ANALYSIS AND DESIGN OF STRUCTURAL COMPONENTS OF (G+10) STOREY RCC BUILDINDING USING ETABS

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Submission date: 14-May-2020 08:52PM (UTC+0530)

Submission ID: 1324178605

File name: TURAL_COMPONENTS_OF_G_10_STOREY_RCC_BUILDINDING_USING_ETABS.docx (7.67M)

Word count: 5990

Character count: 28645

ANALYSIS AND DESIGN OF STRUCTURAL COMPONENTS OF (G+10) STOREY RCC BUILDINDING USING ETABS

GALGOTIAS UNIVERSITY

(Established under Galgotias University Uttar Pradesh Act No. 14 of 2011)

By

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Acknowledgements

First of all thanks to Galgotias university who helped me to complete the practicum work and the practicum report, leading to the M Tech Structural Civil Engineering degree.

I would like to thank every families and friends that participate on my life and get me in this intensity and individuals who support and share idea and also helping me to be like this.

I would like to pay my gratitude to our respected Chair, Department of Civil Engineering, Prof. Dr. Vishwash who gave me the opportunity to do the report on Analysis and Design Multi Story Building (G+10) Using Etabs

I would also thank my respective supervisor Dr. Manju Dominic his endless support in his office. All teachers of civil engineering department who brought me to my present

ABSTRACT

The main objective of this project is to analyze and design a multi-story building 3D-dimensional reinforce concrete frame the design of reinforced concrete slabs, beams, columns, footings and Staircase were made by hand calculations according to ACI code and compare the results by using ETABS. In order to design. it is important to first obtain the plan of the particular building that is, positioning of the particular rooms Drawing room, bed room, kitchen toilet etc. such that they serve their respective purpose and also suiting to the requirement and comfort of the inhabitants

An office building were used with reinforced concrete frame consists of three floors where the maximum area of floor. Each floor consists of twelve offices. We used AutoCAD programs to complete the architectural design, ETABS to design and analyze the structure of building. Finally, we arranged the results as architectural and structural maps for this building.

ETABS has a very interactive user interface which allows the user to draw the frame and input the load values dimensions and materials properties. Then according to the specified criteria assigned it analysis the structure and design the members with reinforcement details for reinforced concrete frames.

The design process of structural planning and design requires not only imagination and conceptual thinking but also sound knowledge of science of structural engineering besides the knowledge of practical aspects, such as recent design codes, bye laws, backed up by ample experience, intuition and judgment.

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Abbreviations:

FF = Floor finish

DL = Dead load

LL = Live load

DW = Distributed wall load

PW = Partition wall load

SD = Super Dead load

lb = Pound force

psf = pound per square ft.

kip = kilo-pound force

RCC = Reinforced Cement Concrete

UBC = Uniform Building Code

ETABS = Extended Three Dimensional Analysis of Building System

RCC = Reinforced Cement Concrete

List of Symbols:

ρ = Reinforcement ratio

Ø = Strength reduction factor

fy = Yielding stress of steel

$f\ c'$ = Compressive strength of concrete

Mu = Factored moment

Pu = Factored load

Ast = Area of reinforcing steel

Ag = Total gross area

Av = Total steel area of web reinforcement

CHAPTER 1 : Introduction

1.1. Origin of the report

This report has been prepared as an integral part of the internship program for the Master of Science in Civil Engineering under the Department of Civil Engineering Galgotia University. Nominated as the organization for the practicum while honorable Prof. Dr. Md. Manju Domonic.

1.2 Objectives:

The main objectives of this report is to show the Analysis and Design of a RCC Building by USD- Ultimate Strength Design Method and also by an integrated building design software ETABS where all the Design consideration has been taken from the Indian standard Code. To learn about the practical design concept of a RCC building. To get idea about the implementation of theoretical and practical design specification of RCC materials. To get some ideas about the handling of an integrated building design software like ETABS.

1.3 ETABS

ETABS is a special purpose computer program developed specifically for building structures. It provides the Structural Engineer with all the tools necessary to create modify analyze design and optimize building models. These features are fully integrated in a single Windows based graphical user interface that is unmatched in terms of ease of use productivity and capability

1.4. Background

1.4.1. Concrete and Reinforced Concrete

Concrete is a mixture of sand , gravel, crushed rock. Sometimes one or more admixtures are added to change certain characteristics of the concrete such as its workability durability and time of hardening. As with most rocklike substances, concrete has a high compressive strength and a very low tensile strength steel reinforcement provides the tensile strength lacking in the concrete. Steel reinforcing is also capable of resisting compression forces and is used in columns as well as in other situations.

1.4.2. RCC as building materials

There is no simple answer to this question, inasmuch as both of these materials ha When a particular type of structure is being considered, the designer may be ve many excellent characteristics that can be utilized successfully for so many types of structures. In fact, they are often used together in the same structures with wonderful results. The selection of the structural material to be used for a particular building depends on the height and span of the structure, the material market, foundation conditions, local building codes, and architectural considerations in our country RCC is mostly used because constituent materials are locally available and low cost, can be produced in any desired shapes and a lower grade of skilled labor is required than steel construction

Literature Review:

In this paper, structural behavior of multi- story building for different plan configurations like rectangular C L and I-shape. Modeling of 10- story R.C.C. framed building is done on the ETABS software for analysis. Post analysis of the structure, maximum shear forces, bending moments and maximum story displacement are computed and then compared for all the analyzed cases.

CHAPTER 2: Methodology and Specification

2.1. Methodology

A ten-storied building has been analyzed and designed by using Ultimate Strength Design (USD) following the necessary codes from IS CODE.

2.2. Design Specification

I have considered to complete my analysis and design of the residential building project. In some points specifications have been simplified for the purpose of ease of calculation. Each and every information has been used here as per the direction of IS456 CODE-2000 and also following the references book.

Live load = 40 psf (for slab) = 120 psf (for stair)

DW = 45 psf (assuming 5 in. brick wall)

PW = 450 plf (assuming 5 in. thick & 9' height brick wall.)

FF = 20 psf (for Slab) = 30 psf (for Stair)

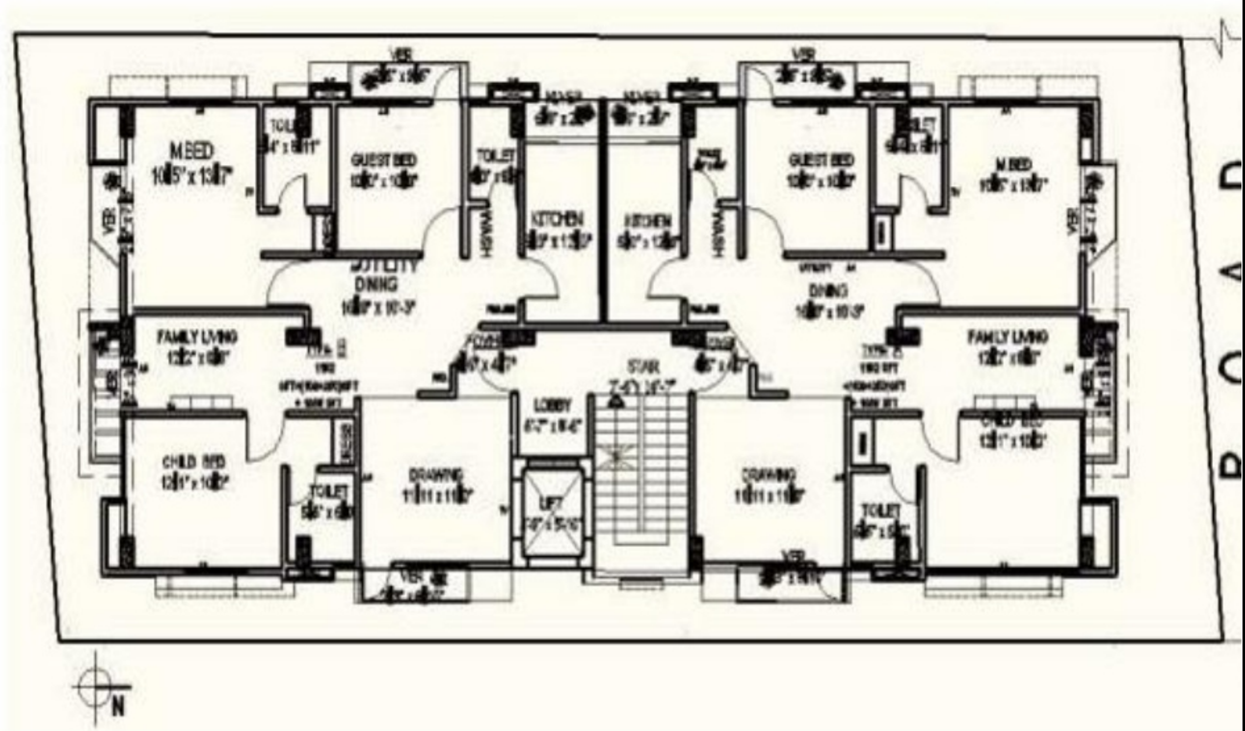
$f\ c'$ = 2800 psi (for Slab & Beam) = 3500 psi (for Column)

fy = 60,000 psi

Ø = 0.65 (for Column)

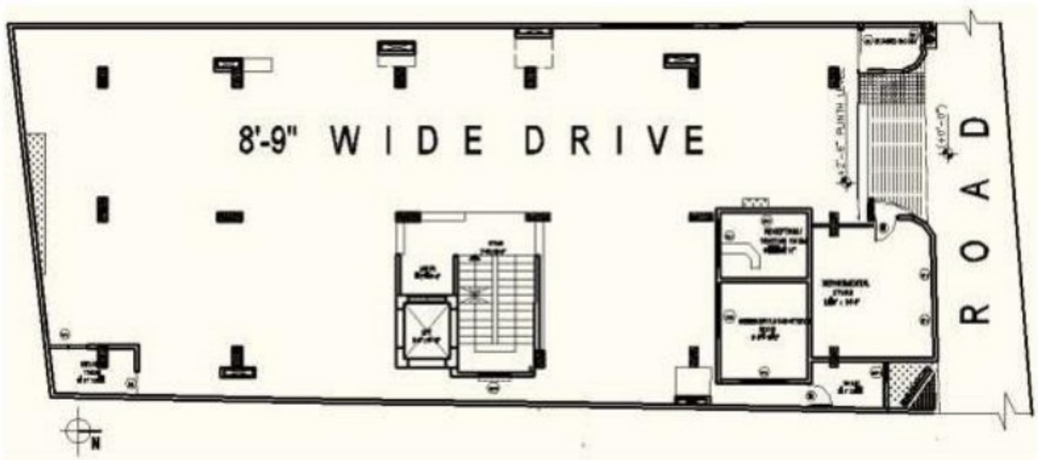
CHAPTER 3 : Assigned Project View

3.1. Architectural floor plan



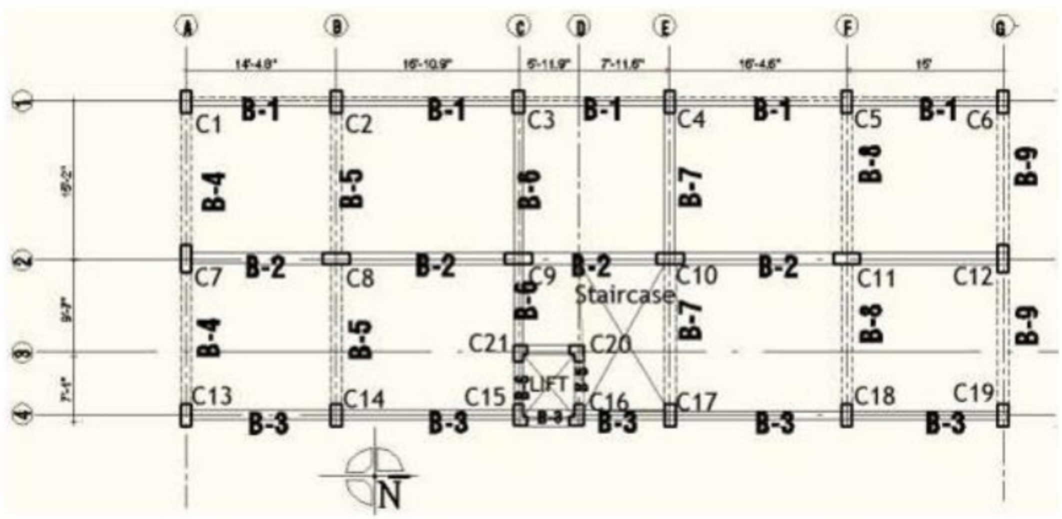
Typical Floor Plan

Ground Floor Plan



Ground Floor Plan

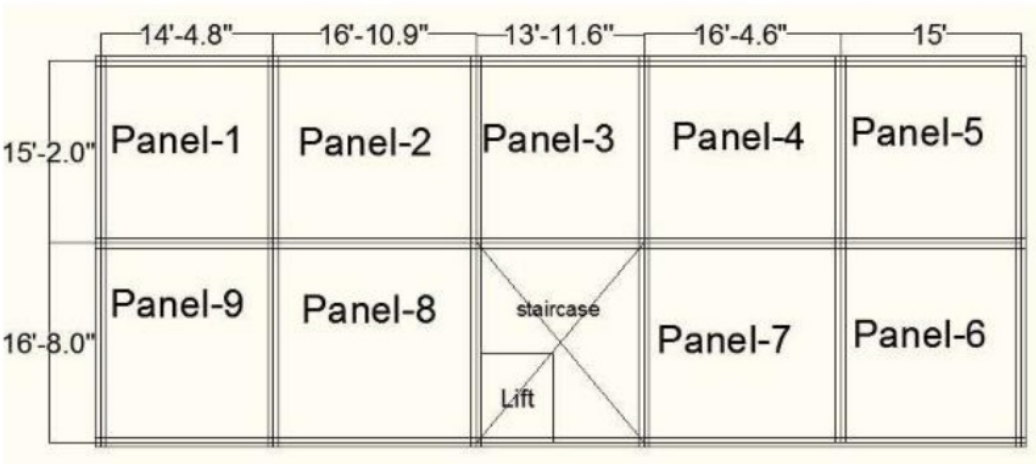
Beam-Column Layout Plan



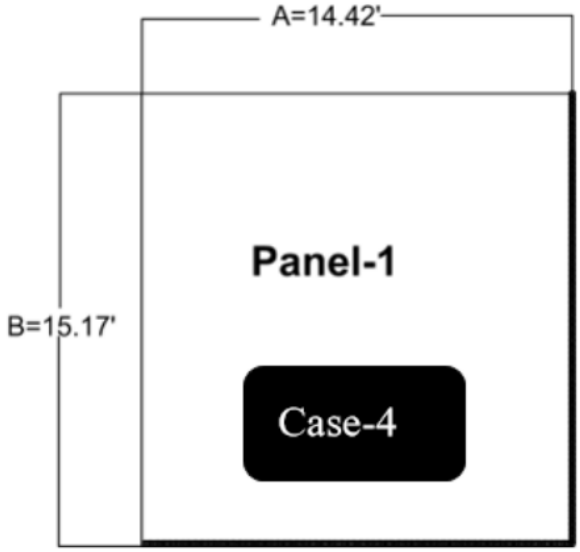
CHAPTER 4 : Slab Design

Slab Design

Panel View



Design of Panel-1



4.2.1. Slab Thickness Calculation

Here,

the ratio of longer to shorter span = B A = 15.17 14.42 = 1.052 < 2 ; So, the panel will be design as two way slab.

Here,

For calculating the desirable slab thickness a trial value of h= 6 inch will be introduced and beam section of 10x18 in. and 10x20 in. correspondingly for the short-span and long -span direction.

Moment of Inertia of beam strips:

For short-span direction I = 1/12∗ 10 ∗ 183 >> 4860 $in4$ 15

For long-span direction I = 1/12*10*203 >> 6666.67 $in4$

Moment of inertia for slab strips:

For 7.63 ft. edge width (along long span), I = 1/12*7.63*12*63 >> 1665.36 $in4$

For 8 ft. edge width (along short span) I = 1/12*8*12*63 $in4$ >> 1746.36 $in4$

For 14.42 ft. width (short direction) I = 1/12*14.42*12* $in4$ >> 3114.72 $in4$

For 15.17 ft. width (long direction) I = 1/12*15.17*12*63 $in4$ >> 3276.72 $in4$

Relative stiffness of beam to slab αf =$EcbIb$ $EcsIs$

>>αf = Ib Is [Ecb&Ecs are the modulus of elasticity of beam & slab; usually are same]

For, edge beam(long-span direction): $\alpha f1$ = 6666.67 1665.36

>> 4 For, edge beam (short-span direction): $\alpha f2$ = 4860 1746.36

>> 2.78 For, the 14.42 ft. beam (short-span direction):$\alpha f3$ = 6666.67 3276.72

>> 1.48 For, the 15.17 ft. beam (long-span direction): $\alpha f4$ = 6666.67 3114.72

>> 2.14

So Average of αfm = 4+2.78+1.48+2.14 4

>> 2.6

The ratio of longer to shorter clear span, β = 14.33 13.58

>>1.05

Then the minimum thickness of the Panel will be,

h =

ln (0.8+

fy 200

)

36+9β

>> 14.33∗12(0.8+

60 200

)

36+9∗1.05

>> 4.16 in.

We will consider 5 in. depth for further calculation.

4.2.2. Load Calculation:

Live load : 40 psf

4

Dead load : (FF + DW + Self Weight) >> (20 + 45 + 62.5) >> 127.5 psf

As per IS CODE [3] Factored dead load, WDL = 1.4*DL >> 1.4*127.5 >> 178.5 psf

[3] Factored live load, WLL = 1.7*LL >> 1.7*40 >>68psf Total factored load, W = (178.5+68) >>246.5 psf

4.2.3. Moment Calculation

We will follow the Moment coefficient Method

Ratio of shorter to longer clear span of the panel, m= $A\ B$

>> 14.003 14.753

>> 0.95 ; Case-4 (As per conditions of continuity)

Negative moment at continuous edge: $MA\ neg$= $CA\ neg$*W*A2 >> 0.055*246.5*14.422 >> 34996 in-lb

$MB\ neg$ = $CB\ neg$*W*B2 >> 0.045*246.5*15.172 >> 31689 in-lb

Positive moment at mid-span: $MA\ pos\ DL$ = $CA\ DL$*WDL*A2 >> 0.030*178.5*14.422 >> 13362 in-lb

$MA\ pos\ LL$ = $CA\ LL$* WLL*A2 >> 0.035*68*14.422 >> 6681 in-lb

Total positive moment at short direction , $MA\ pos$ = 20043 in-lb

$MB\ pos\ DL$= $CB\ DL$*WDL*B2 >> 0.024*178.5*15.172 >> 11830 in-lb

$MB\ pos\ LL$ = $CB\ LL$* WLL*B2 >> 0.029*68*15.172 >> 6127 in-lb

Total positive moment at long direction, $MB\ pos$ = 17957 in-lb

Negative moment in Discontinuous edges: $MA\ neg$= 1/3($MA\ pos$) >>1/3* 20043 >> 6681 in-lb

$MB\ neg$ = 1/3 ($MB\ pos$) >> 1/3*17957 >> 5986 in-lb

5.2.4. Selection of Reinforcement:

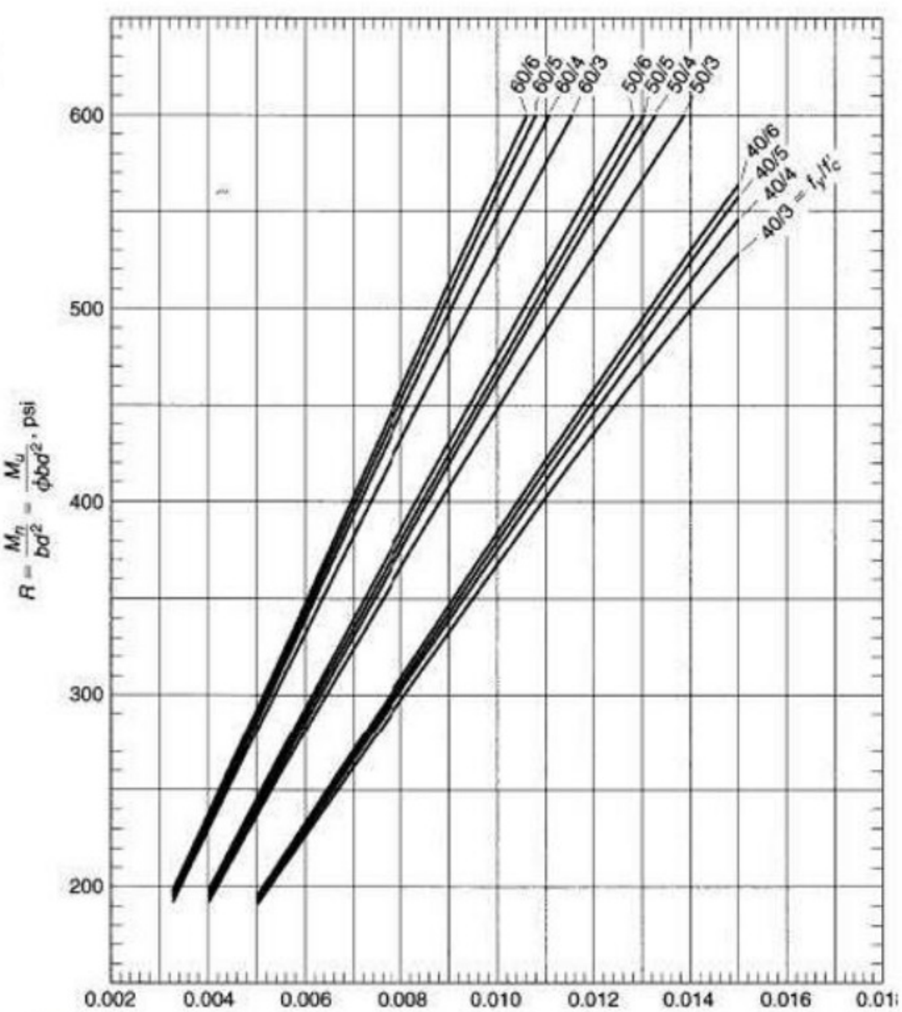
Short direction: Mid-span (Positive Moment): Flexural resistance factor, R = Mu $\Phi bd2$

= 20043 0.9∗12∗42

= 116 ; ρ = 0.0033

AS = ρbd = 0.0033*12*4 >> 0.16 in2/ft. Spacing, S = 0.11 ∗ 12/0.16 >> 8.25 in

Using, 8.00 inch.



Graph for Moment Capacity of Rectangular Sections.

Continuous edge (Negative Moment):

R = $Mu\ \Phi bd2$ = 34996 0.9∗12∗42 = 203 ; ρ = 0.0034 AS = ρbd = 0.0034*12*4 >> 0.16 in2/ft.

Spacing,

S = 0.11 ∗ 12/0.16

>> 8.25 in. Using, 8.00 inch

Discontinuous edge (Negative Moment): Negative moment at the discontinuous edge is one-third of the positive moment in the span. So it will be adequate to bend up every third bar from the bottom to provide negative moment steel at the discontinuous edge. But it will make 24 in. spacing which is larger than the maximum spacing of 3t = 15 in. permitted by the code.

Hence for the discontinuous edge every alternate bar will be bent up from the bottom steel.

Long direction: Mid-span (positive moment):

R = $Mu\ \Phi bd2$

= 17957 0.9∗12∗3.62

= 128 ; ρ = 0.0033

AS =   ρbd = 0.0033*12*3.6 >> 0.14   in2/ft. [Positive moment steel of long span direction is placed above the positive moment steel of short span direction. So, d= 4 - (10/25.4) =   3.6 in.]

Spacing, S = 0.11 $*$ 12/0.14

>> 9.4 in. Using, 9 inch.

32

Continuous edge (Negative moment): R = Mu $\Phi bd2$ =   31689 0.9$*$12$*$42   = 227; ρ = 0.004 AS = 0.004*12*3.6 >>0.173   in2/ft.

Spacing,

S = 0.11 $*$ 12/0.173 >> 7.63 in. Using,   7.5 inch

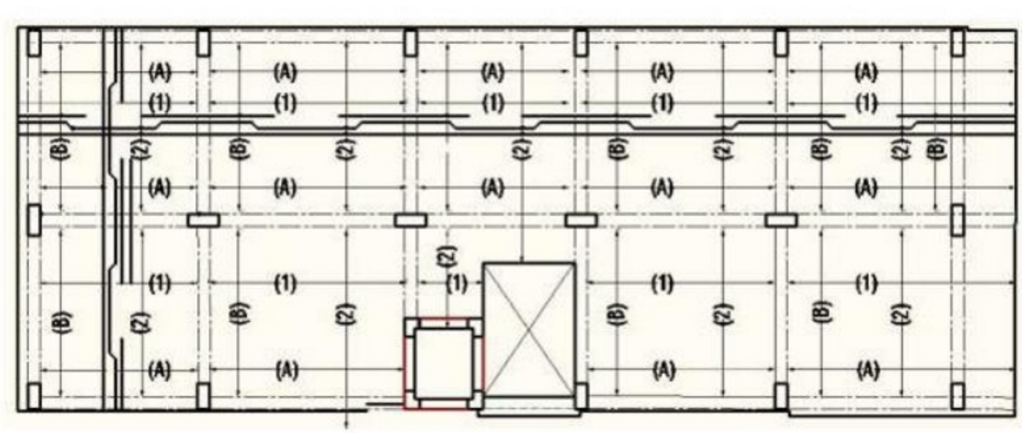
Discontinuous edge (Negative Moment): Every third bar of   the positive steel will be bent up for providing the negative moment steel   in the discontinuous edge but Maximum spacing will be less than 3t = 15 in.

Hence for the discontinuous edge every alternate bar will be bent up from the bottom steel.

4.2.5. Reinforcement bar designation for slab

(1) = 10 mm Ø @ 8" c/c Alternate Cranked (A) = 1- 10 mm ØExtra Top between Cranked Bar

(2) = 10 mm Ø @ 9" c/c Alternate Cranked (B) = 2-10 mm Ø Extra Top between Cranked Bar



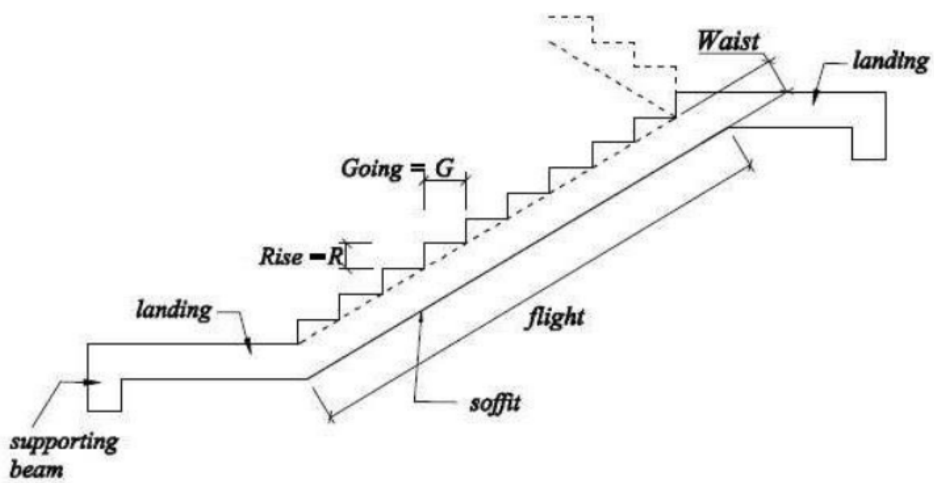
Reinforcement arrangement of slab

CHAPTER 5 : Stair Design

Stair Design

6.1. **General Introduction**:

Staircases provide means of movement from one floor to another in a structure. Staircases consist of a number of steps with landings at suitable intervals to provide comfort and safety for the users.



Main technical terms of Stair

For purpose of design, stairs are classified into two types

(a) Transversely supported (transverse to the direction of movement)

Transversely supported stairs include:

Simply supported steps supported by two walls or beams or a combination of both.

Steps cantilevering from a wall or a beam.

Stairs cantilevering from a central spine beam.

(b) Longitudinally supported (in the direction of movement):

These stairs span between supports at the top and bottom of a flight and unsupported at the sides.

As a common practice in Bangladesh we will design a “Two-flight longitudinally supported Stair”

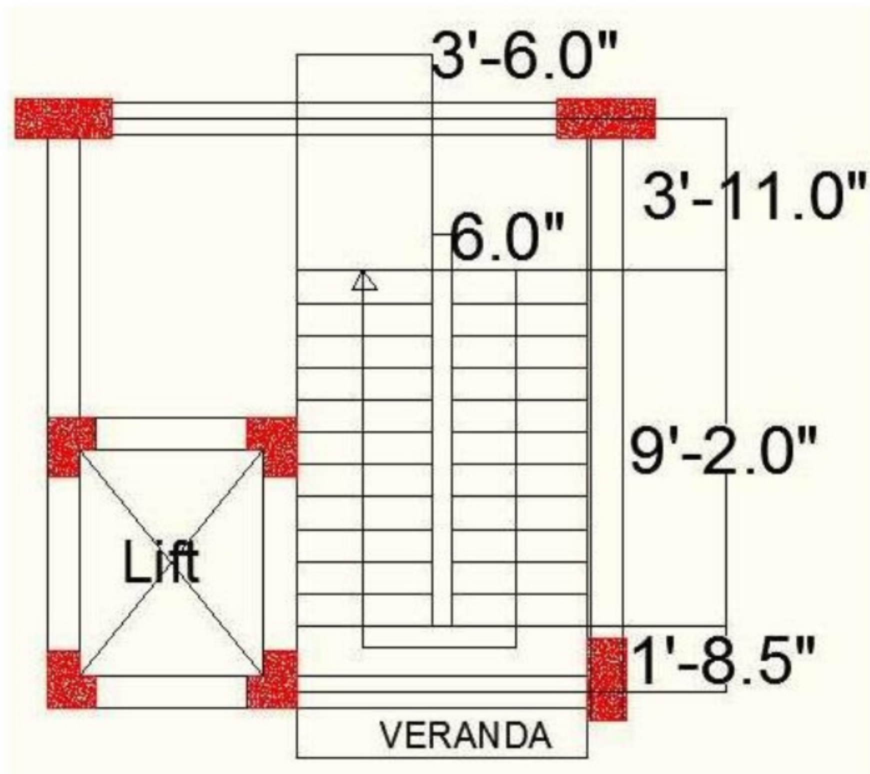
Here,

Considering, $f c' = 2.8$ ksi and $f y = 60$ ksi Riser = 6 in. and Tread or Going = 10 in.

Minimum thickness of stair = $0.85*(l \, 20 \,/\,)$

>>0.85*14.792*12/20 >>7.54 in.

Considering waist slab thickness 7.5 in.



Staircase plan view

6.2. Loading on Flight:

1) Dead Load:

a) Self weight of step: 0.5*6/12*3.5*150

>> 131.25 $lb\, ft\,/$

b) Self weight of waist slab: 7.5/12*3.5*150/cos30.95°

>> 328.125 $lb\ ft/$

c) Weight of plaster finish and floor finish : (2+1)/12*3.5*120/ $cos 30.95°$

>> 122.52 $lb\ ft/$

2) Live Load: 120 psf *3.5 ft

>> 420 $lb\ ft/$

According to BNBC-2006,

Total factored load: 1.4(Dead Load) + 1.7(Live Load) >>
1.4(131.25+382.88+122.52) + 1.7*420 >> 1605.31 $lb\ ft/$

5.3. Loading on Landing:

(1) Dead load :

(a) Self weight of landing: 7.5/12*3.67*150

>>344.0625 $lb\ ft/$

(b) Plaster finish and floor finish: (2+1)/12*3.67*120

>> 110.1 $lb\ ft/$

(2) Live Load : 120*3.67

>> 440.4 $lb\ ft/$

Total factored load: 1.4(344.0625 + 11.01) + 1.7* 440.4

>> 1384.5075 $lb\,ft/$

5.4. Checking shear force:

$Vu,\ \ max$= (1384.5075*3.922/2 + 1605.31*9.17 *8.505 + 1384.5075*1.375*13.7775)/14.792 >> 11203.94991 lbs.

Shear resisting capability of the concrete section:

$\Phi V\ c$ = Φ2λ√$f\ c'bd$ Here,

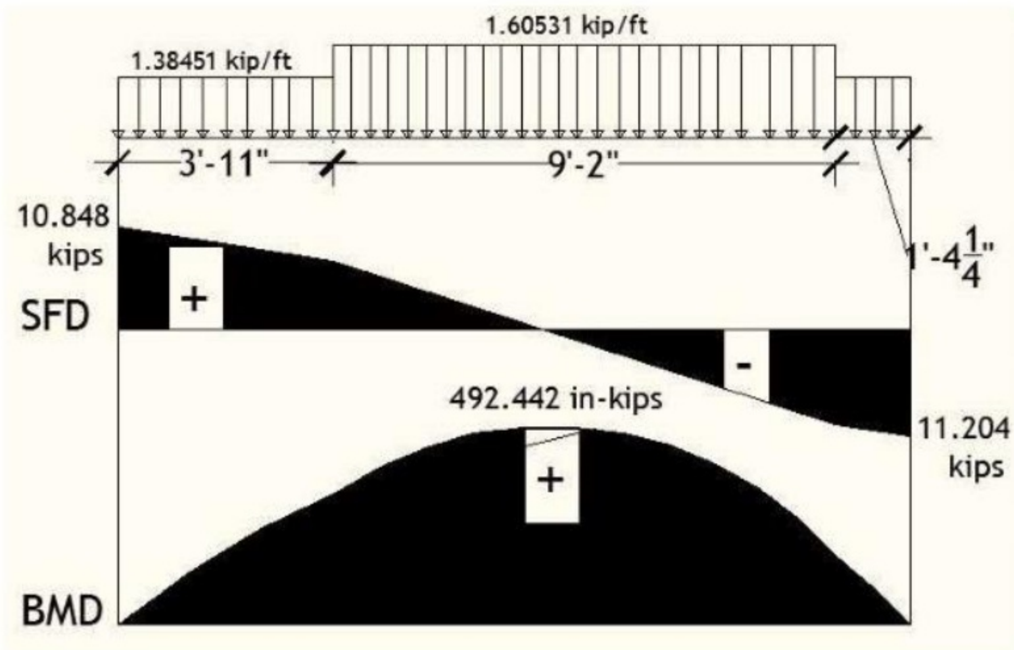
d = 7.5 – 1.5 – 0.47 – 0.31 = 5.22 and λ = 0.75

for lightweight concrete (Being in the conservative side)

>> 0.75*2*0.75*√2800*3.5*12*5.22

>> 13051.22 which is less than $Vu\ \ max$ So, slab thickness is adequate for resisting shear force without using shear reinforcement.

5.5. Bending Moment:



Bending Moment Diagram of stair

From the moment diagram we got maximum moment 492.442 $in-kips$

ρmax = 0.85 β1fc' fy

$\in u$ $\in u$+$\in y$

>> 0.85*0.85*2.8 60 $*$ 0.003 0.003+0.005

>> 0.01264

Mu = $\emptyset \rho fybd2$(1 – 0.59 ρfy fc') >> 492.446 = 0.9*ρ*3.5*12*5.222 (1 - 0.59 $\rho *$60 2.8

)

>>ρ = 0.00899 and ρ = 0.070 >ρmax

So, Flexural steel area : $As = \rho bd$

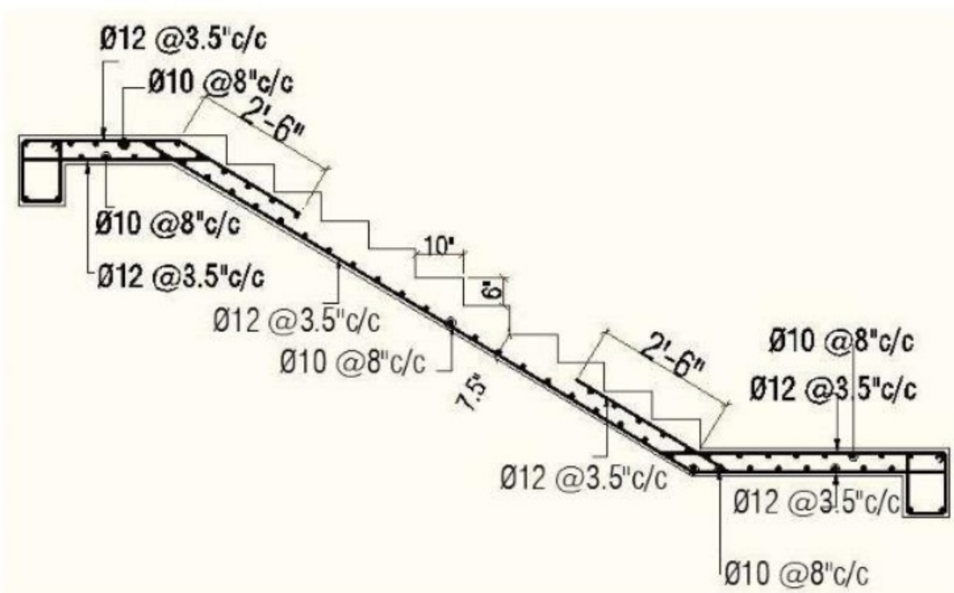
>> 0.00899*3.5*12*5.22

>> 1.97 $in2$ Using,12 Ø12 mm @ 3.5 " in the longitudinal direction.

Shrinkage and temperature reinforcement : As = 0.0018bt

>> 0.0018*14.465*12*7.5

>> 1.63 $in2$ Using,24 Ø 10 mm @ 8 " in the transverse direction.



Reinforcement detailing of stair

CHAPTER 6 : Beam Design

Beam Design

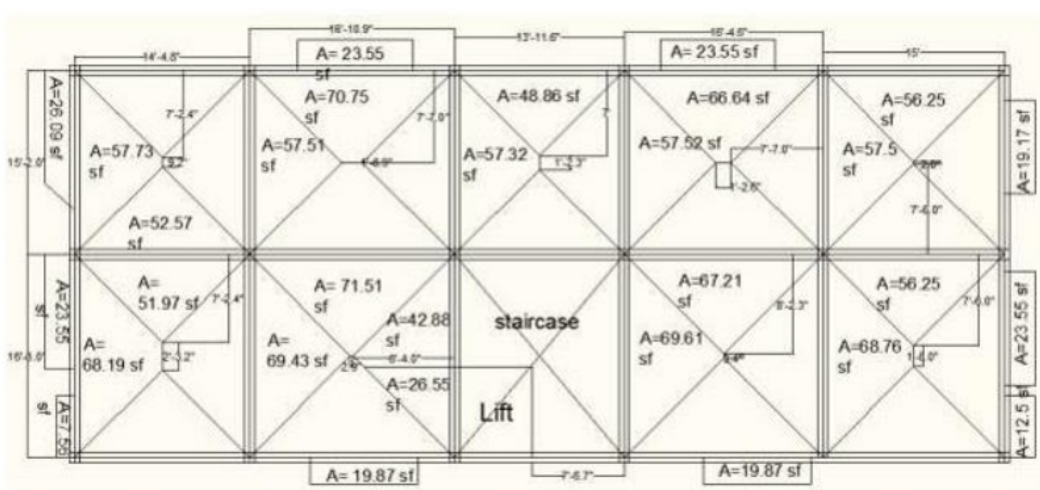
6.1. General Introduction

Beam is the horizontal structure components which transfer load from the slab to column. In terms of environmental loading like earthquake and wind force beam provides most significance reliability.

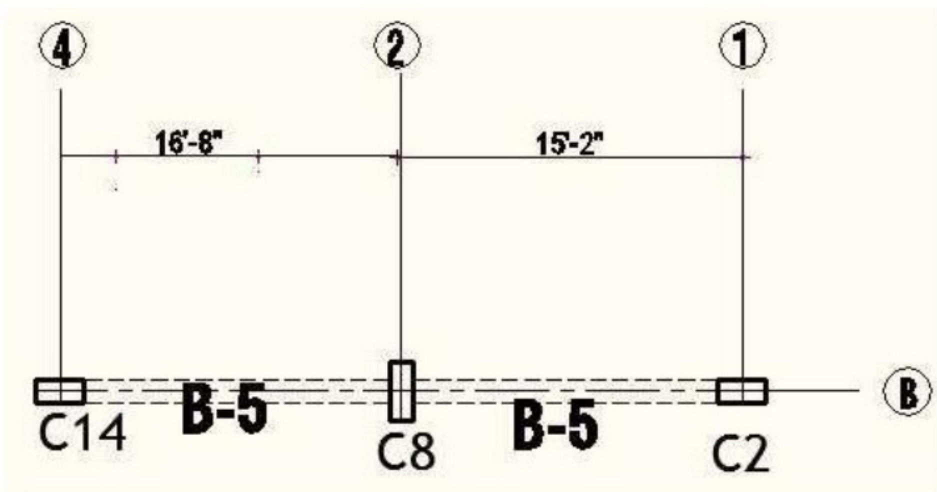
Generally in reinforced concrete structure depending on reinforcement arrangement we use 3 types of beam.

Singly reinforced beam Doubly reinforced beam.

T-beam



Tributary area distribution for beam



Beam B-5 layout view

6.2. Design of beam B-5

6.2.1. Gravity Load on beam B-5in Grid-B:

Self weight of beam = 21/12*150 lb/ft3 = 262.5 lb

In Beam segment B4-2:

DL = (137.62*127.5/15.13*1000) + 0.262 + 0.45

>> 1.815 kip/ft

LL = (137.62*40/15.13*1000)

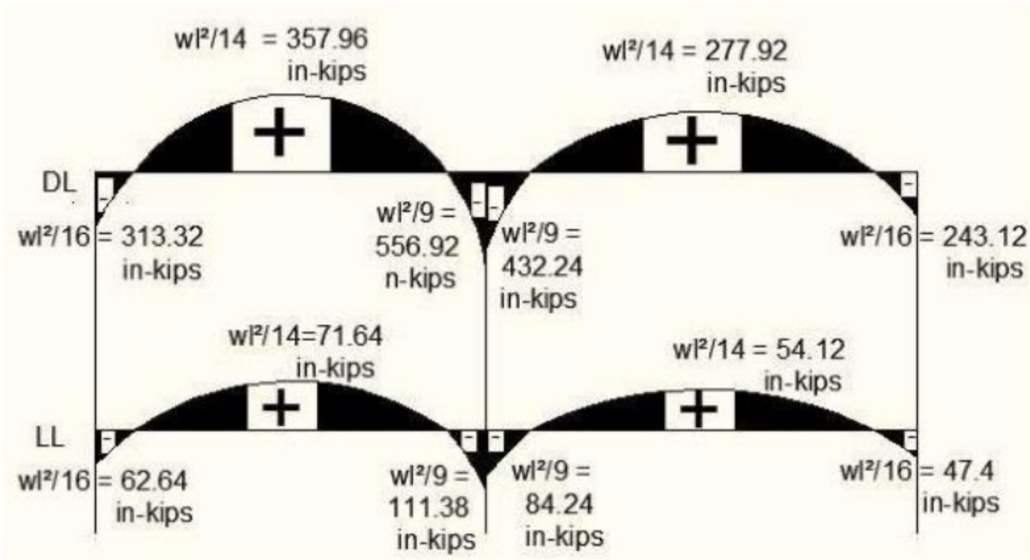
>> 0.363 kip/ft

In Beam segment B2-1: DL = (115.44*127.5/13.63*1000) + 0.262 + 0.45

>> 1.735 kip/ft

LL = 115.44*40/13.63*1000

>> 0.338 kip/ft



BMD of beam B-5 for DL and LL

6.2.2. Seismic load analysis:

According to BNBC-2006,

3

Total base shear,

V = $ZIC\ R$

W

Here,

Z = Seismic zone co-efficient = 0.15(Zone-II)

I = Structure Importance Co-efficient = 1.00 (Standard occupancy structure)

R = Response modification co-efficient = 8 (IMRF, Concrete)

W = Total seismic dead load

Calculation of total seismic dead load, W

Service dead load =(FF + DW + PW)* Floor area

>> [20 psf + 45 psf + 450 plf/(207.46 ft +144.62 ft)]*2319.5452

>> 66.28 psf * 2319.5452 $ft2$

>> 153739.46 lb

Self weight of Slab = (5/12*150)* 2319.5452

>> 144971.58 lb

Self weight of Beam = [(144.62*10*20/144) + (207.46*10*18/144)]*150

>> 69027.92 lb

Self weight of column = 4667.92 $ft3$* 150 $lb\ ft3/$

>> 700187.5 lb

Over-Head water tank weight = Weight of water + Self weight of Tank

>> 58152.11 lb + 44349.78 lb

>> 102501.89 lb

Self weight of Stair = Weight of Flight + Weight of Landing

>> 11434.24 lb + 6421.14 lb

>> 17855.38 lb

25% of Live load = 0.25*40 psf *2319.5452 $ft2$

>> 23195.452 lb

Weight of Grade Beam = (144.62+207.46)*12*24/144)*150

>>105624 lb

3

Total dead load of the Building excluding roof level:

(Service dead load *9) + (Slab weight*9) + (Beam self weight*9) + (Column self weight)

+ (Stair self weight*9) + (Grade Beam weight) + (25% Live load's weight*9)

+ (Water tank weight)

>> (153739.46*9)+ (144971.58*9) +(69027.92*9) + (700187.5) + (17855.38*9) + (105624)

+ (23195.452*9) + (102501.89)

>> 4546900.52 lbs.

>> 59.32 $kip\ ft/$ [For long direction, 76.655 ft]

>> 865.71 $kn\ m/$[1 $\mathbf{kn\ m/}$= 68.522 $\mathbf{lb\ ft/}$]

Total dead load in the Roof level:

18

>> (Slab self weight) + (Stair weight) + (Beam self weight) +

18

[(Lift case including Machine + self weight of machine room)]

+(Self weight of Boundary wall)

>>(5/12*150 + 2.5/12*120)2319.5452 + (17855.38) + (69027.92) + [(6615.215 + 6372)]

+ (40685.625)

>> 355114.075 lbs.

>> 4.63293 $kip\ ft/$ [For long direction, 76.655 ft]

>> 67.612 $kn\ m/$

C = Numerical Coefficient,

Where,

C = 1.25 $S\ T$2/3

S = Site co-efficient = 1.5 (Being in the conservative side)

T = Fundamental period of Vibration in seconds.

T = Ct*(hn)3/4

Here,

Ct = 0.073, for Reinforced concrete moment resisting frame.

hn = Height in meters above the base to level n.

>> 30.5 m

T = 0.073*(30.5)3/4

>> 0.94743

So,

C = 1.25∗1.5 (0.94743)2/3

>> 1.944

Now Seismic load analysis for for beam B-2 (Beam B4-2& B2-1)

W = 865.71*15.655/3.28 + 67.612*15.655/3.28

>> 4554.621 KN

[459.102 KN for each floor]

We get,

V = (0.15∗1∗1.944 /8)4554.621

>> 166.016 KN

Ft = 0.07TV since, T = 0.94743 sec > 0.7 sec

>> 0.07*0.94743*166.016

>> 11.01 KN< 0.25 V

(V - Ft) = (66.016 - 11.01)

>> 155.006 KN

$\sum Wx\ hx$ = 459.102(3.05+6.1+9.15+12.2+15.25+18.3+21.35+24.4+27.45) + (322.703*30.5)

>> 72854.191 KN

F1= $(V - Ft)W1h1/\sum Wxhx$

>> (155.006−11.01)459.102∗3.05/ 72854.191

>> 2.98 KN

F2 = $(V - Ft)W2h2/\sum Wxhx$

>> (155.006−11.01)459.102∗6.1 /72854.191

>> 5.96 KN

Similarly,

we get

F3 = 8.94 KN ; F4 = 11.92 KN ; F5 = 14.9 KN ; F6 = 17.88 KN

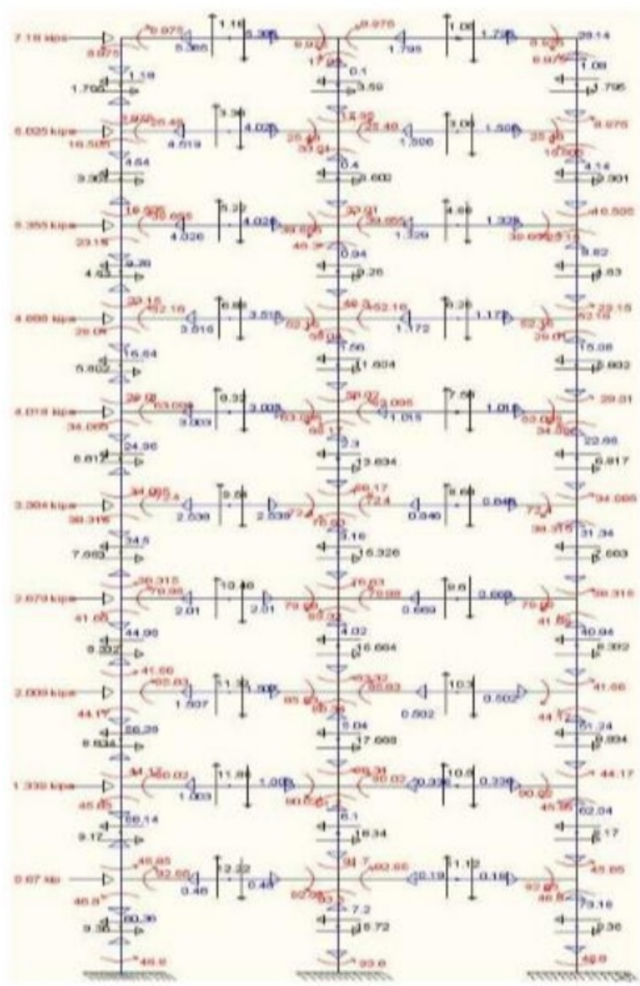
F7 = 20.86 KN ; F8 = 23.83 KN ; F9 = 26.81 KN

And,

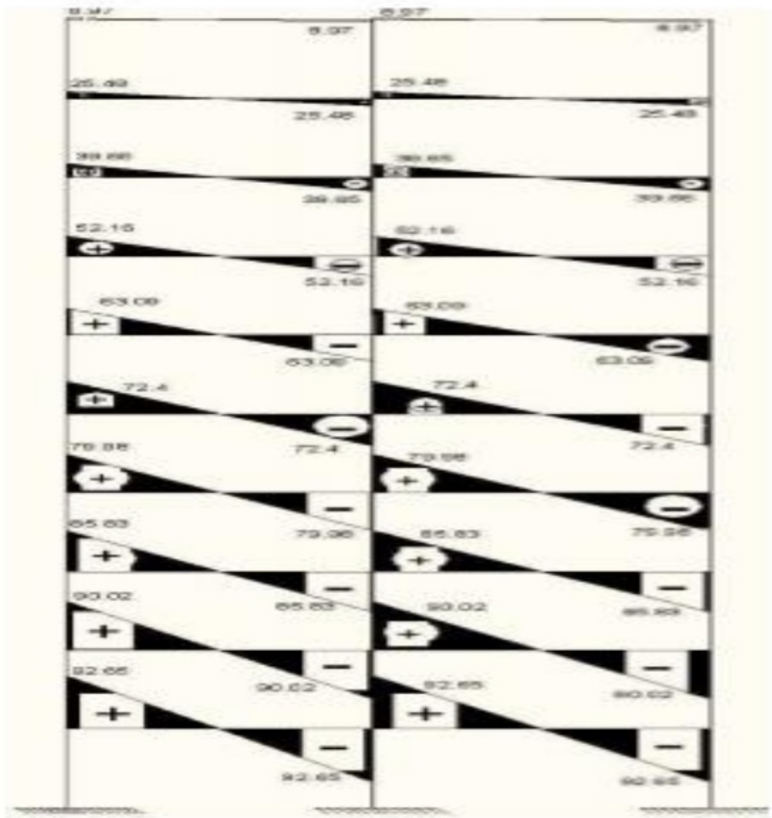
Froof = $(V - Ft) W10h10 \sum Wxhx$ + Ft

>> 20.941 + 11.01

>> 31.95 KN



Portal-frame analysis of beam B-5 for seismic load



BMD (kip-ft) of beam B-5for seismic load

6.2.3. Wind load analysis

According to IS456-2007

Sustained wind pressure at height z

$qz = Cc\ CI\ CZ\ Vb2$

Here,

Sustained Wind Pressure at height

z = qz KN/m2 Velocity to Pressure Conversion Coefficient, Cc = 47.2× 10−6 Structure importance coefficient (Table 6.2.9),

CI = 1.0

Combined height and exposure coefficient (Table-6.2.10)CZ, (Exposure-B) Basic Wind Speed,

Vb = 210 Km/hr (for INDAI)

$\therefore qz = CcCICZVb2$ >>47× 10−6 ∗ 1 ∗ 2102 ∗ CZ

≫ 2.08 CZ Design wind pressure,

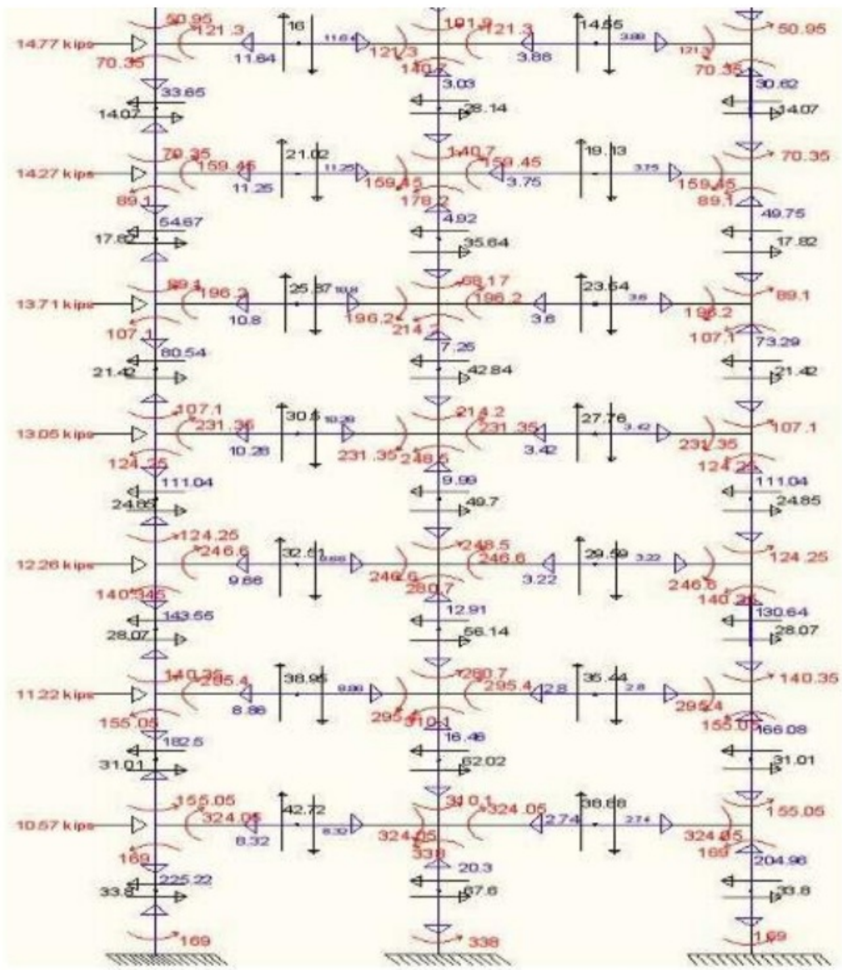
pz = CG Cp qz pz = Design wind pressure . KN/m2 CG = gust co-efficient

CG = Gh [Gust Response Factor, Gh for Non-slender (h < 5L) buildings and structures]

Cp = Overall Pressure co-efficient

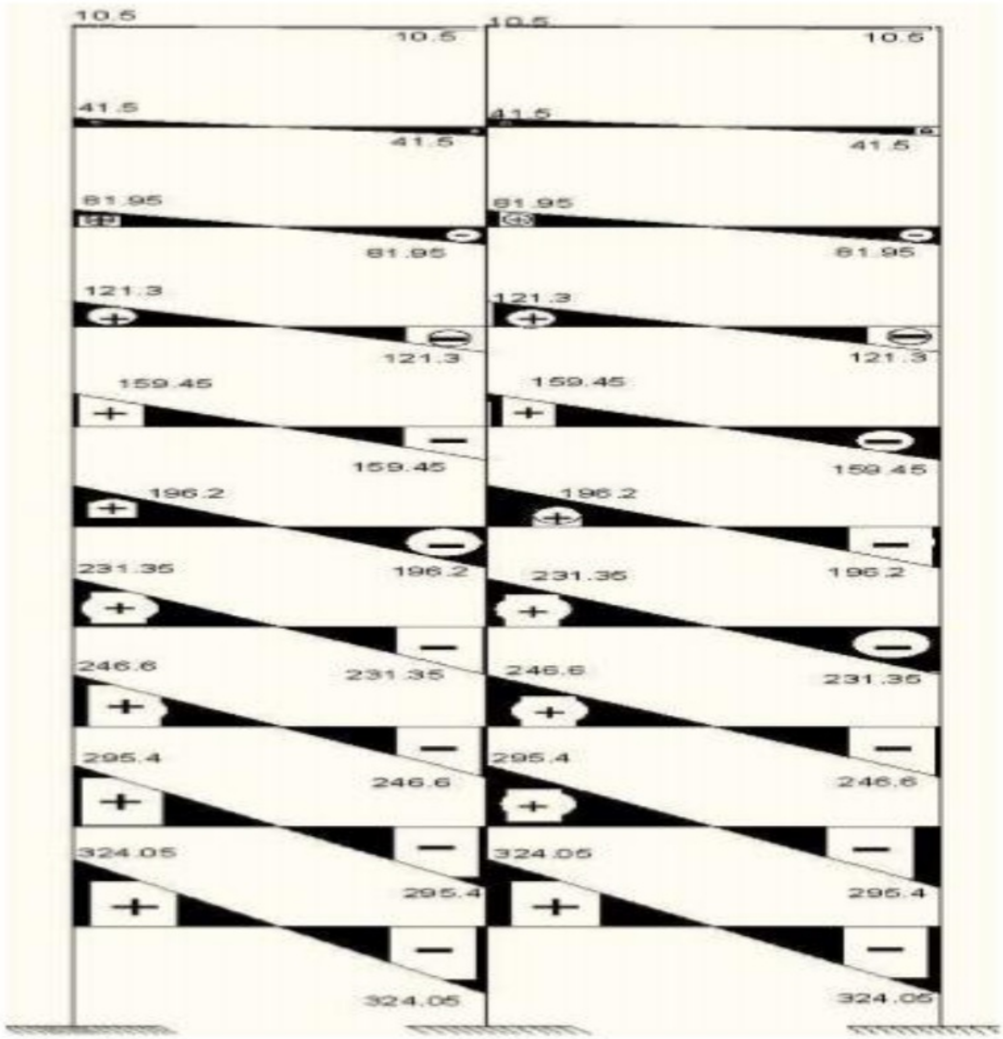
Wind load calculation

Floor Level	Height Z (m)	C_Z Table 6.2.10	$q_z = 2.08C_Z$ $\frac{kN}{m^2}$	$C_G = G_h$ Table 6.2.11	C_p Table 6.2.15	$P_Z = C_G\ C_p q_z$ $\frac{kN}{m^2}$	A_Z (m^2)	$F = \frac{P_Z * A_Z}{4.45}$ Kips
1	3.05	0.801	1.666	1.321		3.233	14.56	10.57
2	6.1	0.869	1.807	1.293		3.43	14.56	11.22
3	9.15	0.976	2.03	1.257		3.748	14.56	12.26
4	12.2	1.06	2.204	1.232		3.989	14.56	13.05
5	15.25	1.13	2.35	1.214		4.191	14.56	13.71
6	18.3	1.19	2.475	1.2	1.469	4.363	14.56	14.27
7	21.35	1.244	2.588	1.188		4.516	14.56	14.77
8	24.4	1.292	2.687	1.177		4.646	14.56	15.20
9	27.45	1.336	2.779	1.169		4.772	14.56	15.61
Roof	30.5	1.377	2.864	1.161		4.884	7.28	8



Portal frame analysis of beam B-5 for wind load

555



BMD (kip-ft) of beamB-5 for wind load

6.2.4. Flexural design of beam B-5

For the flexural design purpose there are numbers of load combinations for Reinforced Concrete Structures are recommended by our code BNBC.

Among those following load combinations will be used here

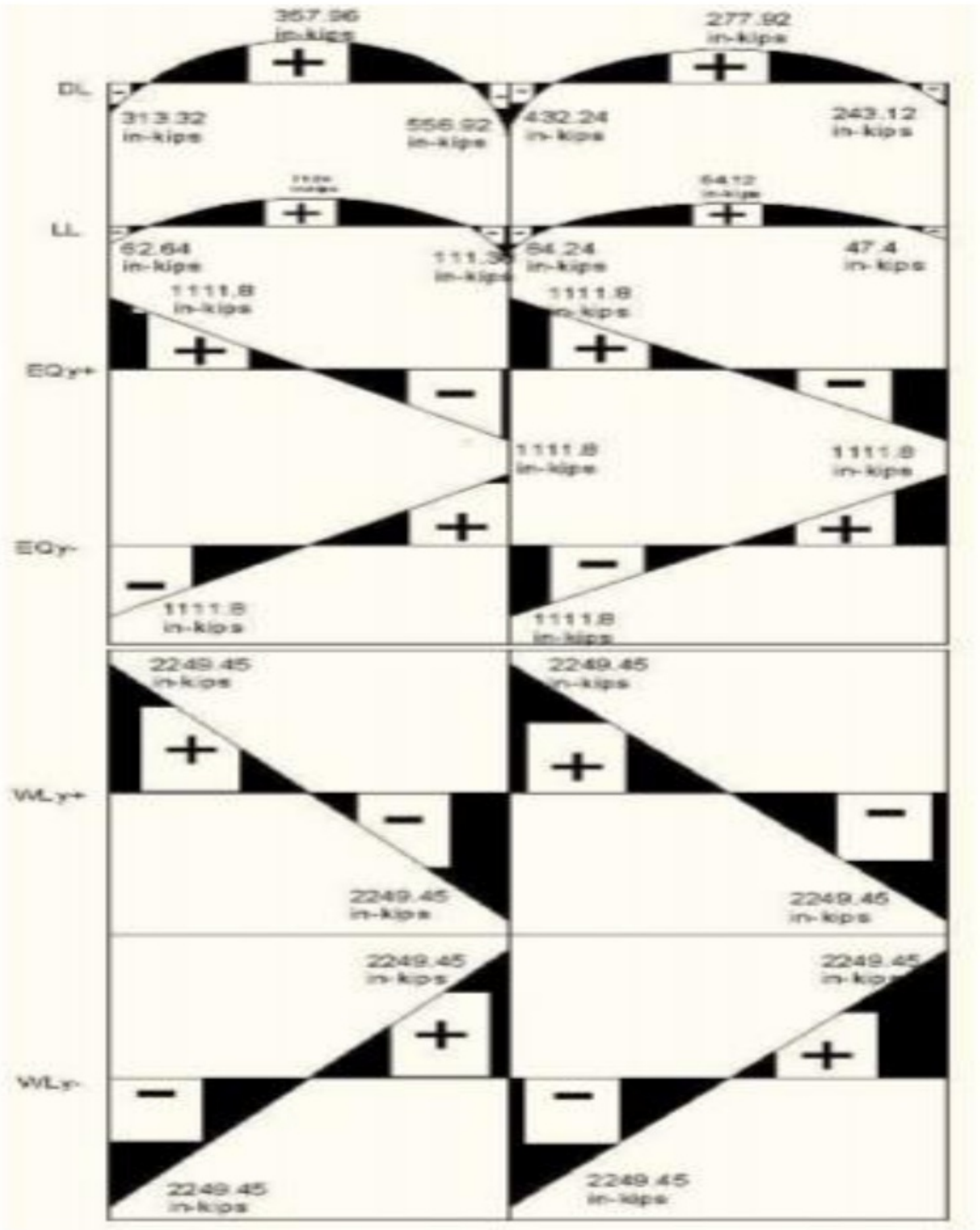
1.4 DL + 1.7 LL

0.75[1.4 DL + 1.7 LL + 1.7 (1.1 Ex+)] [Same as (1.05 DL + 1.275 LL + 1.4025 Ex+)]

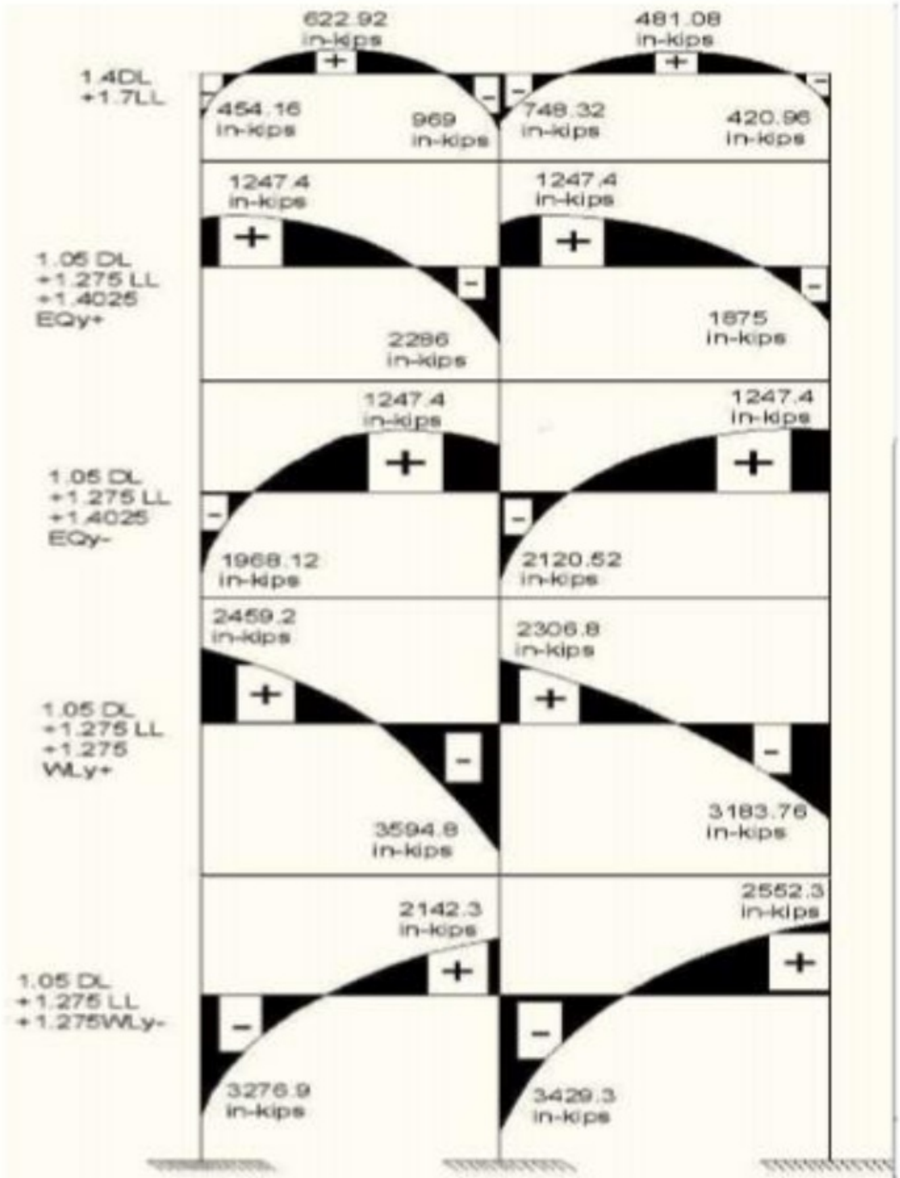
0.75[1.4 DL + 1.7 LL + 1.7 (1.1 Ex -)] [Same as (1.05 DL + 1.275 LL + 1.4025 Ex-)]

0.75[1.4 DL + 1.7 LL + 1.7 Wx+] [Same as (1.05 DL + 1.275 LL + 1.275Wx+)]

0.75[1.4 DL + 1.7 LL + 1.7 Wx-] [Same as (1.05 DL + 1.275 LL + 1.275Wx-)]



BMD of beam B-5 (1st Floor) for different service load



BMD of beam B-5 (1st Floor) for different load combinations

Maximum reinforcement ratio,

ρmax = 0.85β1 fc'/ fy(ϵu/ ϵu+ϵt)

>>0.85*0.852.8/ 60 (0.003/ 0.003+0.005)

>> 0.01264

Minimum reinforcement ratio,

ρmin = 200/ fy

>> 0.00333

Now,

Mu = $\emptyset \rho fybd2$(1 – 0.59 ρfy /fc ')

>>3594.8 = 0.9*0.01264*60*bd2 (1 - 0.59 0.01264∗60/ 2.8)

>>$bd2$ = 3594.8 /0.573483011

in3

>> d = 22.86 in. [Taking b= 12 in.]

Now, considering total depth, t = 26 in.

So,

d = 26 – 1.5 – 10/25.4 – (20/25.4*2)

>>23.71 in.

Considering, d = 23.71 in. we get,

Mu = 0.9*ρ*60*12*23.712(1 - 0.59 ρ∗60 /2.8)

Mu = 364282.34 ρ - 4605569.54 ρ2

Solving quadratic equation,

ρ = 0.01156 ; 0.0675>ρmax

[When,Mu = 3594.8 in-kips]

Steel area for different position of beam B-5

Moment M_u (in-kips)	Reinforcement ratio, ρ	Steel Area, A_s (in^2) = ρ*b*d >> ρ*12*23.71
−3296.6	0.0104	2.96
+2459.2	0.00745	2.12
− 3429.3	0.01092	3.11
−3594.8	0.01156	3.29
+2552.3	0.00777	2.21
−3268.73	0.0103	2.93

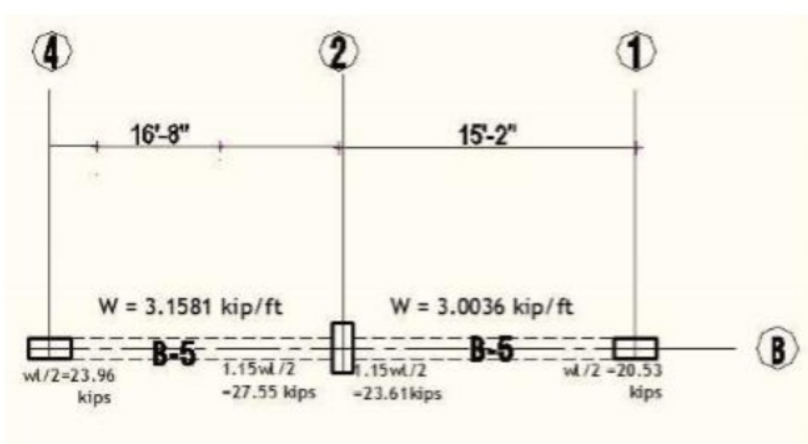
Shear design for beam B-5

Total factored load in beam segment B4-2= 1.4*1.815 kip/ft + 1.7*0.363 kip/ft
>> 3.1581 kip/ft

Total factored load in beam segment B2-1 = 1.4*1.735 kip/ft + 1.7*0.338 kip/ft

>> 3.0036 kip/ft

From figure-4.10, we get maximum shear of Vu = 27.55 kips.



Shear resisting capability of concrete

$\boldsymbol{V\,c} = \emptyset 2\lambda\sqrt{f\,c\,'}bd$ >>$0.75* 2* 1\sqrt{2800}* 12* 23.71$

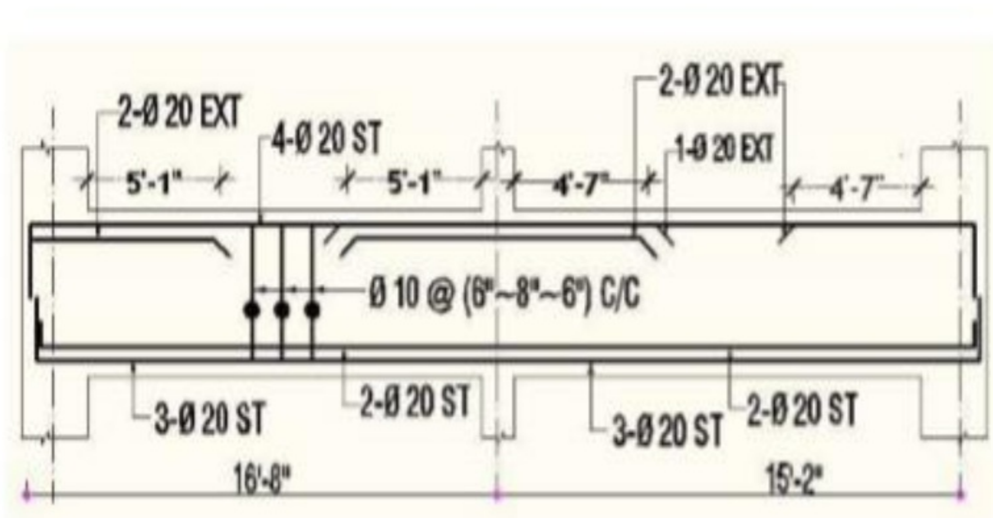
>> 14.12 kips

With Ø10 mm stirrups the required spacing of web reinforcement is for vertical stirrups:

$s = \emptyset Av f y / Vu - V\,cd$

>>$0.75*0.22*60*17.79 / 27.55-14.12$

>> 13.11 inch. Using 8 inch spacing at the middle zone and 6 inch for the support ends



Reinforcement detailing of beam B-5

CHAPTER 7 :

Column Design

7.1. General Introduction

the structure which carries the whole structures load from slab and beam and transfer to the soil through foundation.

17

Three types of reinforced concrete compression member are in use:

1. Members reinforced with longitudinal bars and lateral ties (Tied column)

2. Members reinforced with longitudinal bars and continuous spiral (Spirally reinforced column)

1

3. structural steel shapes, pipe or tubing, with or without additional longitudinal bars and various types of lateral reinforcement.

Types 1 and 2 are by far the most common practice.

7.2. Design of column C8

7.2.1. Loads on column C8

4

Floor Finish (FF) : 20 psf

Distributed wall Load (DW) : 45 psf

Partition wall Load on Beam (PW) : 450 plf

Slab Thickness = 5 inch.

Slab Weight = (5/12*150) psf

= 62.5 psf

Total Dead Load = FF + DW + Slab Weight

= 20 + 45 + 62.5

= 127.5 psf

Live Load : 40 psf

According to ISCODE-

Factored Load = (1.4* Dead Load) + (1.6* Live Load)

= 1.4*127.5 + 1.7*40

=246.5 psf

For the simplicity purpose of the design we are considering that the 5 inch. Partition Wall Load (450 lb) will be suggested al through the beam span.

Load on beam B-5 (Lower segment):

(246.5 psf*137.62 sf/15.13 ft *1000) + (0.262 kip/ft + 0.45 kip/ft)*1.4

>>3.1581 kip/ft

Load on beam B-5 (Upper segment):

(246.5 psf*115.04 sf/13.63 ft *1000) + (0.262 kip/ft + 0.45kip/ft)*1.4

>>3.0036 kip/ft

Load on beam B-2(Left):

(246.5 psf*104.54 sf/12.65 ft *1000) + (0.156kip/ft + 0.45 kip/ft)*1.4

>> 2.9kip/ft

Load on beam B-2 (Right):

(246.5 psf*142.26 sf/14.41 ft *1000) + (0.156 kip/ft + 0.45 kip/ft)*1.4

>> 3.34 kip/ft

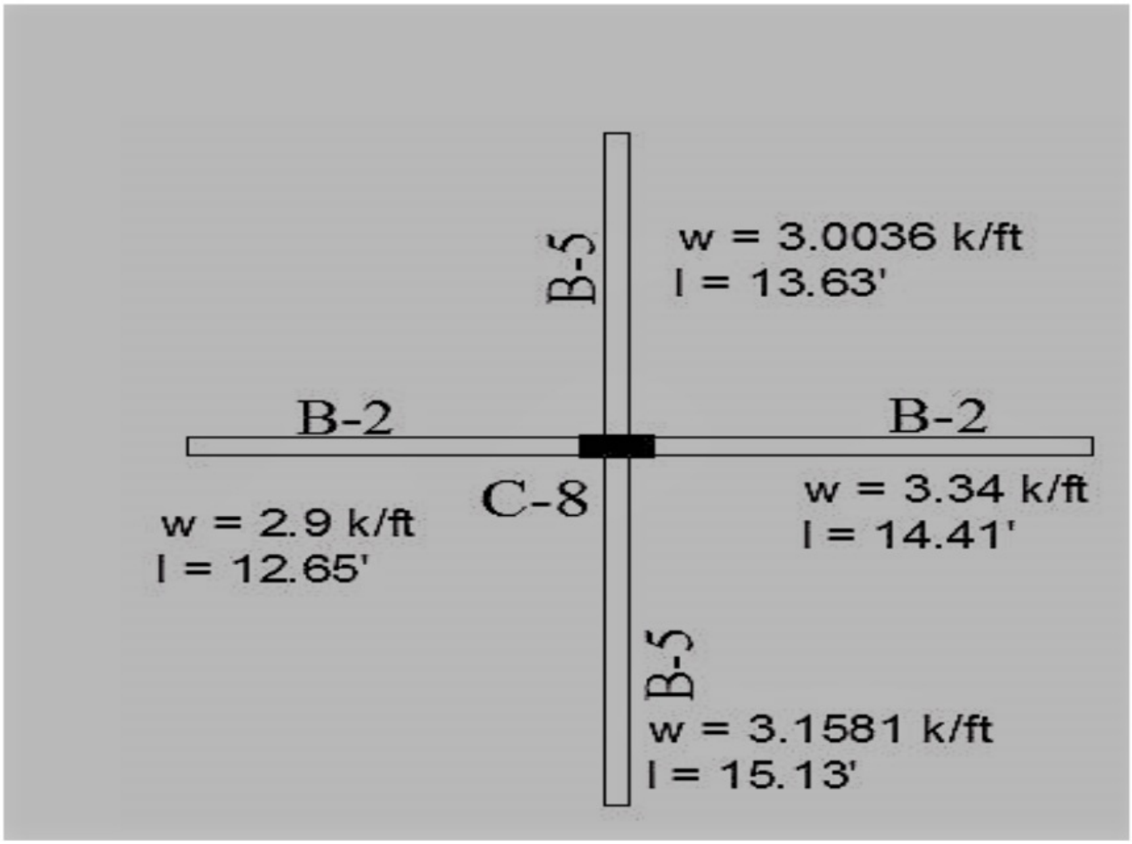


Figure 7.1 : Column C8 with corresponding beams

Load on the Column C8:

(3.0036*13.63’/2) + (3.1581*15.13/2) + (2.9*12.65/2) + (3.34*14.41/2)

>> 88.9 kips (For each floor)

Self weight of the column :

1.25’*2.5’*108’*150

>>50.625 kip

>> (50.625*1.4)

>>70.87 kips

For the Ground Floor column Total load on the Column:

88.9*10 + 70.87

>> 959.87 kips

Table 8-1 :Loads on Column

Column Name	Dimension (inch x inch)	Total load including self weight (kips)	Group Type
C1	15x25	342.5	
C6	15x25	409.59	
C13	15x25	368.43	
C15	15x15	334.6	Group-1
C16	15x15	298	
C19	15x25	378.97	
C20	15x15	130	
C21	15x15	142	
C2	15x25	557.09	
C3	15x25	549.12	
C4	15x25	543.05	Group-2
C5	15x25	558.54	
C17	15x25	445.59	
C7	15x25	608.7	
C12	15x25	695	Group-3
C14	15x25	587.89	
C18	15x25	590.36	
C8	15x30	959.87	
C9	15x30	844.03	Group-4
C10	15x30	921.74	
C11	15x30	971.92	

7.2.2. Design of column Group-4 (C8, C9, C10, C11)

According to ACI Code,

For tied columns, Ø = 0.65

Total factored load Pu = 1037.43 kips (ETABS)

Let, Column size 15”x30” (Ag = 450 in2)

Pu = Φ0.8[0.85fc’(Ag - Ast) + fyAst]

Considering, ρ = 0.03 ; (3% of Ag)

So, Ast= 0.03Ag

>>1037.43 = 0.65*0.8[0.85*3.5(Ag – 0.03Ag) + 60*0.03Ag]

>>1037.43 = 1.50059 Ag + 0.936 Ag

>> Ag= 425.77in2< 450 in2 (ok)

So, Total steel area comes, Ast = 0.03*15*30 >> 13.5 in2

7.2.3. Checking the column strength with interaction diagramfor lateral load

Considering moment about Y-axis

My = 4201.09 in-kips (ETABS-M3: moment about Cyan axis)

γ = (30 – 1.5 – 1.5 – 45/25.4)/ 30 = 0.841

$Kn = Pu/\ Øfc'\ Ag$

= 1037.43/ 0.65∗3.5∗450

= 1.013

$Rn = \ Mu/\ Øfc'Ag\ h$

= 4201.09/ 0.65∗3.5∗450∗30

= 0.1367

Using the column strength interaction diagram for rectangular section with bars on four faces and considering γ = 0.80 (smaller value will require more steel area

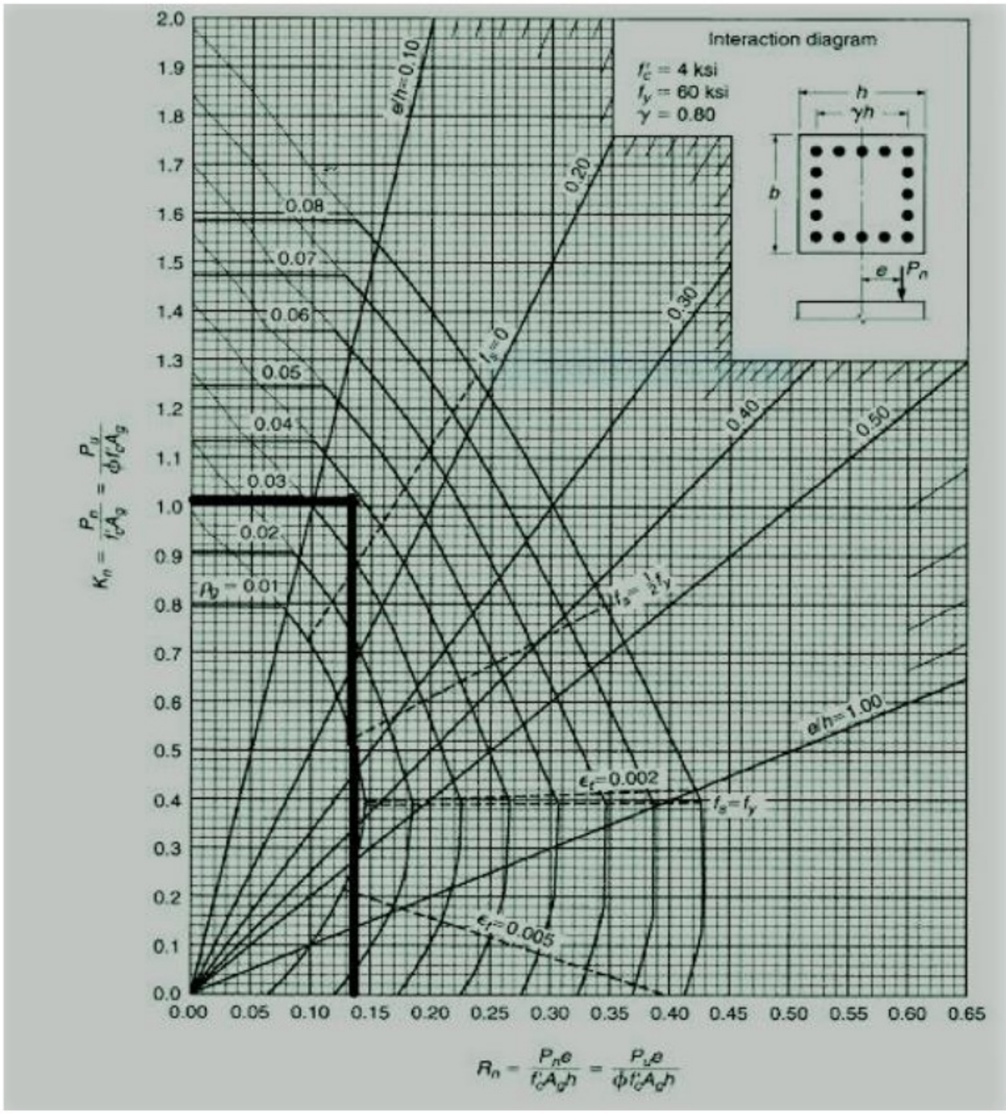


Figure 7.2 :Steel ratio checking for moment about Y-axis (γ = 0.80)

From interaction diagram, ρ = 0.038 (3.8%)

>ρ (actual) = 0.03 (3%)

Considering moment about X-axis

Mx = 2358.28 in-kips (ETABS- M2: moment about white axis)

γ = (15 – 1.5 – 1.5 – 45/25.4)/ 30 = 0.682

$Kn = Pu/\emptyset fc' Ag$

= 1037.43/ 0.65∗3.5∗450

= 1.013

$Rn = Mu/\emptyset fc' Ag\, h$

= 2358.28/ 0.65∗3.5∗450∗30

= 0.153

Using the graph of column strength interaction diagram for rectangular section with bars on four faces and considering γ = 0.70

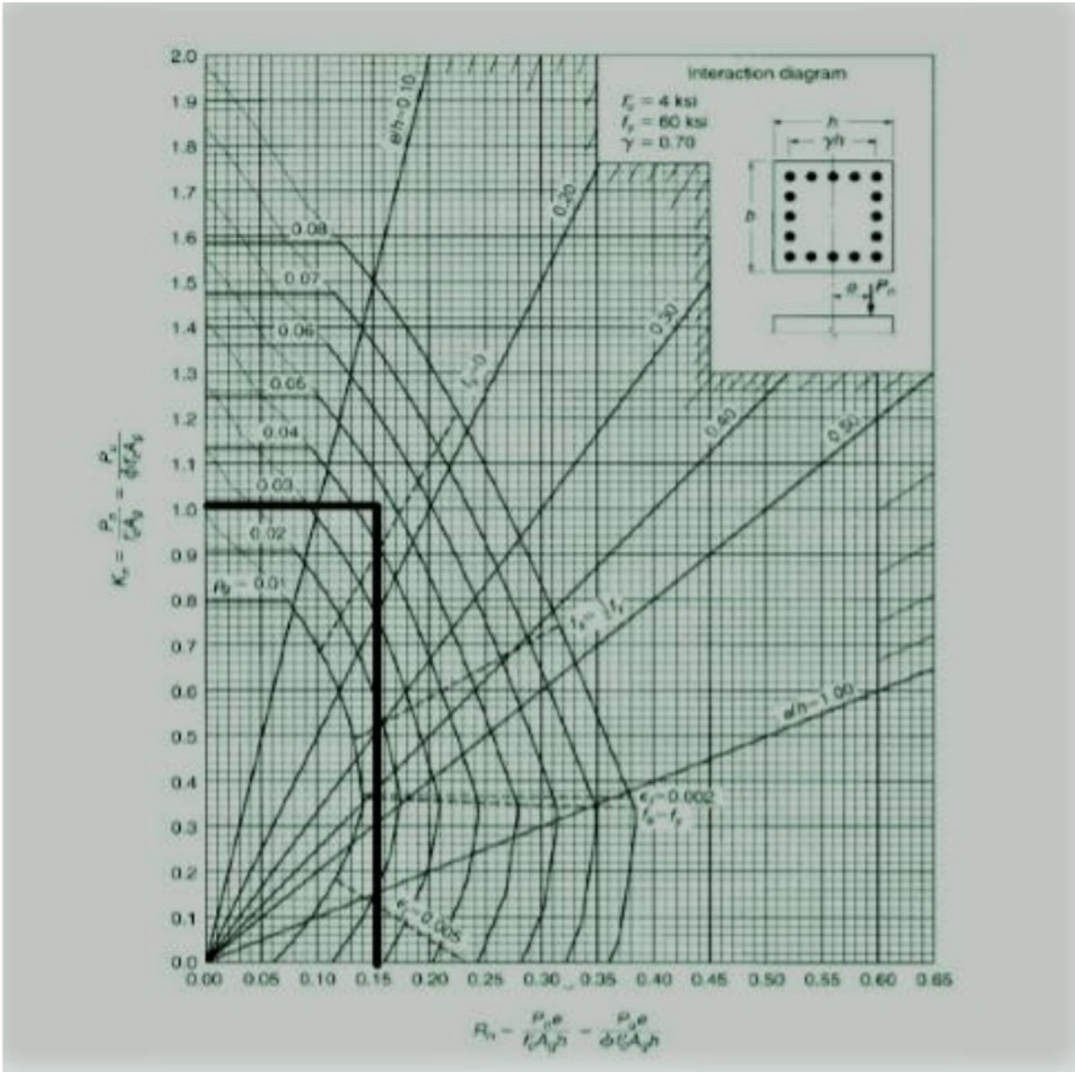


Figure 8.3 :Steel ratio checking for moment about X-axis (γ = 0.70)

From interaction diagram, ρ = 0.044 (4.4%) >ρ (actual) = 0.03 (3%)

From the column strength interaction diagram we got for both of the cases, assumed steel ratio is lesser than the required steel ratio. So, further modification is required with our preliminary assumed steel ratio.

We will use ρ = 0.044 (4.4%) for calculating the total steel area.

Ast = 0.044*15*30

>> 19.8 in2

26 Ø $\mathbf{25\ mm}$ bar will provide the required steel area.

Calculation of maximum spacing of Tie :

According to IS456 -2007, Section: 8.3.10.5 Maximum Tie spacing will be smaller of these four criteria

Taking,Ø10 mm bar for Tie,

1. $s0$ = 8 times the diameter of the smallest longitudinal bar enclosed

= 8× 0.76 = 6.08′′

2. $s0$ = 24 times of tie bar

= 24× 0.393 = 9.43′′

3. $s0$ = One-half of the smallest cross sectional dimension of the frame member

=15/ 2

= 7.5′′

4. s0 = 300mm = 11.811″

Using, Ø10 mm @ 6′′ c/c

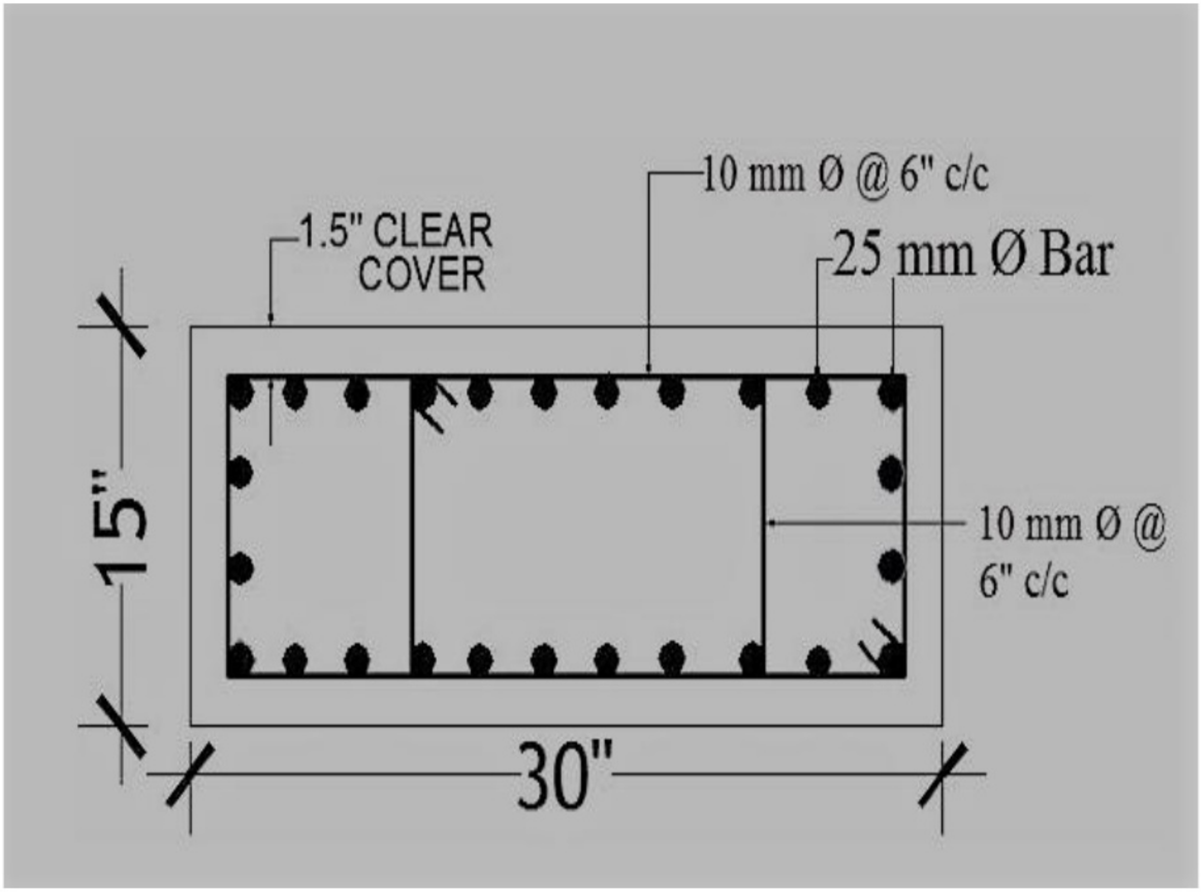


Figure 8.4 : Reinforcement detailing of column Group-4

CHAPTER 8 :

Analysis by ETABS and Comparison with Manual Results

8.1. Introduction

ETABS- Extended Three dimensional Analysis of Building Systems is an integrated building design software. A sophisticated, special purpose analysis and design program developed specifically for building systems. ETABS Version 9.6 features an intuitive and powerful graphical interface coupled with unmatched modeling analytical and design procedures, all integrated using a common database. Although quick and easy for simple structures, ETABS can also handle the largest and most complex building models, including a wide range of nonlinear behaviours, making it the tool of choice for structural engineers in the building industry.

8.2. ETABS Inputs

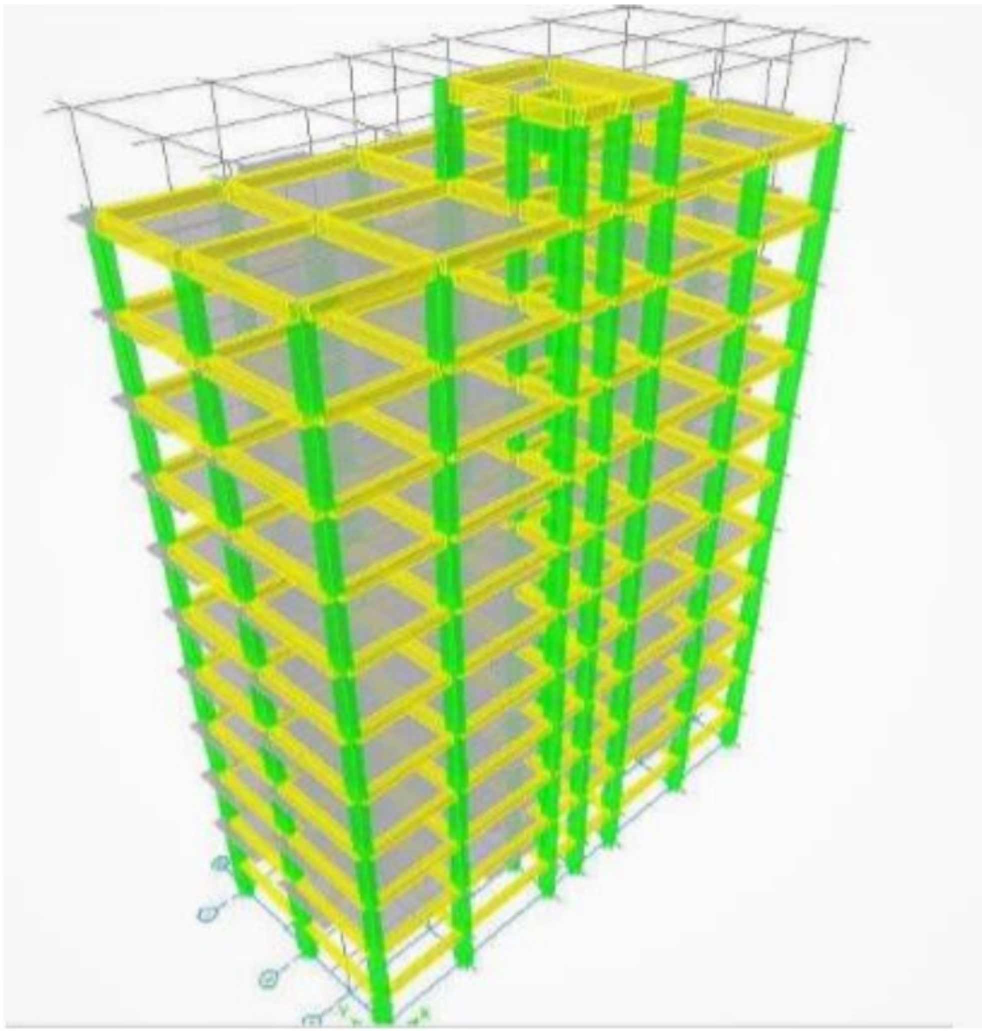
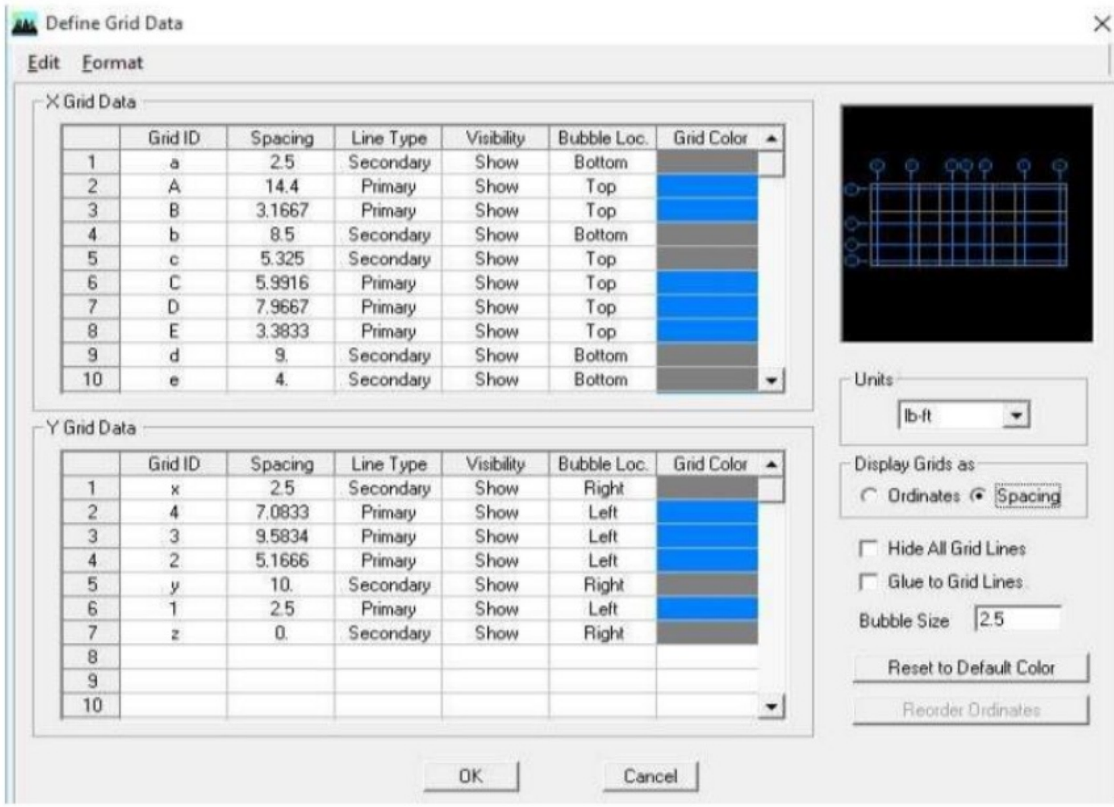


Figure 0.1 : ETABS input frame



X Grid Data

	Grid ID	Spacing	Line Type	Visibility	Bubble Loc.	Grid Color
1	a	2.5	Secondary	Show	Bottom	
2	A	14.4	Primary	Show	Top	
3	B	3.1667	Primary	Show	Top	
4	b	8.5	Secondary	Show	Bottom	
5	c	5.325	Secondary	Show	Top	
6	C	5.9916	Primary	Show	Top	
7	D	7.9667	Primary	Show	Top	
8	E	3.3833	Primary	Show	Top	
9	d	9.	Secondary	Show	Bottom	
10	e	4.	Secondary	Show	Bottom	

Y Grid Data

	Grid ID	Spacing	Line Type	Visibility	Bubble Loc.	Grid Color
1	x	2.5	Secondary	Show	Right	
2	4	7.0833	Primary	Show	Left	
3	3	9.5834	Primary	Show	Left	
4	2	5.1666	Primary	Show	Left	
5	y	10.	Secondary	Show	Right	
6	1	2.5	Primary	Show	Left	
7	z	0.	Secondary	Show	Right	
8						
9						
10						

Figure 0.2 : Insertion of Grid Data

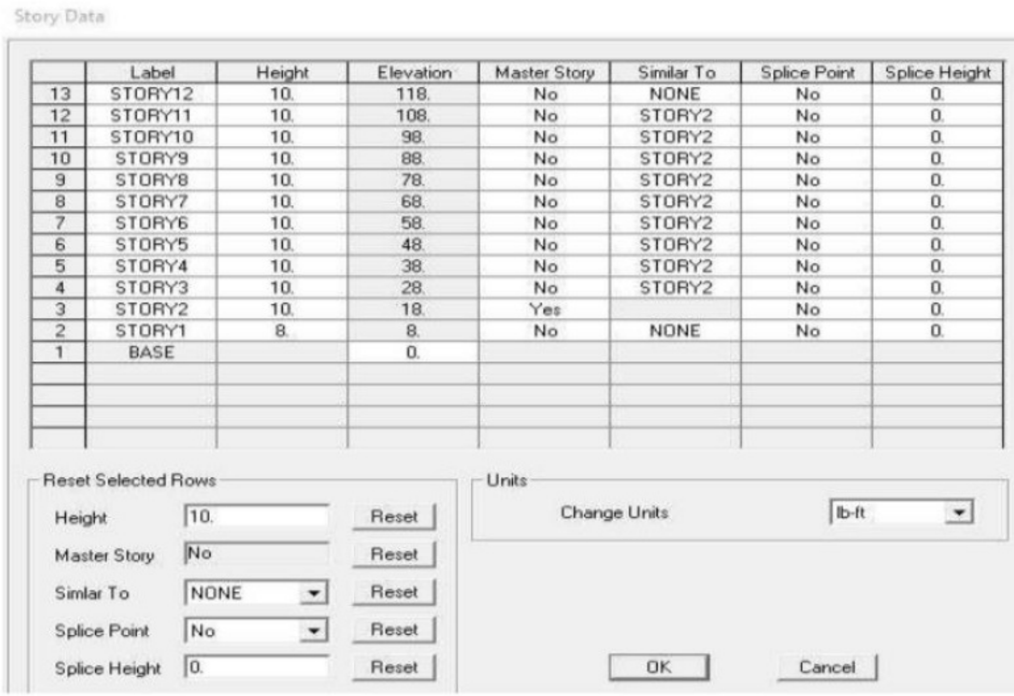
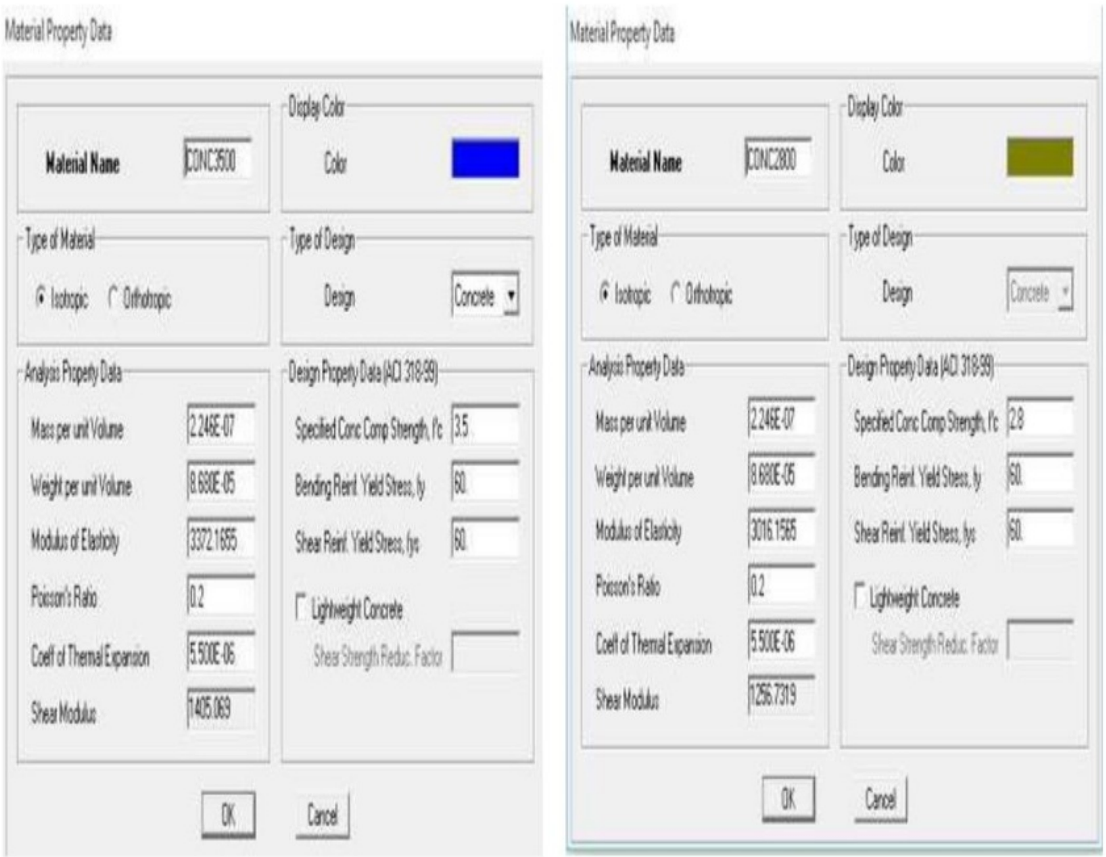


Figure 0.3 : Defining Story Data



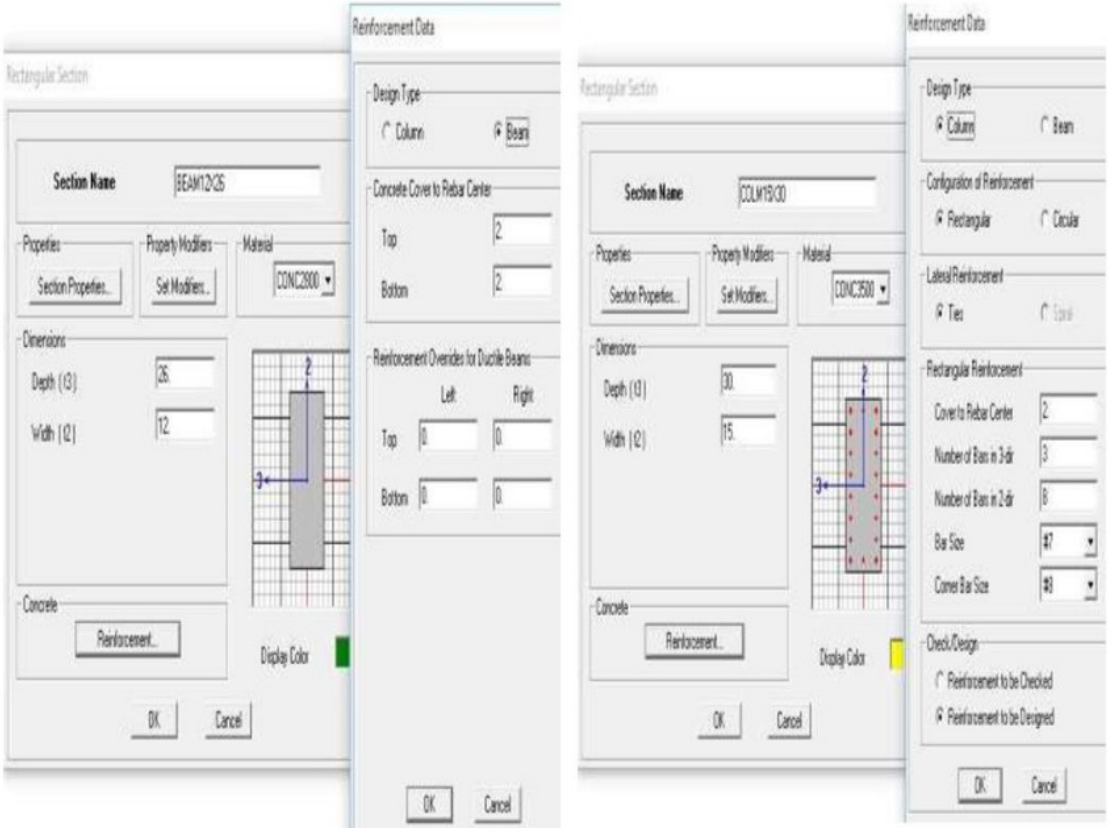
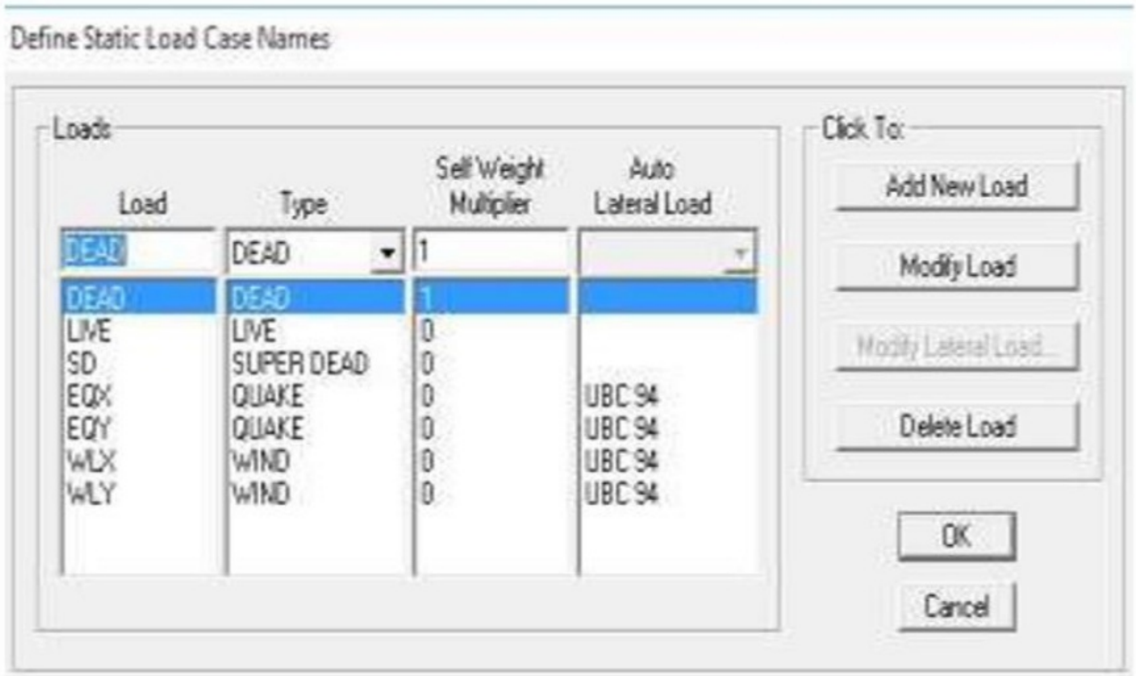

Reinforcement Data
Section Name
Properties
Property Modifiers
Material
Dimensions
Concrete
Display Color
OK
Cancel
Design Type
Column
Beam
Concrete Cover to Rebar Center
Top
Bottom
Reinforcement Overrides for Ductile Beams
Left
Right
Configuration of Reinforcement
Rectangular
Circular
Lateral Reinforcement
Ties
Rectangular Reinforcement
Cover to Rebar Center
Bar Size
Corner Bar Size
Check/Design
Reinforcement to be Checked
Reinforcement to be Designed

Figure 0.4 : ETABS input windows for Material properties and Frame section



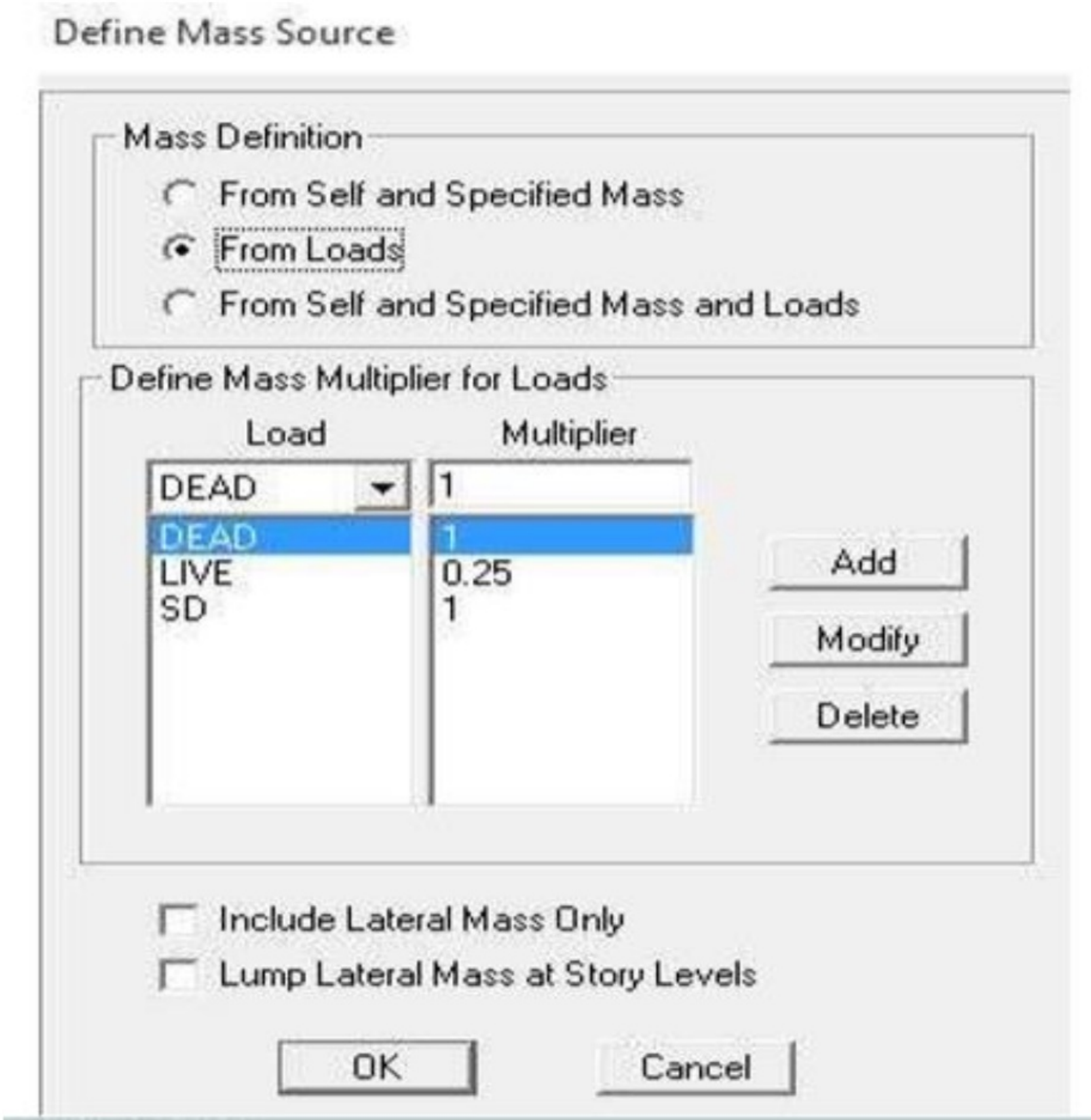
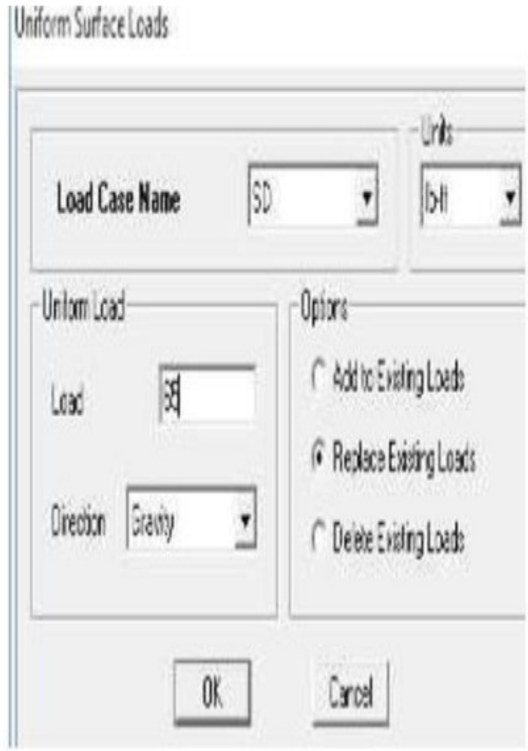

Define Mass Source
Mass Definition
From Self and Specified Mass
From Loads
From Self and Specified Mass and Loads
Define Mass Multiplier for Loads
Load
Multiplier
DEAD
1
DEAD
LIVE
SD
1
0.25
1
Add
Modify
Delete
Include Lateral Mass Only
Lump Lateral Mass at Story Levels
OK
Cancel

Figure 0.5 : Load assigning windows for

Static load cases

Mass Source

Wind loading data for X and Y direction



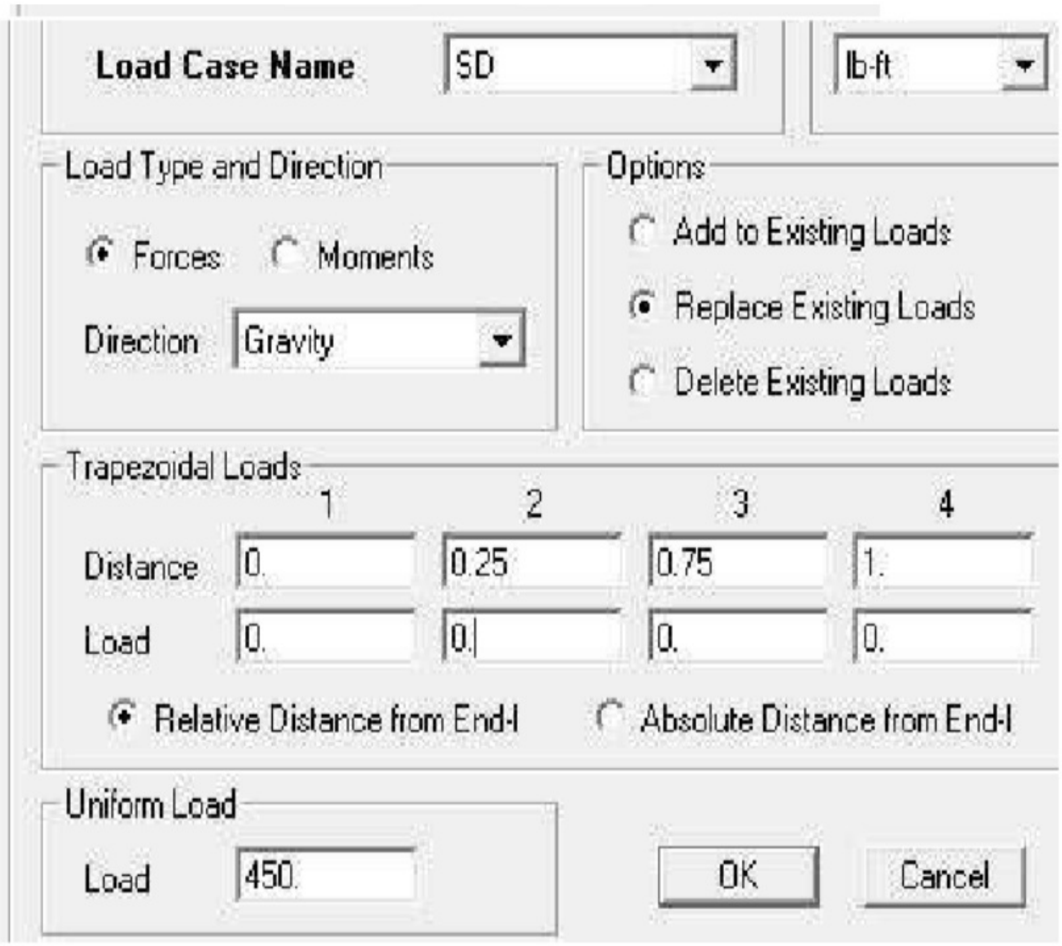

Load Case Name
SD
lb-ft
Load Type and Direction
Forces
Moments
Direction
Gravity
Options
Add to Existing Loads
Replace Existing Loads
Delete Existing Loads
Trapezoidal Loads
1
2
3
4
Distance
0.
0.25
0.75
1.
Load
0.
0.
0.
0.
Relative Distance from End-I
Absolute Distance from End-I
Uniform Load
Load
450.
OK
Cancel

Figure 0.6 : Wind load assigning data and Inputs for load assigning on beam and floor

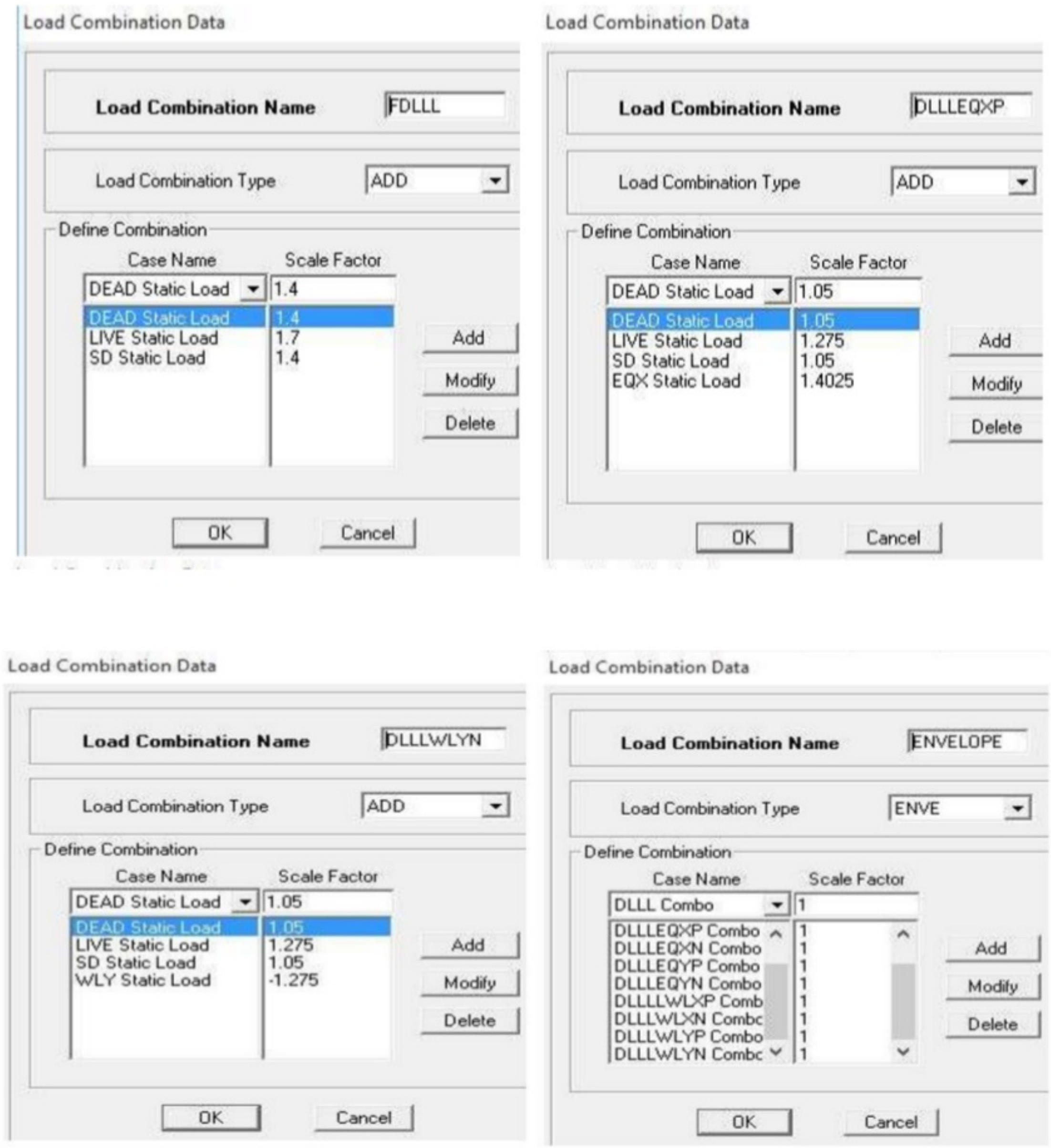


Figure 0.7 : Construction of different Load Combinations

8.3. Outputs from ETABS

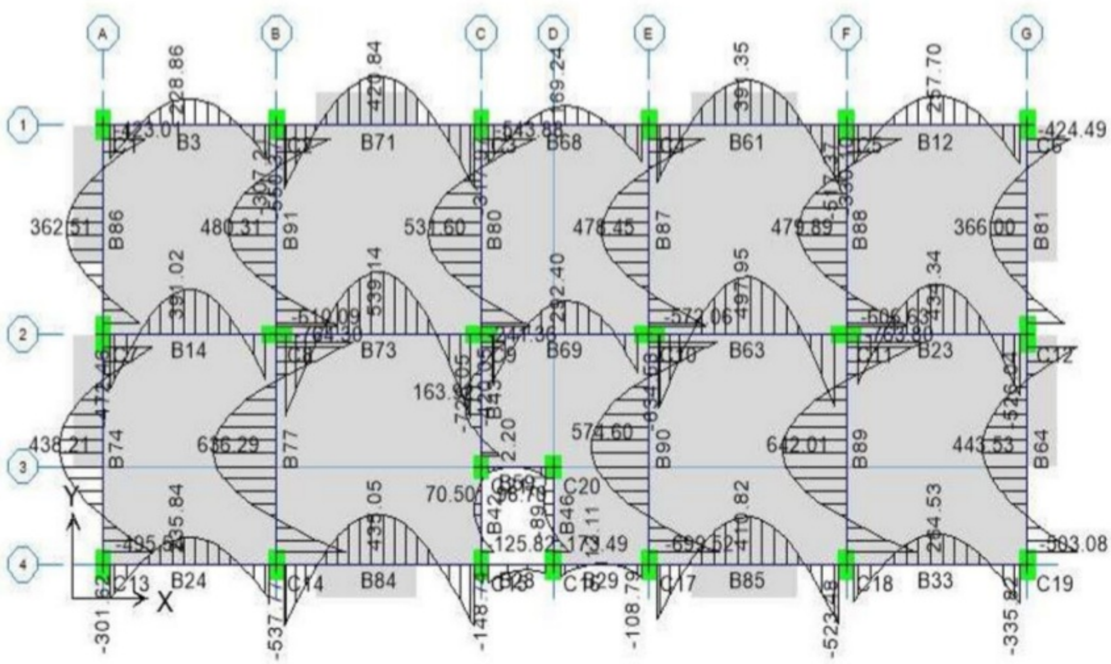


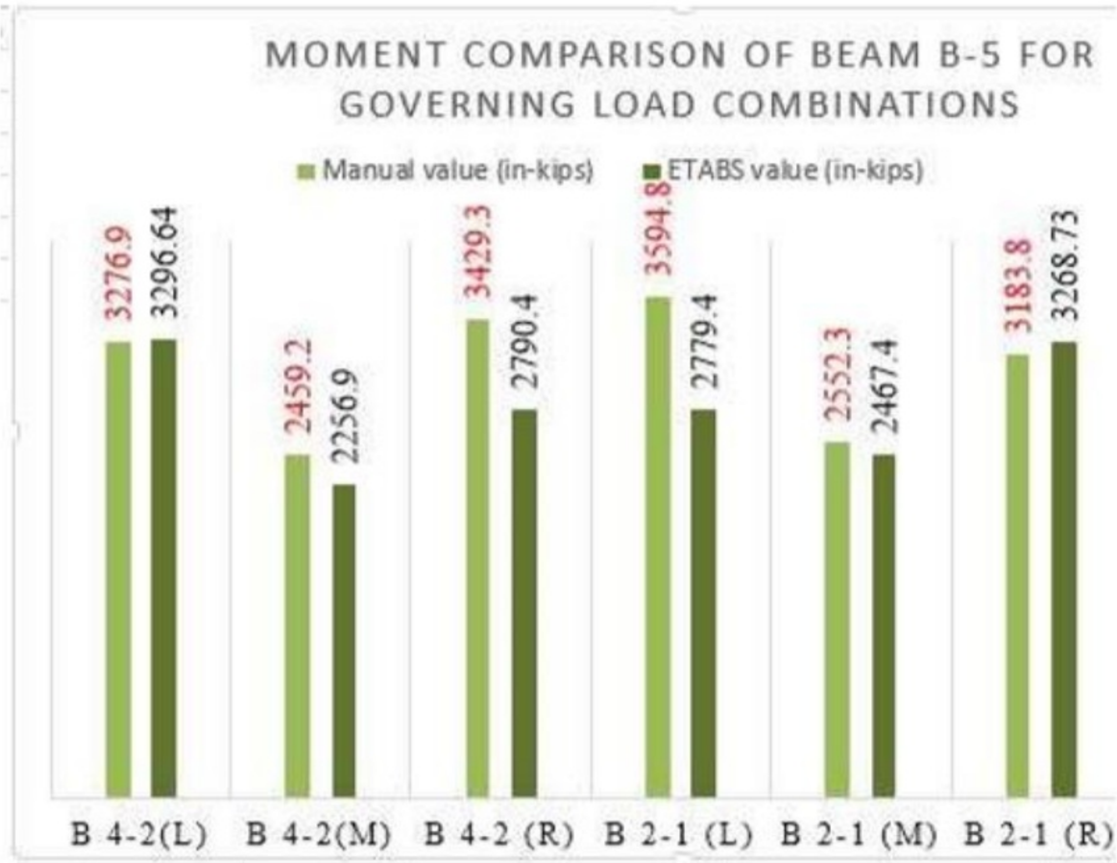
Figure 0.8 : BMD(in-kips) of beams for factored DL & LL in story 2

8.4. Comparison between manual and ETABS result

8.4.1. Moment Comparison of beam B-5:

Moment position	Manual value (in-kips)	ETABS value (in-kips)
B 4-2(L)	3276.9	3296.64
B 4-2(M)	2459.2	2256.9
B 4-2 (R)	3429.3	2790.4
B 2-1 (L)	3594.8	2779.4
B 2-1 (M)	2552.3	2467.4
B 2-1 (R)	3183.8	3268.73

(a) Table



(b) Graph

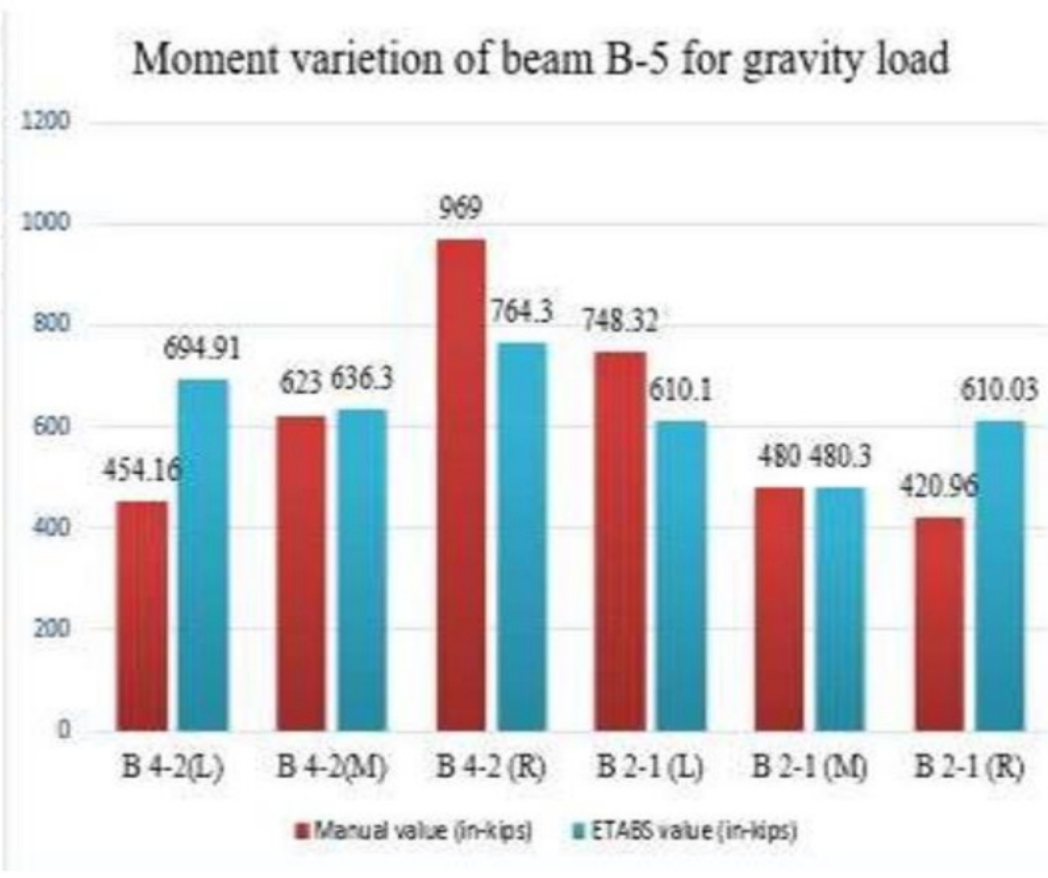
Figure 0.9 : Moment comparison of beam B-5 for governing load combinations between Manual and ETABS output

a) Comparison table

(b) Graphical representation

Moment position	Manual value (in-kips)	ETABS value (in-kips)
B 4-2(L)	454.16	694.91
B 4-2(M)	622.92	636.29
B 4-2 (R)	969	764.3
B 2-1 (L)	748.32	610.09
B 2-1 (M)	481.08	480.31
B 2-1 (R)	420.96	610.03

(a) Table



(b) Graph

Figure 0.10 : Moment comparison of beam B-5 between Manual and ETABS output

For gravity load.

(a) Table

(b) Graph

8.4.2. Comparison of factored gravity load of column

Column ID	Manual value (kips)	ETABS value (kips)
C8	959.87	995.74
C9	844.03	945.46
C10	921.74	1037.43
C11	971.92	1008.27
C12	695	736.63

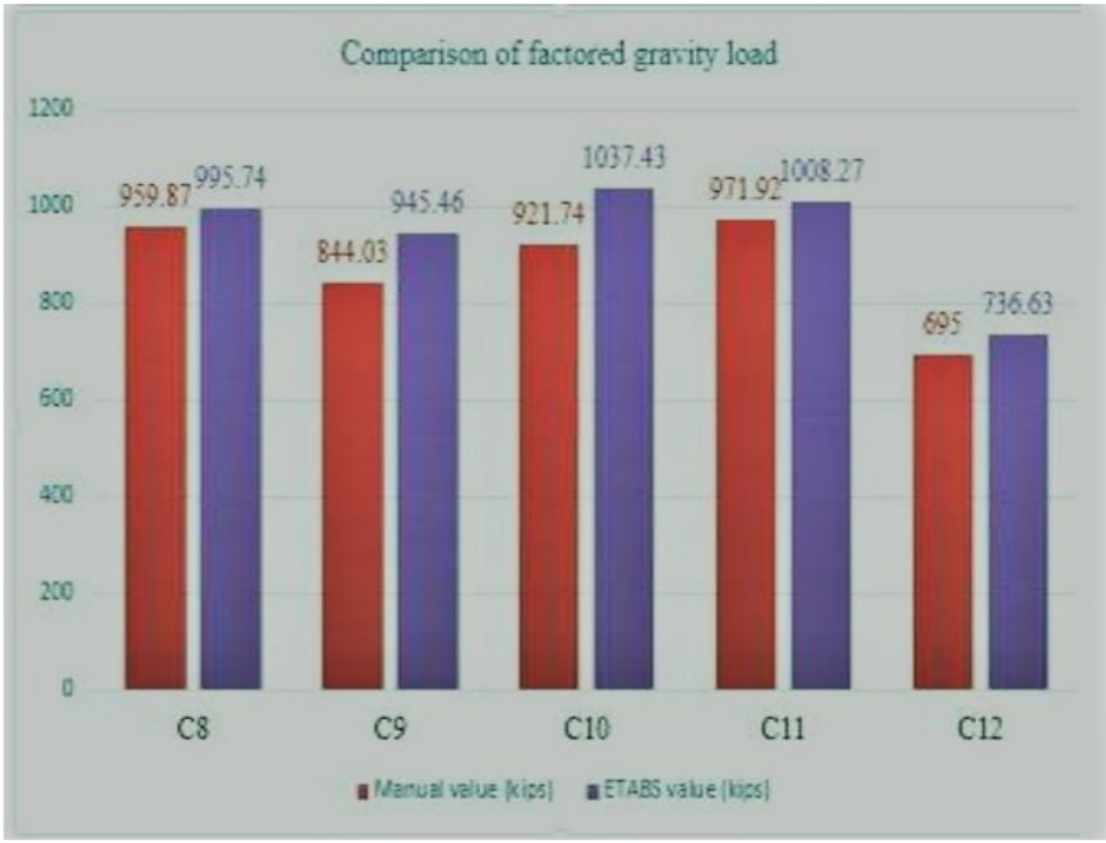


Figure 0.11 : Graphical comparison of factored gravity load between Manual & ETABS result

(a) Table

(b) Graph

8.5. Discussion on the results

we experienced the fluctuation between the manual results and ETABS outputs. But the satisfactory things is that the variation governed either by the manual result or by the ETABS output each of them are close enough to other. One exceptional case happened regarding wind load effect. My manual result is far away beyond the ETABS result. In manual calculation I am following Projected Area Method (Method-2; IS456-2000Section: 2.4.6.4). Consulting with the technical experts I have used the ETABS result in the preceding calculation. This can be a findings as limitation of my study. It seems to me that the main reason behind the variation is actually the method the procedure of the analysis is being used by ETABS is not same with what I am using. In terms of beam design, for the case of gravity loading I am following co-efficient method and I am not sure actually which method is being used by ETABS program. For the calculation of lateral load effect I am following portal frame analysis in order to convert the lateral force into vertical effect while in the ETABS program's is performing Finite Element Method (FEM). The lateral load i.e. for seismic and wind loading the effect on the structure have significantly varied. This could be because of individual code has followed for each cases. For manual calculation I have follow the data and procedure from ISCODE and in ETABS I have selected a code called IS875 PART3. In case of factored gravity load calculation on column for manually I have used tributary area method considering that all loads from floor area will comes on beams and from the beams will pass the loads on column. And how ETABS calculate the total load on column, I don't have clear idea about that procedure. However, actually the comparison of the result is shown above is basically a simple justification on manually driven work with a integrated building design software. Now-a-days professional designer depends a lot on the software result as they are vastly experienced on design field. As a fresh designer I have tried to gather some basic knowledge about how practically the things are working. I am in a very much initial level to discuss about actual reason due to which the manual results and ETABS output have differed from each other. Those parameters seems to me that yes this can be the reason have stated above.

CHAPTER 9 : Conclusions And Recommendations

Recommendation And Conclusion

10.1. Recommendations

More emphasize should be given on the communication phase between the architecture and the structural designer in the period of final design. Some points appeared important to me that due to architectural demand, there is a tendency of reducing the dimension of the crucial components of the structure. All other consideration should be to maintain keeping the safety issue as the most important parameter for design work.

10.2. Conclusion

We have covered many of the basic features of the ETABS program in this tutorial, and yet there are many more we have not had time to explore. You have been exposed to enough of the program to become quickly productive. The additional features will make certain tasks easier, allow you to model more complicated buildings, and provide many additional capabilities for analysis and design. A fresh designer more often gets in trouble to make a balance of his theoretical design knowledge with practical workmanship. But this is something which a design engineer must know before delivering a structural design hand out. Throughout this practicum period I got the opportunity to work under a well reputed company with the help of the professional structural designers. There I got the Scope to study a structure and make comparisons of its design. Here all the findings and discussions are done in consultation with professional people. So fresh learners who are interested in structural design they may follow this report to get some clear basic idea about the manual process of designing structural components of a bu

Reference: -

1. P. Kalpana R.D. Prasad and B. Kranthi Kumar (2016) “Analysis of Building with and without Shear Wall at Various Heights and Variation of Zone III and Zone V,” IJERA, Volume 6, Issue 12, Part-2, pp.05-11.

2. Ravi Kumar, K. Sunder Kumar and BadipatiAnup (2017), “Analysis and Design of Shear Wall for an Earthquake Resistant Building using ETABS”, IJIRST, Volume 4 Issue 5, PP 5-11.

3. Jay Kumar Sash, Perm Shankar Singh and Chimney Kumar Kudu (2018), “Analysis and design of the seismic resistant multistory building with shear walls and raft foundation,” International Journal of Advance Research, Ideas and Innovations in Technology, Volume 4, Issue 2, PP 2348-2356.

4. Purnima D., Sanjay S.J. and Yajnodbhavi H.M. (2017), “Seismic analysis of RC residential multi-story building for different locations of shear wall”, International Journal of Science Technology & Engineering,” Volume 3, Issue 11, PP 54-62

5.Piyush Tiwari, P.J. Saluki, “Earthquake Resistant Design of Open Ground Story Building”, International Research Journal of Engineering and Technology, ISSN: 2395 0056, Volume: 02 Issue: 07-Oct-2015.

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