8m, = same as Gs-4
Is-3b h= 3.0m, same as Gs-4b
Is-Jc l3= 3.0m, same as Gs-4c
Is-Sa 14=3.15m, same an Gs-4a

3.22 Ribbed slab Is-4 same as G-s5

Edge Rib 1s-2d (e, f) and slab I-s2 q2= 15.5 *kN/M*



Loading pattern

As Ib-2d

Pillars of block wall F2 = (3.3 + 0.53) x 2.85 x 1.4 xO.323 = 3.51kN

F I = 100mm partition perp. to span

(1.5 + 0.53) x [(0.45-023) + 0.23/2) X 1.4 x 2.85 =2.71kN

For slab Gs-ls-l

Dead - From Ground floor *7.81kN/m*

Live Ground floor (Offices) 4.00 "

11.8kN/m

PI - load from slab Gs - 1 = 11.6 x (0.45 - 0.023) = *2.60kN/m*

Self-weight of rib = 0.23 x 0.3 x 24 x 1.4 = *0.55kN/m*
2.4 + 0.74 + 1.11

Weight of LOrn 9" wall under window = 3.51 / (0.23 x 2.85) =*5.35kN/m*

q = *8.50kN/M*

3.23 q_2 = weight of 9" wall between window

(3.3 + 0.53) x 2.85 x 1.4 *18.3KN/M*

REACTIONS

$6R_A$ = 8.5 x *62/2+* 3.51 x 4.762 + 15.3 x 3.45 x 2.63 + 2.71 *xr3.45* + 3.51 x 2.13

153 + 16.71 + 138.82 + 6.64 + 7.51, RA =53.5KN

RD = 8.5 x 6 + 3.5l+2.71+3.51+15x2.63-53.8=97.l7KN

Position of BMmax (where SF = 0

53.8 - 8.5x - 3.51 - 15.3x + 19 = 0

23.8x = 69.29, : x=2.91m

- 15.3 ex - *1.24il2*

= 56.8 x 2.91- 3.51 (1.67) - 8.51 x 4.23 -15.3 (l.67f12

157 - 5.86 -36-21.34

= 94KNM

$K = \frac{M}{f_{cu}bd^2} = \frac{94 \times 10^6}{30 \times 450 \times 262^2}$ = 1 (at section B-C) .

0.10<0.156

Z d(0.5+ 1(0.25-0.110.9) = 0.87d

As *94xl061* 0.87x410xO.87x262

1156mm2

Provide 4Y20 ($1260mm^2$) bottom

AT SECTION AB AND CD

Point B, Bm = RA x 1.24^2 - 8.5 1.24^2

2

= 53.8 x 1.24 - 8.5 x 1.242 *12* = *60kN/m*

Point C, Bm = RA x 2.14 - 8.5 x 75 D^2 *12*

=47.17x2‌14-19.5 82kNm

82kNm>60kNm, use *82kN/m* for 230mm section

b = 230, d = 262

K= = 82 x 10^6 0.17>0.156
30x230x2622

Compression reinforcement
AI s = d' 20 + 8 + 6 =34n'lm

A's = (82-73.9) x 10^6 = 98mm2
0.87x 410 x (262-34)

Provide 2Y12 Top (113mm2)

A' = 73.9x 10^6 +98
72428 .
= 11118mm2

Provide 4Y20 (1260mm2) Bottom

SHEAR
V = 53.8KN,. v = *0.89N/mm2*
100AS lCl0 x 1260 = 2.11
Bd = 230 *X 262*
vc = *0.99N/mm2*
Nominal links is required

ASv = *OAb* = *004* x 230 = *00423*
Sv 0.87fyv 0.87 x 250

Sv max=0.75d=197mm

Hence Provide R8@190%
Anchorage into slab against torsion

e = 225-] 15 = 110mm

T= (l5.3+0.225XO.3X24X104)   X 0.l1 = 1.936kNMlm X 0.11

$$A_T = \frac{M}{0.87 f_{yv} d} = \frac{1.93 \times 10^6}{0.87 \times 2.50 \times 262} = 34mm^2/m$$

Provide R8 @ 300 ($167mm^2/m$)

Hence R8 @ 15'0% controls both the nominal links reinforcement and torsion

LOADS
From slab s - 2b : 11.81 x 6/2 = 35.43KN/M = ql selfwt
From slab s - 3c : 11.81 x 3/2 + 1.19 = 18.91kN/M = q2
REACTIONS
Rl = 18.91 x 4.24^2 + Y2 15.43 x 4.24 x 2/3 (4.24)/4.24 f
= 170 + 212/9.24
R2 = Y2 35.43 x 4.24 + 18.91 x 4.24 - 90
Bm_{max} = where SF = 0
65 - Y2 35.43/4.24x X x'- 18.91x =
65 - $4.18x^2$ - 18.91x = 0
X^2 + 4.52x-15.6 0
X = -4.52~,jL5i~ - 4x 1x (-15.6)

2x1
= - 4.52 + 9 - 10/2 2.29m
Bm_{max} = 65 x 2.29 - 4.18 x 2.29^3 x 1/3 - 18.91 x 2.23^2/2
149 - 16.70 - 49.6 83kNM
K = m/fcubd^2 = 83 x 10^6/30 x 500 x 262^2 0.08<0.156
Z = d (0.5 + .25 - 0.08/0.9
As = 83 x 6/0.87 x 410 x 0.90 x 262 = 987mm^2
Provide 5Y16 (1010mm^2)
Hanger bars - 0.2%(500 x 300) = 300mm^2
Provide 3Y12 (339mm^2) Top
SHEAR
Provide y% *30qc/c* doubled

Bean IB - 8 L = 3.8m 300 x 300

3.8m 45.0KNIM

LOADING

From slab 1 s - 2 : 11.81 x *6/2* = *35.43kN/M*

From slab 1 s - 2: 11.81 x *1.5/2* 8.86

Selfwt: 0.33 x 0.3 x 24 x *1.4/2.4* + 1.11 + 0.74 0.71

q = *45.0kN/M*

m = *ql2/8* = 45 x *4218* = *90kN/M*

k = 90 x 10^6 x *10₆/300* x 300 x 262^2 0.014<0.156

z = 0.95d

.. AS = 90 x *10^6/0.87* x 410 x 0.95 x 262 = 1014mm^2

Provide 4Y20 (1260mm^2)

SHEAR

R_1 = V = q*il2* = 45 x *4/2* 90kN

..J = 90 x *10^3/300* x 262 = 1.15N/mm^2

100ASlbd = 100 x *12601300* x 262 1.60

By interpolated — Provide

V_c = *0.85N/mm2* *v*, + 0.4 = 1.25<1.60 — 5Y16(10

Mosley — Shear rtf in required — Omrrr')

table 6.5 — AsvlSv = b(V - *V_c)/0.87fy* -00 (1.15 - *0.85)/0.87* x 250 = *90/218* — Bottom

Page 116 — 0.414 — Provide

Provide *Y:225c/c* links — 3Y12

4.8 — (339) To

IB-9 L = 3.5m 230 x 300mm

4.9 t|-_ 3_.5_m _,l~ ~ 24.78kN/M

LOADING

From slab Is - 4: 11.81 x *1.5/2*			*8.86kN/M*
From 230mm wall :			*15.38kN/M*
Selfwt	*0.7110.3* x 0.23		0.54
	q	=	*24.78kN/M*

m = 0.3125 x 3.5~ x 24.78 ,38kN/M

k = *38/30* x 230 x 2622x 10^6 0.08

Z = d (0.5~.25 - *0.08/0.9)* 0.9d Bottom

As = 38 x 10^6*10.87* x 410 x 0.9 x 262 451mm2

Provide 3Y16 (603mm2)

SHEAR

V = *qI/2* = 24.378 x *3.5/2* 43kN

v= 43 x 10^3*/230* x 262 0.1*71N/mm2*

IOOAs/bd = 100 x *60*$_3$*/230* x 262 = 1.0

From table

vc = *0.72N/mm2*

Provide nominal rtf

Asv ISv=- OAb/0.87fyv = 0.4 x *23010.87* x 250 = *D2/218* = 0.4223

Provide R: @ 200c/c (Sv = 0.75d = 197mm)

4.14 1B-14 450 x 300

F

3.670 t.40'

Provide some reinforcement as in bean IB - 11

Span 6Y20

Support 4Y20

Link Y:@150c/c

4.15 1B-15 230 x 450mm

L = 4.24 + 0.57 = 4.813 .

F = 21KN/M

2= 18.60KN/M

.700

LOADING

Loadsfrom slab IB - 3B : 11.81 x 3.15/2 = 18.60

From slab wall = 15.28

Self weight of beam : 0.55 + 24 + 1.4 x .23 x .15 = 1.71KN/M

ql =. 35.59KN/M

from slab s - 3a 11.81 x 3.15/2 = 18.(iOKN/M q2

F = reaction from IB - IIa = 21.0KN

$$AS = \frac{51 \times 10^6}{0.87 \times 410 \times 0.8d} = 682mm^2$$ Bottom

Provide 2y16 + Iy20 (716mm2)

From hanger bars provide 20% of both bars 0.2 x 716 = 143 Top

Provide 2Y12

SHEAR

V = 40.75 X 3.15 = 64.2kn
2

V = 64.2 X 10^3 = 1.07N/mm^2
230 x 262

100AS = 100 x.716 = 1.19
Bd 230 x 262

By interpolation v, 0.76

Reymyds v; + 0.4 = 1.16

H/brok. (Vc + 0.4) < V < 0.87 f~

table 3.33 0.5Vc< V ~..vc+ 0.4 .. become of the don may in

. ASV = 230 x 0.4 = 0.4230
SV 0.87 x 250

R: @ 200% in R: @ 200%

4.17 (Balustrade), 150 x 1700mm

< = 13500mm q = 28.18

900 RA 1.~0 IRs

500 d = 1700 - 20 - 2 - 20 .. 10
= 1700 - 58 = 1642mm

lb0

LOADS

From slab: s - 3a : 11.81 x 3.15/2 = 18.60KN/M

Self wt : 7 x 0.15 x 24 x 1.4 + 0.03 x 22 x 1.4 = 9.58KN/M

q = 28.18KN/M

m = 28.181 /8 = 28.1 x 13.50^2 = 642KN/M

k = 6842 x 10^6/30 x 150 x 1642^2 = 0.05<0.156 2'12

z = (0.5 +-J.25 - 0.05/0.9)d
= 0.94d

As = 642 x 10^6/0.87 x 410 x 0.94 x 1642 = 1169

Provide 2Y25 + 2Y12 (1208mm^2)

SHEAR

V = 28.18 X 13.5/2 = 190.2KN = R 2Y25

Y = 190.2 x 10^3/150 x 1642 = 0.77N/mm^2

100AS/bd = 100 x 1208/150 x 1642 = 0.49

By interpolation Yc = 0.526N/mm^2

0.5Vc<Y~.vc+ 0.4

ASV = 0.4 x 150,0.87 x 250 = 0.276

Provide links at spacing 0.75d = 0.75 x 16.421

Provide R: @ 300 c/c links

1b-19 l= 2.00M, 230 X 300MM

RA ...------":;>"""""":::...- q = 54.33 RB

2.00

From slab 1s - 2 : 11.81 x 6/2 + 11.81 x 0.052/2 = 38.50KN/M

Self wt of slab = 0.55

Wt of 9.of blockwall = 15.28

ql = 54.33KN/M

m = 54.33 x 22/8 = 27.2KN/M

z = 0.95d

AS = 27.2 x 106/0.87 x 0.95 x 262 x 410 = 306mm2

Provide 3Y12 botton (339mm2)

Top - pm 20% (332) = 68mm2

SHEAR

V = 54.33 x 2/2 = 54.33KN/M

V = 54.33 x 103/230 x 262 = 0.90N/mm2

100AS/bd = 100 x 339/230 x 2E? = 0.56

By interpolation Vc = 0.59

0.5Vc<V<Vc + 0.4

. . ASV/SV = 230 x 0.4/0.87 x 250 = 0.4230mm2

Provide R8@230c/c

REACTION

4.7RA = 35.59 x 4.72/2 + 18.6 x 1 x (3.7 + 0.5) + 21 x 3.7 Provide

= 393.1 + 78.12 + 77.7 2YI0

. . RA= 549/4.7 = 117KN

RB= 18.6 x 1 + 21 + 35.59 x 4.7 x 4.7 -117 = 90KN

Position of Bmmax

RB- *35.9x*

. . x 90/35.59 = 2.53m from B

Bmmax 90 x 2.53 - 35.59 x 2.53 2/2

227.6 -114 = 114KN/M

K = 4 x 106/30 x 230 x 4122 = 0.10 d = 450 - 20 - 8 -10 =
412mm2

Z = d (0.5 +..£l25 - 0.1/0.D) = 0. 88d

$AS = 114 \times 10^6/0.87 \times 410 \times 0.88 \times 412$ = $885mm^2$

Provide 3Y20 (94~mm) ,

Hanger bars- provide 20% = 0.2 x 943 = 187

Provide 2Y12(226m2)

SHEAR

V 117KN," = $117 \times 10^3/230 \times 412 = 1.23N/mm^2$

100/bd = 100 x 943/230 x 412 = 11.00

From table, "c = 0.67N/mm

Vc+ 0.4 = 1.07 < 1.23

ASV/SV= 230(1.23 - 0.67)/0.87 x 250 = 0.592

4.16 Provide Y8 @ 250c/c

Beam IB -'16 L = 3.15m, 230 x 300m

~r.

RAt 3.15

LOADING

From slab ls - 3 : 11.81 x 3.8/2 = 22.44KN/M

From slab ls - 3c: 11.81 x 3.0/2 = 17.7_2

40.20KNLM

Self weight = 0.55

q = 40.75KNLM Bottom

$m = 40.75 \times 3.15^2/8$ = 51KNM

$k = 51 \times 10^6/30 \times 230 \times 262^2$ = 0.11<0.156

z = (0.5 + ~25 - 0.11/0.9) = 0.86d

7.0 . 3RD FLOOR SLABS AND BEAMS

Must of slabs are repeated from 1st floor

~dditional Slabs

7.1 3s - 2: additionally in the areas M - Ll15 - 16 & I - II *129* - 30 (reinforcem (Ah stair case) - L = 6.000m as for Is - 2)

7.2 3s - 2g - is as above, but with variable length from 6.0m to 3.50m "

7.3 3s -s3c -in area A - *B/5* - 6 & 9 - 10

7.4 3s - 3d - is as above with span around 1.5m. K - *J/5-6* & 9 to 10 (re 2YI0)

8.0 3RD FLOOR BEAMS

Mostly beams are repeated from 1st floor.

8.1 beam 3B - I lb L = LOOm, 150 x 300mm

q=26KN/M

1.000

Loads

From slab 3s - 3d: 11.81 x *1.5/2* = *8.86KN/M*

Selfwt. of beam: 0.15xO.3x24x1.4 =1.51 "

RIC. wall- (0.15x24+0.03x22) x2.8x 1.4 =16.7 "

q= 25.92KN 1M q = 26 *KN/M*

M = 26 x 12*18* = 3.25 *KNIM*

V = *26x1/2* = 13 KN

As = 3.25 x *106/0.87x410xO.95x262*

Min rft by code = 0.13%bh = ~.13x 150x3001l00 = 59mm2

Provide 2YI0 (157mm2) . bottom 2 Y 10 bottom

Shear:

V= 13 KN proved R 8 @ *250c/c* R8 @250 *c/c*

8.2 beam 3B - 11 C L = 1.5, 1150 x 300

~F= 13KN R'~O.13 *KN/M*

1.5 -p

Loads

From slab 35 - 3d: 11.8 x *0.52/2* 3.07

RIc. + selfwt.: 17.06

20.13KN!M

F = load from beam 3b - 11b 13KN

M = . (13x1.~~0.13x1.52/2) = - 42 *KN/M*

K = 42 x *106/30x150x2622*= 0.133 < 0.156

Z = (0.5+ ,(J.25 - 0.136/0.9) 262 - 212.2

Provide 2Y 20 (628 mrrr') top 2Y 20 Top

SHEAR

V= 13+20.13x 1.5 =43KN.

V= 43 x 10^3/150 x 262 = 1.09 N/mm^2

100 As /bd = 100x 628/ 150x 262 = 1.60

By interpolation v_c = 0.85 N/mm^2

0.5 v_c < v > (0.4 + v_c)

: - Asv/sv = O.4b/O.87 fy, = *OAx* 150/0.87x250 = 0.276

Provide R 8 @ 300 C/c R 8 @ 300 C/c

8.3 BEAM 3B - 1 (Curve) L = 3357, 250 x 300mm

JC?93b

18.6 = P

1.5

3.357

From the 3b - 3c - varies - 11. 81 x ½ 3.15 = ql= 18.60KN/m ql = 18.60 KN/m

Selfwt. of beam - *0.25xO.3x24xlA/2A+ 1.11+0.74*

RIC wall - as in 3b - 11 q,. t 6.7 :: q2 = 17.06 KN/m

RA = 13 x (3.357 - 1.5) + ½ 18.6x3.357x 1133.357+

17.06X3.357^2/2)?>57 = 46.23KN

RB = 13 + ½ 3.357 x 18.6+ 17.06x.057-46.23

= 55.26

... ofBmm

RB - 17.06u - 17.06u - 18.6/3.357 x ½ u

---+46.23 - 17.06u - $2.77u^2$

U^2 + 6069 = 0

10.5Vc <Vc < Vc to .4

:- Abv = *0.4x2501* = 0.460

Sv *O.8'lx250*

Proved R8 @ 220 *C/c*

B cam 3B - 13 L= 2.4 . 230x300

= *35.84KN/m*

Loads

From slabs 38 - 3b *18.60 KN/m*

Self wt of beam : 0.05 x *0.23/0.25* = 0.54

Ric wall: 16.7

F = from beam

F = from beam: (3b - 13) 46.23KN

RA = 46.23 x 1.7+112(1.7x18.6)x *1I3.(1.7)+35.84x2.4₂/2.4* =79.5KN

RB = 46.23+35.8x2.4+ 112 186x1.7-79.4 = 67.12KN

Position of beam

97.4-35.84x = 0, :- x:h 2.22m

'BMmax=79.4x2.22 - 46.23x (2.22 - 0.7) - 35.84x2.222

- *18.60/1.7x* (2.27- 0.7) x 1I2(2.22-0.7)x1l3(2.22-0.7) = 10.84 KNM

Provide same reinfmt. as 1B - 13 (2Y16 bottom), links = R8 at 230 *C/c.*

Beam 3b - 12 & 3b - 14 L = 6.88M, 450m

F= 88.66m

~ ....----40.67(m) n = 92 60 KN = q2

50-20-8-10

| -----:'=.1--'1 ~ *' ~ = 412 mm

~. R2

2.00m x 4.88n1

4.00

Load

From slab 3b - 1: 11.81 * *6.3/2* = *37.20KN/m*

230mm wall

100 wall tr to slab: *4.14/6:3* = 0.66

Selfwt of beam 452 x 24 x 1.4 = 6.80 *59.94KM/m* V *60KN/Mql*

. F = re of Beam 3b - 11 55.26 KN = 55.26KN
RIC wall as a callion: 16.7 (1+1) : 33.40
8·-;:-8.-::-66=KN~-

Reaction R. = 609.98+ 1420/6.88
From slab 35- C : 11.81 x 3.15/2 = 18.59
Self wt of beam . = 6.90
block wall = 40.67KN/m q2

reaction
4.88Rc = 60x4.88 = 40. 67 x 22/2 - 88.6x2
714.43-81.34-177.2
:: R2 = 93. 42 KN

Rl = 88.67+ 40.67x2+60x488- 93
=370KN

Provide ofBM Mix
R2 x 1.56 - 60x1.562
= 93.42 x 1.56- 60x 1.52 = 73 KN m
K=73xl0 6
30x 450x 4122
7 = ~.5+ vr-""2"'-5~O."'I""I1'---d"--~J
0.9'
= 0.95 d
Ab = 73 x l0b
0.87x410xO.95x412 544

Provide
3Y16(bottom)

Provide 4 y16 Bottom (603mm2)
Support Mt = mount at A
= 88- 6 6 x 2 + 40 - 22/2 = 259 KN
K= 259x 106 =
30x450x912 2 0.11<0.156

7= ~.5+ *v-25* - 0.11 d
0.9 J
=.0.85d
As=259x 10 6
O.87x410xO.83x412 =
= 2067mm2 3Y15+2Y20'
Provide (2098)tw

Provide
3Y25+2Y20
(2000mm2)
Top

Shear

V=370v= 370xl03
V = 350x412 = 2- OON/mm2

100 As = 100x 603 = 033
bd 450x 412

:- Vc = *0.45N/mm2*

(O.4+Vc) < v < *0.~/~4/Nrmm2*

Asv = b (V-Vc) = 450(2.0 - 0.455)
0.87fyv 087x410

.= 1.955

Y8 @ 120 Yc doubled

Stirrups
Y8 @120Yc
Doubled

From slab: 33 - 3b -11. 81 x 3.15
2 18.60KN/m ql

From slab 230mm block wall = 15.2g

Suit wt of hear 0.45x02324x1.4 = 3.48 "
37.36KN/m'

q I = 37-36 *KN/m*

From slab 3b - 3d = 11.81x1.5 = 18 - 60 - q2
-2-

From slab 3b - 3a = 11.81 x 3.15 = 18- 60- q3

F= Beam 3B - 11 46.23 KN

Reaction

4.77RA = 37.36 x 4772 + I8.6x2x5.77
2
+ 46.23x2.77+8.58x2.772
2

4.77RA = 425.02 = 214.64 128.06+32.92

RA=800.64 = 168 KN
4.77

RB= I8.6x2+37.36x4.77+46.23+8.58x2.77 - I6B

= 37.2+ 178.21+46 - 23+23.77-168

= 1I7.41KN

Position of BMmax

RB - 37.36 u - 8.58u=0

II7.45.94u 8. u = 2.55m

BMmax= RB'x 2.55 - (8.58x37.35)x2.56

117. 41x2.56 - (45.93) x2.56

- (1 1'7 II 1 / II" 01'\ v ') '\h= v IIn *KN2*

K= 183x1 =0.156
30x230x4122

Provide minimal compression reinforcement.
Provide 2Y12 as A's (226mm2) top — 2Y12 top

As = 0.156 fcvubd2 + A's — 2Y25+2Y20 (Bottom)
0.87fy7
= 0.156x410x6.775x 412
= 16004mrn2 provide (2Y25+ 2Y20)(161Omrrr')

2Y16 top

.Top bar = 0.2 % x 230x450= 207 mrrr' provide 2Y16 top (402mm2)

V= 168KN
v= 168 x 10^3 = *1.77N/mm2*
230x412
vc = *0.79N/mm2*
(0.4+Vc)<v<0.8 ~
:- Asv = b (V-VcM3-0(1-77-0.97)
SV D87fYv V.87x410
= 0.632
Provide Y8@150 *C/c*

8.2 Beam 3B - 20
C = 230x310

17.60KN/m

<7

A

~Y---f-/----ly
3.00 1.50

Load
(i) 230mm beam wall:- ....:::..*1=5.28KN/m*
Selfwt: 0.23 xO.3x24x1.4 2.32 "
Q = 17.60 *KN.60KN/m*

RA = *17.6x32/2* 17.6 x *1.521*
79.2 - *19.8/3* = u 20 KN
RB = 17.6x4.5-20 = 59KN

Max (=m) = 17.6x32 *18* = 19.8 KNm

Max-m = 17.6x1.52 /2 = 19.8KNm

K = 19.8xl06 *10.87x410xO.95x262* = 223mm2

.Provide 3Y12 top & Bottom (336mm2)

Shear

V = 59, v = 59 x *103/230* x 262 = *0.98N/mm2*

100 Ab lbd = 100x3391230x262 = 0.56

Vc _= 0.60 *N/mm2*

'. 0.5 Vc = 0.60 *N/mm2*

0.5VC<V<0~

Asv = 0.4 x 230 = 0.56

SV 0.87x250

An change Wall

For ..

.... .amount T = 17.6xO.23

= 18.22KNm

3B - 19 I = 3.000, 250 x 300

Table 8.35

37.92

RB

loads ~.

from slab 35 - 29: *11.81x6/2* = *35.43KN/m*

self of beam: 0.25xO.3x24x1.4 = 2.52

-:-37--. 9~2-:':K=Nm"1

RA= Y2 37.92x3-18.96=37.92KN
Position of Bm Max

18.96 = Y2 v 37 .... ?v 0
3
V = 1.73m

BMmax= 18.96x1.73 - Y2 v x 37.92 v x *1/3* v
3
= 32.80 - 1.73 3 x 12.46
O-
21.90KNm v 22KNm

K = 22 x 10 6 = 0.04 < 0.156 = 238mm2
087x410xO.95x262
Proved 2Y16 (402rnm2) Bottom
Proved 20% x 150 x 360 = 90 provide 2Y10 top

Shear
V = 33.84 KN. V = 33.84xI0 3 = *1.37N/mm2*
150x262

]00 Ab = 100x40 2 = 1.02
Bd 150x262

B~ interpolation Vc = *0.69N/mm2* .
Vc + O.4:Sv:s O~

Asv = 150(1.37 - 0.69) = 0.469
Tv O.87x250
Proved R8 @ 220 *C/c* *i'*
8.15 beam 3B -lIe 1= 1.575, 250x300mm2

F = 33.84KN
q = 21.28 KN?m
~

1.575

Loads:
From slab: 35 - 3d = 11.81xO.26 = 3.07 *KN/m*
RIC wall + self wt: . = 18.21 "
q = *21.28KN/m*
F = load from beam 3b - I1d = 33.84
Moment at the support (M support) = *33.84x1.58 2/2*
= 7 v 75 KNm

K = 75x10 6 = 0.146 < 0.156 Proved
250x262 2x30 2y20+2Y16
As 7 = 0.80d (Top)
As = 75x10 6 I003mm2 2YlO
(h,...

Proved 2Y20+2Y16 (1030mm2) top
Bottom = 20 % (250x300) = 150mm2
Proved 2yl0(157m) bottom

Shear
V2 33.8 + 21.28x1.5=66KN
V = 66xl03 - *1.0KN/mm2*
250x262

100As = 100xl030 = 1.37
. Bd 250x300
Vc = 0.80 *N/mm2*
0.5 Vc < v< O.~/cu

Asv = 0.4x250
Sv 0.87x250

= 0.460

Proved R8@200 *C/c*
_--=8=.1:..:::.6 beam 3B - 11a (= 3.35?m)

18.60KN~ F = 33.8
18.21KN/m

3.36m

Loads
From slab 35 - 3a: 11.81 x 3.15 = *I8.60KN/m* - ql
Selfwt + RC wall: :' = *18.21KN/m* - ql
F = reach of beam 36 - 11d = 33.84KN

Rb = 18.2x3.362 *12* + 33.8 x 3.36/2 +1123.36 x 18.6
X *2/3* 3.36 = 0
102.7 + *56.7+56.78+70/3.36*
= 68.30KN
RA= 33.8 + 18.21x3.36+ 12 18.6x3.36 - 68.30
= 57.93KN

Position Max
RB - 18.21 v - 18.60 v x 112ux = 0
3.36
68.30 - 18.21u - 2.77 v2

-_., V2 + 6.57v - 24.6

I'

R - 6.57 + ~ - 4xlx24.6 93
~ =--2xl
= 6.57 + :v = 2.66m

2 3.36

6~.64 - 64.42 - 17.36 - 33.12 = 46.26KNm

K 49.26xl0 6 = 0.10 < 0.156
250x262<Zx30

Z 0.88d

_S = 49.26xl0 6 = 600mm2

""8=7x-1.4Ox"8""8-"x2""6=-2

Provide
2YIO

Provide 4Y16 (802mm2) Bottom
Hanger 0.2 % (250 x 300)
Shear
V = 68.30 V = 68.30xl0 3 = 1.13 *N/mm2*
230x262
100 As = 100 x 804 1.33
Bd 230x 262
Vc = 0.78 *N/mm2*
0.5Vc<0.4x vc<~

Links
R8@22C/c

= 0.4 x 230 = 0.4230
Sv . 0.87x250

Prove R 8 @ 220 *c/c*

: 25 - 4 : 11 81 x *1.512*

11.93

=2.50

Weight of self 9 wall 1:1 .

RA = RB = 219.71 x 3 .5 *R'/2* = 52 KN = 29.71

Mmaxx = 29.71x 6.5^2 *t* = *54KN/m*

K = 45.5 x 10^6 = 0.08

$30x230x262^2$

Z = a.90d = 236

As = 45x10^6 = 541 mrrr'

0.87x236x410

Provide 3Y16 (603 mrrr') bottom ~

Shear

V= 54 KN - proved same stirrups 1B- 9 (R 8 @ 200 *C/c*

SOO

350

2S00

--I- IB.18

Loading from slab IS - 31 - 11.8xO.S2/2 = 3.07

Selfwt: 0.3xO.3x24x1.4 = 3.02

Q = 1S.67 KN/m

q2 = 3.07 + 3.02xl.S7S + J\12X9.S8 - IS.67 KN/m

03

3.07 + 1S.89+ IS.0S = 18.31KN/m

Em=O

3R) = y; 18.31xl.S(2+ 1)+16.67x3211

Rl = 38.7KN

M = 38.7x1.S - 18.31x1.Sx2/3 (1.S)- 16.67xl.S212

S8.0S - 27.47 - 18.7S = 11.83 KN/m

Provide 3Y12 (339mm2) hanger = 0.2x300x300

Shear - R - 8 @ 2S0 C/c provide 2YI0 top

Beam IB - 19

L = 2.8m 230 x 230

The same on beam 3B - 19 1.3Y 12 both Stirrups R 8 @ 220 C/c

8.18

3B - 9 1= 9 L= 3.Sm, 230x,300mrri

1 3.Sm i

Loading

"Rl"f\l'Yl c,J.,h1" ,)V. 11 01 ~ f\ c2,,,,

DESIGN OF MACHINE FROM PLATFORM

503.70

2330 if if if

2330 , 2330 2330

Loading

From slab: O.l5x24x1.4 = 5.04

Improve load: = 5.0 x 1.6 = 8.00 "

q, = 13.04 *kN/m2*

Reinforcement: d = 150- 20 - 5 = 152mm

Mmax = 0.125x13.04x2.33^2 = 8.85kNm

M/feubl =8.85x10^6/30xlOOOx255^2 = 0.02

Z = 0.95d = 119

As = 8.5x10^6/0.87x410x119

provide

Bottom

Distribution bar = 0.13x 1OOxl50/1 00 = 195mm^2

provide *Y8@250clc*

provide *Y8@250clc* Bottom

At the supports

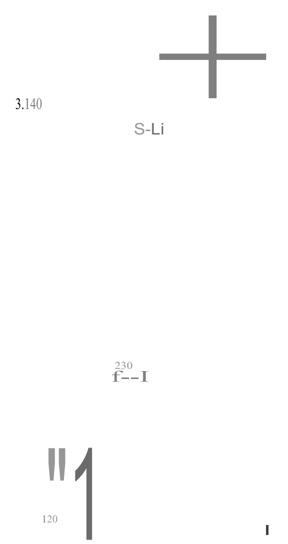
Provide 50% (252) = 126mm^2

Provide *Y8@250c/e*

Deflection:

Fs = *5/8* x410 *(208/252)* = 212m.f= 0.55+ *(4777-212)/120(0.9+0.57):S2*

The slab thickness is O.k.



section A-A

LOADING

120mm slab; 0.12*24*1.4 = *4.03kN/m2*

Cement Screed

Bitumen = *2.88kN/m2*

Conc. Plaster

Live load for roof= 0.75*1.6 = *1.20kN/m2*

(without access) 8.*llkN/m2*

amy = 0.070 fixed edges

amx = 0.022 Span

amy ~ 0.032 Span

Span Moments

'm~= 0.022*8.11 *3.142 = *3.10kN/m*

my = 0.032*8.11 *3.142 = *4.50kN/m*

d = 120 - 20 - 4 = 96mm

Longitudinal bending

k= *(3.10*10₆)1* (30*1000*962),= 0.01 .

z = 0.954 = 91.2

As= *(3.10*106)1* (0.87*410*91.2) = 95.3mm2

As min = (0.13*120*1000)1100 = 156mm2

Y1 *0@300c/c* both ways

Max spacing = 3d = 3*96 = 288mm

Hence, *provide yl0@250e/c (314mni)*

Traverse bending

d = 97 -10 = 86mm

k= *(4.50*106)1* (30*1000*862) = 0.02

z = 0.95d = 81.7

As = *(4.50*106)1* (0.87*410*81.7) = *154mm2/m* < *156mm2/m*

Provide Y10 @250 *c/c*

At Support

Mmy= 0.070*8.11 *3.142 =*6.0kNmlm*

As = *(6.0*106)1* (0.87*410*86) = 196mm2(Top)

Provide Y10 @250 cle (262mm2)

Deflection

m/(bd2) = *(3.10*106)1* (1000*962) = 0.,34

mf= 1.68

Limiting; span/effective depth= 26* 1.68 = 43.7

Actual; span/effective depth = *3140/96* = 32.7

Therefore 43.7 > 32.7

The thickness (120mm) is okay

10.18 TANK BEAM

L-B3 (230x450mm)

Tank size = 3.66*2.44*1.22 rh

Empty ofbraith wate tank = 1.626kg

Density of water = 1000*9.81 = *9.81kN/m3*

LOADING

Live load from water: *{(3.66*2.44*1.22*9.81)1* (3.66*2.44)}*1.6 = *19.2kN/m2*

Live load from tank: *{(1.626*9.81)1* (3.66*2.44)}*1.6 = *2.86kN/m2*

Self weight of beam: 0.23*0.33*24*1.4 = 2.55kNm2

Load from slab: 8.11**1/4* = *2.03kN/m2*

q = *36.45kN/m2*

ql = 36.45*3.14= *114.45kN/m*

M = *(114.45*3.14²)/8* = 141.06kNm

D = 450-20-8-8 = 414mm

RA=Rs=*(141.06*3.14)/2* = 179.7kN

k= *(141.06*10⁶)/* (30*230*414²) = 0.12 < 0.156

z=0.76d=317

As = (141.06*10⁶)/ (0.87*410*317) =.1248mm²

Provide 4Y20 (1260mm²) Bottom

For top bars, provide 2.0% 1260 = 250mm² > 0.13bh/100

Provide 3Y12 (339mm²) Top

SHE4R

DTI

V = 180kN

v = 180*10³/(230*414) = *1.89N/mm²*

$$\frac{100As}{bd} = \frac{100*1260}{230*414} = 1.32$$

By interpolation v_c = *0.73N/mm²*

$$Asv = \frac{b\,(v-vc)}{0.87fyv} = \frac{230\,(1.89-0.73)}{0.87*250} = 1.228$$

Asv = R8@180 c/e double links

Deflection:

Fs = (5/8)fy (Asreqd/Asprov,)

F;= (5/8)*410*(1248/1260) =j 254N/irun2

M.f= 0.55 + (477 - fs)/120(0.9+ M/db2) S 2.0

= 0.55 + (477- 254)/120 (0.9 + 3.58) = 0.88 < 2.0

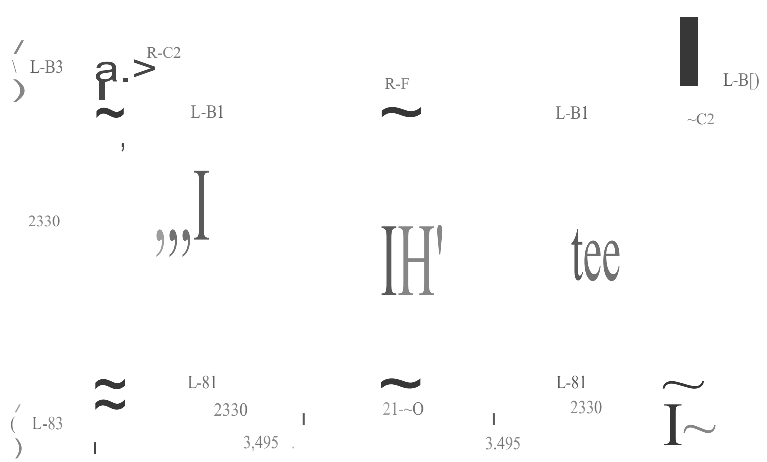
Basic Span/effective depth ratio = 0.86*26 = 22.40

limiting Span/effective depth ratio = 22.4*0.88 = 19.7

Actual Span/effective depth ratio = 3143/414 = 7.60 < 19.6

Deflection is satisfied

1.1.13 DESIGN OF SLAB S- L2



LOADING

Slab load: Item 10.17 = *8.11kN/m2*

Span reinforcement

M = 8.11*$2.330^2/8$ = 5.50kNm

As = $(5.50*10^6)/$ (0.87*410*96*0.95) = 169mm^2

Provide Yl0@250 e/c (314mm2) Bottom

Distribution = (0.13*1000*120)1100 = 156mm2)

Provide Y8@300 c/e (168mm~) Bottom

At the Support

Provide 50% (314mni) = 157mm2

Provide YI0@250 c/e Top

Design Of Slab S- I,

M = *5.50*3.012.33* = 7.08kNm

Provide same reinforcement as in S-h

Provide Yl0 @250 c/e (314mni) Bottom -main bar

Distribution- Provide Y8@ 30,0c/e (168mm^2) Bottom

Deflection:

Fs = (5/8)fy(Asreqd/Asprov,)

Fs = (5/8)*410*(280/314) = 229N/mm^2

M.f= 0.55 + (477 - fs)/120(0.9+ M/db^2) S 2.0

= 0.55 + (477- 229)/ 120 (0.9 +5.70) = 1.41 < 2.0

Basic Span/effective depth ratio = 1.41*26 = 21.10

Actual Span/effective depth ratio = 3000/96 = 31.25 < 36.6

Deflection is satisfied

DESIGN OF COLUMN R-C1

Column R - Cl = 225x 630mm

LOADS

From beams L-b3 (item 10.18) = 179.7kN

From beams L-b3 (item 11.14) = 54.lkN

Self weight col: 0.225*0.63*2.3*24*1.4 = 11.0kN

N=244.8kN

= 0.02*245 = 4.9kNm

Le.zb= (0.75*2300)/230 = 7.5<15

Leylh= (0.75*2300)/630 = 2.7<15

The colmnn is short column

N/bh = (245*10^3)/ (230*630) = 1.69

Mlbh^2 = (4.9*10^6)/ (230*630^2) = 0.05

Provide minimum reinforcement

Asc = (0.4*bh)/100 = (0.4*225*630)1100 = 567mm^2

Provide 6Y12 (679mm2)

DESIGN OF COLUMN R-C3,

Column R - C1 = 225x 225mm

LOADS

From (item 11.14) RA = Rs = (54.113.495)* 3.637 = 56.3kN

From (item 11.14) RA = Rs = *(54.1/3.495)** 3.637 = 56.3kN

Self weight col: 0.225*0.225*2.3*24*1.4 = 3.9kN

N = 116.5kN

M = 0.02*245 = 4.9kNm

Le.zb= Ley/h = *(0.75*3200)/225* = 7.7<15

Design as short column

M/bh^2 = *(117*0.02*10^6)1* (225*225^2) = 0.21

Provide minimum reinforcement

Asc = (0.4*bh)1100 = *(0.4*225*225)/100* = 203mm^2

Provide 4Yl0 (314mm2)

11.0 c/c COLUMNS

11.1 Cols. *E,Hl4,11* C-l (600 x 600)

From beam:	RB -1 item 3.7	276kN	
450 x 450mm	RB - 3 item 10.2	97KN	{Roof Beam}
	RB - 5 item 10.3	134kN	
	RB - 10 item 4.10	49KN	
From beam:	3B - 1 item 4.1	276kN	
450 x 450mm	3B - 3 item 4.3	161KN	{3rd floor}
	3B - 5 item 4.5	143kN	
	3B - 10 item 4.10	49kN	
From beam:	2B - 1 item 4.1	276kN	
600 x 600mm	2B - 3 item 4.3	161KN	{2rd floor}
	2B - 5 item 4.5	143kN	
	2B - 10 item 4.10	49kN	
From beam:	B-1 item 4.1	276kN	
600 x 600mm	B-3 item 4.3	161KN	{1st floor}
	B - 5 item 4.5	143kN	
	B - 10 item 4.10	~9kN	
From beam:	GB - 1 item 3.6	270kN	
600 x 600mm	GB - 2 item 3.7	276KN	{Grd floor}
	GB - 5 item 3.11	147kN	
	GB -7 item 3.13	237kN	

11.14 DESIGN OF BEAM (Lb-1 & Lb-2)

~ Size = 230x300mm

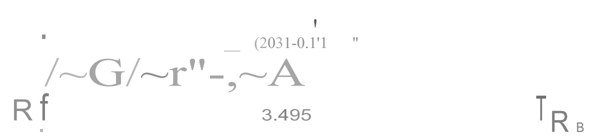
d= 300-20-8-8 = 264mm

LOADING

From slab: 8.11 *2.330 = *18.90kN/m*

Selfweight of Beam: 0.23*0.18*24*1.4 = *1.39kN/m*

q = *20.3kN/m*



Mmax= *(20.3*3.495^2)/8* = 31kNm

RA = RB = 20.3*3.495*0.5 = 54.1kNm

k= (31 * 10^6)/ (30*230*264^2) = 0.06

z = 0.93d = 245

As = (31 *10^6)/0.87*410*245* = 355mm^2

Provide 2Y16 (402mml) Bottom

For top bars provide *(0.13bh)/100* = 90mm^2

20% (402) = 80mm^2 < (0.13bh)1100 = 90mm^2

Hence,provide 2Y10 Top (157mm1)

At the Support, (31 *8)/12* = 20.7kNm

SHEAR

V= 54kN

$v = (54*10^3)/(230*264) = 0.89N/mm^2$

$\frac{100A_s}{bd} = \frac{100*402}{230*264} = 0.66$

By interpolation $v_c = 0.68N/mm^2$

$A_{sv} = \frac{b(v-v_c)}{0.87f_{yv}} = \frac{230*0.4}{0.87*250} = 0.423$

Asv = R8@230 c/c

At the Support

$A_s = (402/31)*20.7 = 268mm^2$

Provide 3Y12 (339mm²)

Deflection:

$M/db^2 = (31*10^6)/(230*264^2) = 1.93$

$F_s = (5/9)f_y (A_{sreqd}/A_{sprov},)$

$F_s = (5/9)*410*(355/402) = 201N/mm^2$

$M.f = 0.55 + (477 - f_s)/120(0.9 + M/db^2) \le 2.0$

$= 0.55 + (477 - 201)/120 (0.9 + 1.93) = 0.81 < 2.0$

Basic Span/effective depth ratio = 0.81 *26 = 21.10

Actual Span/effective depth ratio = 3000/264 = 11.4 < 21.1

Deflection is satisfied

Self weight of column:

Roof 2.85 x 24 x 0.45 x 0.45 x 1.4

Ditto'

Ditto

Ditto

Grd 3.30 x 24 x 0.6 x 0.6 x 1.4

Basement 4.2 x 24 x 0.6 x 0.6 xl.4

At the Top (4thfloor)

Total load v = 558 + self weight.

=556 + 19.39 =575.1kN

Self ~eights of columns

(a) Roof - 3rd floor, ht = 3.15 - 0.6 = 2.55m

Self weight =2.55 x 0.45 x 0.45 x 24 x 1.4 = 17.40kN

(b) 3rd floor - 2nd floor = 2.55 x 0.6 x 0.6 x 24 x 1.4 = 30.8kN

(c) 2nd floor= 1" floor= 2.55 x 0.6 x 0.6 x 24 x 1.4 = 30.8kN

(d) lst- Grd floor ht = 3.6 - 0.6 = 3.0m
Self weight = 3 x 0.6 x 0.6 x 24 x 1.4 = 36.3kN
(e) Grd Basement: ht=4.15 -0.6 = 3.55
Self weight = 3.55x 0.6 x 0.6 x 24 x 1.4 = 3.55kN

Design of Column C1

(1) At the basement.
10 = 4.15 - 0.6 = 3.55m
Effective length Ie = Blo

Nmax = 556 + 3(629) + 930 + 17.4 + 2(30.8) + 36.3 + 43

=556 + 1887 + 61.6 + 1009.3 ~ 3452.3kN,

C-1,4 Nos, C-2b 2 Nos,

Floor	N kN	M KVM	N bh	M Bh2	100ASe Bh	ASc rnm '	
Roof	556						450
u/s	17.4		1.6	0.053	Min=O.4	%	4S0D]
3rd Floor	57.4	11.5					6716
u/s	629						(1210mm2)
2nd Floor	30.8						
tIs	1233.2	25.0	3.4	0.16	0.4	11440m	8y16
u/s	629						G00iDl
i" floor	30.8						600
tis	1873	36.0	5.3	0.18	04	1140m	8y16
u/s'	629						
Grd/floor	36.3						8716
	2556.3	51.0	7.1	0.24	04	1440mm	
	930						8716
Basement	43	71.0	9.8	0.33	04	1440mm	(1610M2)
found	3531.3						

ABC = BH X 0.4 = 600 X 600 X 0.4
100 100

ABC2 = 450 X 450 X 04 240mm2
100 1440m2

LINKS =
Min Size XO =.1 x 16 = 4mm
4 A Rupt R8-

"'j ,

Max sputing = 12 x Omin

= 12 x 16 = 192mm2

Provide R8 @ 17Syc

~ 11.2 C- 3, E3a

Cys E, H/I, 13; 5/18,0/21, y/24, v/27

From beam: Rs- 5 item 10.3 134.4kn

Rs- 5 Ditto 134.4kn Roof

Rs- 5 Ditto 134.4kn

403.2kn

Self Wright: 0.45^2 x 24 x 1.4 x (3.15-0.03) = 19.4kn

From beam 3B - 5 Item 4.5 143kn

3B - 5 " " 143kn 3rd Floor

3B - 5 " " 143kn

429kn

Self Not 0.602 x 24 x 1.4 (3.15-0.3) = 345kn

From beam 2B - 5

2B - 5 Ditto = 429kn 2nd Floor

2B-5

Self not: 0.6^2 x 24 x 1.4 x 2.85 = 34.5kn

From beam B- 5

B- 5 Ditto = 429kn 1st Floor

B-5

Self Not - 0.6^2 x 24 xl .4 X 3.3 = 40kN

From beam GB- 6 Item 285kn

GB-7Item 237kn Ground Floor

522kn

Self not = 0.6^2 x 24 x 1.4 x (4.15-0.3) = 47kn

(Basement)

At the basement

10 - 4.15 - 0.3 = 3.85m

Nmax = 40.3 2+ (429 x+3) + 10. 4 + 34.5 + 2 + 522 + 40 + 47 = 2388kn

Floor	N kN	M KVM ExN	N bh	M Bh2	100Abc Bh	ABC mm2	
Roof u/s	403.2						
3rd Floor u/s	+19.4 422.6 429.0	8.5	2.1	0.039	Min	810	6716
2nd Floor t/s' u/s	34.5 886.1 429.0	17.7	2.5	0.082	0.4	144'0	
1st floor t/s u/s	34.5 1349.6	27	3.8	0.13		1440	8716
Grd/floor tiS u/s	40.00 1818.6 522	36.4	5.1	0.17		1440	
Fdn	47 2388	47.8	6.6	0.22		1440	

11.3 col. F/M G/16/ F/29, G/ll
1273

(−5

'l

From beams:	Rb-2	Item 230	
	Rb-9	Item	Roof
	Rb-19	9.13	

Self net = Yz $0.9_2 \times 24 \times 1.4 \times 2.85$ = 39kN

From beam	38-9	52kN	
	38-19	38kN	3RD floor
	38 - 2	320	

Self not 39kN

From beam	3-2	320kN	
	28-9		2ndfloor
	28-19		

Self not 39kN

From beam	28-2		
	28-9	320	1ST Floor
	28-19		

Self weight Yz $0.9_2 \times 24 \times 3.3 \times$ '1.4 45kN

G8 - 3	Item 3.0	238kN
G8-9	Item 3.9	121kN

Self not: Yz $0.9_2 \times 24 \times 1.4 \times (4.15-0.3)$ = 53kN

M = Nemin d = 900 - 40-8 -10
h 900

Floor	N kN	M KVM ExN	N bh	M Bh^2	100Abc Bh	ABC rnm'
Roof					0.4	
u/s	293					Area =
					Provide	3240
3rd Floor	39				min reft	
Illi	332					
u/s	320	13.0	0.8	0.02		Provide
						10720
						($3140mm^2$)
2nd Floor	39					
tis	691					
u/s	293	20	1.2	0.03		
i'' floor	39					
tis'	1023					
u/s	320					
Grd/floor	45					
tis	1380					
u/s	359					
Fdn	53					
	1000	36	2.2	0.05		

1273

900

Links

Min size = X 6 = X 20 = 50mm

Max-sputing = 120 = 12 x 20 = 240

Provide R8@200o If

Same rfl

Floor	N kN	M KVM NXO.02	N bh	M Bh_2	100Abc Bh	ABC mm'	
Roof u/s	 275	 5.5	 1.4	 0.06		0.4 X 810_2/100	 6716 (1210mm)
3rd Floor ill. u/s	194 294.4 275.0	, 5.9	 1.5	 0.06	 MIN = 0.4 X 450/100 = 810	 =810	41>0 D 450
2nd Floor tis u/s	19.4 589 275	 11.8	 2.9	 0.13			
i'' floor t/s' u/s	19.4 883 275 23	17.7	4.4	0.19			
Grd/floor tls u/s	23 1181 278	, 23.6	 5.8	 0.26			
Fdn	26.2 1485	 29.7	 7.3	 9.33			

Links

Min size = 4mm

Max sputing = 120 = 12,x 16 ='19_2

Provide R8 @ 175

~

~

11.15 DESIGN OF SLAB S-L3

LOADING

From slab item 10.17 - *8.11kN/m2*

Span Reinforcement

M = 8.11 $*3^2/8$ = 9. 12kNm

z = O.95d = 91.2

As = $(9.12*10^6)/$ (0.87*410*91.2) = 280mm^2

Provide Y10@250c/c (314mm2)

Distribution = *(0.13*1000*120)/100* = 156mm2)

Provide Y8@250 c/c *(201mm2) Bottom*

At the Support provide 50% (314mm2)

Provide Y10@250c/c (314mm2)

11.5 provide same ret for colour on
GL F, G/ 3, 12 - C- 6a
FG/l, 14 - C - 6c
15/m, P, R, S
L/16, 19, 20, 21,

11/24, 25, 26, 29   M/22, T/16   *C-6*
30/V, W, V, V, I   U/29, 1/23.
AKL/6,9
T/15, L/22, 11/23, K/30   C-6b

Col. N/15, L/17, 2/28 Z/30 - 450 X 1430 C-7

From beam:	RB- 8 item 4.8	90kN	
	Rb- 2c item 9.3	56.4kN	Roof
	RB-2c	56.4kN	
		202.8kN	

Self net: 0.45 X 1.43 X 24 X.8 5 X 1.4 = 61.6kN

3B - 8 item 4.8	90kN	
3B 2c item 3.23	53.7kN	3rd Floor
	143.8kN	

.lo(_.

2B-8
B2-2C
} nd Floor

Self net
B-8
S-2C
f 1st Floor

Self Net 61.6kN
Gb - 12 142.4kN

Self net - 0.45 X 1.43 X 24 X 3.3 X 1.4 = 71.~roU~d floor

Foundah - 71.4 x 3.83 = 82.8kN
3.3

Floor	N kN	M KVM NXO.02	N bh	M Bh2	100Abc Bh	ABC rnrn"	
Roof					Provide min ret = 04	Age = 0-4 x 1430 x 450 = 257	Provide 14y16 (2814m2)
u/s	202.8						
3rd Floor	61.6						
s/s	264.4						
u/s	143.8						
2nd Floor	61.6						
tis	469.8						
u/s	143.8						
i" floor	61.6						
tis'	675.2						
u/s	143.8						
Grd/floor	71.9						
tis	890.4	17.8	1.38	0.05			
u/s	142.4						
Fdn	82.8						
	1115.6KN	22.3	1.73	0.08			

4'0

1430

y

y 143b

x

1430

y

Ley = 0.9 x 3.2 x 10³ / 1430 = 2 < 15

Leu = 0.9 x 3.2 x 10³ / 450 6.4 < 15

Short column

Max spacing 120
= 12 x 25 = 300

11.6 Columns A, K/4, 11 C-8

132 318 132 225 450 318 318 225 fSO132

d = 300 - 40 - 870
h 300 = 0.8

Substituted section: h = 450 + 318 + 132 = 300
3
b = 450

from beam:

	RB- 1 item 3.7	276kN	
	RB- 5 item 10.3	134.4k}	Roof
	45 - 2f item 3.23	47.2kN	
	45-2d	53.8kN	
		511.4kN	

Self net 06 beam 0.3 x 45 x 24 x 1.4 x 2.85 = 12.92kN

3B- 1 item 3.7	2766kN	
3B- 5 item 4.5	143kN	
35 - 2f	47.2	3RD Floor
35-2d	53.8	
	520kN	
22nd	520kN	
Selfnet	12.92kN	2nd floor
1st floor	520kN	
Selfnet	12.92kN	1ST floor
From beam GB-	335kN	
Self net 12.92 x 3.3 / 3.85'	15kN	

Basement fundat self net 15 x 3.85 / 3.3 = 17.5kN

Floor	N kN	M KVM NXO.02	N bh	M Bh2	100Abc Bh	ABC rnrn"	
Roof							
u/s	511.4						
3rd Floor	12.9		3.9	0.36	0.4	0.4X300X450	Provide
ill	524.3	10.5				100	5 y12
u/s	520.0	21.1	7.8	0.5	0.4	=540	566
							Provide
2nd Floor	12.9	31.8	11.8	1.0	0.4	540	6y20
tis	1057.2						(1800)
u/s	520.0						
i" floor	12.9						Provide
tis'	1590.1						7y25
u/s	520.0	42.2	15.6	1.0	1.2	0012 X300X450	(1890)
						1620	
Grd/floor	15.0						
tis	2125.1						Provide
u/s	335.0	50.0	18.4	1.2	2.4	0.012 X300 X	7y25
						450=2340	(3440)
Fdn	17.5						
	2377.6	50.0	18.4	2.4	2.4	0.024X300X450	
						32400	

Max 120 =

Links

Provide R8 @250

Leu = 3200 = 10.7 < 15
300

short Column

Ley = 3200 = 7.1< 15
H 450

11.7 Column A, K/5, 10 C-9 230 X 450mm

From beam RB - 13 item 10.9
45- 2F item 3.23 Proof

Self net: 0.23 x 0.45 x 24 x.85 x 1.4 = 9.9kN
From beam 3B - 13 item 8-y 79.50
3b-2f 47.20
126.70kN

Self net 9.90kN

From beam 2B - 13 item 4.13 50.09 2nd Floor
2B - 2f items 3.23 47.20

Self net 9.90kN

AB-13 1st Floor
AS-2F

ground floor

Fdn: self net: 11.5'0x 3.85 = 12.5kN
3.3

Max N = 472kN

Floor	N kN	M KVM NXO.02	N bh	M Bh2	100Abc Bh	ABC Mm2	
Fdn	472	9.4	4.6	0.2	Min 0.4	0.4x450x230= 414mm2	Provide 4y16 (80mm2)

Column C- 10 450 x 600mm

In comparison to the above provided

ALC AMIN = 0.00 4 x 450 x 600

= 1080mm2

Provide 6y716 (1206)

Links R8 @ 200yc

11.8 Columns E, H /6,,9 C- i3

450 X 600 mm, h=450

b-600

Fro'mbeams RB-12 Item 10-7

RB- 18 item 10.11

Self net: 0.45 x 1.2 x 2. 85 x 24 x 1.4 =

	3B- 12 item BI5	370kN	3rd floor
	2B-12 item 4.12	246kN	2 floor
	B-12 item 4.12	246	
Self net		26	
	GB- B item 3.14b	139kN	

Ground floor self net: 0.45 x 0.6 x 3.3 x 24 x 1.4 = 30 kN

Basement fdn 30 x 3.85 = 35kN
3.3

Max N 638kN

M = 1630 X 0.02 = 33kN

N = 1638 X 10^3 6.1
bh 450 X 600

M = 33 X 10^6
bh^2 6000 x 450^2 = 0.3

but min asi = 0.4

rF= O.4bh = 0.004 x 600 x 450
100 = 1080mm^2

Provide 6y16 (1210mm^2)

Links:

R8@2000/C

11.9 Colum *i/6,* 9 C- ,11

450 X 1200

From beam AB - 3 item 10.2 97kN Roof

RB- 12 item 10.8 84kN

Self: 0.45 x 1.2 x 24 x 2.85 x 1.4 52kN

3B - 3 item A.3 161kN

3B - 12 item 8.5 93.4kN 3RD floor

3B -17 item 4.7

2B - 3 item 4.3 161kN

2b -12 item 4.12 115kN

2b -17 item 4.7

Self net

B-3 item 4.3 161kN

B-12 item 4.12 115kN 1st floor

B-17 item 4.7 190.2

Self net 52kN

GB- 4 item 3.10 277kN

GB- 5 item 3.11 112kN

GB- 8b item 3.146 139kN

Self net 0.45 x 1.2 x 3.33x 24xl.4 60kN

Basement fdn 60 x,3.85 -= 70kN

3.3

MAX N = 2372kN

$N =$

Bh $M = 2372 \times 0.02 = 47.4kN$

N

Bh 2372×10^3

1200×450

$M = 47 - 4 \times 10^6 = 0.2$

bh^2 $1200 \times 450 = 0.2$

hence provide min reft $= 0.004bh$

$= 0.00\ 4 \times 1200 \times 450$

$= 2160mm^2$

/-Provide 8yzo

Links

Provide R8 @200 c/o

Column 1/6,9450 x 800

"'~ C-12

Treat as C- 13

Provide 6716

Links ~8 @ 200c/c

11.10 Column J/7, 8 C-14

600 X 800mm

h = 800

b = (500

fdn grad	4.15m
grand	3.60m
1st _ 2nd	3.15m
2nd _ 3rd	3.15m
3rd _ 4th	3.15m
	17.75
Dot	40
	17.35m

Le = 0.9 x 17 - 35 = 26 > 15
h 6

Ley = 0.9 x 17.35 = 19.5 > 15
b 0.8

Me = mi + madd
< Mi + Nau

When

Mi = initial moment in the column
Madd = moment caused by the deflect of the column
Au = deflection of the column

Bx= 1. (le)2
200

= 1 x (Cl.g X 17.35)2
2000 0.6
= 0.339

au = Bx kh value K= 1
= 0.339 x 1 x 0.8 = 0.27

Madd = Nau h2

From beam	RB- 18 Item 10.11	227kN
	RB- 18a Item 10.11	114kN
		341kN

Selfnet: 17.35 x 0.6 x 0.8 x 24 x 1.4 = 280kN

Nmax = 341 + 280 = 62ikN

Madd = 621 x 0.27

Madd = 168kNm

Mi = Nxem = 621 x 0.02 = 12.4 kNm

Mi = 12.4 + 168 = 180kNm

N = 621 x 103 = 1.3
Bh 600 x 800

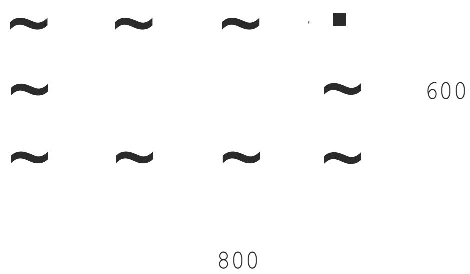
M = 170 X 106 = 0.47
bh2 600 X 8002

d 800 - 40 -8 - 10 = 0.9 = 0.9
h 800

from the chart
provide min rfl
0.004 bh = 0.004 x 600 x 800 = 1920mm2

Provide 8 y20 (25YOmm2)

Links: R8 @200yc



ROOF FLOOR SLAB AND BEAMS

9.0 CC ROOF FLOOR RIBBED SLABS

Some parts of the roof floor serve as a roof (without access):

5cm gravel ballast	0.5 x 20 x 1.4	$=1.4kN/m^2$
Cement screed to slope (max. 9cm)		$=2.77kN/m^2$
4mm bituminous felt	:(0.4)/20 x 4 x 1.4	$= 0.11kN/m^2$

Ribbed Slab:

Slab:	0.1 x 24	$= 2.4kN/m^2$
Ribs:	{(0.1 x 0.2)/0.52} x 24	$= 1.11kN/m^2$
15mm cone. Plaster	= 0.0015 x 22	$= 0.33kN/m^2$
Pots	(0.077)1 0.2x 0.52	$= 0.74kN/m^2$
		$= 4.58kN/m^2$
	1.4 x 4.58	$=6.412kNm^2$
Live Load:	0.75 x 1.6	$=1.20kNm^2$
		$11.89kN/m^2$

9.1 SLABS:

Rs-1, Rs-2, Rs-2a, Rs-2b, Rs-3, Rs-3a, Rs-3b, Rs-3c are just the same as Is-I, 1s-2, 1s-2a, 1s-2b, 1s-3c--------respectively

9.2 SLAB Rs-3d

Span L varies from 3.00m to 0

Reinforcement- the same as for Rs-3c.

9.3 EDGE RIBS OF SLABS 4S-2

9.3.1 Edges rib Rs-2d:- reinforcement provided as for 1s-2d. i.e 4y20

9.3.2 Edge rib Rs-2c:- in comparison to 1s-2d. Provide reinforcement

4Y20 bottom

Links R8 @ 230 *c/c*

Anchorage reinforcement against torsion

R8 @ *300c/c* (as in Is-2d)

o
o
U1

o
o
f'1

Loading

Weight ofwall-0.15 x 1.5 x 24 1.4 *7.56kN/m*

Form slab- 11.89 x Y2(1.5) *8.92kN/m*

Self weight ofrib-0.23 x 0.3 x1.4 x 24 = *2.32kN/m*

18.8kN/m2

is.SOkN/M

RA~-------6-.0-0-f'l-------R-B~

R_A= 18.8 x 6 = 56.4kN= RB

2

BMmax = We = 18.8 x 6^2 5kN

8 8

K = .85 x 10^6 = 0.18 > 0.15 (compression reinforcement required)

30 x 230 x 262^2

d= 20 + 8 + 8 = 36mm

9.4 RIBBED SLAB RS - 4

550 550

20

Wooden Box

b = 550mm, d = 400 - 20 - 8 - 10 = 362

LOADS

Finished layer	0.05 x 20 x 1.4		*1.54kN/m2*
100mm slab	0.1 x 24 x 1.4		*3.36kN/m2*
Pots	$\frac{0.077}{0.55 \times 0.23} \times 1.4$	=	*0.61kN/m2*
Plaster	0.015 x 22 x 1.4		*0.46kN/m2*
Ribs	$\frac{0.15 \times 0.30}{0.55} \times 24 \times 1.4$		*2.75kN/m2*
Wooden box	= $\frac{0.1 \times 0.4 \times 4}{0.55} \times 1.4$	-	*0.41kN/m2*
Live load	0.75 x 1.6	=	*1.20kN/m2*
Scm Gravel ballat, cement screed to Slope and 4mm bituminous felt		=	*4.28kN/m2*
q		=	*14.61kN/m2*
ql = 14.61 x 0.55		=	*8.04kN/m*

F =230mm block work = (3.3+ 0.53) x 2.85 x 1.4 x 0.55 =8.40kN

Fl = *7.56kN/m*

$Als = \frac{M-0.156f_{cu}bd^2}{0.87 \times (262-36) \times 410} = \frac{(85-74) \times 10^6}{0.87 \times (262-36) \times 410} = 136mm^2$

Provide 2Y10 ($157mm^2$) top

$As = \frac{0.156f_{cu}bd^2}{0.87f_yz} + Al\,S = \frac{74 \times 10^6}{0.87 \times 410 \times 0.775 \times 262} + 136$

= 1158mm2 provide 4Y20 (Bottom)

SHEAR

V = 56.4kN

$v = \frac{56.4 \times 10^3}{230 \times 262} = 0.94\ N/mm^2$

$\frac{100\,AS}{bd} = \frac{100 \times 1158}{230 \times 262} = 1.92$

By interpolation vc = 0.9

$\frac{Asv}{Sv}$ = 0.4 x *23010.87* x250 = 0.4230

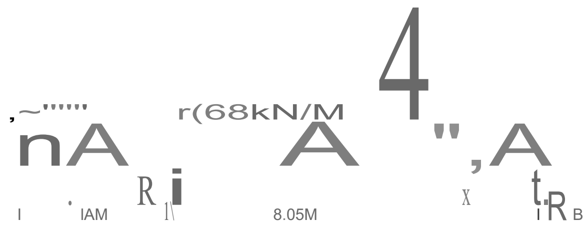
Provide R8 @ 230 *c/c* R8 @230 *c/c*

Anchorage into slab against torsion

e=115-75=40m_2

$T = \frac{(7.56+2.32) \times 0.04}{2} = 0.35kN/m$

Provide R8 @ *300c/c* R8 @*300c/c*



$8.05R_A = 5.68(8.05+1.4)^2 \times 0.5 + 7.56(8.04+1.4)$

$$R_A = \frac{253.1 + 71.37}{8.06} = 40.6kN$$

$R_B = 7.56 + 53.62 - 40.6 = 20.6kN$

Position of $BM_{max} = 20.6 - 5.68x = 0$

$x = 3.62m$

$BM_{max} = R_B x - (5.68x^2)/2$

$= 20.6 * 3.62 - (5.68 \times 3.62^2)/2$

$= 74.57 - 37.22 = 37.4kNm$

At the cantilever,

$M = 7.56 \times 1.4 + 5.68 \times 1.4^2/2 = -16.2kNm$

Positive moment

$$K = \frac{37.4 \times 10^6}{30 \times 550 \times 362^2} = 0.02$$

$z = 0.95d = 344$

$$A_s = \frac{37.4 \times 10^6}{0.87 \times 410 \times 344} = 305mm^2$$

Provide 2Y16 ($402mm^2$) bottom *2Y16 Bottom*

At support A

Negative moment

M= -16.2kNm

$K = \dfrac{16.2 \times 10^6}{30 \times 150 \times 362^2} = 0.03$

Z = 0.95d = 344

$As = \dfrac{16.2 \times 10^6}{0.87 \times 410 \times 344} = 132mm^2$

Provide 2Y10 (157mm²) Top *2Y10 Top*

SHEAR

V= 40.6kN

$v = \dfrac{40.6 \times 10^3}{150 \times 362} = 0.75N/mm^2$

$\dfrac{100AS}{bd} = \dfrac{100 \times 402}{150 \times 362} = 0.74$

For table *v« = 0.60N/mm2*

$\dfrac{Asv}{Sv} = \dfrac{0.4 \times 150}{0.87 \times 250} = 0.276$

Provide R8 @300 c!e *R8@300c!e*

Deflection: fs = *5/9* x 410 x *305/402 = 173N/mm2*

$\dfrac{M}{bd^2} = \dfrac{37.4 \times 10^6}{550 \times 362^2} = 0.52$

m.f= 0.55 + (477-173)/120(0.9+0.52) = 2.33

Basic span/effective depth = 2 x 26 = 52

Actual span/effective depth = *(8050)/264* = 30.5

52> 30.5 deflection o.k

10 CC ROOF FLOOR BEAMS

10.1 BEAMS:RB-1, RB-2, RB-3, RB-4, RB-40

RB-5, RB-6, RB-7, RB-8, RB-9,RB-:10,

4B-16, 4B-17, 4B-19, are just the same

as 1B-4A, 1B-5,lB-6,lB-7,lB-8,lB-9,lB-10,

1B-16,3B-10,respectively.

10.2 BEAMS

L = 6.3m, 500 x 300mm

Loading

For AS-I: 11.89 x *3.0/2 = 22.59kN/m*

RS-1: 11.89 x *0.52/2 =3.00*

Self weight of beam =5.04

q= *30.74kN/m*

RA ~ RB =30.74 x 6.3 x 1/2 = 96.77kN

Bmmax= $30.74 \times 6.3^2 \times 1/8$ = 152.51kNm

$$K = \frac{152.51 \times 10^6}{30 \times 500 \times 262^2} = 0.148,\ Z = 0.79 \times d$$

$$As = \frac{152.51 \times 10^6}{0.87 \times 410 \times 0.79 \times 262} = 2060mm^2$$

Provide 3Y25+2Y20 bottom (2098m2)

~Hanger bars 0.25 (500 x 300) = 300mmm = 339mm^2
3Y12(339)

SHEAR .

V = 97kN, v = $\frac{97 \times 10^3}{500 \times 262}$ = *0.74N/mm2*

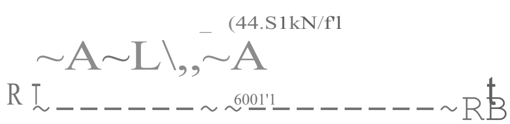
$\frac{100As}{bd}$ = $\frac{100 \times 2098}{500 \times 262}$ = 1.60

By interpolation *v« = 0.85N/mm2*

ASv/Sy =$\frac{0.4 \times 500}{0.87 \times 250}$ = 0.920

Provide R8@195 c/c stirrups.

10.3 RB-5



From slab R-S2: 11.89 x *6/2* = *23.78kN/m*

" " 11.89 x *0.52/2 = 3.09kN/m*

= *38.76kN/m*

Selfweight:- 6 x 3 x 24 x 1.4= *6.05kN/m*

q = *44.81kN/m*

RA= RB= 44.81 x *6/2=134.4kN*

Mmax = 44.81 x *6/8* = 202kNm

K = $\frac{202 \times 10^6}{30 \times 600 \times 262}$ = 0.16 compression reinforcement required

$A1s = \frac{M-0.156fcubd^2}{0.87 \times 410(d-dl)} = \frac{(202-193) \times 10^6}{80614} = 112mm^2$

Provide 2Y12 Top (226mnl).

$As = 0.156fcud^2/0.87 \times 410 \times 0.775 + A1s = 2777mm^2$

Provide iY25 + 3Y20 (290mm2)

SHEAR

$V = 134.4$, $V. = \frac{134.4 \times 10^3}{600 \times 262} = 0.85N/mm2$

$\frac{100AS}{bd} = \frac{100 \times 2903}{600 \times 262} = 1.85N/mm2$

$v_c = 0.89N/mm2$

Provide min reinforcement.

$\frac{Asv}{Sv} \quad \frac{OA \times 600}{0.87 \times 250} = 1.103$

Provide double R8@l60 C/C *double links.*

10.4 RB-9

RB-9 same as 3B-9

4B-lld

LOADING

From slab RS-3d: 11.89 x *1,575/2* = *9.36kN/m*

For parapet wall: 0.15 x 1.5 x 24 x 1.4 = *7.56kN/m*

" self weight: 0.23 x 0.3 x 24 x 1.4 = *2.32kN/m*

q = *19.24kN/m*

RA=RB = 19.24 x 2.5 = 24.1kN

2

2

K=_ =

30 x 230 x 262

Provide same as reinforcement as3B-9 i.e 3Y12Bottom ($339mm^2$)

SHEAR

Provide R8 @230 C/C asfor 3B-9.

10.5RB-llc & 4B-lle

F=24.1kN

hA.L~:AAA.

. 1.575,.., ~

LOADING

Same Loading as 4B-11d = *19.24kN/m*

Mmax = 24.1 x 1.58 + 19.2 x $1.58^2/2$ = 62kNm

Provide some reinforcement as 3B-lle (2Y20 + 2y16) Top (1030mni)

10.6RB-ll curve

~ LOADING

For slab AS-3d, AS-3c (average): (11.89 x *1.5/2* x 0.5) x 2·= 9.24mmm

For parapet & self weight =9.*88kN/m*

=19.*12kN/m*

Mmax = 19.12 x 3.357^2 + 24.1 x 1.575 x 1.76^2= 26.93 + 20.15 = 47.08 kNm

8 3.357

Alternatively,

RA= 24.1 x 1782 + 19.12 x 3.336 = 44.90kN

3.357

Rs = 24.1 x 19.12 x 3.36 - 44.9 = 43.44kN

Position ofBMmax= Rs-19.12x =0

$x = \frac{43.44}{19.12} = 2.27m$

BMmax = RBX $-19.12X^2/2$ = 49.35kNm.

$K = \frac{49.35 \times 10^6}{30 \times 250 \times 262^2} = 0.10$

As = 591mm^2 *provide 3Y16 Bottom (605mm2)*

Hanger bars- 0.2(603) = 121mni

Provide 2Yl0 Top (157mm2)

i.e. Reinforcement same as 3B-ll (a)

Stirrups same as 3B-lla

i.e R8@220 clc.

107 RB -lIb L = 1.00,230 x 300m^2

Reinforcement same as in 4B-lld.

i.e 3Y12 Bottom.

Stirrups R8@230 clc.

10.8RB -12 & 14 L = 450 x 400mm

F= 43.44KN

LOADING

From slab RS-1: 11.89 x *6.3/2* = *37.45kN/m*

" RS -4c: 10.33 x *0.55/2* = *2.69kN/m*

Self weight of beam: 0.45 x 0.4 x 24 x 1.4\ = *6.05kN/m*

q = 46.*19kN/m*

For slab 4b - 4c: 10.33 x *0.55/2* = 2.*84kN/m*

" 45 - 3c: 11.89 x *3.15/2* = *18.73kN/m*

Self weight of beam = *6.05kN/m*

= 27.*62kN/m*

4.88RA= 46.19 x *4.88212* - 27.62 x *2212* - 43.44 x 2

RB = 83.38kN.

RB = 46.19 x 4.88 + 27.~62x 2 + 43.44 - 83.58 = 241kN

Position of beam:

RA-46.19x = 0 :- x = 83.58 = 1.81m

46.19

BMmax = 83.58 x 1.81-46.19 x 1.8f= 76kNm

Provide same reinforcement as 3B -12 & 14*(3Y16) Bottom-titlsmm'*

At the support (3Y16 + *2Y20) Top-2067mm2*

Links Y8@200 clc

Deflection:

10.9RB -13 L = 2.4m, 230 x 300mm

tr---Tl"i~""·73'IT:1kNIT7I-::-" ~t

RA [M 2.4M R B

LOADING

From slab RS - 3b- 11.89 x *3.15/2* =: 18.73

Selfweight of beam 0.23 x 0.3 x 24 x 1.4 = 2.31

"

=*21.04kN/m* qi

From slab RS - 3C 11.89 x *3.1S0/2* = *18.73kN/mq2*

From slab RS - 3C1 11.89 x *3.1S0/2* = *18.73kN/m*

RA= 2.4^2 x 21.04^2 *12* + 18.75x 0.7^2 *12* x *2.0S* + ~ x 1.7 x 18.73 x *1/3* x *1.7/2.4*

60.60 + *4.S8* + *9.02/2.4* = 30.91kN

RB= 18.73 x 0.7 + ~ (18.73) x 1.7 + 21.07 x 2.4 -119.88

= 13.11 + *lS.92* + *SO.S7* - 30.91 = 48.69kN

Position of BMm

R.t\ - 21.04x - 18.73x = 0

x = *30.91/39.77* = 0.70m

BMmax = 30.91 x 0.7 - 18.73 x o.i *12* - 21.Q4x *0.82/2*

= 21.64 - *4.S9* - *S.1S* = 11.90kNm

Provide 2Y16 Bottom.

Links Y8 @230 cle.

10.10 RB -15 L = 4.77,300 x 300mm.

F=44.90kN

IS.73kN/M

9.36kN/M

21.04kN/M

2.00M

4.77M

RB

LOADING

From slab RS-3b - 11.89 x *3.1S/2* = *18.73kN/m*

Self weight = *2.31kN/m* q2

q2 = *21.04kN/m*

From slab RS - 3d - 11.89 x *1.S7S/2* = 9.36kN/m=Ql

From slab RS - 3c- = *18.73kN/m=Q3*

$R_A = 21.'04 \times 4.77^2/2 + 18.73 \times 2 \times 3.77 + 44.9 \times 2.77/4.77$

$+ 9.36 \times 4.77^2/2 = 239.36 + 141.22 + 124.37 + 35.91$

$R_A = 113.4kN$

$R_B = 18.73 \times 2 + 44.90 + 9.36 \times 2.77 + 21.04 + 4.77 - 113.4 = 95.1kN$

$R_B = 95.1kN$

Position of BMmax

$R_B - (21.04+9.36) x = 0$

$x = 3'13m$

$M_{max} = 95.1 \times 3.13 - 21.04 \times 3.13^2/2 - 9.36 \times 2.77 \times (3.13-2.77) - 44.9 \times (3.13 - 2.77)$

$= 297.7 - 103.06 - 9.33 - 16.16$

$= 169kNm$

$$K = \frac{169 \times 10^6}{30 \times 230 \times 262^2} = 0.36$$

Increase breadth to 300mm.

Hence, beam size = 300 x 300

$$K = \frac{169 \times 10^6}{30 \times 300 \times 262^2} = 0.27$$

$$A_{1S} = \frac{M - 0.156 f_{cu} b d^2}{0.87 f_y (d - d_1)}$$

$d_1 = 20 + 8 + 8 = 36mm$

$$A_{1S}' = \frac{(169 - 96.38)10^6}{0.87 \times 410 \times (262 - 36)} = 901mm^2$$

Provide 3Y20 Top (943 *mm²)*

$$A_{1s} = \frac{96.38 \times 10^6}{0.87 \times 410 \times 0.775 \times 262} + 901$$

$= 2331 mm^2$

5Y25 Bottom (2450 mnl)

SHEAR

V = *113AkN*

v = 113*A* x 10^3 = *1A4kNi* mm^2

300 x 262

100 As = 100 x 2450 . = 3.11

bd 300 x 262

Asv = btv-v.) = 300(1.44 r: 1.04) =0.5517

0.87fy 0.87 x 250

.*Provide RS@lSOcic*

10.11RB -18 (18a) L = *5040* 800 x400mm

~ j74.lJkN/fYI

c\'£\'~AAA

~~i ~5~.4o=fYI ~T RB

LOADS

From slab RS-4c: 14.61 x *(8.0512* + 1.0) = *73.41kN/m*

Self weight of beam: 0.8 x *004* x 24 x 1..4 = *10.70kN/m*

=*84.11kN/m*

RA:::: 84.11 x *504/2* = 227kN = Rs

M = 84.11 x *5040*$_2$*/2* = 307kNm

b = 800, d = 400-20-8-12.5=359.5 = 360mm

=0.10 z = 0.87d = 315

30 x 800 X 360[6]

307 x 10[6] = 2733mm2

0.87x410x315

Provide 6Y25 (2950) bottom

20% (2950) = 590mni
Provide 3Y16 (603mm2) Top

SHEAR

V = 227kN, v = 227 x 10[6] = *0.79N/mm3*

800 x 360

100AS = 100 x 2950 = 1.02

bd 800 x 360

By interpolation

v« = 0.675NI mnr'

Asv = 0.4(800) = 1.4713

0.87 x 250

Provide doubled R8@l35c!c

Deflection:

DESIGN OF LIFT BASE

--------------------~-----~

13143 1233°12330 1233°1 3143 |

ul _1_

ail iM il ..~

~-.u--5~ W31

LOAD ESTIMATION

WALLA

1. From Upper Roof - Col. RG-245 = 490 KN
RG-117 X 1-117KN
ii. G Wall- 3.82 X 1.4 X 1.3 X 6.99 - 49KN
iii. 6mm Platform Slab - 13.04 X 2.33 X 6.99 - 106KN
2
iv. Power Roof Slab RS - 4C: 10:44 X 8.36 X 6.99 - 305KN
2
v. 1a - 3rd Flr. Slab (S - 3a, 25 - 3a, 35 - 3a): 11.89 X 3.8 X. 6.99 X 3 - 471KN
,\ri. Groud Floor: GS - 4b: 11.89 X 3.8 X 6.99 - 160KN
2
vii. SelfWt Of Wall: 0.23 (17.35 + 1.3) X 24 X 6.99 X 1.4) - 1007KN
2,708KN

-4 lY~
~elfWeyin: 1067 X 2.1 = 303.!>·-;
6.99

WALL')
From Slab: Roof - Ground Floor (Rs - 4d, S - 2c, 25- 2c, 35 - 2c, Gs - 4c)
(11.89X3.14)X5X3.14 = 147KN
4
From 9" Wall- 3.82 X 1.4 X 1.3 X 3.143 = 22KN
SelfWt: 303 X 3.143 = 453KN
2.1 622KN

WALL 4
Wt Of Wall 3 + 0.26 X 11.89 X 3.143 X 5 = 671KN

WALL 5
From Slab: Grd - 3rd Fir (Gs - 4, S - 3a, 25 - 3a, 35- 3a)
11.89 X 3.8 X 3.143 X 4 = 284KN
2
From Lower Roof Slab 455-4:5
= 10.89 X 8.36 X 3.143 = 143KN
2
FrQ,~9" Wall_________ = 22KN
~S~el~f~W~t~:============~ ~4S3KN

From Upper Roof Slab Cal RC2 180KN
1,082KN

Load Summation:
Wall 1 - 2,708 X 2 = 5,416KN
Wall 2 - 302 X 2 = 604KN
WaJl3 - 622 X 2 = 1244KN
Wall 4 - 671 X 2 = 1342KN
Wall 5 - 1082 X 4 = 4328KN
IN = 12,934KN
Ar 12934 x 1.1
1.47 x (300 - 0.55 x 20)
,. Provide base 13.90 x 3.54m x 0.4 = 49.2m2
qc (Earth pressure) 12934 =

~11[1Jlm.~

3.147m

2.33Om

ly = 1 for both a & b
lu
Panel a - case 7
x = 0.043 Span
Panel a y = 0.043 Span
xy = 0.058 support
mx=0.Q43 x263 x3.1432=
my= 0.043 x 263 x3.1432 = *112KNmlm*

At the continious edge
mKY = 0.058 x 263 x 3.1432 = -151KNm
d = 400 - 50 -10 = 340mm
k = _m_ 112 x 106 0.03
feubj' 30 x 1000 x 3402
7 = 0.950 = 323
As = 112 x 106 972mm2/m
0.87 x 410 x 323
Provide yI6@200c/c (1010mm2)
Bothways

At support
As = 151 x 972 = 1310mm2
112
Provide 16@150% (1340rnm2)
(under walls)

Provide
Y16@200
c/e
i.e. mid-
span top
bothways

Provide
Y16@150
"t, bottom
at support
(i.e.
continious
edge)

PANEL B

oon - 0.34

ooy - 0.034 pr spam

ooxy - 0.46 Support

fin = 0.034 x *112/0.043* = 88KNM - my

mny = -0.046/0.058 x 151 - 1200NM

Support

support

span m - 88KNM

As = 88/110$_2$ x 972 - 764mm^2/m

Provide Y16@250c/c(804mm^2)bathways (bottom)

(mid-spam top bath ways)

Provide Y16@250c/cbottom (mid-span top)

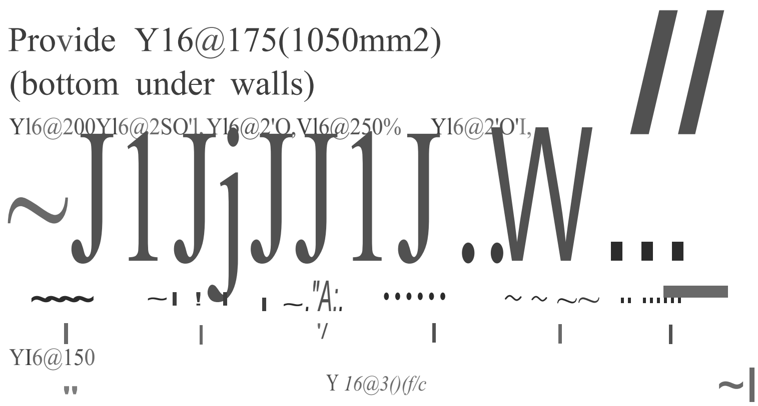
Support 120KNM

As = *120/151* x 1310 1041mm^2

Top

Provide Y16@175(1050mm2)

(bottom under walls)



"Runners for the bottom bar

Provide V.13bh = 0.13 x 1000 x 400 = 520m^2

=YI6@300mm

Checks

1. Punching shear: punching shear cannot be checked since the critical perimeter
11. 1.5d from the wall face (1.5 x 340 = 510mm) lies outside the base.
iii. Shear: lies outside base

~

22/CC FOUNDATIONS

PAD FOUNDATION FOR COLUMNS C1 &: C-26

22.1

N = 3531.3KN F-2

Allowed bearing pressure of soil = *300KN/m2*

.. AF = $\frac{3531.3 \times 1.1}{1.46 (300 - (0.55 \times 20)}$

provide3.Ox30

t.

Provide footing : 3.0 x 3.0x 0.7m

qc(Em1hpressure) = *367.51* 3.12= *392.4kn1m2*

At the col. face of the critical section: b = 3000mm

M = 367.5 x 3 x *1.22/2* = 1367 *kN/m*

Min.d = 700-50-2-10 = 520mn
h=700mm
k = 847 x *106/30x300x620*
z = 0.95x620 = 637
As = $\frac{847 \times 10^6}{0.87 \times 410 \times 494}$ = 6025mm2

Provide 13Y25both ways
bar spacing >750 or 3d whichever is smaller
0) Check for min reinforcement
$\frac{100 As}{bh} = \frac{100 \times 6025}{3000 \times 700}$ = 0.29 > 0.13

Max spacing = 750mm. therefore the reinforcement provided meets the requirement specified by the code for minimum area and maximum bar spacin In a slab

(ii) Check for punching shear

Critical perimeter - u=col. Perimeter + 8 x 1.5 col.

0.6x4+8x1.5xO.670 = 10.44m

Area with perimeter == *(0.6+3xO.620i* = 6.81m2

Punching shear force v = *367.5(32-(6.81/4)2* = 2242kN

Punching shear stress v = 2242x10[3] = *0.35N/mm2*

10440x620

by interpolation *v«= 0.47N/mm2*

:. 0.35<0.47, hence the chosen depth 700mm is okay

Check for shear 1.2-.67 1.0d = 670mm

1.2-0.62 = 0.48, 1.0xO.620= 0.620m

shear

At the critical section for shear, 1.0d from the column face

V = 367x3x0.48 = 529kN

v= 529x10[3] = *0.28N/mm2*
3000x620

100As = 0.32
bd
Shear stress Vc = *0.44N/mm2*

:. *0.28<0.44N/mm2,* hence the section is adequate i.e 3000x3000x750mnl is okay.

22. ..PAD FOUNDATION FOR COLUMN C-ll, C-12, & C-13

N 2372KN F-3

AF = 2372x11.1

1.47(300-0.55x20

provide base 2.20x2.80 = 6.75m2

AF= 6.75m2

qc(Earth pressure) = *2372/6.75=351.4K.N/m* 2

Assume h = 6001mn

d = 600-50-10= 540mm

At the face of column b=2250 mm

M = 351.4x2.25x2.8 =889.5kNm

z = 0.95x540 = 394mm

As= 889.5x106 =4861mm2
0.87x910x394

(16Y20 (S030m2)

I-{ Provide (16Y20 (S030'm2)

My = 351.4x3xl.132 = 667 kNm

d = 600-50-20-10 = 520mm

d=600-50-20-10 = 52vmm

z = 0.95x520 = 494mm

As = 667x106 = 3785mm2
0.87x410x494

Provide 13Y20 (4083mm2)

Provide 13Y20

Checks

i) Min reinforcement 100x5030 =0.45 > 0.13 OK.
2250x500

Spacing 750mm max. OK.

~

Punching shear

ii) Critical perimeter - v = col. Perimeter +8xl.Sd

=2(1.SxO.S40x2+ 1.2+1.SxO.52x2+0.45)

= 9.66m

Table S.ſ mosley

V = 351.4(2.25x3-(9.66/4)2) = 323kN

v = 323x103 = *0.06N/mm2*

9660x540

100As = 100xS030 = 0.3

bd 3000x540

vc = *0.44N/mm2*

0.06< 0.44N/mm2, the chosen depth is adequate

(iii) check for shear 0.90-0.54 = 0.46m

.45

.1.2

2250

V = 351.4x2.25x0.46 = 364kN

v = 364x103 = *0.43N/mm2*

2250x540

By vc from table 5.1 mosley = *0.43N/mm2*

:. 0.42 < *0.43N/mm2*

Hence section is OK

i.e 22S0x3000xSOO is OK

22.4 PAD FOUNDATION FOR C-2, C-2a C-3, C-3a & C-4

N=2388kN

AF=*1.1x2388/1.47(300-(0.55x20)* = 6.18mm2

Provide base 2.5x2.5xO.6m, Area = 6.25mm2

d = 600-50-20-10 = 520 d

qc(Earth pressure) = *2388/6.25* = *382kN/m2*

At the face of column b =2500

M= *382xO.952x2.5/2* = 431kNm
Z = 0.95x520 = 494
As= $\frac{431\times10^{6}}{0.87\times410\times494}$ = 2446mm2

Provide 13Y16(2613mm2) i.e Y16@180 both ways Provide 13Y16

Checks

(i) Min reinforcement-, $\frac{100\times2613}{2500\times600}$ = 0.17>0.13

(ii) Punching shear:-

Critical perimeter = v = (l.5xO.52x2+0.6)4

= 8.64m

V = 382 (2.52_ *(8.64)2/4)*
= 605KN
v = $\frac{605000}{8640\times520}$ = *0.13N/mm2*

$\frac{100As}{bd}$ = $\frac{100\times2613}{2500\times520}$ = 0.20 .

by interpolation IlC = $0.4 N/mm^2$ >0.13

hence, the depth is adequate.

Check for shear 0.95-0.52 = 0.43

1.0d = 520

0.95-0.52=0.43

I.Od=520 ~-+|-+~--+-|

shear

/

V = 382x2.5x0.43 = 411Kn

$v = \frac{411 \times 10^3}{2500 \times 520} = 0.32 N/mm^2 < V_c = 0.4 N/mm^2$

Hence, the seetin is okay

i.e 2.5x2.5xO.6m is okay

22.5 Combined footings for column C-6a, C-6b

F-6

Loads:

2N = 1485x = 2970

$AF = \frac{2970 \times 1.1}{1.46 \times 300} = 7.46 m^2$

Provide footing 1.8x4.8 = $8.64 m^2$

$\%(\text{earth pressure}) = \frac{2970}{8.64} = 344 Kn1m^2$

~

O.9m 3.0m O.9m

O.9m

1.8

O.9m

4.8m

 251

PAD FOOTING FOR ~OLUMNS C-S, C-=6a, C-6b

F -7 & F-7a

N = 1800Kn :.2N = 3600Kn

characteristic = 1.lx3600 = 2694Kn

1.47

Ap = 2694 = 9.32nl

300-(0.55x20) 066 80

1.0

1.0

1.0 3,Om

Area of the rectangle = (3+2x)(2x) = 9.38

By solving for x = 0.96m

Hence, adopt base area 2x5=10m2 =provide 2x5xO.6m

%(earth pressure) = 3600 = *360kN/m2*

10

REINFORCEMENT (a) (longitudinal bending)

(i) Cantitever: M\ = 3600 x *2.0x(1.oi* = 360kNm

2

b = 2000, d=600-50-8 = 542Il1.J11.

As = 360xl06 = 1960mm2

0.87x410xO.95x542

Provide 7Y20(2200mm2) bottom

7Y20@290% bottom

Transverse bending

b = 5.0m, d = 600-50-20-10 = 5comm

M/ = 360x12x5 = 900Knm

2

=

0.87x410xO.95x520

Provide 17Y20(5340rnm2) bottom

Provide 17Y20@bottom

Min reinforcement - O.l3x5000x5340

100

= 3380rnm2 <5108mm2

Checks

(i) Punching shear:-

Critical perimeter u (1.5xO:542x2+0.91z)+

(1.5xO.520x2+0.9z)x2

= (2.08+2.01)x2 = 8.18m

V = 360 [2x5-(1.5xO.52x2+0.9)(1.5xO.520x2+0.9)x2]

2 2

360 =(10-[(2.08)(2.01)x2]

.. V = (10-8.36)360 = 589kN

v = 589000 = 0.13N/mm2

81880x542

by interpolation v.= 0.40 N/mm2 >0.13N/mm2

the depth is Ok

(ii) Shear

$V = 360x2x0.458 \quad = \quad 330KN$

$v = \frac{33000}{2000x542} \quad 0.30N/mnl$

$\frac{100\,As}{bd} = 0.2$

$v_c = 0.40N/mm^2 > 0.30Nm$

The chosen section is Ok

1.2-0.62 = 0.48, 1.0xO.620= 0.620m

shear

At the critical section for shear, 1.0d from the column face

V = 367x3x0.48 = 529kN

$$v = \frac{529x10^3}{3000x620} = 0.28 N/mm^2$$

$$\frac{100As}{bd} = 0.32$$

Shear stress $v_c = 0.44 N/mm^2$

.. $0.28 < 0.44 N/mm^2$, hence the section is adequate i.e 3000x3000x750mm is okay.

22.3 PAD FOUNDATION FOR COLUMN C-ll, C-12, & C-13

N2372KN F-3

$$AF = \frac{2372x11.1}{1.47(300-0.55x20} \quad 6.14m^2$$

provide base 2.20x2.80 = $6.75m^2$

qc(Earth pressure) = $2372/6.75=351.4 KN/m^2$

Assume h = 600mm

100As = 100x5030 = 0.3

bd 3000x540

vc = *0.44N/mm2*

0.06< 0.44N/rrun2, the chosen depth is adequate

(iii) check for shear 0.90-0.54 = 0.46m

.45

.~: 1.2

2250

V = 351.4x2.25x0.46 = 364kN

v = 364x10^3 = *0.43N/mm^2*

2250x540

By vc from table 5.1 mosley = *0.43N/mm^2*

:. 0.42 < 0.43N/mm2

Hence section is OK

i.e 2250x3000x500 is OK

22.4 PAD FOUNDATION FOR C-2, C-2a C-3, C-3a & C-4

N=2388kN

AF=1.1x2388/1.47(300-(0.55x20) = 6.l8mm2

Provide base 2.5x2.5xO.6m, Area = 6.25mm2

d = 600-50-20-10 = 520 d

qc(Earth pressure) = 2388/6.25 = *382kN/m^2*

At the face of column b =2500

M= 382xO.952x2.5/2= 431kNm

V = 382x2.5x0.43 = 411Kn

v = 411xl03 = $0.32 N/mm^2$ < Ve = $0.4 N/mm^2$

2500x520

Hence, the seetin is okay

i.e 2.5x2.5xO.6m is okay

22.5 COMBINED FOOTINGS FOR COLUMN C-6A, C-6B

F-6'

Loads:

2N = 1485x = 2970

AF = 2970xl.l = 7.46m2

1.46x300

Provide footing 1.8x4.8 = 8.64m2

%(earth pressure) = 2970 = $344 Kn1m_2$

8.64

O.9m 3.0m O.9m

O.9m

1.8

O.9m

4.8m

1485KN 1485K

251

2000x542

100 As = 0.2
bd .

$v_c = 0.40 N/mm^2 > 0.30 Nm$

The chosen section is okay

~

22/CC FOUNDATIONS

PAD FOUNDATION FOR COLUMNS C_1 & C-26

22.1

N = 3531.3KN F-2

Allowed bearing pressure of soil $300 KN/m^2$

.. AF = 3531.3x1.1

1.46 (300 - (0.55 x 20)

$= 9.21 m^2$

pr vide

3.1

3.10xO.7m

Provide footing : 3.1 x 3.1x 0.7m

qc(Earth pressure) = *367.51* 3.1^2 = *392.4kn1m2*

At the col. face of the critical section: b = 3000mm

M = 367.5 x 3 *x1.22/2* = 1369 kNm

Min.d = 700-50-20-10 = 620mn

h=700mm

k = 847 x $10^6/30x300x620$

z = 0.95x620 = 637

As= $\frac{1369 \times 10^6}{0.87 \times 410 \times 637}$ = $6025 mm^2$

".PAD FOOTING FOR COLUMN C6 & C8

re-cs N = 2478kn
AF 2478 x 1.1 _. 6.42m2
1.47 (3N vcbv v vcv vc vc b bv b db b vb
bvc BN BN BBN BBNB BN NNB N BN/ NB NN B ? - 0.55 x 20)
Provide base 2.6 x 2¥6 x 0.5 = 6.76m2
q, (Earth Pressure) = 247_~ = 366.6kn/m2

Provide base 2.6x2.6x 0.55

1.oe

h
1.3
1.3

At the face of column, b = 2600mm
Moment M, = 366.6 x 2.6 x 1.08² x 0.5 ._ 55Iknm
d = 550 - 50 - 20 = 4~Omm
7r (I.Q5)(430·; 4%
As= 551 x 10⁶__ = 3388mm2
0.87 x 410 x 4085
Provide 11Y20 (3454mm2) both ways

d::: 480mm

f' <K ide 11 Y20 both ways

Checks

1. Min. reinforcement = 0.13 x 2600 x 400 + 100
- 1352mm.

ii. Punching shear.
Critical perimeter (1.5 x 0.48 x 2·+ 0.45) x 4
= 6.96m.

V = 366.6 (2.6² -7.56f)
= 1368 1169kN

At the column face
Shear stress, VI) = 2478 x 10³ =.= 2.22 < O.g-feu 2.22< 4.38·
2600 x 480

Punching shear stress = v = 1169 x 10³ = 0.32N/m7.
7560 x 480

=

= 100 x3768 = 0.30
2600 x 480
.by interpolation, Vo 0.44 > 0.32N/mm
Depth 550mm is okay.
'--- __ --J.._@i) Shear Stress

~

Shear force v = 367.5 x 3.1 x 0.610

= 695KN

v = 698.4 x 10^3 / 3100 x 640 = 0.35N/mm2

100As / bd = 0.3, vc = 0.44N/mm2

v, > 0.35N/mrn2

The chosen section is okay

dL

22.3 PAD FOUNDATION FOR COLUMN C-ll, C -12 & C -13

h=700mm

N 2372KN F-3

AF = 2372 XI.I / 1.47 (300 - 0.55 x 20)

b = 3100mm

Provide base 2.25 x 3.00 = 6.75m2

4.l0 IB -10 L = 3.8m, 300 x 300

d=637mm

t

Loading

From slab 15 - 2b:	11.81 x 6/2	=	35.44KN/m
From slab 15 - 3:	11.81 x 0.52h	=	2.99KN/m
Sub rnfllobean			0.71
	Q	=	39.14KN/m

Provide
13Y25
both ways

Reinforcement
okay

Reactions

3.8 R, = ½ 3.8 x 39.14 x 't, 3.8

R) = 25KN

R2 = ½ 39.l4 x 3.8 - 25

= 49KN

Post ofBMM.AX

25-39.14XxY2x=O

5.l5x^2 = 25

.: x= 25 / 5.15 =

BMM.AX

25x- 39.14 xlxlx / 3.8 2 3

25 x 2.2 - 1.7 x 2.2^3

55 -18.28 = 37KNM

h = 700 okay

Hence

Provide 50% of the reinforcement of beam IB - 8.

Provide 13Y25both ways (6380mm)

bar spacing >750 or 3d whichever is smaller

(i) Check for min reinforcement

$$\frac{100\,As}{bh} = \frac{100 \times 6025}{3000 \times 700} = 0.29 > 0.13$$

Max spacing = 750mm. therefore the reinforcement provided meets the requirement specified by the code for minimum area and maximum bar spacing in a slab

(ii) ,Check for punching shear

Critical perimeter - u=col. Perimeter + 8 x 1.5 col. ,

$$0.6x4+8x1.5xO.670 = 10.44m$$

Area with perimeter == $(0.6+3xO.620)^2$= $6.81m^2$

Punching shear force v = $367.5(3^2-(6.8114)^2$ = 2242kN

Punching shear stress $v = \frac{2242x10^3}{10440x620}$

by interpolation $v_c = 0.47 N/mm^2$

0.35<0.47, hence the chosen depth 700mm is okay

Check for shear 1.2-.67 1.0d = 670mm

~

$M1Y = \frac{360xix5}{2} = 900Knm$

$As = \frac{900 \ x10^6}{0.87x410xO.95x520} = 5108rnm^2$

Provide 17y20(5340rnm2) bottom

Provide 17y20@bottom

Min reinforcement - $\frac{0.13x5000x5340}{100}$

= 3380rnm2 <5108rnm2

Checks

(i) Punching shear:-

Critical perimeter u (1.5xO.542x2+0.9h)+

(l.5xO.520x2+0.9h)x2

= (2.08+2.01)x2 = 8.18m

$V = 360 \ [2x5-(\frac{1.5xO.52x2+0.9)(1.5xO.520x2+0.9)}{2}x2]$

360 =(10-[(2.08)(2.01)x2]

:. v = (10-8.36)360 = 589KN

$V = \frac{589000}{81880x542} = 0.13N/rnm2$

by u,= 0.40 N/rnm2 >0.13N/rnm2

the depth is okay

(ii) Shear

V ~ 360x2x0.458 =330KN

U = 33000 = 0.30N/rnm2

22.6 PAD FOOTING FOR COLUMNS C-S, C-=6a, C-6b

F-7&F-7a

N = 1800Kn :.2N = 3600Kn

characteristic = 1.lx3600 = 2694Kn

1.47

AF·=2694 = 9.32

300-(0.55x20)

566 80 y

1.0

LO

1.0 3.0m

Area of the rectangle = (3+2x)(2x) = 9.38

By solving for x = 0.96

Hence, adopt base area 2x5=10m2 = provide 2x5xO.6m

%(earth pressure) = 3600 = *360Knlm2*

10

REINFORCEMENT (a) (longitudinal bending)

(i) Cantitever: M\ = 3600 x 2.0x(1.0)2 = 360Knm

2

b = 2000, d=600-50-8 = 542mm

As = 360x106 1960mm2

0.87x410xO.95x542

Provide 7Y20 (2200mm2) bottom

7y20@290% botton

Tranverce bending

b = *5.qm,* d = 600-50-20-10 = 5comm

Z = *0.9SxS20* = 494

As= 43lxl06 = 2446mm2

0.87x410x494

Provide 13Y16(2613mm2) i.e Y16@180 both ways

Provide 13'· 6

Checks

(i) Min reinforcement - 100x2613 = 0.17>0.13

2S00x600

(ii) Punching shear:-

Critical perimeter = v = (l.SxO.52x2+0.6)4

= 8.64m

V = 382 *(2.S2_(8.64)2/4)*

= *60SKN*

v = *60S000* = *0.13N/mm2*

8640x520

100As = 100x2613 = 0.20

bd *2500xS20*

by interpolatin uc = *0.4N/mm2 >0.13*

hence, the depth is adequate.

Check for shear *0.9S-0.52* = 0.43

1.0d = *S20*

0.9S-0.52=0.43

1.0d=S20 ~l ~l

shear

d = 600-50-10= 540mm

At the face of column b=2250 mm

M = 351.4x2.25x2.8 =889.5kNm

z = 0.95x540 = 394mm

$$As = \frac{889.5x10^6}{0.87x910x394} = 4861mm^2$$

16Y20(5030m')

Provide (16Y20 (5030m2)

$My = 351.4x3xl.13^2$ = 667kNm

d ='600-50-20-10 = 520mm

d=600-50-20-10 = 52vmm

z = 0.95x520 = 494mm

$$As = \frac{667x10^6}{0.87x410x494} = 3785mm^2$$

Provide 13Y20 (4083mm2)

rovide 13Y20

Checks

i) Min reinforcement $\frac{100x5030}{2250x500}$ =0.45 > 0.13 OK

Spacing 750mm max. OK

Punching shear

ii) Critical perimeter - v = col. Perimeter +8x1.5d

=2(1.5xO.540x2+ 1.2+ 1.5xO.52x2+0.45)

= 9.66m

Table 5.1 mosley

V = *351.4(2.25x3-(9.66/4)* 2) = 323kN

$$v = \frac{323x10^3}{9660x540} = 0.06N/mm2$$

V = 366.6 x 2.6 x 0.6 572kn

v = $\frac{572000}{2600 \times 480}$ *0.46N/mm2*

EXTERNAL *R/W* AT CORNER WELLS

PI

~ P2

Assuming surchange load *of2.5KN/m* ^{2} & 8_5 =

he = $\frac{\sim}{r,}$ = $\frac{2.5}{20}$ = 0.125

PI X, = $\frac{1- \text{Sin"Q}}{\text{l+Sin'Q}}$ = $\frac{1- \text{Sin } 33°}{1-\text{Sin}33°}$ 0.295

PI = Karhe = 0.295 x 20 x 0.125 = *0.74KN/m2*

P_2 = Ka.rH = 0.295 x 20 x 3.65 = *21.5 KW/m2*

MA = 0.74 x 3.65^2 x 0.5 + 21.5 x 3.65^2 xl_- 3.5x

/23

0.62 x 0.5 xL = 4.929 + 47.739 - 0.2

3

= 52.5 KNm/m

h = 1000mm, d=230-30-10 = 190mm

k $\frac{52.5 \times 10^6}{30 \times 1000 \times 190^2}$ 0.05

:. :6 = 0.95d = 180.5

m = $\frac{52.5 \times 10^6}{0.87 \times 410 \times 180.5}$ = *8.l5mm2/m*

Provide proviye y12 @ 135 't,- External N.F

Provide min. reinforcement for Runners

= $\frac{0.13 \times 1000 \times 230}{100}$ = *299mm2/m*

Provide y10@280 %

Mo = 52.5 + (0.74 x 3.65 + 2l.5 x 3.65 x 0.5 - 3.5 x 0.65 x 0.5) x 0.4 + 3.65 x 20 x 0.25 = 59.2KNm/m

N 1.1 x 3.65 x 20.0 + 0.27 x 0.6 x 20 +0.23 x 3.65 x 24.0 + 1.6 x 0.4 x 24

1115.7KN

e $\frac{59.2}{115.7}$ 0.5Im >.LQ = 0.267

6

max = $\frac{2 \times 115.7}{3 \times 1.0 \text{ (L.§\_ - } 0.51)}$ = $266 KN/m^2$ < 300 KN/m^2

2

FOUNDATIONS

22/CC PAD FOUNDATION FOR COLUMN

£1 & C6b F-2

22.1 N 3531.3 KN

A footiy = 3531 xLI / 1.47 (300 - 0.55 x 20) = 9.21m^2

Provide base 3.10 x 3.10 = 9.61m2

a = 3000 - 600 = 2500/2 = 1250

h > 0.5a = 0.5 x 1250 = 625min

Hence provide h = 700

Earth pressure qc = 3531.3 / 3.12 = 367.5KN/M2

At the face of column which is critical

m = 367.5 x 3.1 x (3.1 x 0.5)2 / 2 1369KNm

d = 700 - 50 - 20 - 10 = 640mm

k = 1369 x 106/30 x 3100 x 6402

= 0.04

..:Z = 0.95d = 608

Ab = 1369 x 106/0.87 x 410 x 605

= 6312mm2

'.'Provide 11'-:25 both ways

'"~80mm2)

CciE~XS

1. Min. L,;.:,f(' ·-:rf.'-n' = O.13bh%

= 0.13 x 3100 x /50/100 =

3023mm2 < 63&Omm

11. ';")ul' ~\,.,.,

critical perimeter - v = (1.5 x .64 x 2 + 0.6) 4

= 10.08m

Punching shear force V = 367.5 (3.12 - (J Q.9~i) / 4

367.5 (9.61 - 6.35)

1198KN

v = 1210 x 103 / 10040 x 640 = 0.19N/mm2

100AS / br = 100 x 6380 / 3100 x 640 = 0.3

By interpolation,

vc = 0.44> 0.19 N/mrn2

Hence depth 750mm is okay

iii. Shear: (a-l.Od) 0.64

Shear perimeter = 1.25 - (1 x 0.637)

= 0.610

CHAPTER FOUR

4.0 DISCUSSION OF RESULTS

This project revealed the advantages of ribbed slab over the solid slab. The design of the 300mm thick ribbed slab with the ribs spaced at 520mm center - center would have taken an equivalent of 200mm thick solid slab and is considered being uneconomical.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

5.1 CONCLUSIONS

The aim of structural design is to have a safe and economical design and detailing of this project, it can be said that the aim of this project has been achieved. The building was designed as reinforced concrete structure with ribbed slab of300mm thick (200~ clay pots with lOOmm thick topping). The stair cases and lift walls are introduced as shear walls to neutralize the effect of wind.

5.2 RECOMMENDATION

Specifications should be strictly adhere to, qualified and experienced professionals should be commissioned to execute the project. The structural engineers should be allowed free hand to exercise his discretion in the execution of any projects.

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