CHAPTER 1

INTRODUCTION

1.1 Background

Today, Pokhara is a rapidly urbanizing city with building construction at just about every corner of the city that one can see. Nowadays, with the awareness level of the building owners increasing than in the past, the trend of having a building analyzed scientifically before it is actually constructed is growing popular, especially in case of medium to large commercial buildings, which is a good thing because such a practice helps construction of more safer buildings which can eventually lead to avoidance of loss of lives and property in case of a structural failure.

A designer has to deal with various structures ranging from simple ones to more complex multistoried frame buildings, shell roofs bridges etc. These structure are subjected to various load like concentrated loads uniformly distributed loads, uniformly varying loads live loads, earthquake loads and dynamic forces. The structure transfers the loads acting on it to the supports and ultimately to the ground. While transferring the loads acting on the structure, the members of the structure are subjected to the internal forces like axial forces, shearing forces, bending and torsional moments.

Structural Analysis deals with analyzing these internal forces in the members of the structures. Structural Design deals with sizing various members of the structures to resist the internal forces to which they are subjected during their effective life span. Unless the proper Structural Detailing method is adopted the structural design will be no more effective. The Indian Standard Code of Practice should be thoroughly adopted for proper analysis, design and detailing with respect to safety, economy, stability and strength.

The projected selected by our group is a RCC complex building located at Damside, Pokhara.

According to IS 1893:2002, Pokhara lies on V^{th} Zone, the severest one. Hence the effect of earthquake is pre-dominant than the wind load. So, the building is analyzed for Earthquake as lateral Load. The seismic coefficient design method as stipulated in IS 1893:2002 is applied to analyze the building for earthquake. Special reinforced concrete moment resisting frame is considered as the main structural system of the building.

The project report has been prepared in complete conformity with various stipulations in Indian Standards, Code of Practice for Plain and Reinforced Concrete IS 456-2000, Design Aids for Reinforced Concrete to IS 456-2000(SP-16), Criteria Earthquake Resistant Design Structures IS 1893-2000, Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces- Code of Practice IS 13920-1993, and Detailing SP-34. Use of these codes have emphasized on providing sufficient safety, economy, strength and ductility besides satisfactory serviceability requirements of cracking and deflection in concrete structures. These codes are based on principles of Limit State of Design.

This project work has been undertaken as a partial requirement for B.E. degree in Civil Engineering. This project work contains structural analysis, design and detailing of a purposed RCC complex building located in Kaski District. All the theoretical knowledge on analysis and design acquired on the course work are utilized with practical application. The main objective of the project is to acquaint in the practical aspects of Civil Engineering. We, being the budding engineers of tomorrow, are interested in such analysis and design of structures which will, we hope, help us in similar jobs that we might have in our hands in the future.

1.2 Theme of Project work

This group under the project work has undertaken the structural analysis and design of RCC complex building. The main aim of the project work under the title is to acquire knowledge and skill with an emphasis of practical application. Besides the utilization of analytical methods and design approaches, exposure and application of various available codes of practices is another aim of the work.

1.3 Building Description

Building Type	:	Residential Building, purposed location at Damside pokhara
Structural System	:	RCC Space Frame
Plinth area covered	:	1129 sqft.
Type of Foundation	:	Raft footing
No. of Storey	:	4
Floor Height	:	10’ for all floors
Bearing capacity	:	130 KN/m^2 (assumed)
Expansion Joints	:	no expansion joint has been provided

The design is intended to serve for residential purpose.

1.4 Code of Practices

Following codes of practices developed by Bureau of Indian Standards were followed in the analysis and design of building:

1. IS 456:2000 (Code of practice for plain and reinforced concrete)
2. IS 1893 (part 1):2002 (Criteria for earthquake resistant design of structures)
3. IS 13920: 1993 (Code of practice for ductile detailing of reinforced concrete structures subjected to seismic forces)
4. IS 875 (part 1):1987 (to assess dead loads)
5. IS 875 (part 2):1987 (to assess live loads)
6. IS 875 (part 5):1987 (for load combinations)

7. SP 16 and SP 34 (design aids and hands book for detailing)

CHAPTER 2

STRUCTURAL SYSTEM AND PRELIMINARY DESIGN

2.1 Structural System

Any structure is made up of structural elements (load carrying, such as beams and columns and non-structural elements (such as partitions, false ceilings, doors). The structural elements put together, constitute the structural systems. Its function is to resist effectively the action of gravitational and environmental loads, and to transmit the resulting forces to the supporting ground without significantly disturbing the geometry, integrity and serviceability of the structure.

2.2 Structural Arrangement Plan

The planning of the building has been done as per available land area, shape, space according to building by laws. The positioning of columns, staircases, toilets, bathrooms, elevators etc are appropriately done and accordingly Beam arrangements is carried out so that the whole building will be aesthetically, functionally and economically feasible.

The aim of design is the achievements of an acceptable probability that structures being design will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and adequate resistance to the effect of misuse and fire.

2.3 Need of Preliminary Design

It is necessary to know the preliminary section of the structure for the detail analysis. As the section should be given initially while doing analysis in almost all software, the need of preliminary design is vital. Only dead loads and live loads are considered while doing preliminary design. Preliminary design is carried out to estimate approximate size of the structural members before analysis of structure.

2.4 Preliminary Design

Preliminary design is carried out to estimate approximate size of the structural members before analysis of structure. Grid diagram is the basic factor for analysis in both Approximate and Exact method and is presented below.

2.4.1 For Slab

$$Depth\ of\ slab, (d) = \frac{Shorter\ Span}{Basic\ Value\ x\ Modification\ Factor}$$

Taking Modification Factor, MF = 1.4

$$d = \frac{3900}{26\,x\,1.4} = 107.143\text{mm}$$

Say d = 107 mm

Taking Clear cover = 20mm

Overall Depth of Slab, D = 127mm

Floor Finish thickness = 48mm

Total Thickness = 175mm

2.4.2 For Beam

Straight Beam

$$Total\ Depth, (D) = \frac{Longer\ Span}{12} = \frac{4500}{12} = 375mm$$

Say, D = 380mm

Width of Beam B = 9 inches =225mm

2.4.3 For Column

Square Column

Approximate Vertical load = 1000 KN

Factored Vertical load = 1.5 x 1000 = 1500 KN

Considering Earthquake load, total axial load = 1.5 x 1500 = 2250KN

Required area of cross-section = 2250 x 1000 / 25 = 90000mm^2

Required size of square column = $\sqrt{90000}$ = 300 mm

Say, D = 14 inches=350 m, B = 14 inches=350 mm

CHAPTER 3

ANALYSIS

For the analysis of structure, first of all calculations of live load and earth quake load is done, secondly these loads as well as frame structure is introduced in sap2000.

3.1 Load Combination

In the course of analysis, different load cases and combinations are considered to obtain the most critical stresses in the element of the structure. The load cases considered for the structural analysis are:

i. Dead Load (DL)

ii. Live Load (LL)

iii. Earthquake load in X (EQ_x) static

iv. Earthquake load in Y (EQ_y) static

Following load combination as per IS 1893(Part I):2002 are adopted:

i. 1.5(DL + LL)

ii. 1.2(DL + LL + EQ_x)

iii. 1.2(DL + LL - EQ_x)

iv. 1.2(DL + LL + EQ_y)

v. 1.2(DL + LL - EQ_y)

vi. 1.5(DL + EQ_x)

vii. 1.5(DL - EQ_x)

viii. 1.5(DL + EQ_y)

ix. 1.5(DL - EQ_y)

x. 0.9DL+1.5EQ_x

xi. 0.9DL-1.5EQ_x

xii. $0.9DL+1.5EQ_y$

xiii. $0.9DL-1.5EQ_y$

After checking the results, it was found that the stresses developed are most critical for the following load combinations:

i. 1.5 (DL + LL)

ii. 1.2 (DL + LL + EQ_x)

iii. 1.2 (DL + LL - EQ_x)

iv. 1.2 (DL + LL + EQ_y)

v. 1.2(DL + LL –Eqy)

3.2 Load calculations:

Calculation of lumped masses (weights) at the floor levels.

The earthquake forces are calculated for the full dead load plus 25% of live load.

The live load on roof is assumed to be zero. [Clause 7.3.2 of IS 1893]

3.2.1 Lateral load calculation:

Lateral load calculation								
S.N	**mass**	**weight**	**floor height(hi)**	**Ah**	**Vb**	**wi hi2**	**wi hi2/(∑wi hi2)**	**Qi(KN)**
1	160.97	1579.116	3.048	0.09	536.7237	14670.46	0.038199949	20.50
2	167.37	1641.9	6.096			61014.99	0.158874956	85.27
3	167.37	1641.9	9.144			137283.7	0.357468651	191.86
4	103.1	1011.411	12.192			150341.1	0.391468196	210.11
5	9.1	89.271	15.24			20733.87	0.053988248	28.98
		5963.597				384044.1		

3.3Storey Drift

It is the displacement of one level relative to the other level above or below. According to IS 1893:2002 Clause 7.11.1, the storey drift due to the minimum specified design lateral force with partial load factor of 1.0 shall not exceed 0.004

times the storey height. Hence, our building is safe against the storey drift.

TABLE: Joint Displacements								
Joint	**OutputCase**	**CaseType**	**U1**	**U2**	**U3**		displacement in x-	
No.		Text	ft	ft	ft	height	direction	drift ratio
top floor	eqx	LinStatic	0.036323	0.0023	0.000766	3.048	0.005398	0.001771
4th floor	eqx	LinStatic	0.030925	0.002059	0.000747	3.048	0.005955	0.001954
3rd floor	eqx	LinStatic	0.02497	0.001285	0.000689	3.048	0.011251	0.003691
2nd floor	eqx	LinStatic	0.013719	0.000282	0.000559	3.048	0.003257	0.001069
1st floor	eqx	LinStatic	0.010462	-0.000001518	0.000328	3.048	0.010462	0.003432
ground floo	eqx	LinStatic	0	0	0			

CHAPTER 4

SAFETY OF BUILDING AGAINST EARTHQUAKE

4.1 Building construction trends in Nepal

In Nepal, the informal sector of owner builder produces at least 98% of the dwelling in country. As Nepal lies in seismically very active region, it is important to make effort towards earthly quake safety of such building construction.

Until yet, the owner take advice from friends, neighbors etc professional advise in rarely sought, even in urban areas and if solicited is limited to the preparation of submission drawing for municipality permits. Therefore owner builds buildings are almost non-engineered. Building owner is always oriented towards cost minimization. Building labors are also traditional, they can play pivotal roles in construction attributes as they provided over all technical and organizational support. Development of mass understanding of the cause and effect of earthquake obtain mainly from observation of structure performance of building designing part earthquake construction makes now possible in pin pointing about our weakness of building construction and helps to identify the solution to avert the failure against earthquake. This is particularly true for non engineered building since there earthquake resistant design is mostly based on observed behavior of building during past earthquake and engineer judgment.

In Nepal 1st category of building is those which are built according to tradition using indigenous wisdom, experience and skill got through a long run of time in history.

It is observed that building generally have a good performance in past earthquake. Indigenous buildings are gradually funding and replace with the second category of non engineered building that do not adopt traditional skill and craft in detailing and material use. Whereas in urban areas, with the advent of modern materials like cement and steel. R.C and masonry building are being constructed as the use of concrete and steel in building is total structural solution. The modern material is imported but the technology required for making proper use of them is not being adopted at least in consideration of earthquake force.

Absence of earthquake consideration in building planning and design deficiencies construction practices seems main cause of structural failure of the building.

4.2 Failure in masonry building

Wall fall apart, failure at corner of wall, failure at corner of opening, failure gable walls, roof and floor separate from their support walls shear off diagonally, building and separation of stone masonry walls.

4.3 In case of reframe with masonry in few buildings

Failure of columns with excessive bucking of rods, out of plane failures of in full walls, shear failure of columns. Brittle failure of columns, brittle failure of building with pan cape village failure at beam column collection severe damage in wall and column of extreme corner hammering between two adjacent buildings.

The main structural and constructional weakness due to

- ❖ Geometry of building
- ❖ Lateral load resisting system
- ❖ Ductility
- ❖ Construction practice

From the history it is observed that irregular building with large offsets and unsymmetrical from sustain more damage in earthquake loading. Which is cause due to high torsion developed in extreme corner of building? In unsymmetrical or irregular building, high eccentricity developed due to non coincidence of centre of mass and centre of rigidity cause large torsion moments. Which thereby produces stress concentration at extreme edges in buildings? Uneven distribution of stiffness which mainly achieved by columns and walls causes severe damages in buildings.

Absence of lateral load resisting system in non engineered building is another cause of failure. Important factor ductility is the capacity of building elements to undergo displacement without loss of strength. Therefore

building members and their connections should be such that they allow plastic deformation without failure.

Construction practice is the most important aspects of the non engineered construction in owner built houses. Connection of building elements, proper wall to wall construction, roof/floor connection, integration of wall are lacking in masonry construction. Therefore poor construction practices, poor workmanship, poor materials are the weakness of non engineered building.

4.4 Seismic Improvement

Thus to overcome the earthquake effects, building should be earthquake resistance. In spite of inherent weakness in material and building forms which could be alters abruptly due to financial and cultural reasons, better engineering inputs from prudent judgment can be incorporated in these buildings to achieved better performance in seismic events. Interconnection should be well between orthogonal wall of masonry structure, „L" and „T" shaped covering specified length of wall in each direction can be provided in joints in regular vertical intervals. Besides these sill and lintel bands continuously running overall wall in same level would make structure act as integral units. To provide tensile strength minimum reinforcing bars in recommended in corners, joints and sides of openings running vertically from foundation to roof. If steel bars are not available well seasoned bamboo of similar wooden log can replace such bars. To make roof/floor effective in diaphragm action to X or V bracing in each panel could be applied anchor beam action diagonally. Since gable walls in longitudinal run are very

effective in earthquake resistance action. Roof system should be anchored with wall to ensure not be detached in earthquake. In stone masonry, bond stone covering whole wall width should be provided in class internal in vertical wall area to avoid delimitation. The extra cost by in corporate these features in traditional houses would not be more than 10%.

In RCC building, detailing of re-bar should be maintained to produce ductile response. In addition besides shear reinforcement, even distribution of in-filled wall with proper anchoring which are directly concerned to lateral loading, better workmanship is equally important to make building safer in earthquake.

CHAPTER -5

SECTION DESIGN

5.1 Design

Design of each member is done in different sub-section under the main section. In the design, limit state method is used.

5.1.1 Limit State Method

In this method, design is based on limit state concept, the structure shall be designed to withstand safely all loads liable to act on it throughout its life; it shall also satisfy the serviceability requirements, such as limitations on deflection and cracking. The acceptable limit for the safety and serviceability requirements before failure occurs is called a 'limit state'.

The object of design based on limit state concept is to achieved an acceptable probability that a structure will not become unserviceable in each life time for the use for which it is intended i.e. it will not reach a limit state. Thus this method is designed as a method which limits the structural usefulness of the material of the structure up to a certain load at which acceptable limit of safety and serviceability are applied so that the failure of structure does not occur. The acceptable limit applied for safety requirement before failure of structure takes place is termed as limit state. The most important of these limit state which must be examined in design are as below:

- Limit state of collapse
- Limit state of serviceability

a) Limit state of collapse:

This state corresponds to the maximum load carrying capacity. Violation of collapse limits state implies failure in the sense that a clearly defined limit state of structural usefulness has been exceeded. However it does not mean a complete collapse. This limit state may corresponds to

- ➢ Flexure

- Compression
- Shear
- Torsion

b) Limit state of serviceability:

This method corresponds to a development of excessive deformation and is used for checking for members in whom magnitude of deformation may limit the use of structure or its components. This limit states may corresponds to

- Deflections
- Cracking
- Vibration

To satisfy this limit state deflection, cracking and vibration must not be excessive. The aim of design is to achieve acceptable probabilistic that the structure will not become unfit for the use for which it is intended, that is, that it will not reach a limit state.

Assumptions for flexural member:

i. Plane sections normal to the axis of the member remain plane after bending.
ii. The maximum strain in concrete at the outermost compression fiber is 0.0035.
iii. The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoidal, parabola or any other shape which results in prediction of strength in substantial agreement with the result of test. For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor γ_m = 1.5 shall be applied in addition to this.
iv. The tensile strength of concrete is ignored.
v. The design stresses in reinforcement are derived from representative stress-strain Curve for the type of steel used. For the design purposes the partial safety factor γ_m =1.15 shall be applied.

vi. The maximum strain in the tension reinforcement in the section at failure shall not

be less than: $f_y/1.15E_s + 0.002$

Where,

f_y = characteristic strength of steel

E_s = modulus of elasticity of steel

Assumptions of limit state of collapse for compression:

In addition to the assumptions given above from i) to v), the following shall be assumed:

i) The maximum compressive strain in concrete in axial compression is taken as 0.002.

ii) The maximum compressive strain at highly compressed extreme fiber in concrete subjected to axial compressive and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at the least compressed extreme fiber.

iii) The limiting values of the depth of neutral axis for different grades of steel based on the assumptions are as follows:

Table 5: Limiting values of depth of neutral axis

F_y	$X_{u,\,max}/d$
250	0.53
415	0.48
500	0.46

Materials adopted in our design:

M20 (1:1.5:3)

Fe415

Use of SP16, IS456-2000, IS1893-2002, IS13920-1993, SP34:

The code we use for the design is IS456-2000; IS1893-2002, IS13920-1993 and Design aids are SP16 and SP34. Suitable material, quality control, adequate detailing and good supervision are equally important during implementation of the project.

5.2 Design of Structural Elements

The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and adequate resistance to the effects of misuse and fire.

In the limit state design concept, the structure shall be designed to withstand safely all loads liable to act on it throughout its life; it shall also satisfy the serviceability requirements such as limitation on deflection and cracking. The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state. The aim of limit state design is to achieve acceptable probabilities that the structure will not become unfit for the use for which it is intended, i.e. it will not reach a limit state. The design is in compliance with clearly defined standards for materials, production, workmanship, and maintenance and use of the structure in service.

CHAPTER 6
SLAB

6.1 Introduction

Slab is a plate element forming floors and roofs of buildings and carrying distributed loads primarily by flexure. Slab may be simply supported or continuous over one or more supports and is classified according to the manner of support:

- One way slabs spanning in one direction , that is , supported on two opposite edges
- Two way slabs spanning in both directions that is, supported on four edges.

If the cross-sectional areas of the three basic structural elements: beam, slab, column are related to the amount of steel reinforcement provided, it will be seen that present of steel is usually maximum in a column than in beam and the least in a slab.

6.2Worth worthy point about slab

- A slab is essentially a beam or a flexure member. The minimum span should not be less than four times the overall depth of a slab.
- Slabs are analyzed and designed as having a unit width, that is, 1m wide beam.
- Compression reinforcement is use only in exceptional case in a slab.
- Shear stress is usually very low and shear reinforcement is not provided in slabs, its preferred to increase depth instead.
- Temperature reinforcement is invariably provided at right angles to the main longitudinal reinforcement in a slab.
- No earthquake force acts in slab it is taken as rigid diaphragm.

6.3Design of restrained slab

A slab have its few or all edges restrained, the degree of restrained may vary depending whether it is a continuous over supports or it is cast monolithically with its supporting beams. A hogging or negative bending moment will develop in the top face of the slab at the supported sides. In these slabs the corners are prevented from lifting and made provision is made for torsion, the maximum moment Mx and My at mid span on strips of unit width for spans lx, and ly are as:

$Mx = \beta x \ w \ lx^2$

$My = \beta y \ w \ ly^2$

Where, βx and βy = moment coefficient

6.3.1Design of slab BCEF

Here, effective size 4.5m*4.2m

Ly = 4.5m

Lx = 4.2m

Moment along short span Mx and along long span My are given by,

$Mx^+ = \beta x^+ \ wlx^2$

$Mx^- = \beta x^- \ wlx^2$

$My^+ = \beta y^+ \ wly^2$

$My = \beta y^- \ wly^2$

For two adjacent edge discontinuous and ly/lx at 1.071

$\beta x^+ = 0.04$

$\beta x^- = 0.053$

$\beta y^+ = 0.035$

$\beta y^- = 0.047$

Effective depth of slab = 107mm

Overall depth of lab = 127mm

Dead laod od slab = 25*0.127 = 3.18 KN/m^2

Live load + Dead load due partition load =2+1 = 3KN/m^2

Total load = 3.18 + 3 =6.18KN/m^2

Factored load = 1.5*6.18 = 9.27KN/m^2

From above equation,

Mx = 6.5KN-m

Mx = 8.7KN-m

My = 5.7KN-m

My = 7.7KN-m

Hence, BMmax = 8.7 KN-m

Effective depth of slab d = $\sqrt{\frac{BMmax}{0.138\sigma b}}$

$= \sqrt{\frac{8.7*10^6}{0.138*20*1000}}$

$=56 \not> 102$mm OK

Now, $Mx^+ = 0.87*\sigma y*Ast^{x+}[d - \sigma y*Ast^{x+}/(\sigma ck*b)]$

$6.5*10^6 = 0.87*415*Ast^{x+}[100 - 415*Ast^{x+} /(20*1000)]$

Solving,

$Ast^{x+} = 145.2mm^2$

Similailly,

$Ast^{x-} = 196.01mm^2$

$Ast^{y+} = 126.94mm^2$

$Ast^{y-} = 172.81mm^2$

Choose bar,

Ast^{x+} = 5-Φ8mm@250mm per m c/c

Ast^{x-} = 4-Φ8mm@330mm per m c/c

Ast^{y+} = 3-Φ8mm@496mm per m c/c

Ast^{y-} = 4-Φ8mm@330mm per m c/c

Provision of torsion bar in case of corner held down

Ast(corner) = $0.75*Ast^{x+provided}$

= 0.75*251

= $188.25mm^2$

Choose bar, 4-Φ8mm@140mm c/c

Ast(corner provided) =201mm

Check

1) **Deflection criteria**

$fs = 0.58*\sigma y*(Ast^{x+required}/Ast^{x+provided})$

fs =0.58*415*(145.2/150) = 233Mpa

$\%Ast = 100*Ast^{x+provided} /b*d$

= 100*251 / 1000*107

=0.234%

Now, $\gamma = 1.55$

$L/d \leq \alpha\beta\gamma\lambda\delta$

$4200/107 \leq 26*1*1.55*1*1$

39 < 40.33 (Ok)

2) Minimum and Maximum Reinforcement

Ast_{max} = 0.04*b*D = 0.04*1000*127 =5080mm^2

Ast_{min} = 0.85*b*D/σy =0.85*1000*107/415 = 219mm^2

$Ast_{provided}$ = 251mm^2

Ast_{max} >$Ast_{provided}$ >Ast_{min} Hence ok.

3) Development length

Ld = 0.87*σy*Φ/4τbd

But for deform bar τbd = 1.6*1.2

= 0.87*415*8/(4*1.2*1.6)

= 376.1mm

Designs of slabs are shown in table below:

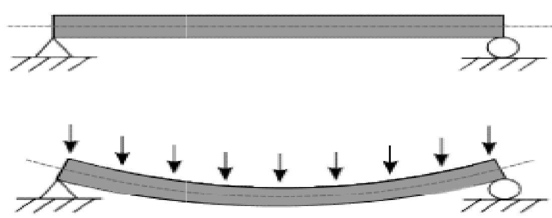
For floor1, floor 2. floor3												
S.N	slab	Lex(m)	Ley(m)	Ley/lex	Slab	d(m)	adopt d	adopt D	L.L	D.L	Wu(KN/m)	Slab Type
1	ABDE	3.9	4.5	1.15	Two way	0.094	107	127	2	3.18	9.2625	Three edge discont.(one long edge continious)
2	BCFE	4.2	4.5	1.07	Two way	0.101	107	127	2	3.18	9.2625	Two adj edge discontinious
4	EFIH	2.4	4.2	1.75	Two way	0.058	107	127	2	3.18	9.2625	Two short edge discontinious
5	GHKJ	3.9	4.5	1.15	Two way	0.094	107	127	2	3.18	9.2625	Three edge discont.(one long edge continious)
6	HILK	4.2	4.5	1.07	Two way	0.101	107	127	2	3.18	9.2625	One long edge discontinious
7	KLMN	1.2	4.2	3.50	One way		107	127	2	3.18	9.2625	One way slab
Top floor												
S.N	slab	Lex(m)	Ley(m)	Ley/lex	Slab	d(m)	adopt d	adopt D	L.L	D.L	Wu(KN/m)	Slab Type
1	ABDE	3.9	4.5	1.2	Two way	0.0938	107	127	1.5	3.18	8.5125	Three edge discont.(one long edge continious)
2	BCFE	4.2	4.5	1.1	Two way	0.101	107	127	1.5	3.18	8.5125	Two adj edge discontinious
4	EFIH	2.4	4.2	1.8	Two way	0.0577	107	127	1.5	3.18	8.5125	Two short edge discontinious
5	GHKJ	3.9	4.5	1.2	Two way	0.0938	107	127	1.5	3.18	8.5125	Three edge discont.(one long edge continious)
6	HILK	4.2	4.5	1.1	Two way	0.101	107	127	1.5	3.18	8.5125	One long edge discontinious
7	KLMN	1.2	4.2	3.5	One way		107	127	1.5	3.18	8.5125	One Way slab

For floor1, floor 2. floor3																
slab	βx+	βy+	βx-	βy-	Mx+	My+	Mx-	My-	d required	result	minAst	Astx+(mm2)	Asty+	Astx-	Asty-
ABDE	0.0505	0.043	0.0675	0	7.1	6.1	9.5	0.0	58.70613	**OK**	214.71	158.97	136.06	214.71	0
BCFE	0.04	0.035	0.047	0.047	5.7	5.7	7.7	7.7	52.75519	**OK**	172.81	126.94	126.94	172.81	172.81
EFIH	0.049	0.035	0.065	0	2.6	1.9	3.5	0.0	35.45152	**OK**	77.31	57.24	41.72	77.31	0
GHKJ	0.0505	0.043	0.0675	0	7.1	6.1	9.5	0.0	58.70613	**OK**	214.71	158.97	136.06	214.71	0
HILK	0.028	0.028	0.037	0.037	4.6	4.6	6.0	6.0	46.80768	**OK**	133.78	102.02	102.02	133.78	133.78
KLMN									50.51	**Ok**	157.59	157.59	152.4		
Top floor															
slab	βx+	βy+	βx-	βy-	Mx+	My+	Mx-	My-	d required	result	min Ast	Astx+	Asty+	Astx-	Asty-
ABDE	0.0425	0.035	0.0565	0.047	5.5	4.5	7.3	6.1	51.48969	**OK**	179.75	133.78	108.8	179.75	147.49
BCFE	0.04	0.035	0.053	0.047	6.0	5.3	8.0	7.1	53.70549	**OK**	196.01	145.2	126.94	196.01	172.81
EFIH	0.048	0.028	0.064	0.037	2.4	1.4	3.1	1.8	33.7235	**OK**	75.07	57.24	32.89	75.07	43.93
GHKJ	0.0425	0.035	0.0565	0.047	5.5	4.5	7.3	6.1	51.48969	**OK**	179.75	133.78	108.8	179.75	147.49
HILK	0.033	0.028	0.044	0.037	5.0	4.2	6.6	5.6	48.93359	**OK**	161.27	120.12	102.02	161.27	133.78
KLMN									50.51	**Ok**	157.59	157.59	152.4		

CHAPTER 7

BEAM

7.1 Introduction

- Horizontal member locating X,Y-Axis
- Structural members that is capable of withstanding load primarily by resisting bending.
- 

The design of beam requires the determination of steel for the section fixed from the preliminary design. The design of the section may result as singly or doubly reinforcement which may be ascertained by comparing the design moment (M_u) with the moment of resistance of balanced section (M_{ul}) and the section is usually design as under reinforced section.

7.2 Design

Design of beam is attached in Appendix.

CHAPTER 8

CHAPTER 8

COLUMN

8.1 Introduction

Columns are the vertical members that are subjected to axial loads and moment acting from two directions (Bi-axially). All columns are subjected to some moment which may be due to accidental eccentricity or due to end restraint imposed by monolithically placed beams or slabs. The strength of column depends upon the strength of the material, shape and size of the cross section, length and the degree of positional and directional restraint at its ends.

The column section may be rectangular, square or circular shaped depending upon the architectural or structural requirements.

A column may be classified as follows based on types of loading:

a. Axially loaded column

b. A column subjected to axial load and uniaxial bending and

c. A column subjected to axial load and biaxial bending

The design of column section for given axial load and biaxial moments can be made by pre assigning the section and then checking adequacy. The design of column depends upon the eccentricity of loading and the moment acting in different directions. The minimum eccentricity specified by the IS 456-2000(clause 39.2) is:

e_{min} = Lo/500 + D/30 but not less than 20mm

where, Lo = unsupported length of column

D = lateral dimension in plane of bending

If e_{min} is less than 0.05D, then column is designed as axially loaded column .If the eccentricity exceeds 0.05D, then column is designed for both moment and axial load.

Assumptions:

The following are made for the limit state of collapse in compression:

i. Plane section normal to the axis remains plane after bending.

ii. The relationship between stress-strain distributions in concrete is assumed to be parabolic. The maximum compressive stress is equal to 0.67x fck/1.5 or 0.446fck

iii. The tensile strength of concrete is ignored.

iv. The stress in reinforcement is derived from the representative stress-strain curve for the type of steel used.

v. The maximum compressive strain in axial compression is taken as 0.002.

vi. The maximum compressive strain at the highly compressed extreme fiber in concrete subjected to axial compression and bending, but when there is no tension on the section is taken as 0.0035 minus 0.75 times the strain at the highly compressed extreme fiber.

vii. The maximum compressive strain at the highly compressed extreme fiber in concrete subjected to axial compression and bending, when part of the section in tension is taken as 0.0035.

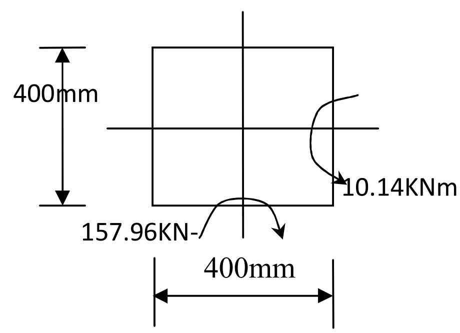
8.2 Design (Square Column)

Detail design of all column members is attached in Appendix.

Column type: Square column

Concrete Grade = M20

Steel Grade = 415 (TOR)

Reference	Step	Calculations	Output
		Column Identification : **Column No.64** , First Floor	
			
	1.	***Known data:***	
		Overall Depth of Column, D = 16” = 400mm	
		Width of Column, B = 16” = 400mm	
		Height, L = 3.048m	
		Clear height, l = 2.648m	**D =400mm**
		Assume following data:	**B =400mm**
		Clear cover, d= 27.5mm	**L = 3.048**
		diameter of longitudinal reinforcement, ø = 25mm	**effective cover**
		So, effective cover, d’= 27.5+25/2 = 40mm	**d’= 40mm**
	2.	***Check for Axial Stress:***	
		Maximum Axial Load = 1230 KN	
		Factored Axial Stress =	

IS 3920:1993 cl.7.1.1 **IS 456:2000, cl.26.5.3.1**	 **3.**	$\frac{1230x1.5x1000}{400x400} = 11.53Mpa$ Axial Stress = 11.53 > 0.1fck(2.5) Hence, design as Column Member. Check for Member Size: Width of Column, B =400 mm > 200mm Depth of Column, D = 400 mm B/D = 400/400 = 1 > 0.4 Hence, OK Effective Length, le = 0.65 x l = 0.65 x 2.648= 1.72m Check for Short and Slender Column: le/D = (1720)/400 = 4.30<12,(short column),ok **Limiting Longitudinal Reinforcement:** Min. Reinforcement, = 0.8% of BD = 0.8 x 400x400/100 = 1280 mm^2 Max. Reinforcement, Max. Asc = 4% of BD = 0.04*400*400 = 6400 mm^2	 **le = 1.72m** **Min. Asc=1280mm²** **Max. Asc = 6400mm²**

	4.	***Design for section:*** Pu= 488.263KN (Value corresponding to Max. absolute sum of Mx and My) Mz= 10.14 KNm My= 157.96 KNm $\frac{d'}{D} = \frac{40}{400} = 0.1$ Assume reinforcement is uniformly distributed on four sides, $\frac{\mathrm{p}}{\mathrm{f_{ck}}} = 0.1$ p = 2%	
SP16 chart 44		$\frac{P_u}{f_{ck}BD} = \frac{732.39\mathrm{x}1000}{20x400x400} = 0.23$ $\frac{M_u}{f_{ck}BD^2} = 0.15$, Mux1=Muy1=192KN	**Muxl=192 KNm** **Muyl =192 KNm** **Puz =2436 KN**
IS456:2000 cl.39.6		***Check for Biaxial Moment*** $p_{uz} = 0.45 f_{ck} A_g + 0.75 f_y \ A_{sc}$ = 2436 KN Pu/Puz = 0.30	**αn = 0.83**

		αn = 0.83	
		$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.83 < 1, ok$$	
		Providing 8- ø 25 mm	**Asc=3200 mm^2**
		Asc provided = 3927mm^2	**Asc(Actual)= 3927mm^2**
		$$p(provided) = \frac{3927}{400x400} x100\% = 2.45\%$$	
IS 456:2000 cl 26.5.3.2.c.1		***Design of Lateral Ties:***	
		Pitch of the Ties:	
		Sv ≤	
		i)The least dimension of the member.	
		B=D=400mm	
		ii) Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied.	
		16*25=400mm	
		iii)300mm	
		Thus, provide 8mm ø lateral ties @ 100mmc/c at L/4 from each corner and @150c/c in central part.	
IS 456:2000 cl 26.5.3.2.c.2		**Diameter of ties:**	
		øt ≥ not less than 6mm	
		≥ ¼ *maximum diameter of longitudinal	

		reinforcement = 0.25*25 =6.25mm Hence, adopt ties of 8mmø	

CHAPTER 9

FOUNDATION

9.1 Introduction

Foundations are structural elements that transfer loads from the building or individual columns to the earth below. If these loads are to be properly transmitted, foundations must be designed to prevent excessive settlement and rotation, to minimize differential settlement and to provide adequate safety against sliding and overturning.

It is necessary to provide a continuous footing under all the columns and walls if the loads transmitted by the columns in a structure are so heavy or the allowable soil bearing pressure small. Such a footing is called a raft or Mat Foundation. The raft is divided into series of continuous strips centered on the appropriate column rows in the both directions as shown in figure below. The shear and bending moment diagrams may be drawn using continuous beam analysis or coefficients for each strip. The depth is selected to satisfy shear requirements. The steel requirements will vary from strip. This method generally gives a conservative design since the interaction of adjacent strips is neglected. We have provided a mat foundation without beams (i.e. solid slab)

Taking the base of footing to be 1.5m below the ground level and safe bearing capacity of the soil as 130KN/m^2 and load factor 1.5, design of raft foundation is as:

Total vertical column load

= 636+1082+682+893+1230+724+885+1219+736+625+952+640+328+256

= 10888KN

$$\overline{X}_{\text{load}} = \frac{3039*0.2+481 \;*4.1+3038*8.3}{10888} = 4.183$$

$$\overline{Y}_{\text{load}} = \frac{584*0.2+221 \;*1.4+284 \;*5.9+2847*8.3+240 \;*12.8}{10888} = 6.826$$

C.G. of the figure (x = 4.532, y = 6.761) as shown in figure

e_x = -0.169m

e_y = 0.065m

$M_x = P_{total}$*ey = 10888*0.065 = 707.72KN-m

$M_y = P_{total}$*ex = 10888*(-0.169) = -1840.072KN-m

I_{xx} = 1378.004m^4

I_{yy} = 632.368m^4

Soil pressure@different point is given by

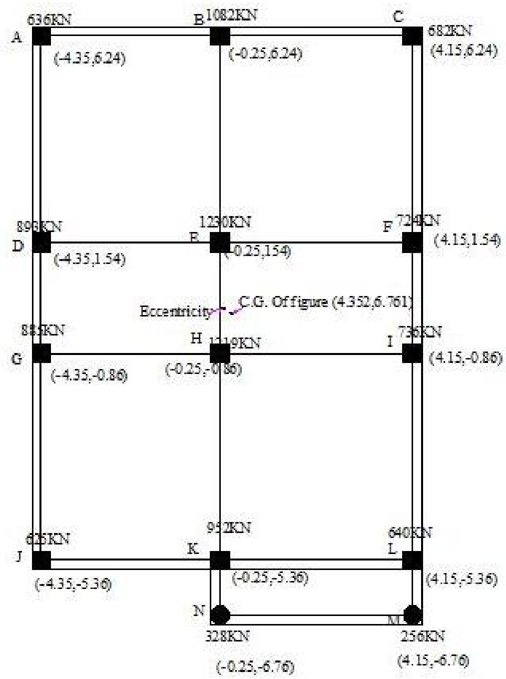
$\sigma = P/A \pm M_x*y/I_x \pm M_y*x/I_y$

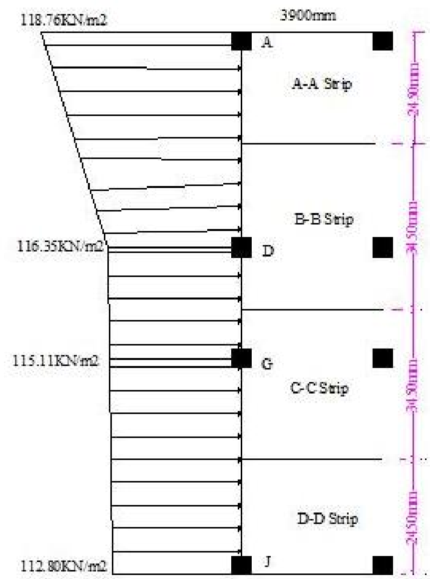
Calculation has been done in excel as shown below:

Joint name	Total loadP(KN)	Area(m2)	Mx	My	Ix	Iy	X-coordinate	Y-coordinate	Positive σ
J	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-4.352	-5.361	112.80
G	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-4.352	-0.861	115.11
D	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-4.352	1.539	116.35
A	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-4.352	6.239	118.76
N	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-0.252	-6.761	100.15
K	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-0.252	-5.361	100.87
H	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-0.252	-0.861	103.18
E	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-0.252	1.539	104.42
B	10888.00	105.82	707.72	-1840.07	1378.00	632.37	-0.252	6.239	106.83
M	10888.00	105.82	707.72	-1840.07	1378.00	632.37	4.148	-6.761	87.35
L	10888.00	105.82	707.72	-1840.07	1378.00	632.37	4.148	-5.361	88.07
I	10888.00	105.82	707.72	-1840.07	1378.00	632.37	4.148	-0.861	90.38
G	10888.00	105.82	707.72	-1840.07	1378.00	632.37	4.148	1.539	91.61
C	10888.00	105.82	707.72	-1840.07	1378.00	632.37	4.148	6.239	94.03
								Maxm value	118.76

Maximum pressure =118.76KN/m^2< 130KN/m^2, OK

Pressure diagram for calculation:





For strip A-A with 2.45m width and span 3.9m having soil pressure 118.76KN/m^2

M $_{A\text{-}A}$= $wl^2/10 = 118.76*2.45*3.9^2/10 = 442.553$KN-m

Similarly for strip B-B, C-C, D-D, with width 3.45m, 3.45m and 2.45m respectively and span 3.9m

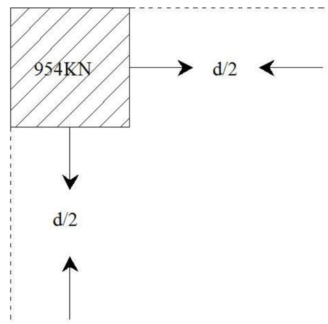
M $_{B\text{-}B}$ = 616.864KN-m

M c-c = 607.287KN-m

M $_{D\text{-}D}$ = 424.648KN-m

M $_{max}$ = 616.86KN-m

Calculation of depth of footing considering punching shear (2-way shear effect)



$Pu = 1.5*636 = 954KN$

$\tau v = Pu/b_o d$

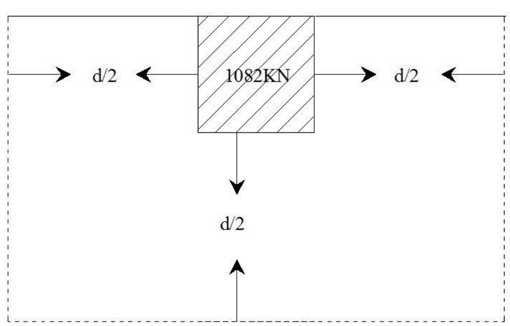
$b_o = 400 + d/2 + 400 + d/2 = 800 + d$

$\tau_v = 954*1000/(800+d)*d$, $\tau_c = 0.25\sqrt{\sigma ck} = 0.25\sqrt{20} = 1.118$

therefore, $954*1000/(800d+d^2) = 1.118$

solving, d = 606.633mm

again,

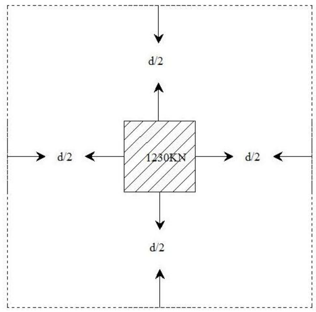


$Pu = 1.5*1082 = 1623$ KN

$b_o = 1200 + 2d$

solving, as in above, d= 603.244mm

again,



Pu = 1.5*1230 = 1845 KN

b_o = 1600 + 4d

Solving, d= 472.731mm

So adopt d = 610mm

Taking top and bottom cover of 50mm, total depth = 710mm

Calculation of Ast :

Mmax = 0.87* σy*Ast(d- σy*Ast/ σck *b)

$616.86*10^6$ = 0.87*415*Ast(610 -415*Ast/20*3450)

Ast = 4391mm^2

Choose bar, 15-Φ20mm@230mm c/c

Check for minimum Ast

0.12%bd = 0.12% * 610 *3450 = 2525mm^2 < 4391mm^2, OK

9.1 Summary of design of mat foundation

Concrete grade=20 N/mm^2

Steel grade = 415 N/mm^2

Safe bearing capacity=130 KN/m^2

Total depth of foundation=710mm with clear cover =50 top and bottom each.

CHAPTER 10

STAIRCASE

10.1 Introduction

Staircase is a structural member, which joins one floor to another. The design of staircase is same as design of slab.

10.2 Design of staircase

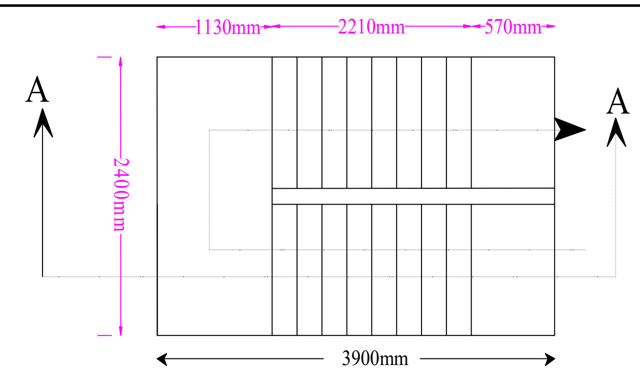
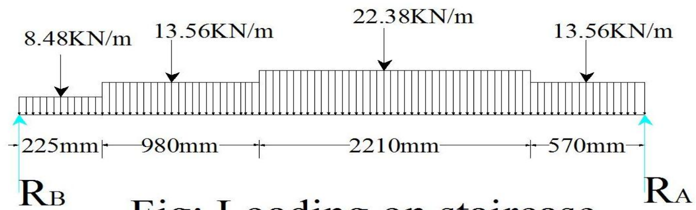


Fig1: Plan

Steps	**Ground floor**		
	Calculation	**Output**	**Unit**
A)	**Known Data:**		
	Floor Height=	3.00	m
	Number of flight=	2.00	
	Height of each flight=	1.50	
	Height of Riser(R.)=	0.15	m
	Number of riser in each flight=	10.00	

	Number of tread in each flight=	9.00	
	Length of going=	2.21	m
	length of Tread (T)=	0.245	m
	Width of stair=	1.13	m
	Length of landing=	1.13	m
	Effective length ((l)	3.91	m
	Over all depth of waist slab(D)	0.20	m
B)	**Load on going:**		
	Area of step section=0.5*R*T	0.02	m2
	Area of inclined slab=0.2*0.28727	0.06	m2
	Area of floor finish	0.02	m2
	Total area=	0.10	KN/m2
	Unit weight of concrete=	25.00	KN/m
	Dead load of step,1m in width and 245 mm in plan length	10.20	KN/m2
	Dead load per m2 on span=	2.50	KN/m
	Live load=	3.00	KN/m^2
	Total factored load	22.38	KN/m^2
	Total load per unit length = 13.2x1.13	14.92	KN/m
C)	**Load on landing:**		
	Self weight of slab	5.00	KN/m2
	Live load=	3.00	KN/m^2
	Total factored load	13.56	KN/m
D)	**Checking of Depth:**		
	Reaction at A(R_A)	38.24	KN
	Reaction at B (R_B)=	34.15	KN

	 Fig: Loading on staircase		
	Maxium bending moment(M)	40.39	KNm
	Required depth($d_{req.}$) $=\sqrt{[M/(0.138 x f_{ck} x b)]}$		
	$=\sqrt{[40.39x10^6/(0.138x20x1.13x1000)]}$		
	=113.82 mm < d, ok		
E)	**Reinforcement Required:**		
	Main bar:		
	$M=0.87fyA_{st}[d\text{-}fy\ A_{st}/(fckb)]$		
	or,$40.39x10^6$ =0.87x415xAst[185-415xAst/(20x1.13x1000)]		
	or, Ast =646.13mm2	646.13	mm^2
	Using 6- 12 mm diameter bar,		
	c/c spacing of bar	218.00	mm
	Provide 12 mm dia. bar @218 mm c/c		
	Ast (provided)	678.00	mm^2
	>646.13 mm2,ok		
	Distribution bar:		
	Area of steel=0.12% of b*D=	250.86	mm^2
	Using 8mm dia. Bar per meter		
	c/c spacing of bar	218.00	mm
	Provide 8 mm dia. bar @ 218 mm c/c per meter		
	Ast (provided)	301.00	mm^2
	>250.86mm2, ok		
	Design of landing:		
A)	**Load on landing:**		

	Self weight of slab= 25x0.20	5.00	KN/m^2
	Live load=	3.00	KN/m^2
	Total factored load per unit length	13.56	KN/m
	Caculation	Output	
B)	**Reinforcement Required:**		
	Main bar:		
	$M=0.87fyA_{st}[d-fyA_{st}/(fckb)]$		
	or, $28.27x10^6$ =0.87x415xAst[185-415xAst/(20x1130)]		
	or,Ast =442.69mm2	442.69	mm^2
	Using 10 mm diameter bar,		
	c/c spacing of bar	218.00	mm
	Provide 10 mm dia. bar @28 mm c/c		
	Ast(provided)	471.00	mm^2
	471 >442.69 mm2,ok		
C)	**Check for deflection:**		
	From Graph, $(l_{effective}/d)$<= $\alpha\beta_{\gamma}\lambda$đ, ok		
	20<30 ($_{\gamma}$=1.5,α=20),ok		
	percentage of steel used = 100xAst(provided) /(b*D)=	0.32	%
	From Graph, ᵞ=1.5		
D)	**Check for shear:**		
	Maximum shear force =R_A =	38.24	KN
	Shear stress($ح_v$) =R_A/(b*D) =	0.18	N/mm2
	percentage of steel used = 100xAst(provided) /(b*D)=	0.32	%
	Shear strength of M20 at 0.32% steel ($ح_c$)= 0.39 N/mm2	0.39	N/mm2

	For over all depth 200 mm,k=1.2		
	so, $\tau_{c'}$ = kxτ_c =1.2x0.39 =0.47 N/mm2		
	Maxium shear stress for M20 concrete =2.8 N/mm2		
	So, $\tau_v < \tau_{c'} < \tau_{c\ max}$ ok		
E)	**Check for Development Length:**		
	Design bond stress (τ_{bd}) =1.6x1.2 =	1.92	N/mm2
	Design bond stress (τ_{bd}) =1.6x1.2 =	1.92	N/mm2
	Development Length for 12 ǿ bar,		
	L_d=ǿ$\sigma_s/4\tau_{bd}$=12x0.87x415/(4x1.92)=	564.14	mm
	Moment of resistance of 12 ǿ bar=		
	M=0.87fy$A_{st(provided)}$[d-fy$A_{st(provided)}$/(fckb)]	42.24	KNm
	Shear Force (V) =	38.24	KN
	Lo=effective depth or 12ǿ,whichever is greater=	185.00	mm
	L_d<=1.4*1.2*1.6(1.3M/V+Lo)/0.87*415	12	mm
	Distribution bar:		
	Area of steel=0.12% of bxD=	250.86	mm^2
	Using 8mm dia. Bar per meter		
	c/c spacing of bar	218.00	mm
	Provide 8 mm dia. bar @ 218 mm c/c per meter		
	Ast(provided)	301.00	mm^2
	>250.86 mm2, ok		

The type of staircase is doglegged and other reinforcement details are shown in drawing section.

CHAPTER 11

COST ESTIMATION

The dimensions, length, breadth and height or depth are to be taken out from the drawings (plan, elevations, and sections) From the study of the drawings, the building is to be imagine and pictured in the mind and the dimensions are to be taken out correctly. There is no hard and fast rule for finding out dimensions but the dimensions are to be taken out accurately. Here, estimation is carried out using plinth area method. Plinth area is the built-up covered area of a building measured at floor level of any storey. Plinth area is calculated by taking the external dimensions of the building at the floor level excluding the plinth offsets if any.

Plinth area of our building =1129 sqft.

Rate per sqft. = Rs.2000 as per pokhara sub-municipality

Total cost per floor = Rs 2000*1129 = Rs.22, 58,000

CHAPTER 11

CONCLUSION

Hence we have successfully completed the project entitled "SESMIC ANALYSIS AND DESIGN OF EARTHQUAKE RESISTANCE STRUCTURAL BUILDING". This project work has helped us to acquire knowledge and skill with an emphasis of practical application. The exposure and application of various available codes of practices was possible besides the utilization of analytical methods and design approaches. We were acquainted to different software related to structural analysis and design during this project.

Design and layout of the building services like pipeline, electrical appliances, sanitary and sewage system were not covered in this project. The environmental, social and economical condition of the locality was not taken into consideration. The project work was only related with the practical application of the studied courses in the field. Detail cost estimate of the project was not included in this report.