CHAPTER THREE
MANUAL ANALYSIS AND DESIGN

3.1Introduction:

The first function in design is the planning carried out by the architect to determine the arrangement and layout of the building to meet the client`s requirements. Architect and engineer should work together at this conceptual design stage. The design of different structures is achieved by performing, in general, two main steps: (1) determining the different forces acting on the structure after estimation of loads using proper methods of structural analysis, (2) proportioning all structural members economically, considering the safety, stability, serviceability, and functionality of the structure [12].

3.2 Description of Case Studied:

A forty story concrete framed tube building,

-Basic wind speed: 100mph (45m/s)

- Terrain: flat

-Plan Dimension: 20*15m^2

-Building Height: 128.8m

-Story height: 3.2m and 4m for ground floor

-Building lateral system: perimeter tube with exterior columns typically spaced at 5m, with spandrel beams and core at centre of the building.

-Building Use: health care facilities.

-Typical Floor Live Load: 2.4kN/m^2

-RoofLiveLoad:0.96 kN/m^2

-Super Imposed Dead Load:

Floor: 1.5kN/m^2(ceiling load and partition)

Roof: 1.075kN/m^2(0.48kN/m^2+200kN/m^2 for penthouse)

- Member Section:

Beam section: 0.9*0.3m^2

Column section:0.8*0.8m^2

Wall thickness: 300mm

Slab depth: 200mm

- Material Properties:

W_c = 24kN/m^3

W_m = 22KN/m^3

f_y= 420N/mm^2

f'_c = 35N/mm^2

f_{ys}= 250N/mm^2

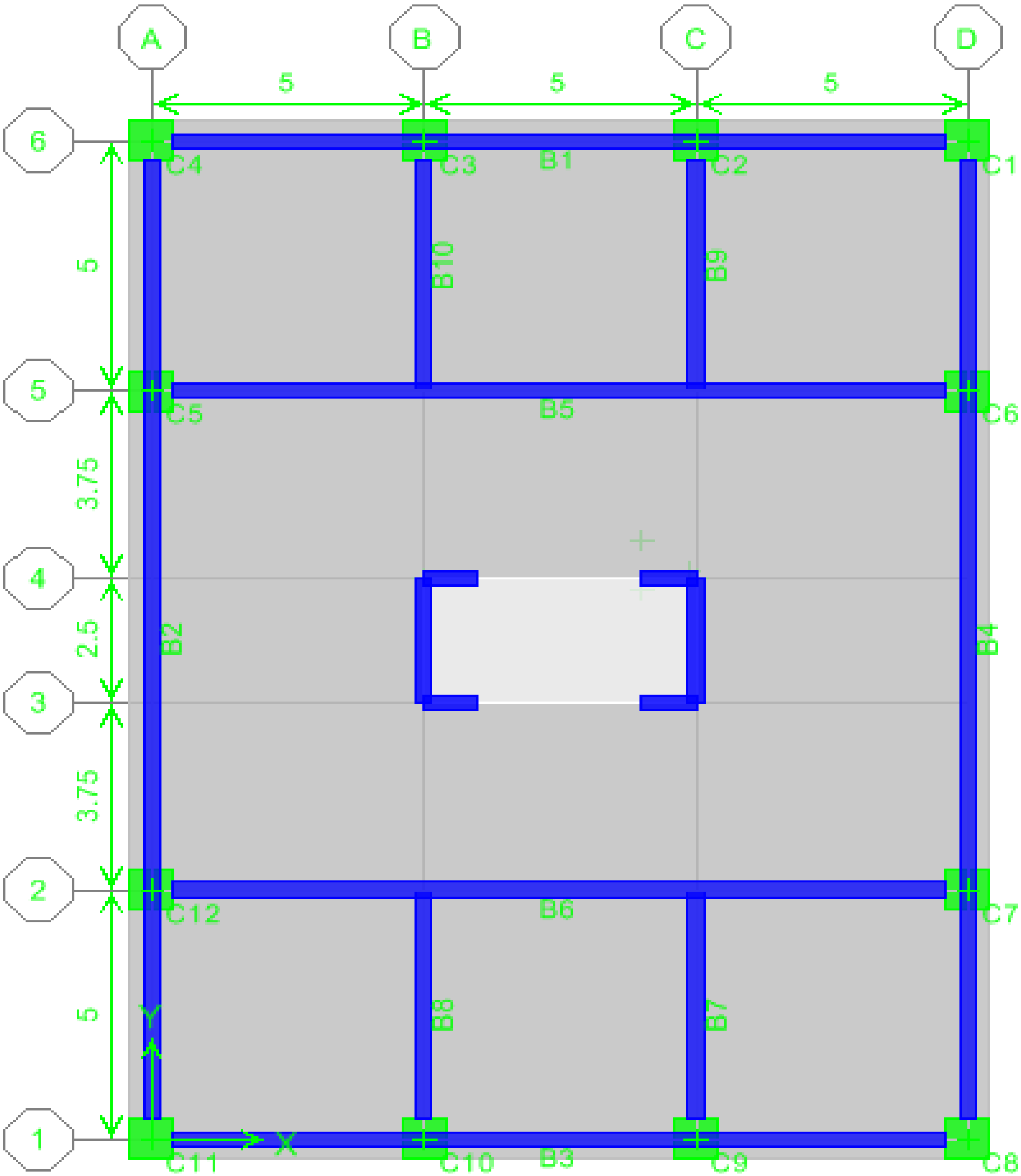

Fig. (3.1): Floors Plan

3.3 Analysis results:

3.3.1 Wind Loads Calculation:

- Wind design data is as follows:

$V = 45m/sec$

Assuming, the building is a health care facilities with a capacity of 50 or more resident patients, so, occupancy category is III (ASCE7-05 table 1-1) and the exposure categories is D (the ground surface roughness ASCE7-05-6.5.6.3)

For exposure (D), I = 1.25 according to ASCE7-05 table 6-1(Appendix A)

k_d = 0.85 for main wind force resisting systems of buildings according to ASCE7-05 table 6-4 (Appendix A)

$k_z = 2.01(z/z_g)^{2/e}$

e = 11.5 for exposure D according to ASCE7-05 table 6-2 (Appendix A)

z_g = 213 for exposure D according to ASCE7-05 table 6-2 (Appendix A)

k_z at the 40th story $= 2.01(128.8/213)^{2/11.5} = 1.842$

k_{zt} = 1 assuming the terrain is flat (Appendix A)

$G_f = 0.925((1+1.7I_z\sqrt{(g^2{}_QQ^2+g^2{}_gR^2)}/(1+1.7g_vI_z))$ Equation 6-8 from ASCE7-05

$G_f = 0.94$

$q_z = 0.613k_zk_{zt}k_dV^2I$

$q_z = 0.613\times1.842\times1\times0.85\times1.25\times45^2 = 2429N/m^2$

Assuming the building is enclosed (ASCE6.5.9), $GC_{pi} = \pm0.18$

C_p for wind in E-W direction (ASCE7-05 figure 6.6), (Appendix A)

C_p(wind ward wall) = 0.8

C_p (lee ward wall) = -0.5

C_p(side wall) = -0.7

C_p (over entire roof) = -1.3

$P_{windward}$(external pressure) $= qGC_p = 2429\times0.94\times0.8 = 1827N/m^2$

P (internal pressure) $= qGC_{pi} = 2429\times(\pm0.18) = 437N/m^2$

$P_{leeward}$(external pressure) = 2429×0.94×0.5 = 1142N/m^2

$P = P_{wind\ ward} + P_{lee\ ward} = 98.8kN$

Wind loads at story level are shown in table (3.1).

Table (3.1): Summary of Wind Loads at Story Level

Level	Story height (m)	Position (m)	Tributary area (m)	Design Wind Load (kN)
40th	3.2	128.8	1.6	98.8
39th	3.2	125.6	3.2	197
38th	3.2	122.4	3.2	195.8
37	3.2	119.2	3.2	195.7
36	3.2	116	3.2	195.3
35	3.2	112.8	3.2	194.6
34	3.2	109.6	3.2	194.1
33	3.2	106.4	3.2	193.4
32	3.2	103.2	3.2	192.7
31	3.2	100	3.2	192.3
30	3.2	96.8	3.2	191.6
29	3.2	93.6	3.2	190.8
28	3.2	90.4	3.2	189.8
27	3.2	87.2	3.2	189.5
26	3.2	84	3.2	188.7
25	3.2	80.8	3.2	188
24	3.2	77.6	3.2	187.2
23	3.2	74.4	3.2	186.4
22	3.2	71.2	3.2	185.5
21	3.2	68	3.2	184.7
20	3.2	64.8	3.2	183.7
19	3.2	61.6	3.2	182.9
18	3.2	58.4	3.2	181.9
17	3.2	55.2	3.2	180.8

16	3.2	52	3.2	179.7
15	3.2	48.8	3.2	178.6
14	3.2	45.6	3.2	177.3
13	3.2	42.4	3.2	176.1
12	3.2	39.2	3.2	174.6
11	3.2	36	3.2	173.2
10	3.2	32.8	3.2	171.8
9	3.2	29.6	3.2	170
8	3.2	26.4	3.2	168.2
7	3.2	23.2	3.2	165.3
6	3.2	20	3.2	163.6
5	3.2	16.8	3.2	161.2
4th	3.2	13.6	3.2	158.2
3rd	3.2	10.4	3.2	154.3
2nd	3.2	7.2	3.2	149.5
1st	4	4	3.6	162

3.3.2 Distribution of Wind Loads:

Wind load percentage carried by core = $EI_{core}/(EI_{core}+GA_{frames})$ =

$14.29\times2.78\times10^7/(14.29\times2.78\times10^7+128.2\times10^7)$ = 24%

Wind load percentage carried by four frames = 1-0.24= 76%

Table 3.2 shows the distribution of wind loads

Table (3.2): Distribution of Wind Loads between Core and Frames

Level	story height (m)	Position (m)	Load resist by core (kN)	Load resist by Frames (kN)
40th	3.2	128.8	30	70
39th	3.2	125.6	47	150
38	3.2	122.4	47	149
37	3.2	119.2	47	148
36	3.2	116	47	148

35	3.2	112.8	47	147
34	3.2	109.6	47	146
33	3.2	106.4	46	146
32	3.2	103.2	46	146
31	3.2	100	46	146
30	3.2	96.8	46	146
29	3.2	93.6	46	145
28	3.2	90.4	46	144
27	3.2	87.2	46	144
26	3.2	84	45	144
25	3.2	80.8	45	143
24	3.2	77.6	45	142
23	3.2	74.4	45	141
22	3.2	71.2	45	141
21	3.2	68	44	141
20	3.2	64.8	44	140
19	3.2	61.6	44	139
18	3.2	58.4	44	138
17	3.2	55.2	43	138
16	3.2	52	43	137
15	3.2	48.8	43	136
14	3.2	45.6	43	134
13	3.2	42.4	42	134
12	3.2	39.2	42	133
11	3.2	36	42	131
10	3.2	32.8	41	131
9	3.2	29.6	41	129
8	3.2	26.4	40	128
7	3.2	23.2	40	125
6	3.2	20	40	124
5	3.2	16.8	40	121
4^{th}	3.2	13.6	38	120

3rd	3.2	10.4	37	117
2nd	3.2	72	36	114
1st	4	4	39	123

3.3.3 Load Resisted by Frame (1) in Fig. (3.1):

Shear rigidity of frames (1,2,5,6)

$GA_{(1,6)} = 12E/h((1/C)+(1/G)) = 19.3\times10^7$

Shear rigidity of frames (2) and (5)

$GA_{(2,5)} = 44.8\times10^7$

Rigidity of frame (1) = $GA_1/(GA_1+GA_6+GA_2+GA_5) = 20\%$

Load resisted by frame (1) is shown in table (3.3)

Table (3.3): Load Resisted by Frame (1)

Level	Position (m)	Wind Load (kN)	S.F at mid story (kN)
40th	128.8	15	15
39th	125.6	30	45
38th	122.4	30	75
37	119.2	30	95
36	116	30	125
35	112.8	30	155
34	109.6	30	185
33	106.4	29	214
32	103.2	29	243
31	100	29	272
30	96.8	29	301
29	93.6	29	330
28	90.4	29	359
27	87.2	29	388
26	84	29	417
25	80.8	29	446

24	77.6	29	475
23	74.4	28	504
22	71.2	28	533
21	68	28	562
20	64.8	28	591
19	61.6	28	620
18	58.4	28	649
17	55.2	28	678
16	52	27	707
15	48.8	27	736
14	45.6	27	765
13	42.4	27	794
12	39.2	27	823
11	36	26	852
10	32.8	26	881
9	29.6	26	910
8	26.4	26	939
7	23.2	25	968
6	20	25	997
5	16.8	25	1026
4^{th}	13.6	24	1055
3^{rd}	10.4	24	1084
2^{nd}	72	23	1113
1^{st}	4	25	1142

3.3.4 Analysis of Gravity Loads:

Gravity Loads on beam (C11-C10)

DL_{floor} = 45kN/m

LL_{floor} = 5kN/m

DL_{roof} = 35kN/m

LL_{roof} = 2kN/m

Fixed end moments = $wL^2/12$, L = 5m

By moments distribution method

Stiffness ($k_{(beam\ or\ column)}$) = $I/L_{(beam\ or\ column)}$

Moment of inertia ($I_{(beam\ or\ column)}$) = $bh^3/12$

Distribution factor (DF) = $k/\Sigma k$

Bending moments and shear forces under gravity loads are shown in table (3.4)

Table (3.4): Summary of Bending Moments and Shear Forces under Gravity Loads for beam (C11-C10) and beam (C10-C9)

Cases		Location		B.M (kN-m)	S.F (kN)
For Roof	Span (C11- C10)	Support	Exterior	-116	-68
			Interior	42	55
		Mid span		51	
	Span (C10-C9)	Support	First	-53	-58.4
			Second	-53	58.4
		Mid span		25	
For Floors	Span (C11- C10)	Support	Exterior	-128	-100
			Interior	68	76
		Mid span		57.7	
	Span (C10-C9)	Support	First	-71	-60
			Second	-71	60
		Mid span		40	
For First Floor	Span (C11- C10)	Support	Exterior	-85	-80
			Interior	65	72
		Mid span		49	
	Span (C10-C9)	Support	First	-69.3	-82.5
			Second	-69.3	82.5
		Mid span		35	

- Beams shear force and bending moment diagrams due to gravity loads are shown in Figures bellow.

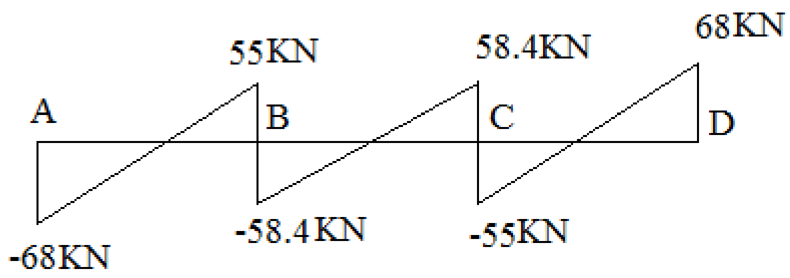

Fig. (3.2): Shear Forces Diagrams due to Gravity Loads for Beams at Roof Level

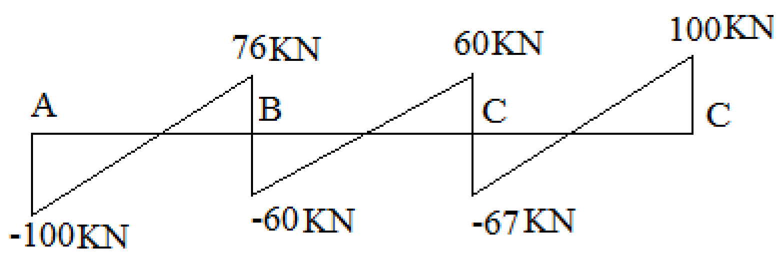

Fig. (3.3): Shear Forces Diagrams due to Gravity Loads for Beams at Floors Level

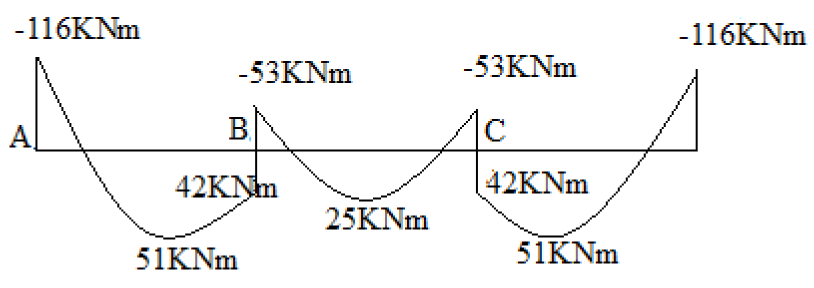

Fig. (3.4): Bending Moments Diagrams due to Gravity Loads for Beams at Roof Level

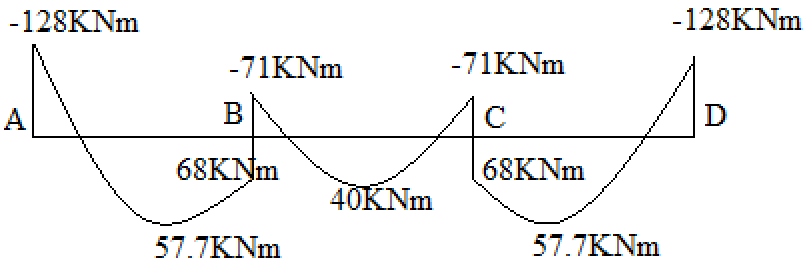

Fig. (3.5): Bending Moments Diagrams due to Gravity Loads for Beams at Floors Level

3.3.5 Analysis of Wind Loads:

By cantilever method

Wind load at 40th story level (frame 1) = 15kN

External moment due to wind = 15×1.6 = 24 kNm

Second moment of area

$= 1\ (7.5^2+2.5^2+2.5^2+7.5^2) = 125\text{m}^4$

Column(C11) axial forces

$= 24\times7.5/125 = 1.5\text{kN}$

Shear forces at beam (C11-C10) = 1.5kN

Moment at left end of beam (C11-C10) = 1.5×2.5 = ±4kNm

Bending moments and shear forces under wind loads are shown in table (3.5) and (3.6)

Table (3.5): Summary of Bending Moments and Shear Forces for Beam (C11-C10) under Wind loads

Level	Location		B.M (kN-m)	S.F (kN)
40th	Support	Exterior	±4	±1.5
		Interior	±4	±1.5
39th	Support	Exterior	±15	±5

		Interior	±15	±5
38th	Support	Exterior	±28.8	±11.5
		Interior	±28.8	±11.5
37	Support	Exterior	±43.3	±17.3
		Interior	±43.3	±17.3
36	Support	Exterior	±57.6	±23
		Interior	±57.6	±23
35	Support	Exterior	±70	±28.8
		Interior	±70	±28.8
34	Support	Exterior	±86.5	±34.6
		Interior	±86.5	±34.6
33	Support	Exterior	±100.5	±40.2
		Interior	±100.5	±40.2
32	Support	Exterior	±113.5	±45.4
		Interior	±113.5	±45.4
31	Support	Exterior	±130	±52
		Interior	±130	±52
30	Support	Exterior	±142.5	±57
		Interior	±142.5	±57
29	Support	Exterior	±157.5	±63
		Interior	±157.5	±63
28	Support	Exterior	±170	±68
		Interior	±170	±68
27	Support	Exterior	±183	±73
		Interior	±183	±73
26	Support	Exterior	±198	±79
		Interior	±198	±79

25	Support	Exterior	±213	±85
		Interior	±213	±85
24	Support	Exterior	±225	±90
	Support	Interior	±225	±90
23	Support	Exterior	±240	±96
		Interior	±240	±96
22	Support	Exterior	±253	±101
		Interior	±253	±101
21	Support	Exterior	±268	±107
		Interior	±268	±107
20	Support	Exterior	±280	±112
		Interior	±280	±112
19	Support	Exterior	±291	±117
		Interior	±291	±117
18	Support	Exterior	±307	±123
		Interior	±307	±123
17	Support	Exterior	±317.5	±127
		Interior	±317.5	±127
16	Support	Exterior	±325	±130
		Interior	±325	±130
15	Support	Exterior	±352	±141
		Interior	±352	±141
14	Support	Exterior	±360	±144
		Interior	±360	±144
13	Support	Exterior	±367	±147
		Interior	±367	±147
12	Support	Exterior	±398	±159

		Interior	±398	±159
11	Support	Exterior	±411	±165
		Interior	±411	±165
10	Support	Exterior	±440	±180
		Interior	±440	±180
9	Support	Exterior	±450	±180
		Interior	±450	±180
8	Support	Exterior	±450	±184
		Interior	±450	±184
7	Support	Exterior	±450	±180
		Interior	±450	±180
6th	Support	Exterior	±450	±180
		Interior	±450	±180
5th	Support	Exterior	±450	±180
		Interior	±450	±180
4th	Support	Exterior	±400	±160
		Interior	±400	±160
3rd	Support	Exterior	±425	±170
		Interior	±425	±170
2nd	Support	Exterior	±450	±180
		Interior	±450	±180
1st	Support	Exterior	±350	±140
		Interior	±350	±140

Table (3.6): Summary of Bending Moments and Shear Forces for Beam (C10-C9) under Wind Loads

level	Location	B.M (kN-m)	S.F (kN)
40th	Support	±8	±3
39th	Support	±25	±10
38	Support	±45	±18
37	Support	±58	±23
36	Support	±75	±30
35	Support	±90	±37
34	Support	±110	±45
33	Support	±115	±50
32	Support	±130	±55
31	Support	±150	±68
30	Support	±180	±75
29	Support	±210	±84
28	Support	±227	±91
27	Support	±242	±97
26	Support	±265	±106
25	Support	±282	±113
24	Support	±300	±120
23	Support	±320	±128
22	Support	±335	±135
21	Support	±355	±142
20	Support	±357	±143
19	Support	±370	±150
18	Support	±390	±157
17	Support	±417	±167

16	Support	±450	±180
15	Support	±470	±190
14	Support	±500	±230
13	Support	±475	±190
12	Support	±500	±200
11	Support	±500	±200
10	Support	±550	±220
9	Support	±575	±230
8	Support	±585	±234
7	Support	±575	±230
6	Support	±562	±225
5	Support	±570	±230
4th	Support	±525	±210
3rd	Support	±475	±190
2nd	Support	±500	±230
1st	Support	±475	±190

- Beams bending moment, shear forces diagrams due to wind loads are shown in figures bellow.

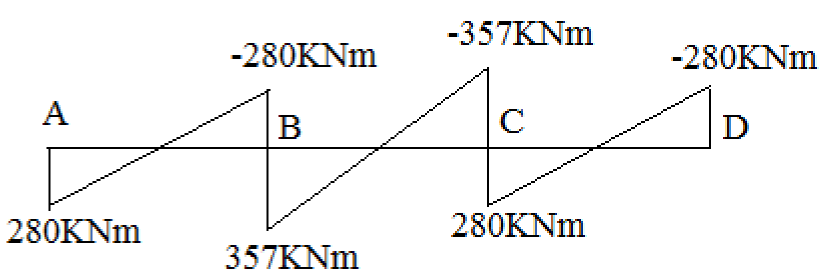

Fig. (3.6): Bending Moment Diagrams due to Wind Loads For Beam at 20th Story Level

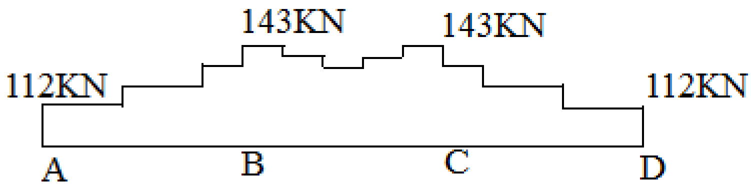

Fig. (3.7): Shear Forces Diagrams due to Wind Loads for Beam at 20th Story Level

3.3.6 Column C11 Axial Loads Calculations (at 40th Story Level)

- Dead load:

Tributary area = 9m^2

Column self weight = 0.8×0.8(3.2-0.9-0.2)×24 = 32.3kN

Slab self weight within the tributary area = 36.3kN

Beam self weight within the tributary area = 29.2kN

Masonry weight = 5× 0.3× 1 ×22 = 33kN

Super imposed dead load for roof =9.7kN

Total axial loads at 40th story level

36.3 + 33 + 29.7 + 9.7 = 118.2kN

- Live load = 0.96 × 9 = 8.6kN

Axial load due to dead, live and wind are shown in table (3.7)

Table (3.7): Summary of Axial Load for Col (C11)

Level	Height (m)	Dead Load (kN)	Live Load (kN)	Wind load (kN)
40th	128.8	118.2	8.6	2
39th	125.6	334.1	30.2	8
38	122.4	554.7	51.8	18.7
37	119.2	775.4	73.4	36
36	116	996	95	59
35	112.8	1216.6	116.6	87.8

34	109.6	1437.3	138.2	122.4
33	106.4	1657.9	159.8	162.6
32	103.2	1878.6	181.4	208
31	100	2099.2	203	260
30	96.8	2319.8	224.6	317
29	93.6	2540.5	246.6	380
28	90.4	2761.1	267.8	448
27	87.2	2981.8	289.4	521
26	84	3202.4	311	600
25	80.8	3423	332.6	685
24	77.6	3643.7	354.2	775
23	74.4	3864.32	375.8	871
22	71.2	4085	397.4	970
21	68	4305.6	419	1079
20	64.8	4526.2	440.6	1191
19	61.6	4746.9	462.2	1308
18	58.4	4967.5	483.8	1430
17	55.2	5188.2	505.4	1559
16	52	5408.8	527	1689
15	48.8	5629.4	548.6	1830
14	45.6	5850	570.2	1974
13	42.4	6070.7	591.8	2121
12	39.2	6291.4	613.4	2280
11	36	6512	635	2400
10	32.8	6732.6	656.6	2580
9	29.6	6953	678.2	2760
8	26.4	7173.9	699.8	2944
7	23.2	7394.6	721.4	3123
6	20	7615.2	743	3300
5	16.8	7835.8	764.6	3480
4th	13.6	8056.5	786.2	3640
3rd	10.4	8277	807.3	3780

2nd	72	8497.8	829.4	3960
1st	4	8718	851	4140
Base	0	8958	872.6	4140

Table (3.8): Summary of Bending Moments and Shear Forces for Col (C11)

Level	Location	Gravity load (Moments distribution method)	Wind load (Cantilever method)	
		B.M (kNm)	B.M (kNm)	S.F (kN)
40th	At Top	63	±5	3.1
	At Bottom	-94	±5	
39th	At Top	36	±13	8
	At Bottom	-89	±13	
38	At Top	36	±18	11
	At Bottom	-89	±18	
37	At Top	36	±25.3	15.8
	At Bottom	-89	±25.3	
36	At Top	36	±30	18.7
	At Bottom	-89	±30	
35	At Top	36	±38	23.7
	At Bottom	-89	±38	
34	At Top	36	±47	29.4
	At Bottom	-89	±47	
33	At Top	36	±53	33
	At Bottom	-89	±53	
32	At Top	36	±60	37.5
	At Bottom	-89	±60	
31	At Top	36	±70	43.7
	At Bottom	-89	±70	
30	At Top	36	±73	45.6
	At Bottom	-89	±73	
29	At Top	36	±84	52

	At Bottom	-89	±84	
28	At Top	36	±86	54
	At Bottom	-89	±86	
27	At Top	36	±95	59.4
	At Bottom	-89	±95	
26	At Top	36	±103	64.4
	At Bottom	-89	±103	
25	At Top	36	±110	68.8
	At Bottom	-89	±110	
24	At Top	36	±115	72
	At Bottom	-89	±115	
23	At Top	36	±125	79
	At Bottom	-89	±125	
22	At Top	36	±128	81
	At Bottom	-89	±128	
21	At Top	36	±139	87
	At Bottom	-89	±139	
20	At Top	36	±142	89
	At Bottom	-89	±142	
19	At Top	36	±149	94
	At Bottom	-89	±149	
18	At Top	36	±158	98
	At Bottom	-89	±158	
17	At Top	36	±162	101
	At Bottom	-89	±162	
16	At Top	36	±166	103
	At Bottom	-89	±166	
15	At Top	36	±180	112
	At Bottom	-89	±180	
14	At Top	36	±180	112
	At Bottom	-89	±180	
13	At Top	36	±192	117

	At Bottom	-89	±192	
12	At Top	36	±200	125
	At Bottom	-89	±200	
11	At Top	36	±200	125
	At Bottom	-89	±200	
10	At Top	36	±203	127
	At Bottom	-89	±203	
9	At Top	36	±215	134
	At Bottom	-89	±215	
8	At Top	36	±220	138
	At Bottom	-89	±220	
7	At Top	36	±227	142
	At Bottom	-89	±227	
6th	At Top	36	±231	144
	At Bottom	-89	±231	
5th	At Top	36	±247	147
	At Bottom	-89	±247	
4th	At Top	36	±255	159
	At Bottom	-89	±255	
3rd	At Top	36	±260	167
	At Bottom	-89	±260	
2nd	At Top	36	±285	178
	At Bottom	-89	±285	
1st	At Top	34	±297.5	186
	At Bottom	-17	±665	

- Column bending moment diagrams due to gravity loads and wind loads are shown in Fig. (3.8)

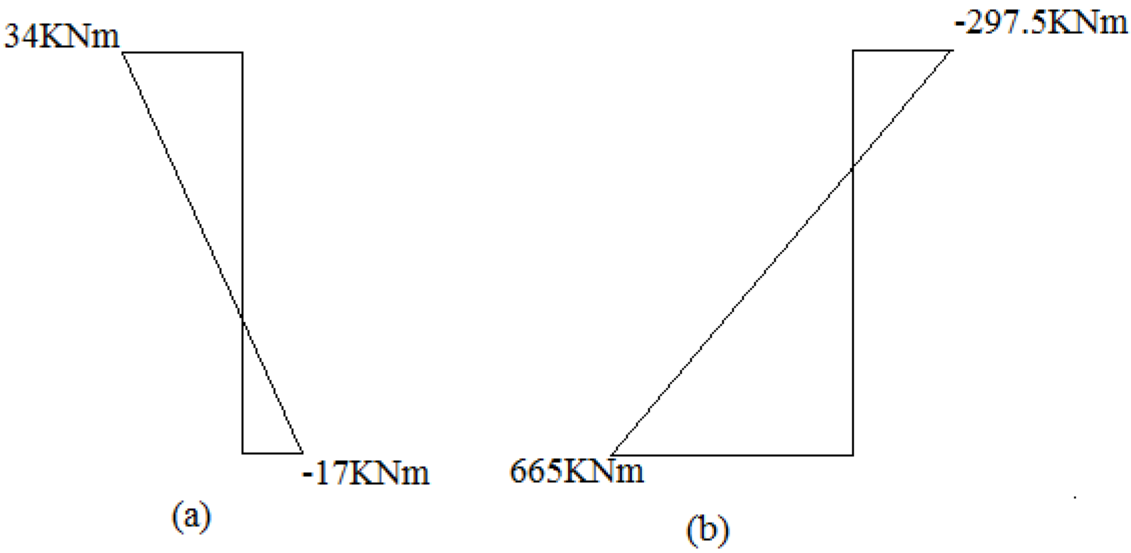

Fig. (3.8): Bending Moment Diagrams for Column (C11) at First Story Level due to (a) Gravity Loads (b) Wind Loads

3.4.7 Column C10 Axial Loads Calculations at 40th Story Level

- Dead load:

Tributary area = 15m^2

Column self weight = 0.8×0.8(3.2-0.9-0.2)× 24 = 32.3kN

Slab self weight within the tributary area = 69.6kN

Beam self weight within the tributary area = 46kN

Masonry weight = 5×0.3×1×22 = 33k

Superimposed dead load for roof = 1.07×15 = 17.2kN

Total axial loads at 40th story level due to dead loads;

70 + 46 + 33 + 17.2 = 167kN

- Live load = 0.96 × 15 = 14.4kN

Axial load due to dead, live and wind are shown in table (3.9)

Table (3.9): Summary of Axial Forces for Col (C10)

Level	Height(m)	Dead Load (kN)	Live Load (kN)	Wind Load (kN)
40th	128.8	167	14.4	1
39th	125.6	433.8	50.4	3
38	122.4	700.6	86.4	6.2
37	119.2	967.4	122.4	12
36	116	1234.1	158.4	20
35	112.8	1501	194.4	29
34	109.6	1767.8	230.4	40.8
33	106.4	2034.6	266.4	54
32	103.2	2301.4	302.4	69
31	100	2568.2	338.4	86
30	96.8	2835	374.4	105.6
29	93.6	3101.8	410.4	126
28	90.4	3368.6	446.4	149
27	87.2	3635.4	482.4	173
26	84	3902.2	518.4	200
25	80.8	4169	554.4	228
24	77.6	4435.8	590.4	258
23	74.4	4702.6	626.4	290
22	71.2	4969.4	662.4	324
21	68	5236.2	698.4	359
20	64.8	5503	734.4	396
19	61.6	5769.8	770.4	436
18	58.4	6036.4	806.4	470
17	55.2	6303.2	842.4	510
16	52	6570	878.4	560
15	48.8	6836.8	914.4	610
14	45.6	7103.6	950.4	650
13	42.4	7370.4	986.4	700
12	39.2	7637.2	1022.4	760

11	36	7904	1058.4	800
10	32.8	8170.8	1094.4	850
9	29.6	8437.6	1130.4	900
8	26.4	8704.4	1202.4	950
7	23.2	8971.2	1238.4	1000
6	20	9238	1274.4	1050
5	16.8	9504.8	1310.4	1100
4th	13.6	9771.6	1346.4	1150
3rd	10.4	10038	1382.4	1200
2nd	7.2	10305.2	1418.4	1300
1st	4	10591.2	1454.4	1380
Base	0	10858	1490.4	1380

Table (3.10): Summary of Bending Moments and Shear Forces for Col (C10)

Level	Location	Gravity load (Moments distribution method)	Wind load (Cantilever method)	
		B.M (kN-m)	B.M (kN-m)	S.F (kN)
40th	At Top	22	±13	±8.1
	At Bottom	-24	±13	
39th	At Top	18	±26	±16.3
	At Bottom	-20	±26	
38	At Top	18	±47	±30
	At Bottom	-20	±47	
37	At Top	18	±54	±34
	At Bottom	-20	±58	
36	At Top	18	±71	±44
	At Bottom	-20	±71	
35	At Top	18	±89	±55
	At Bottom	-20	±89	

34	At Top	18	±100	±63
	At Bottom	-20	±100	
33	At Top	18	±115	±72
	At Bottom	-20	±115	
32	At Top	18	±120	±75
	At Bottom	-20	±120	
31	At Top	18	±150	±94
	At Bottom	-20	±150	
30	At Top	18	±170	±106
	At Bottom	-20	±170	
29	At Top	18	±190	±124
	At Bottom	-20	±190	
28	At Top	18	±207	±129
	At Bottom	-20	±207	
27	At Top	18	±218	±136
	At Bottom	5.7	±218	
26	At Top	18	±238	±149
	At Bottom	-20	±238	
25	At Top	18	±257	±161
	At Bottom	-20	±257	
24	At Top	18	±268	±168
	At Bottom	-20	±268	
23	At Top	18	±290	±181
	At Bottom	-20	±290	
22	At Top	18	±298	±186
	At Bottom	-20	±298	
21	At Top	18	±315	±198
	At Bottom	-20	±315	
20	At Top	18	±322	±200
	At Bottom	-20	±322	
19	At Top	18	±344	±215
	At Bottom	-20	±344	

18	At Top	18	±353	±221
	At Bottom	-20	±353	
17	At Top	18	±380	±237
	At Bottom	-20	±380	
16	At Top	18	±396	±247
	At Bottom	-20	±396	
15	At Top	18	±424	±265
	At Bottom	-20	±424	
14	At Top	18	±436	±272
	At Bottom	-20	±436	
13	At Top	18	±436	±272
	At Bottom	-20	±436	
12	At Top	18	±454	±284
	At Bottom	-20	±454	
11	At Top	18	±476	±297
	At Bottom	-20	±476	
10	At Top	18	±480	±300
	At Bottom	-20	±480	
9	At Top	18	±502	±313
	At Bottom	-20	±502	
8	At Top	18	±518	±324
	At Bottom	-20	±518	
7	At Top	18	±507	±317
	At Bottom	-20	±507	
6	At Top	18	±532	±332
	At Bottom	-20	±532	
5	At Top	18	±463	±289
	At Bottom	-20	±463	
4th	At Top	18	±475	±297
	At Bottom	-20	±475	
3rd	At Top	18	±522	±326
	At Bottom	-20	±522	

2nd	At Top	5	±598	±374
	At Bottom	-6	±598	
1st	At Top	2.3	±622	±389
	At Bottom	-2.4	±622	

Table (3.11): Summary of Design Bending Moments and Axial Force at the Base of Core.

Load Cases	Axial (kN)	Bending (kNm)	Shear (kN)
Dead	35239	0	0
Live	5183	0	0
Wind	0	116620	1715

3.4 Design Results:

3.4.1 Design of Slab:

- Two way solid slab designed by direct design method , it is currently the most common method of analysis in designing concrete floor systems.

Moment transfer between the slab, beam and column , the structure is divided into a series of equivalent frames along support lines. Each frame consists of a row of columns and corresponding slab-beam strip.

Floor slab and roof slab design calculations are shown in tables (3.13) and (3.15)

Table (3.12): Summary of Design Bending Moments for slab (an Edge Panel)

Cases	Location	Column strip moment		Middle strip slab moment
		Beam	Slab	
Floors	Interior negative	55.3	10	22
	Interior positive	45.3	8	20

	Exterior negative	17	3	0
Roof	Interior negative	45.3	8	18.1
	Interior positive	37.4	7	14.2
	Exterior negative	14	2.5	0

Table (3.13): Floor Design Calculations

Reference	Calculation	Out put
	- computing α_1 for both direction	
ACI318-05	Gross moment of inertia of slab 5m wide	
Eq. (13-3)	$I_s = (1/12)(5\times 0.2^3)$	$0.0033m^4$
	Gross I of T beam cross section about centered axis for interior beams	
	$I_b = 1.9\times0.2^3/3 + 0.3\times0.9^3/3$	$0.079m^2$
	$\alpha_1 = EI_b/EI_s = 0.079/0.0033$	23.9
	For edge beams (width = 5/2+0.15 = 2.65m)	
	$I_s = 2.65\times0.2^3/12$	$0.0018m^4$
	I for edge beams = $1.2\times 0.2^3/3 + 0.3\times0.9^3/3$	$0.076m^4$
	$\alpha_2 = 0.076/0.0018$	42.2
	$\alpha_m = (2\times23.9+42.2\times2)/4$	33 > 2
	$h= l_n(0.8+f_y/1400)/(36+9\beta)$	
	$l_{n(\text{long and short})} = 5-0.8$	4.2m
	$\beta = 5/5$	1
	h= 4200(0.8+420/1400)/(36+9)	103mm
	O.K. 200mm be satisfy	
ACI318-05	- Moment for the both spans for floors slab:	

Eq. (9.2)	$w_u = (1.2\times6.3)+(1.6\times2.4)$	$11.14N/m^2$
Eq. (13-4)	$M_u = (wl_2)(l_n^2)/8 = 11.14\times5\times4.2^2/8$	123kNm
	-Check shear strength in the slab at a distance d from the face of the beam, shear is assumed to be produced by the load on the tributary area, working with a 1m wide strip.	
	d = h - cover - half bar diameter	
	= 200-25-6	169mm
	-Design shear forces at critical section:	
Design of Reinforced Concrete ACI318-05 Seven edition	$V_u = W_u(S_n/2\text{-beam thickness}/2\text{-d})$	
	=11.14 (2.5-0.15-0.169)	24.3kN
	Design shear strength	
	$\emptyset V_c = 0.75\sqrt{f'_c}bd/6$	
	$= 0.75\sqrt{35}\times1000\times169/6$	125kN
	$\emptyset V_c > V_u$, (section is satisfactory depended on shear requirements)	
ACI318-05 13.6.3.3	-Dividing this static design moment into negative and positive portions,	
	Interior negative design moment = 0.7×123	87kNm
	Positive design moment = 0.57×123	71kNm
	Exterior negative design moment = 0.16×123	20kNm
ACI13-6-4	-Allotting these moments to beam and column strips,	
	l_2/l_1	1
	$\alpha_1 l_1/l_2$ (in direction of both long and short span)	23.9
ACI318-05 13.6.4.1	The portion of the interior negative moment to be resisted by the column strip,	

	0.75×-87	-65kNm
ACI318-05	This -65 is allotted 0.85% to the beam ,	-55.3kNm
13.5.6	and 15% to the slab, or	-10kNm
	The remaining negative moment, 87-65 is	22kNm
	Allotted to the middle strip.	
ACI318-05	The portion of the exterior negative moment to be	
13.6.4.2	resisted by the column strip,	
	$\beta_t = E_{cb}C/2E_{cs}I_S$	
	$C = \sum(1 - 0.63x/y)x3y/3 = .001 = 0$	0.001
	Exterior negative moment = 1×-20 KN/m, this	-20kNm
	allotted 0.85% to the beam,	-17kN/m
ACI318-05	and 15% to the slab, no moment at the middle strip.	-3kNm
13.6.4.4	The portion of the interior positive moment to be	
	resisted by the column strip, 0.75×71	53.3kNm
	This value 0.85% to the beam,	45.3kNm
	and 0.15% to the slab,	8kNm
	The remaining positive moment,	
	71-53.3 goes to the middle strip	20kNm
	- Moment for the both spans for roof:	
	$w_u = (1.2\times6.36)+(1.6\times0.96)$	$9.2kN/m^2$
	$M_u = (wl_2)(l_n^2)/8 = 9.2\times5\times4.2^2/8$	102kNm
	-Check shear strength in the slab at a distance d	
	from the face of the beam, shear is assumed to be	
	produced by the load on the tributary area, working	
	with a 1m wide strip.	
	d = h - cover - half bar diameter	
	= 200-25-6	169mm
	-Design shear forces capacity at critical section(at	

	distance d from the face of beam:	
	$V_u = W_u(S_n/2\text{-beam thickness}/2\text{-d})$	
	=9.2 (2.5-0.15-0.169)	20.1kN
	Design shear strength	
	$\emptyset V_c = 0.75\sqrt{f'_c}bd/6$	
ACI 9.3.2.1	$\emptyset$ = 0.9 (tension controlled section)	
	$= 0.75\sqrt{35}\times1000\times169/6$	125kN
	$\emptyset V_c > V_u$, (section is satisfactory depended on shear requirements)	
ACI318-05 13.6.3.3	-Dividing this static design moment into negative and positive portions,	
	Interior negative design moment = 0.7×102	71.4kNm
	Positive design moment = 0.57×102	58.2kNm
	Exterior negative design moment = 0.16×102	16.3kNm
	-Allotting these moments to beam and column strips, as per section 13.6.4 of the code:	
	$l_2/l_1 = 5/5$	1
ACI318-05 13.6.4.1	$\alpha_1 l_1/l_2$ (in direction of both short and long span)	23.9
	The portion of the interior negative moment to be resisted by the column strip, 0.75×-71.4This -53.3	-53.3kNm
	is allotted 0.85% to the beam ,	-45.3kNm
ACI318-05 13.5.6	and 15% to the slab,	8kNm
	The remaining negative moment, 71.4-53.3, is allotted to the middle strip.	18.1kNm
	The portion of the exterior negative moment to be resisted by the column strip,	
ACI318-05 13.6.4.2	$\beta_t = E_{cb}C/2E_{cs}I_S$	
	$C = \sum(1 - 0.63x/y)x3y/3 = .001 = 0$	0.001

Eq. (13-5)	Exterior negative moment = 1×-16.5 this allotted	-16.5kNm
Eq. (13-6)	0.85% to the beam, and 15% to the slab, no moment	-14kNm
	at the middle strip.	-2.5kNm
	The portion of the interior positive moment to be	
	resisted by the column strip, 0.75×58.2, This value	44kNm
	0.85% to the beam, and 0.15% to the slab, The	37.4kNm
ACI318-05	remaining positive moment,	7kNm
13.6.4.4	58.2-44 goes to the middle strip.	
	1.Floor slab design	14.2kNm
	a-Design of steel in column strip:	
	*M_u(interior negative)	10kNm
	The minimum reinforcement is that required for	
	control of shrinkage and temperature cracking	
ACI7.12.2.1	A_{smin} = 0.0018bh	
	= 0.0018×1000×200	$360mm^2$
	ρ_{min} (in both direction) = 360/(1000×169)	0.00213
	$M_u/\text{Ø}bd^2 = 10\times10^6/(0.9\times0.25\times5000\times169^2)$	3.2
	m = 14.1	
Design of	$\rho = 1/14.1(1-\sqrt{(1-(2\times14.1\times3.2/420))}) > \rho_{min}$	0.0081
Reinforced	A_s = 0.0081×1250 ×169	$1700mm^2$
Concrete	Use 13-No12 ($A_{sprovide}$= $1700mm^2$)	
ACI318-05	-Moment transfer design:	
Seven	additional bars must be added over the column in a	
edition	width = column diameter + (2)(1.5h)	
	= 0.8+(2×1.5×0.2)	1.4m
	The additional reinforcing needed over the column	
	is to be designed for a moment	
	$\gamma_f = 1/(1+2/3\sqrt{(b_1/b_2)})$	

	$= 1/(1+2/3\sqrt{(0.925/0.925)})$	0.6
	Reminder of the unbalanced moment	
ACI318-05	$\gamma_v = 1-0.6$	0.4
13.5.3.2	$\gamma_f M_u = 0.6\times10$	6kNm
ACI318-05	$M_u/\emptyset bd^2 = 6\times10^6/(0.9\times1400\times169^2)$	0.17
Eq. (11-39)	$\rho_{min}(=0.00213)$	
	$\rho = .0004 < 0.00213$	
	$A_s = 0.00213\times 1400\times 169$	504mm^2
	Add 4-No 12 bars in the 1.4m width and check to see the moment transfer situation is satisfactory.	
	$a = A_sf_y/(0.85f'_cb) = 504\times 420/(0.85\times 35\times1400)$	5.1mm
Design of Reinforced Concrete ACI318-05 Seven edition	$\emptyset M_n = \emptyset A_sf_y(d-a/2)$	
	$= 0.9(504)(420)(169-5.1/2)$	31.7kNm > 6kNm
	-Compute combined shear stress at exterior column due to shear and moment transfer	
	-Nominal moment strength of full column strip with 17-No 12 bars(2205mm^2)	
	$a = 2205\times 420/(0.85\times35\times2500)$	12.45mm
	$M_n = 2205\times420(169-12.45/2)$	150kNm
	-Fraction of unbalanced moment carried by eccentricity of shear =	
	$\gamma_v M_n = 150\times0.4$	60kNm
	*M_u(positive)	8kNm
	$M_u/\emptyset bd^2 = 8\times10^6/(0.9\times1250\times169^2)$	0.21
	$m = 14.1$	
	$\rho = 1/14.1(1-\sqrt{(1-(2\times14.1\times0.21/420))})$	0.00061

Design of Reinforced Concrete ACI318-05 Seven edition	$<\rho_{min}(=0.00213)$ $A_s = 0.00213 \times 1250 \times 169$ Use 4-No12 in two directions($A_{sprovide}=492mm^2$) $*M_u$(Exterior negative) $M_u/\emptyset bd^2 = 3\times10^6/(0.9\times.25\times5000\times169^2)$ m = 14.1 $\rho = 1/14.1(1-\sqrt{(1-(2\times14.1\times0.1/420))})$ $<\rho_{min}(=0.00213)$ $A_s = 0.00213\times 1250 \times169$ Use 4-No12 in two directions($A_{sprovide}= 492mm^2$) b. Design of steel in middle strip: $*M_u$(interior negative) $M_u/\emptyset bd^2 = 22\times10^6/(0.9\times.5\times5000\times169^2)$ m = 14.1 $\rho = 1/14.1(1-\sqrt{(1-(2\times14.1\times0.34/420))})$ $<\rho_{min}(=0.00213)$ $A_s = 0.00213\times2500 \times169$ Use 8-No12 in two directions($A_{sprovide}=985mm^2$) $*M_u$ (interior positive) $M_u/\emptyset bd^2 = 20\times10^6/(0.9\times2500\times169^2)$ m = 14.1 $\rho = 1/14.1(1-\sqrt{(1-(2\times14.1\times0.31/420))})$ $<\rho_{min}(=0.00213)$ $A_s = 0.00213\times2500\times169$ Use 8-No12in both directions($A_{sprovide} = 985mm^2$)	 $450mm^2$ 3kNm 0.1 0.00055 $450mm^2$ 22kNm 0.34 0.00081 $900mm^2$ 20kNm 0.31 0.0007 $900mm^2$

Table (3.14): Required Floors Reinforcement

Strip cases	Location	moment		$A_s(mm^2)$	Number of bars No-12
5m span Two column strip	Exterior negative	3	0.00213	492	4
	positive	8	0.00213	492	4
	Interior Negative	10	0.0081	1700	13
Middle strip	Exterior Negative	0	0	0	0
	Positive	20	0.00213	985	8
	Interior Negative	22	0.00213	985	8

Table (3.15): Roof Design Calculations

Reference	Calculations	Out Put
	2.Roof design	
	a. Design of steel in column strip	
	*M_u(interior negative)	8kNm
ACI7.12.2.1	The minimum reinforcement is that required for control of shrinkage and temperature cracking	
	$A_s = 0.0018bh$	
	$= 0.0018\times1000\times200$	$360mm^2$
	ρ_{min} (in both direction) = $360/(1000\times169)$	0.00213
Design of Reinforced Concrete ACI318-05	$M_u/\text{Ø}bd^2 = 8\times10^6/(0.9\times0.25\times5000\times169^2)$	2.6
	$m = 14.1$	
	$\rho = 1/14.1(1-\sqrt{(1-(2\times14.1\times2.6/420))}) > \rho_{min}$	0.0065
	$A_s = 0.0065\times1250\times169$	$1373mm^2$

Seven edition	Use 11-No12 ($A_{sprovide}$= 1375mm^2) -Moment transfer design: The code 13.5.3.2 state that additional bars must be added over the column in a width = column diameter + (2)(1.5h) = 0.8+(2×1.5×0.2)	1.4m
ACI318-05 13.5.3.2	The additional reinforcing needed over the column is to be designed for a moment γ_f= 1/(1+2/3√(b_1/b_2)) = 1/(1+2/3√(0.925/0.925))	0.6
ACI318-05 Eq. (11-39)	γ_v= 1-0.6 =0.4(reminder of the unbalanced moment) $\gamma_f M_u$= 0.6×8 $M_u/Øbd^2$ = 5×10^6/(0.9×1400×169^2) ρ = .0003 < 0.00213 ρ_{min}(=0.00213) A_s = 0.00213× 1400× 169 Add 4-No 12 bars in the 1.4m width and check to see the moment transfer situation is satisfactory. a= $A_s f_y/(0.85f'_c b)$=504× 420/(0.85× 35×1400)	5kNm 0.14 504mm^2 5.1mm
Design of Reinforced Concrete ACI318-05 Seven edition	$ØM_n = ØA_s f_y(d-a/2)$ = 0.9(504)(420)(169-5.1/2) -Compute combined shear stress at exterior column due to shear and moment transfer -Nominal moment strength of full column strip with	31.7kNm < 5kNm

	15-No 12 bars($1880mm^2$)	
	a= 1880 × 420/(0.85×35×2500)	10.6mm
	M_n= 1880×420(169-10.6/2)	129.3kNm
	-Fraction of unbalanced moment carried by eccentricity of shear	
	$\gamma_v M_n$= 129.3×0.4	52KNm
	*M_u(positive)	7kNm
	$M_u/Øbd^2$ =7×10^6/(0.9×1250×169^2)	0.18
	m = 14.1	
	ρ = 1/14.1(1-√(1-(2×14.1×0.18/420)))	0.0006
	<ρ_{min}(=0.00213)	
	A_s =0.00213×1250×169	450mm^2
	Use 4-No12 in two directions($A_{sprovide}$=492mm^2)	
	*M_u(Exterior negative)	2.5kNm
	$M_u/Øbd^2$ =2.5×10^6/(0.9×.25×5000×169^2)	0.08
	m = 14.1	
	ρ = 1/14.1(1-√(1-(2×14.1×0.08/420)))	0.0005
	<ρ_{min}(=0.00213)	
	A_s =0.00213× 1250 ×169	450mm^2
	Use 4-No12 in two directions($A_{sprovide}$= 492mm^2)	
	b. Design of steel in middle strip	
	*M_u(interior negative)	18kNm
	$M_u/Øbd^2$ =18×10^6/(0.9×.5×5000×169^2)	0.30
	m = 14.1	
	ρ = 1/14.1(1-√(1-(2×14.1×0.30/420)))	0.00075
	<ρ_{min}(=0.00213)	
Design of	A_s =0.00213×2500 ×169	900mm^2

Reinforced Concrete ACI318-05 Seven edition	Use 8-No12 in two directions($A_{sprovide}$ = 985mm^2)	
	*M_u(interior positive)	14.2kNm
	$M_u/\emptyset bd^2$ =14.2×10^6/(0.9×.5×5000×169^2)	0.25
	m = 14.1	
	ρ = 1/14.1(1-√(1-(2×14.1×0.25/420)))	0.00068
	<ρ_{min}(=0.00213)	
	A_s =0.00213×2500 ×169	900mm^2
	Use 8-No12 in two directions($A_{sprovide}$= 985)	

Table (3.16): Required Roof Reinforcement

Strip cases	Location	Moment		A_s(mm^2)	Number of bars No-13
Column strip	Exterior negative	2.5	0.00213	492	4
	positive	7	0.00213	492	4
	Interior negative	8	0.0065	1375	11
Middle strip	Exterior negative	0	0	0	0
	Positive	14.2	0.00213	985	8
	Interior Negative	18.1	0.00213	985	8

Table (3.17): Summary of Design Bending Moments and Shear Forces for Beam C10-C11 at First Story Level

Load Cases	Location	B.M (kNm)	S.F (kN)
Gravity (DL+LL)	Support	-85	80
	Mid span	49	
Wind (WL)	Support	±350	±140
	Mid span	0	0
Load Combination			
1.2DL+1.6LL	Support	-106	100
	Mid span	61	
1.2DL+LL±1.6WL	Support	-660	320

3.4.2 Design of Beams:

Beams must have an adequate safety margin against other types of failure (flexural and shear).

Beams flexural and shear design calculations shown in tables (3.18), (3.20), (3.22), (3.24), (3.26) and (3.28).

Table (3.18): Flexural Beam Design Calculations at First Story

Reference	Calculations	**Out Put**
ACI318-05 21.12.4	1-Flexural Design: The factored axial load on the member, Which is negligible, is less than $A_g f_c'/10$, for beams must be satisfied. All other applicable provisions in ACI318-05 are to be satisfied as well.	
ACI318-05	-Minimum flexural reinforcement	

10.5.1	$A_{smin} = (0.25\sqrt{f_c'}bd)/f_y = 0.25\sqrt{35} \times 300 \times 800/420$	$850mm^2$
	$\geq 1.4bd/f_y = 1.4 \times 300 \times 800/420$	$800mm^2$
	-Maximum flexural reinforcement:	
	$A_{smax} = \frac{0.85\beta_1 f_c' bd}{f_y} \times \left(\frac{0.003}{(.003+.004)}\right)$	
	$= 0.85 \times 0.814 \times 35 \times 300 \times 800/420 \times (0.003/0.007)$	$5930mm^2$
ACI318-05	-Maximum reinforcement percentage	
Fig.R10.3.3	$\rho_{max} = 0.85\beta_1 f_c'/f_y(\varepsilon_c/\varepsilon_c + 0.004)$	
	$= 0.85 \times 0.814 \times 35/420(0.003/0.007)$	0.0247
ACI10.2.3	- Minimum reinforcement percentage	
	$\rho_{min} = \sqrt{f_c'}/4f_y$	0.00352
Design of Reinforced Concrete ACI318-05 Seven edition	-Strain in compression concrete	
	$\varepsilon_c = 0.003$	
	$\beta_1 = 0.85 - 0.05(f_c - 4000)/1000$,	0.814
	$0.65 \leq \beta_1 \leq 0.85$.	
	-Design moment strength	
	$\emptyset M_n = \emptyset[A_{S1} f_y(d-a/2) + A_s' f_s'(d-d')]$	
	$a = A_S f_y/0.85 f_c' b$	
	$A_s = \rho bd$	
ACI9.3.2.1	$\rho = 1/m(1-\sqrt{(1-2mR/f_y)}$	
	$R = M_u/\emptyset bd^2$	
	$m = f_y/0.85f_c'$	
	-Effective depth	
	$d = h-100 = 900-100$	800mm
	$\emptyset = 0.9$ for tension controlled sections	

Table (3.19): Required Beam Reinforcement at First Story

Location	M_u(m-kN)	A_s (mm_2)	Reinforcement	ØMn(m-kN)
Support	-660	2455	5-No25	690
Mid span	61	942	3-No20	260

ACI21.12.4.1: the positive moment strength at the joint be greater than or equal to 33% of the negative moment strength at that location. This is satisfied, since 260 m-KN >690×0.33=227.7m-KN.

Table (3.20): Shear Design Calculation at First Story

Reference	Calculation	Out Put
	2.Shear Design:	
Fig. R21.12.3	Shear demand from nominal flexural capacity	
	V_u= (690 -260)/4.2	102.4kN
ACI318-05	Shear demand from gravity load	
	W_u=1.2W_D+W_L=1.2×45+5	60kN/m
	V_u =$W_u l_n$/2= 60×4.2/2	126kN
	V_u=102.4 +126	228.4kN
	The nominal shear strength provided by	
ACI318-05	concrete (V_c)	
11.3.1	$V_c=0.17 \times \sqrt{f'c} \times b \times d$	
	$= 0.17 \times \sqrt{35} \times 300 \times 800/1000$	241.4kN
	V_u= (228.4KN)> ØV_c = (0.75×241.4)	181kN
	Provide shear reinforcement in assuming No.	
	10 hoops, the required spacing s is determined,	
ACI318-05	S = ($A_v f_{yt}$d)/V_s	
11.5.6	=(142×250×800)/((28800/0.75)-241400) The	200mm

Eq. (11-15) ACI318-05 21.12.4.2 ACI318-05 21.12.4.3	maximum spacing of hoops over the length 2h = 2×900 from the face of the support at each end of the member is the smallest of the following: (1) $\frac{d}{4} = \frac{800}{4}$ (2) 24(diameter of hoop bar) = 24×9.5 For the remainder of the beam, the maximum stirrup spacing is $\frac{d}{2}$ Use No. 10stirrups @ $200mm$ for the remainder of the beam.	1800mm 200mm 228mm 400mm

Table (3.21): Summary of Design Bending Moment and Shear Forces for Beam C10-C11 AT 20th Story Level

Load Cases	Location	B.M (kNm)	S.F (kN)
Gravity (DL+LL)	Support	-128	100
	Mid span	57	
Wind (WL)	Support	±250	±120
	Mid span	0	0
Load Combination			
1.2DL+1.6LL	Support	-160	125
	Mid span	71	
1.2DL+LL±1.6WL	Support	-550	310

Table (3.22): Flexural Design Calculations at 20th Story Level

Reference	Calculation	Out Put
	1.Flexural Design:	
ACI318-05 21.12.4	The factored axial load on the member, Which is negligible, is less than $A_g f_c/10$;thus, the provisions of section for beams must be satisfied.	
	All other applicable provisions in ACI318-05 are to be satisfied as well.	
ACI318-05 10.5.1	Minimum flexural reinforcement	
	$A_{s,min} = (0.25\sqrt{f_c'}bd)/f_y =$	
	$0.25\sqrt{35} \times 300 \times 800/420$	850mm^2
	$\geq 1.4bd/f_y = 1.4 \times 300 \times 800/420$	800mm^2
	Maximum flexural reinforcement:	
	$A_{s,max} = \frac{0.85\beta_1 f_c' bd}{f_y} \times \left(\frac{0.003}{(.003+.004)}\right)$	
	$= 0.85 \times 0.814 \times 35 \times 300 \times 800/420 \times (0.003/0.007)$	5930mm^2
	-Maximum reinforcement percentage	
ACI318-05 Fig.R10.3.3	$\rho_{max} = 0.85\beta_1 f_c'/f_y(\varepsilon_c/\varepsilon_c + 0.004)$	
	- Minimum reinforcement percentage	
	$\rho_{min} = \sqrt{f_c'}/4f_y$	
	-Strain in compression concrete	
	$\varepsilon_c = 0.003$	
ACI10.2.3	$\beta_1 = 0.85-0.05(f_c-4000)/1000$,	
	$0.65 \leq \beta_1 \leq 0.85$.	
	-Design moment strength	

Design of Reinforced Concrete ACI318-05 Seven edition	$\emptyset M_n=\emptyset[A_{S1}f_y(d-a/2)+A_s'f_s'(d-d')]$ $a = A_Sf_y/0.85f_c'b$ $A_s = \rho bd$ $\rho = 1/m(1-\sqrt{(1-2mR/f_y)}$ $R = M_u/\emptyset bd^2$ $m = f_y/0.85f_c'$ -Effective depth $d = h-100$	800mm

Table (3.23): Required Beam Reinforcement at 20th Story Level

Location	M_u(m-kN)	A_s (mm_2)	Reinforcement	ØMn(m-kN)
Support	-550	1964	4-No25	560
Mid span	71	942	3-No20	260

ACI21.12.4.1: the positive moment strength at the joint be greater than or equal to 33% of the negative moment strength at that location. This is satisfied, since 260 m-kN >560×0.33= 185m-kN.

Table (3.24): Shear Design Calculation at 20th Story Level

Reference	Calculation	Out Put
ACI318-05 Fig.R21.12.3	**2.Shear Design:** Shear demand from nominal flexural capacity $V_u= (560-260)/4.2$ Shear demand from gravity load $W_u=1.2W_D+W_L=1.2\times45+5$ $V_u=W_ul_n/2= 60\times4.2/2$	 72kN 60kN/m 126kN

	V_u=72+126	198kN
ACI318-05 11.3.1	The nominal shear strength provided by concrete (V_c) $V_c=0.17 \times \sqrt{f'c} \times b \times d$ $= 0.17 \times 300 \times 800\sqrt{35}/1000$ V_u= (198KN)> $\emptyset V_c(0.75 \times 241.4)$	 241.4kN 181kN
ACI318-05 11.5.6 Eq. (11-15)	Provide shear reinforcement in accordance with ACI318-05-11.5.6 assuming No. 10 hoops, the required spacing S is $S = (A_v \times f_{ys} d)/V_s$ $=(142 \times 250 \times 800/(\frac{241400}{0.75} - 198000)$	 230mm
ACI318-05 21.12.4.2	The maximum spacing of hoops over the length $2h=2 \times 900$ from the face of the support at each end of the member is the smallest of the following: (1) $\frac{d}{4} = \frac{800}{4}$ (2) 24(diameter of hoop bars) = 24×9.5	1800mm 200mm 228mm 400mm
ACI318-05 21.12.4.3	For the remainder of the beam, the maximum stirrup spacing is $\frac{d}{2}$ Use No. 10stirrups @ $250mm$ for the remainder of the beam.	

Table (3.25): Summary of Design Bending Moment and Shear Forces for Beam C10-C11at 40th Story Level

Load Cases	Location	B.M (kNm)	S.F (kN)
Gravity (DL+LL)	Support	-116	68
	Mid span	51	
Wind (WL)	Support	±4	±2
	Mid span		
Load Combination			
1.2DL+1.6LL	Support	-148	85
	Mid span	63.6	
1.2DL+LL±1.6WL	Support	-145	84

Table (3.26): Flexural Design Calculations at 40th Story Level

Reference	Calculation	Out Put
	-Maximum percentage of reinforcement	
	$\rho_{max} = 0.85\beta_1 f_c'/f_y(\varepsilon_c/\ \varepsilon_c+.004)$	
	$= 0.85\times0.814\times35\times(0.003/.007)/420$	0.0247
	-Minimum percentage of reinforcement	
	$\rho_{min} = \sqrt{f_c'}/4f_y = \sqrt{35}/4\times420$	0.00352
	1.Flexural Design:	
ACI318-05 21.12.4	The factored axial load on the member, Which is negligible, is less than $A_g f_c/10$;thus, the provisions of section for beams must be satisfied.	
	All other applicable provisions in ACI318-05 are to be satisfied as well.	
	Minimum flexural reinforcement	
ACI318-05	$A_{s,min} = \underline{(0.25\sqrt{f_c'}bd)}/f_y = 0.25\sqrt{35}\times 300\times$	

10.5.1	800/420	$850mm^2$
	$\geq 1.4bd/f_y = 1.4\times 300 \times 800/420$	$800mm^2$
	Maximum flexural reinforcement:	
	$A_{s,max} = \frac{0.85\beta_1 f_c' bd}{f_y} \times \left(\frac{0.003}{(.003+.004)}\right)$	
	$= 0.85 \times 0.814 \times 35 \times 300 \times 800/420 \times (0.003/0.007)$	$5930mm^2$
	-Maximum reinforcement percentage	
ACI318-05	$\rho_{max} = 0.85\beta_1 f_c'/f_y(\varepsilon_c/\varepsilon_{c+}0.004)$	
Fig.R10.3.3	- Minimum reinforcement percentage	
	$\rho_{min} = \sqrt{f_c'}/4f_y$	
	-Strain in compression concrete	
ACI10.2.3	$\varepsilon_c = 0.003$	
	$\beta_1 = 0.85\text{-}0.05(f_c\text{-}4000)/1000, \quad 0.65 \leq \beta_1 \leq 0.85.$	
Design of	-Design moment strength	
Reinforced	$\emptyset M_n = \emptyset[A_{S1} f_y(d\text{-}a/2) + A_s' f_s'(d\text{-}d')]$	
Concrete	$a = A_S f_y/0.85 f_c' b$	
ACI318-05	$A_s = \rho bd$	
Seven	$\rho = 1/m(1\text{-}\sqrt{(1\text{-}2mR/f_y)}$	
edition	$R = M_u/\phi bd^2$	
	$m = f_y/0.85f_c'$	
	-Effective depth	
	$d = h\text{-}100$	800mm
	$d' = 100mm$	

Table (3.27): Required Beam Reinforcement at 40th Story Level

Location	M_u(m-kN)	A_s (mm_2)	Reinforcement	ØMn(m-kN)
Support	-145	942	3-No20	260
Mid span	64	942	3-No20	260

ACI21.12.4.1: the positive moment strength at the joint be

Greater than or equal to 33% of the negative moment strength at that location.

This is satisfied, since 260 m-kN >260×0.33m-kN.

Table (3.28): Shear Design Calculations at 40th Story Level

Reference	Calculation	Out Put
	2.Shear Design:	
ACI318-05 Fig.R21.12.3	Shear demand from nominal flexural capacity	
	V_u= (260-260)/4.2	45kN/m
	Shear demand from gravity load	
	$W_u=1.2W_D+W_L=1.2\times35+2$	95kN
	$V_u=W_u l_n/2=45\times4.2/2$	95kN
	V_u=0+95	
ACI318-05 11.3.1	The nominal shear strength provided by concrete (V_c)	
	$V_c=0.17\times\sqrt{f'c}\times b\times d$	
	$=0.17\times\sqrt{35}\times300\times800/1000$	241.4kN
	V_u= ($95KN$)< ØV_c(0.75×241.4)	181kN
	There is no reinforcement for shear.	

Table (3.29): Design of Column C11 at First Story Level

Load Cases	Axial load (kN)	B.M (kNm)	S.F (kN)
Gravity (DL+LL)	9569	-34	0
Wind (WL)	±4140	±665	±186
Load Combination			
1.2DL+1.6LL	11823	-42	0
1.2DL+LL±1.6WL	17937	1104	298

3.4.3 Design of Columns:

All columns are subjected to some bending as well as axial forces, and they need to be proportioned to resist both.

Column design calculations shown in tables (3.30), (3.32) and (3.34)

Table (3.30): Column Axial Forces and Bending Design Calculations at First Story Level

Reference	Calculation	Out Put
Design of Reinforced Concrete ACI318-05 Seven edition	1.Design for Axial Force and Bending Since the design strength not investigated requirement of ACI for using interaction chart, so design using basic equations, considered that, balanced failure. -Basic Equations of Short Columns $P_{nb} = 0.85f_c'ab + A_s'f_s' - A_sf_y$ $M_{nb} = 0.85f_c'ab(h/2-a/2) + A_s'f_s'(h/2-d') - A_sf_y(d-h/2)$ Eccentricity $e_b = M_{nb}/P_{nb} \leq 0.03h+15$ -Nominal Design Strength $P_{nb} = P_u/\emptyset = 17937/0.65$	27595kN

	$M_{nb}= M_u/\emptyset = 1104/0.65$	1699kNm
ACI9.3.2.2	$\emptyset = 0.65$ for column sections with tied reinforcement	
	-Strains in compression and tensile steel	
Design of	$\varepsilon_s' = (\varepsilon_u)c\text{-}d'/c$	
Reinforced	$\varepsilon_s' = (\varepsilon_u)d\text{-}c/c$	
Concrete	$c = c_b = 600d/(600+f_y)$	
ACI318-05	$c = 600\times730/(600 + 420)$	429mm
Seven	-Stress in compression and tensile steel	
edition	$f_s' = E_s\varepsilon_s'$	
	$f_s = E_s\varepsilon_s$	
ACI10.2.7.1	$a = \beta_1 c = 0.814\times429$	349mm
	h = 800mm	
	d′= 70mm	
	d = 800-70	730mm
	b = 800mm	
	by Substituting into basic equations above	
	-Compression reinforcement	
	$A_s'= 16612mm^2$ (Use17-No 36)	
	-tension reinforcement	
	$A_s = 3113mm^2$ (Use4-No 36)	
	$A_{sprovide}= 21378mm^2$	
ACI10.9.1	Design is based on the governing load combinations in the table 3.29, a 800×800mm column with 21-No.36 bars ($\rho_g = 3.5\%$) is adequate for column supporting the first floor level. The provided reinforcement ratio is within the allowable rang of 1% and 8%	
ACI21.12.3	2.Design for Shear	

	Columns in intermediate moment frames must satisfy the shear requirements. The first of the two options in that section is utilized here to determine the design shear strength:	
	The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for the factored gravity loads.	
	Because the column is at first floor, and the moment at any column end cannot exceed the average of the nominal moment strengths of the beams framing into that end, shear demand from the lateral forces is calculated from the nominal flexural strengths of the beams.	
	$V_u = (690+260)/1.8$	528kN >
	The shear capacity of the column will be checked for	298kN
ACI318-05	members subjected to axial compression:	
Eq. (11.4)	$V_c = 0.17(1+N_u/14A_g)\sqrt{f_c'}bd$	
	$0.17(1+(17937000/14\times800^2))\sqrt{35}\times800\times730/1000$	1175.8kN
	Since $V_u > \emptyset V_c/2 = 0.75\times1175.8/2$	441kN
ACI11.5.6.1	by minimum transverse reinforcement would be required.	
	$A_{v,min} = .062\sqrt{f_c'}b_wS/f_{yt}$	
	With No. 12 hoops with one cross-site, $A_v = 387mm^2$	
ACI11.5.5.1	$S = 387\times250/(.062\times800\sqrt{f_c'})$	329.7mm
	< d/2 Thus,	365mm
ACI11.5.5.2	$S_{required}$ =330 mm	
ACI21.12.5.	For No 12 rectangular hoops, the vertical spacing s_0	

3	must not exceed the smallest of the flowing: -8(smallest longitudinal bar diameter) = 8×34.5 -24(hoop bar diameter) = 24×11.5 Use No 12 hoops and crossties @ 250mm with the first hoop located at 120mm < $S_0/2=300/2=150$mm; from the joint face below first floor above the base .	 284mm 300mm

Table (3.31): Design of Column C11 at 20th Story Level

Load Cases	Axial load (kN)	B.M (kNm)	S.F (kN)
Gravity (DL+LL)	4967	-89	0
Wind (WL)	±1191	±142	±89
Load Combination			
1.2DL+1.6LL	6136.4	-110	0
1.2DL+LL±1.6WL	7778	332	142

Table (3.32): Column Axial Forces and Bending Design Calculations at 20th Story Level

Reference	Calculation	Out Put
Design of Reinforced Concrete	1.Design for Axial Force and Bending Since the design strength not investigated requirement of ACI for using interaction chart, so design using basic equations, consider this, balanced failure. **-**Basic Equations of Short Columns $P_{nb} =0.85f_c'ab+A_s'f_s'-A_sf_y$ $M_{nb}= 0.85f_c'ab(h/2-a/2) + A_s'f_s'(h/2-d') - A_sf_y(d-h/2)$	

ACI318-05 Seven edition	Eccentricity $e_b = M_{nb}/P_{nb} \leq 0.03h+15$	
	-Nominal Design Strength	
	$P_{nb} = P_u/\emptyset = 7778/0.65$	11966kN
	$M_{nb} = M_u/\emptyset = 332/0.65$	511kNm
	-Strains in compression and tensile steel	
	$\varepsilon_s' = (\varepsilon_u)c\text{-}d'/c$	
	$\varepsilon_s' = (\varepsilon_u)d\text{-}c/c$	
	$c = c_b = 600d/(600+f_y)$	
	$c = 429mm$	
	-Stress in compression and tensile steel	
	$f_s' = E_s\varepsilon_s'$	
	$f_s = E_s\varepsilon_s$	
ACI10.2.7.1	$a = \beta_1 c = 0.814\times 429$	349mm
	$h = 800mm$	
	$d' = 70mm$	
	$d = 800\text{-}70$	730mm
	$b = 800mm$	
	Substituting into the basic equations above	
	-Compression reinforcement	
	$A_s' = 9918mm^2$ (Use13-No. 32)	
	-tension reinforcement	
	$A_s = 9859mm^2$ (Use12-No. 32)	
	$A_{sprovide} = 20100mm^2$	
ACI10.9.1	Design is based on the governing load combinations in the table 3.31, a 800×800mm column with 25-No.32 bars ($\rho_g = 3\%$) is adequate for column supporting the twenty floor level. The provided reinforcement ratio is within the allowable rang of	

	1% and 8%	
	2.Design for Shear	
ACI21.12.3	Columns in intermediate moment frames must satisfy the shear requirements in. The first of the two options in that section is utilized here to determine the design shear strength:	
	The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for the factored gravity loads.	
	Because the column is at twenty floor, and the moment at any column end cannot exceed the average of the nominal moment strengths of the beams framing into that end, shear demand from the lateral forces is calculated from the nominal flexural strengths of the beams.	
	$V_u = (560+260)/1.6$	512.5kN >
	The shear capacity of the column for members	142kN
ACI318-05	subjected to axial compression:	
Eq. (11.4)	$V_c = 0.17(1+N_u/14A_g)\sqrt{f_c'}b_wd$	
	$0.17(1+(7778000/14\times800^2))\sqrt{35}\times800\times730/1000$	1097kN
	Since $V_u > ØV_c/2 = 0.75\times1097/2$	411kN
ACI11.5.6.1	minimum transverse reinforcement would be required.	
	$A_{v,min} = .062\sqrt{f_c'}b_wS/f_{yt}$	
	With No. 10 hoops with one cross-site, $A_v = 213mm^2$	
ACI318-05	$S = 213\times250/(.062\times800\sqrt{35})$	181.5mm
11.5.5.1	$< d/2 = 730/2$	365mm

ACI11.5.5.2 ACI21.12.5.3	$S_{required}$ =181.5 mm For No 12 rectangular hoops, the vertical spacing s_0 must not exceed the smallest of the flowing: -8(smallest longitudinal bar diameter) = 8×31.5 -24(hoop bar diameter) = 24×9.5 Use No -10 hoops and crossties @ 200mm with the first hoop located at 100mm (< $S_0/2$=228/2=114mm; from the joint face below twenty floor level and above the twenty-one level.	 252mm 228mm

Table (3.33): Design of Column C11 at 40th Story Level

Load Cases	Axial load (kN)	B.M (kNm)	S.F (kN)
Gravity (DL+LL)	130	-94	0
Wind (WL)	±2	±5	±3.1
Load Combination			
1.2DL+1.6LL	160	-117	0
1.2DL+LL±1.6WL	178	112	5

Table (3.34): Column Axial Forces and Bending Design Calculations at 40th Story Level

Reference	Calculation	Out Put
	1.Design for Axial Force and Bending Since the design strength not investigated requirement of ACI for using interaction chart, so design using basic equations, consider this,	

	balanced failure.	
Design of Reinforced Concrete ACI318-05 Seven edition	-Basic Equations of Short Columns	
	$P_{nb} = 0.85f_c'ab + A_s'f_s' - A_sf_y$	
	$M_{nb} = 0.85f_c'ab(h/2-a/2) + A_s'f_s'(h/2-d') - A_sf_y(d-h/2)$	
	Eccentricity $e_b = M_{nb}/P_{nb} \leq 0.03h+15$	
	-Nominal Design Strength	
ACI9.3.2.2	$P_{nb} = P_u/\emptyset = 178/0.65$	274kN
	$M_{nb} = M_u/\emptyset = 112/0.65$	173kN
Design of Reinforced Concrete ACI318-05 Seven edition	-Strains in compression and tensile steel	
	$\varepsilon_s' = (\varepsilon_u)c-d'/c$	
	$\varepsilon_s' = (\varepsilon_u)d-c/c$	
	$c = c_b = 600d/(600+f_y)$	
	$c = 429mm$	
	-Stress in compression and tensile steel	
	$f_s' = E_s\varepsilon_s'$	
	$f_s = E_s\varepsilon_s$	
ACI10.2.7.1	$a = \beta_1 c = 0.814\times429$	349mm
	$h = 800mm$	
	$d' = 70mm$	
	$d = 800-70$	730mm
	$b = 800mm$	
	Substituting into the basic equations above	
	-Compression reinforcement	
	$A_s' = 6400mm^2$ (Use13-No 25)	
	-tension reinforcement	
	$A_s = 4056mm^2$	
	Design is based on the governing load combinations in the table, a 800×800mm column	

ACI10.9.1	with 13-No.25 bars (ρ_g = 1%) is adequate for column supporting the first floor level. The provided reinforcement ratio is within the allowable rang of 1% and 8%	
	2.Design for Shear	
ACI21.12.3	Columns in intermediate moment frames must satisfy the shear requirements. The first of the two options in that section is utilized here to determine the design shear strength:	
	The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for the factored gravity loads.	
	Because the column is at roof, and the moment at any column end cannot exceed the average of the nominal moment strengths of the beams framing into that end, shear demand from the lateral forces is calculated from the nominal flexural strengths of the beams.	
	$V_u = (260+260)/1.6$	325kN > 5kN
	The shear capacity of the column w for members subjected to axial compression:	
ACI318-05 Eq. (11.4)	$V_c = 0.17(1+N_u/14A_g)\sqrt{f_c'}b_w d$	
	$0.17(1+(178000/14\times800^2))\sqrt{35}\times800\times730/1000$	599kN
	Since $V_u > \emptyset V_c/2 = 0.75\times599/2$	224.6kN
	minimum transverse reinforcement would be required.	
ACI11.5.6.1	$A_{v,min} = .062\sqrt{f_c'}b_w S/f_{yt}$	

ACI 11.5.5.1 ACI11.5.5.2 ACI21.12.5.3	With No. 10 hoops with one cross-site, A_v = 213mm^2 S = 213×250/(.062×800√35) < d/2 Thus, $S_{required}$ =181.5 mm For No 12 rectangular hoops, the vertical spacing s_0 must not exceed the smallest of the flowing: -8(smallest longitudinal bar diameter) = 8×24.5 -24(hoop bar diameter) = 24×9.5 Use No 12 hoops and crossties @ 200mm with the first hoop located at 90mm (< S_0/2=196/2=98mm; from the joint face below forty floor level.	181.5mm 365mm 196mm 228mm

Table (3.35): Design Axial Forces, Bending Moments and Shear Forces at Base of Shear Wall on Line3

Load Cases	Axial (kN)	Bending (kNm)	Shear (kN)
Dead	8810	0	0
Live	1296	0	0
Wind	0	58310	858
Load Combination			
1.2DL+1.6LL	11417	0	0
1.2DL+LL±1.6WL	11868	93296	1373

3.5.4 Design of Shear walls:

Walls should be designed to resist all loads to which they are subjected, including eccentric axial loads and lateral forces.

Shear wall design calculations shown in table (3.36).

Table (3.36): Wall Shear Design Calculations

Reference	Calculation	Out Put
	1.Check wall thickness	
ACI11.10.3	$V_u = \emptyset 10\sqrt{f'_c}hd$	
ACI11.10.4	$D = 0.8l_w = 0.8\times1000$	800mm
	$V_u = 0.75\times10\sqrt{35000}\times0.3\times0.8$	373 kN
ACI11.30	Minimum value of V_c	
	$V_c= hd(0.6\sqrt{f'_c}+l_w(1.25\sqrt{f'_c}+0.2N_u/l_wh)/(M_u/V_u-l_w/2)$	
	$0.3\times0.8(0.6\sqrt{35000}+(1.25\sqrt{35000}+0.2\times11868/0.3)/(93296/337-1/2)$	30
ACI11.10.5	2.Shear Design	
	The shear strength of the concrete for wall subjected to axial compression,	
	$V_c = 0.17\sqrt{f_c'}hd$	
	$=0.17\sqrt{35}\times300\times(0.8\times1000)/1000$	241.4kN
	Since $\emptyset V_c = 0.75\times241.4<V_u(=1373)$,	181kN
	horizontal shear reinforcement	
ACI11.10.9	Required bar spacing with 2 layers of No. 16:	
	$S = (A_vf_yd)/V_s$	
	$= (398\times420\times800)/(1373000/0.75- 241400)$	85mm
ACI11.10.9.3	spacing of horizontal reinforcement shall not exceed:	
	$-l_w/5 =1000/5$	200mm
	$-3h = 3\times300$	900mm

ACI11.10.9.2	-450mm ratio of horizontal shear reinforcement shall not be less than 0.0025 For 2-No.12 horizontal bar spaced @ 200mm	
ACI11.10.3	$\rho_t = 398/(300\times200)$ Shear strength V_n at any horizontal section must be less than or equal to $0.83(f_c')^{0.5}hd$ (=1178.5KN). In this case, $V_n = V_c + V_s$	0.0066 > 0.002
ACI11.10.9.4	$= 241.4 + (398\times420\times800)/300 = 687.2$ The ratio of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than 0.0025	687.2kN< 1178.5kN
ACI318 Eq. (11-32) ACI11.10.9.5	$\rho_l = 0.0025 + 0.5(2.5 - h_w/l_w)(\rho_t - 0.0025)$ $= 0.0025 + 0.5(2.5 - 128.8/1)(0.0066 - 0.0025)$ spacing of vertical shear Reinforcement shall not exceed(1) $l_w/3 = 1000/3$ For 2 - No. 16 vertical bar spaced at 300mm, $\rho_l = 398/(300\times300) =$ Use 2-No. 16 vertical bars @ 300mm	0.25 > 0.0025 333.3mm 0.0044 > 0.002
ACI14.3.2 and 14.3.2	The provided vertical and horizontal reinforcement satisfy the requirements of sections for minimum ratio of vertical and horizontal reinforcement to gross concrete area. -Design vertical flexural reinforcing:	
ACI318 Eq. (11-32) ACI11.10.9.5	$M_u/\text{Ø}bd^2 = 93296\times10^6/0.9\times300\times800^2$ $\rho = \rho_{max} = 0.0247$ $A_s = 0.0247\times300\times800$	540 5930mm^2