7.2 Design of minaret:
Geometry:
The figure below shows the longitudinal section in the minaret and the cross section will be shown when start calculate the self-weight of each section.

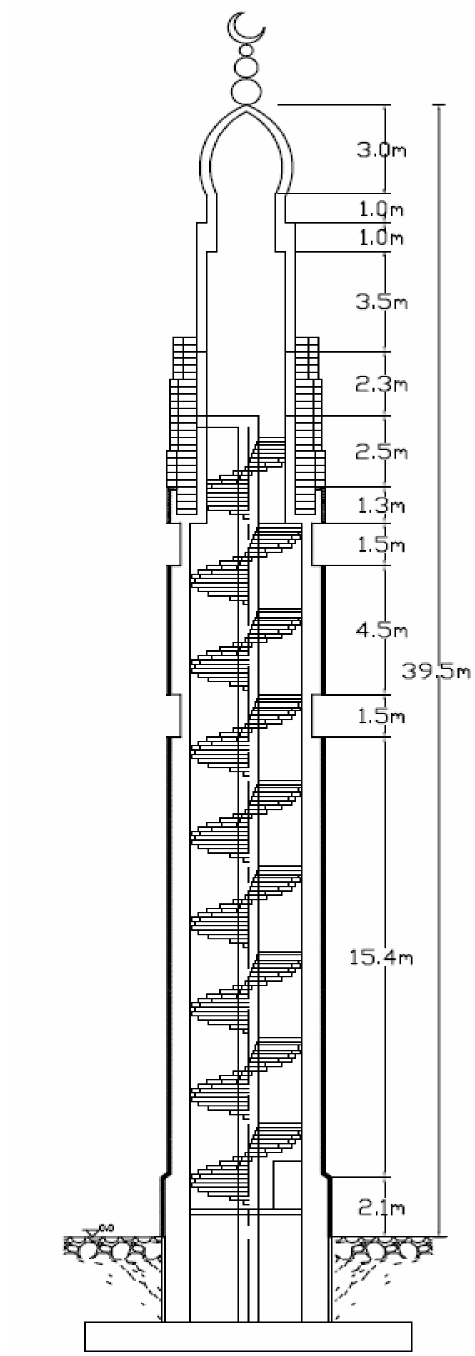

Figure 7.2.1
Longitudinal section in minaret

Analysis and Design:

We should calculate the base shear from the wind and the earthquake load respectively
And decide about the load combination which will be used in analysis and design procedure.

Calculation of the base shear from the wind load:

1- Calculate design wind pressure According to UBC97 Code. As following:

$P = C_e \times C_q \times q_s \times I_w$

$A)\ Basic\ wind\ speed = 1.61x90 = 144.9mph\ From\ table\ A-1(Appendix)$

$q_s = 0.0479x20.8 = 0.996 \approx 1KN/m^2\ From\ table\ A-1(Appendix)$

B) For chimneys and solid tower :

$\mathrm{C_q} = 0.8$ *for any direction. From table* $A-5(Appendix)$

D) For chimneys and solid tower $\mathrm{I_w} = 1$ *From table* $A-6(Appendix)$

E) Choose Exposure C for Mutah University Zone from table A-2(Appendix).

Height above average level of adjoining ground(m)	C_e for EXPOSURE C	q(KN/m^2)
0.0	1.06	0.848
1.0	1.06	0.848
2.0	1.06	0.848
3.0	1.06	0.848
4.0	1.06	0.848
4.57	1.06	0.848
6.09	1.13	0.904
7.62	1.19	0.952
9.14	1.23	0.984
12.19	1.31	1.048
18.29	1.43	1.144
24.38	1.53	1.224
30.48	1.61	1.288
36.58	1.67	1.336

Table 7.2.1
Design wind pressure for exposure C

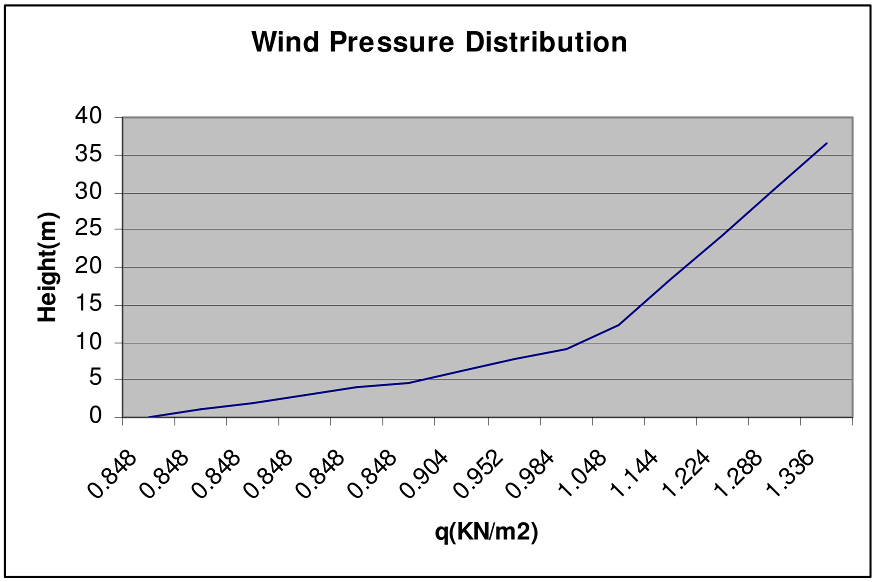

Figure 7.2.2
Design wind pressure for exposure C

2- Distribute the wind pressure for range of height as following:

Height Range(m)	Outer Diameter(m)	Pressure (KN/m^2)	Distribution force (KN/m')
0 - 2.1	4.8	0.848	4.1
2.1 - 17.5	4.6	1.144*	5.26
17.5 - 19	4.0	1.224	4.5
19 – 23.5	4.6	1.224	5.63
23.5 - 25	4.0	1.288	5.15
25 – 28.75	3.0	1.288	3.86
28.75 - 31	3.0	1.336	4
31 – 34.5	3.0	1.336	4
34.5 - 35.5	3.0	1.336	4

*Also use this for the elevated water tank (from 12 - 16) m

Table 7.2.2
Design wind pressure distribution over the range of height

3- Calculation of the Base shear:

$$V = 4.1 \times (2.1 - 0) + 5.26 \times (17.5 - 2.1) + 4.5 \times (19 - 17.5) + 5.63 \times (23.5 - 19) + 5.15 \times (25 - 23.5) + 3.86 \times (28.75 - 25) + 4 \times (35.5 - 28.75)$$

$$=170.9KN$$

After calculating the earthquake base we should decide about the load combination which will be used in analysis and design procedure, because according to UBC97 criteria, we should use the greatest of wind or earthquake load.

Calculation of the base shear from earthquake load:

1- Calculate the weight of the structure:

The minaret will be will be divided to more than one section according to the varying in the section area at each elevation as follow:

Assume the footing is at elevation -3 below the ground level.

Section No.1

Elevation from -3m to 2.1m

From the architectural drawing thickness of the stone facing = 10 cm.

$$W_1 = W_{concrete} + W_{stone} = \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{concrete} \times \gamma_{concrete} + \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{stone} \times \gamma_{stone}$$

$$= \left(\frac{\pi}{4}\right) \times \left[4.8^2 - 3.4^2\right] \times 25 + \left(\frac{\pi}{4}\right) \times \left[5^2 - 4.8^2\right] \times 23 = 266.88 KN/m'$$

$$I = \frac{\pi}{64} \times \left(D_{out}^4 - D_{in}^4\right) = \frac{\pi}{64} \times \left(4.8^4 - 3.4^4\right) = 19.5 m^4$$

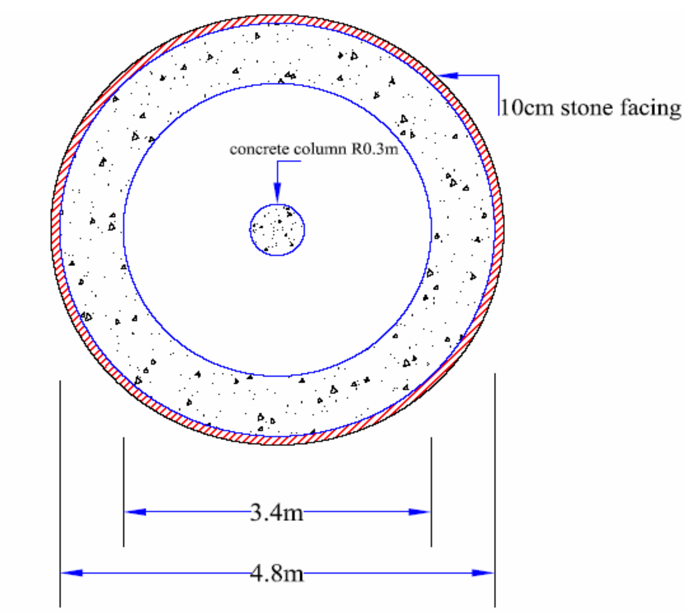

Figure 7.2.3
Section No.1

Section No.2

Elevation from 2.1m to 17.5m and from 19m to 23.5m

From the architectural drawing thickness of the stone facing = 10 cm.

$$W_1 = W_{concrete} + W_{stone} = \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{concrete} \times \gamma_{concrete} + \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{stone} \times \gamma_{concrete}$$

$$= \left(\frac{\pi}{4}\right) \times \left[4.6^2 - 3.4^2\right] \times 25 + \left(\frac{\pi}{4}\right) \times \left[4.8^2 - 4.6^2\right] \times 23 = 222.46 KN/m'$$

$$I = \frac{\pi}{64} \times \left(D_{out}^4 - D_{in}^4\right) = \frac{\pi}{64} \times \left(4.6^4 - 3.4^4\right) = 15.42 m^4$$

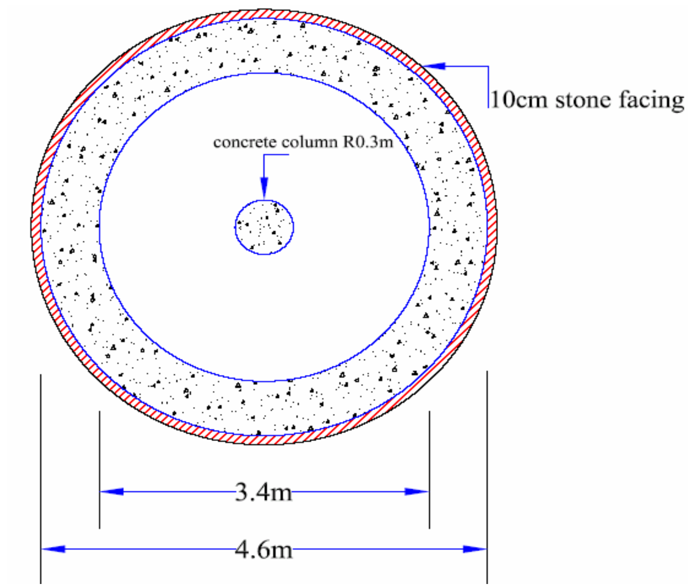

Figure 7.2.4
Section No.2

Section No.3

Elevation from 17.5m to 19m and from 23.5m to 25m

$$W_1 = W_{concrete} = \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{concrete} \times \gamma_{concrete}$$

$$= \left(\frac{\pi}{4}\right) \times \left[4.0^2 - 3.4^2\right] \times 25 = 87.18 KN/m'$$

$$I = \frac{\pi}{64} \times \left(D_{out}^4 - D_{in}^4\right) = \frac{\pi}{64} \times \left(4.0^4 - 3.4^4\right) = 6 m^4$$

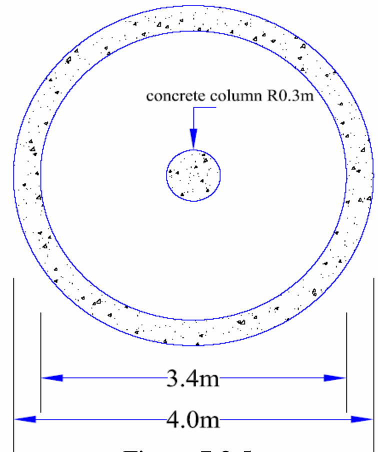

Figure 7.2.5
Section No.3

Section No.4

Elevation from 25m to 28.75m

There is no stone facing but there are stone wall a round the concrete section with more than one diameter.

Use the average diameter of the stone wall and calculate the weight of the wall

$$W_1 = W_{concrete} + W_{stone} = \left(\frac{\pi}{4}\right)\times\left[D_{out}^2 - D_{in}^2\right]_{concrete}\times\gamma_{concrete} + \left(\frac{\pi}{4}\right)\times\left[D_{out}^2 - D_{in}^2\right]_{stone}\times\gamma_{concrete}$$

$$= \left(\frac{\pi}{4}\right)\times\left[3.0^2 - 2.4^2\right]\times 25 + \left(\frac{\pi}{4}\right)\times\left[4.6^2 - 4^2\right]\times 23 = 156.83KN/m'$$

$$I = \frac{\pi}{64}\times\left(D_{out}^4 - D_{in}^4\right) = \frac{\pi}{64}\times\left(3.0^4 - 2.4^4\right) = 2.35m^4$$

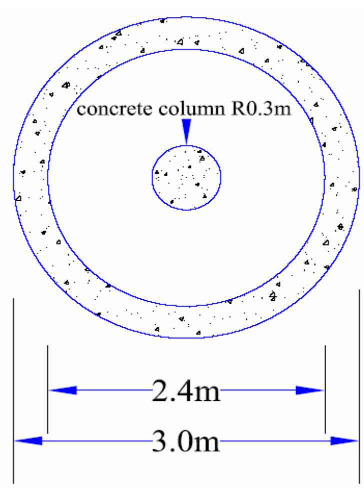

Figure 7.2.6
Section No.4

Section No.5

Elevation from 28.75m to 31m
There is an empty space in this elevation because there are four windows with height equal to 2.25m.

$W_1 = W_{total} - W_{empty\ space}$

$$W_{total} = \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{concrete} \times \gamma_{concrete} = \left(\frac{\pi}{4}\right) \times \left[3^2 - 2.4^2\right] \times 25 = 63.62 KN/m'$$

$$W_{empty\ space} = \left[\left(\frac{1}{2}\right) \times r_{out} \times L_{out} - \left(\frac{1}{2}\right) \times r_{in} \times L_{in}\right] \times \gamma_{concrete}$$

$$L_{out} = r_{out} \times \theta = 1.5 \times 15 \times \left(\frac{\pi}{180}\right) = 0.4m$$

$$L_{in} = r_{in} \times \theta = 1.2 \times 15 \times \left(\frac{\pi}{180}\right) = 0.3m$$

$$W_{empty\ space} = \left[\left(\frac{1}{2}\right) \times 1.5 \times 0.4 - \left(\frac{1}{2}\right) \times 1.2 \times 0.3\right] \times 25 = 3 KN/m'$$

$$W_1 = W_{total} - W_{empty\ space} = 63.62 - 4 \times 3 = 51.62 KN/m'$$

$$I = \frac{\pi}{64} \times \left(D_{out}^4 - D_{in}^4\right) = \frac{\pi}{64} \times \left(3.0^4 - 2.4^4\right) = 2.35 m^4$$

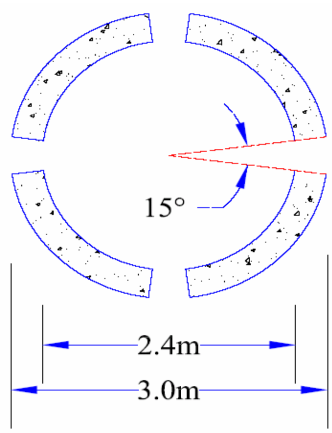

Figure 7.2.7
Section No.5

Section No.6

Elevation from 31m to 34.5m

$$W_1 = W_{concrete} = \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{concrete} \times \gamma_{concrete}$$

$$= \left(\frac{\pi}{4}\right) \times \left[3^2 - 2.4^2\right] \times 25 = 63.62 KN / m'$$

$$I = \frac{\pi}{64} \times \left(D_{out}^4 - D_{in}^4\right) = \frac{\pi}{64} \times \left(3.0^4 - 2.4^4\right) = 2.35 m^4$$

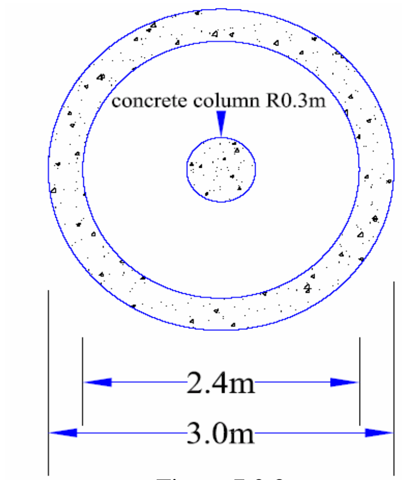

Figure 7.2.8
Section No.6

Section No.7

Elevation from 34.5m to 35.5m

$$W_1 = W_{concrete} = \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right]_{concrete} \times \gamma_{concrete}$$

$$= \left(\frac{\pi}{4}\right) \times \left[3.0^2 - 1.8^2\right] \times 25 = 113.1 KN / m'$$

$$I = \frac{\pi}{64} \times \left(D_{out}^4 - D_{in}^4\right) = \frac{\pi}{64} \times \left(3.0^4 - 1.8^4\right) = 3.46 m^4$$

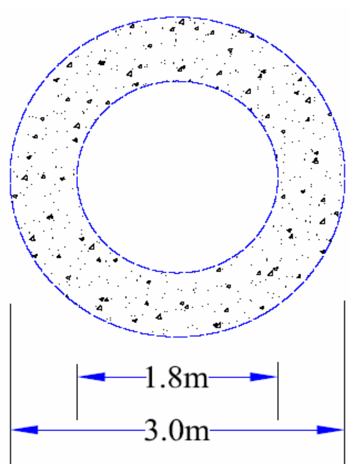

Figure 7.2.9
Section No.7

Total gravity loads:

$$W_{gravity} = 260.82\times(2.1-(-3))+222.46\times(17.5-2.1)+222.46\times(23.5-19)$$
$$+87.18\times(19-17.5)+87.18\times(25-23.5)+156.83\times(28.75-25)$$
$$+51.62\times(31-28.75)+63.62\times(34.5-31)$$
$$+113.1\times(35.5-34.5)$$
$$=7058.7KN$$

2- Quake force parameters:

a) From Jordan seismic hazard map, Karak is on the 2B zone and for this zone get Seismic zone factor Z = 0.2 from table (A-3) (Appendix)

b) There is no available information about the soil, so use S_D soil profile type from table (A-4) (Appendix).

c) The important factor I = 1 from table (A-6) (Appendix).

e) Response modification factor R: for Cast-in-place concrete silos and chimneys having walls continuous to the foundations R = 3.6 from table (A- 7) (Appendix).

f) Seismic coefficient Ca and C_v: based on the soil profile type and the seismic zone factor C_a = 0.28 and C_v = 0.4 from table and table (A- 8) and (A-9) (Appendix).

g) Fundamental period T: for the non-building structure (self supporting structure) like chimney, silo, minaret, we should use method 1 as an initial fundamental period

The initial fundamental period $T = C_t(h_n)^{3/4} = 0.0488\times(35.5)^{3/4} = 0.71 second$

3- Quake force initial base shear:

Based on the initial fundamental period T = 0.75second

$$V = \left(\frac{C_v I}{RT}\right) \times W = \left(\frac{0.4 \times 1}{3.6 \times 0.71}\right) \times 7058.7 = 1104.65KN$$

$$V_{\min} = (0.11 C_a I) \times W = (0.11 \times 0.28 \times 1) \times 7058.7 = 217.4KN$$

$$V_{\max} = \left(\frac{2.5 C_a I}{R}\right) \times W = \left(\frac{2.5 \times 0.28 \times 1}{3.6}\right) \times 7058.7 = 1372.53KN$$

Extra load $F_t = 0.07\, T\, V = 0.07 \times 0.71 \times 1104.65 = 54.9KN$

It should be less than 0.25 V = 0.25 x 1045.73 = 261.4KN

This base initial base shear (V) will be initially distributed over the height as shown in figure below:

according to the following equation:

$$F_x = \frac{(V - F_t)\, w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

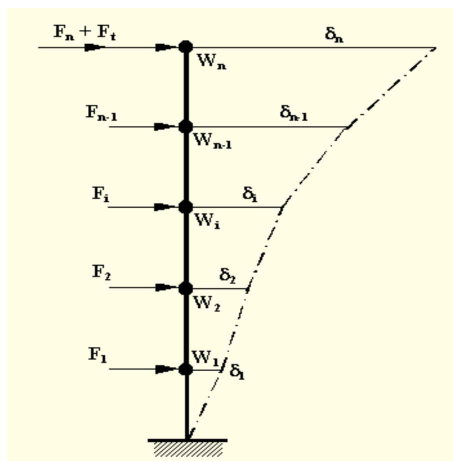

Figure 7.2.10
Lateral force distributed

4- Then calculate the exact fundamental period using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis (method 2).

$$T = 2\pi \sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^{\,2}\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)}$$

We prepare a computer program to calculate the exact fundamental period T that needs trail and error. From the computer program we get T = 0.5 second.

And the table as in the next page.

Mass No.	Weight (KN/m')	Wi(KN)	Accumulative Wi(KN)	I(m^4)	Elevation(m)	Hi(m)	Wi*Hi	Fi(KN)
0	0.000	130.44	7051.786	19.500	-3	0.000	0.000	0.000
1	266.88	266.88	6921.346	19.500	-2	1.000	266.880	3.702
2	266.88	266.88	6654.466	19.500	-1	2.000	533.760	7.403
3	266.88	266.88	6387.586	19.500	0	3.000	800.640	11.105
4	266.88	266.88	6120.706	19.500	1	4.000	1067.520	14.806
5	266.88	146.784	5853.826	19.500	2	5.000	733.920	10.179
6	266.88	113.451	5707.042	19.500	2.1	5.100	578.600	8.189
7	222.460	211.337	5593.591	15.420	3	6.000	1268.022	17.996
8	222.460	222.460	5382.254	15.420	4	7.000	1557.220	22.100
9	222.460	222.460	5159.794	15.420	5	8.000	1779.680	25.257
10	222.460	222.460	4937.334	15.420	6	9.000	2002.140	28.414
11	222.460	222.460	4714.874	15.420	7	10.000	2224.600	31.571
12	222.460	222.460	4492.414	15.420	8	11.000	2447.060	34.728
13	222.460	222.460	4269.954	15.420	9	12.000	2669.520	37.885
14	222.460	222.460	4047.494	15.420	10	13.000	2891.980	41.043
15	222.460	222.460	3825.034	15.420	11	14.000	3114.440	44.200
16	222.460	222.460	3602.574	15.420	12	15.000	3336.900	47.357
17	222.460	222.460	3380.114	15.420	13	16.000	3559.360	50.514
18	222.460	222.460	3157.654	15.420	14	17.000	3781.820	53.671
19	222.460	222.460	2935.194	15.420	15	18.000	4004.280	56.828
20	222.460	222.460	2712.734	15.420	16	19.000	4226.740	59.985
21	222.460	166.845	2490.274	15.420	17	20.000	3336.900	47.357
22	222.460	77.410	2323.429	15.420	17.5	20.500	1586.905	22.521
23	87.180	65.385	2246.019	15.420	18	21.000	1373.085	19.487
24	87.180	154.82	2180.634	15.420	19	22.000	3406.040	48.366
25	222.640	222.46	2025.814	15.420	20	23.000	5116.580	72.643
26	222.460	222.460	1803.354	15.420	21	24.000	5339.040	75.771
27	222.460	222.460	1580.894	15.420	22	25.000	5561.500	78.928
28	222.460	166.845	1358.434	15.420	23	26.000	4337.970	61.564
29	222.460	77.410	1191.589	15.420	23.5	26.500	2051.365	29.113
30	87.180	65.385	1114.179	15.420	24	27.000	1765.395	25.054
31	87.180	65.385	1048.794	15.420	25	28.000	1830.780	25.982
32	87.180	61.003	983.409	15.420	25.5	28.500	1738.586	24.674
33	156.830	117.623	922.406	15.420	26	29.000	3411.067	48.409
34	156.830	156.830	804.783	15.420	27	30.000	4704.900	66.771
35	156.830	137.226	647.953	15.420	28	31.000	4254.006	60.372
36	156.830	65.264	510.727	15.420	28.75	31.750	2072.132	29.407
37	51.620	32.263	445.463	15.420	29	32.000	1032.416	14.652
38	51.620	51.620	413.200	15.420	30	33.000	1703.460	24.175
39	51.620	57.620	361.580	15.420	31	34.000	1959.080	27.803
40	63.620	63.620	303.960	15.420	32	35.000	2226.700	31.601
41	63.620	63.620	240.340	15.420	33	36.000	2290.320	32.504
42	63.620	47.715	176.720	15.420	34.000	37.000	1765.455	25.055
43	63.620	44.180	129.005	15.420	34.500	37.500	1656.750	23.512
44	113.100	56.550	84.825	15.420	35.000	38.000	2148.900	30.497
45	113.100	28.275	28.275	15.420	35.500	38.500	1088.588	15.449
						Sum	**110603.0**	

Table 7.2.3 (lateral force distribution)

Hi (m)	Fx (KN)	Accumulative Fx (KN)	Moment (KN.m)	Deflection (mm)
0.00	0.00	1371.76	30510.16	0.00
1.00	3.31	1371.76	29138.40	0.11
2.00	6.62	1368.45	27769.94	0.28
3.00	9.93	1361.83	26408.11	0.53
4.00	13.24	1351.90	25056.20	0.83
5.00	9.10	1338.66	23717.54	1.19
5.10	7.18	1329.56	23584.58	1.22
6.00	15.73	1322.38	22394.44	1.61
7.00	19.31	1306.66	21087.78	2.11
8.00	22.07	1287.34	19800.44	2.67
9.00	24.83	1265.27	18535.16	3.29
10.00	27.59	1240.44	17294.72	3.96
11.00	30.35	1212.85	16081.87	4.68
12.00	33.11	1182.50	14899.37	5.45
13.00	35.87	1149.39	13749.98	6.26
14.00	38.63	1113.52	12636.46	7.11
15.00	41.39	1074.90	11561.57	8.00
16.00	44.15	1033.51	10528.06	8.91
17.00	46.90	989.36	9538.69	9.86
18.00	49.66	942.46	5896.23	10.83
19.00	52.42	892.80	7703.44	11.83
20.00	41.39	840.37	6863.06	12.85
20.50	19.68	798.99	6463.57	13.36
21.00	17.03	779.31	6073.92	13.89
22.00	42.24	762.28	5311.64	15.00
23.00	63.46	720.03	4591.61	16.11
24.00	66.22	656.57	3935.04	17.23
25.00	68.98	590.36	3344.68	18.36
26.00	53.80	521.38	2823.30	19.51
25.50	25.44	467.58	2589.52	20.08
27.00	21.90	442.13	2368.45	20.66
28.00	22.71	420.24	1948.21	21.83
28.50	21.56	397.53	1749.44	22.42
29.00	42.31	375.97	1561.46	23.02
30.00	58.35	333.66	1227.80	24.26
31.00	52.76	275.31	952.49	25.51
31.75	25.70	222.55	785.57	26.46
32.00	12.80	196.85	736.36	26.77
33.00	21.13	184.05	552.32	28.05
34.00	24.30	162.92	389.40	29.34
35.00	27.62	138.62	250.78	30.63
36.00	28.41	111.00	139.78	31.19
37.00	21.90	82.60	57.18	33.22
37.50	20.55	60.70	26.83	33.87
38.00	26.65	40.15	6.75	34.51
38.50	13.50	13.50	0.00	35.16

Table 7.2.3 (cont.)

From the table in previous page the accumulative Fi = 1371.76KN
Compare to the wind load base shear = 170.9KN the quake force will be used in analysis and design procedure.

Compute the safety factor against overturning:

Over turning moment = 30510.16KN.m
Restoring moment = restoring force x half the base width
Self weight of the minaret (total gravity load) = 7058.7KN
Assume footing dimension = 10 x 10 x 1
Weight of the soil surround by the minaret

$$W = \left(\frac{\pi}{4}\right) \times \left[D_{out}^2 - D_{in}^2\right] \times 18 \times 3 = \left(\frac{\pi}{4}\right) \times \left[4.8^2 - 3.4^2\right] \times 18 \times 3 = 486.88KN$$

Self Weight of column = (0.6^2 x π) / 4 x 25 x 28.75 = 203.22KN
Total restoring weight =7058.7 + 10 x 10 x 1 x 25 + 468.88 + 203.22 = 10230.8KN
Restoring moment = 10230.8 x 5 = 51154KN.m

$$Factor\ of\ \ safety\ against\ overtruning = \frac{\mathrm{Re}\,storing\ moment}{overturning\ moment} = \frac{51154}{30510.16} = 1.7 > 1.5 \quad ok.$$

Check the allowable deflection:

From the above table the maximum deflection equal to 35.16mm
The allowable deflection equal to $0.0025 \times h = 0.0025 \times 35.5 \times 1000 = 88.75mm$
So it is ok.

Design of minaret sections:

From the table 7.2.3 we got the moment at each elevation and we will check the stresses and calculate the reinforcement required for each section as follow:

Note:

$$\text{Allowable tensile strength of concrete } (f_t) = 0.5\sqrt{f_c^{'}} = 0.5\sqrt{25} = 2.5MPa$$

$$\text{Allowable compressive strength of concrete } (f_c) = 0.45 f_c^{'} = 0.45\times 25 = 11.25MPa$$

$$\text{Allowable shear strength of concrete } (V) \approx 0.09\sqrt{f_c^{'}} = 0.09\sqrt{25} = 0.45MPa$$

Elevation from -3m to 2.1m

Vertical reinforcement:

Maximum Moment (M) = 30510.16KN.m. at elevation -3 below the ground.

Weight above elevation -3 (W) = 7058.7KN.

Eccentricity (e) = M / W = 3540.48 / 7058.7 = 4.3m

$$\sigma = \frac{W}{A} \pm \frac{M\times c}{I} = \frac{7058.7}{8} \pm \frac{30510.16\times 2.4}{19.5} = 891.6 \pm 3755.1$$

$$\sigma_{max} = 4646.7KN/m^2 = 4.65MPa < 0.45\times f_c^{'} \rightarrow it\ is\ ok.$$

$$\sigma_{min} = -2863.5KN/m^2 = -2.86MPa\ \text{"Tension stress"}$$

The tensile stress is slightly greater than the allowable tensile strength of concrete
And the minimum reinforcement will be ok.
Area of steel /m = 0.0015 A_g = 0.0015 x (1000 x 700) = 1050 mm^2
The steel will be arranged into two layers
Steel for one layer = 1050 / 2 = 525mm^2
Choose Ø14

$$spacing = \frac{154}{525}\times 1000 = 293.3mm$$

Provide Ø14@ 250mm

Horizontal reinforcement:

$$\text{Applied shear stress} = \frac{V}{A} = \frac{1371.76}{8} = 171.47KN/m^2 = 0.171MPa$$

This is smaller than the shear strength of concrete.
So minimum horizontal reinforcement will be provided
Area of steel /m = 0.002 A_g = 0.002 x (1000 x 700) = 1400 mm^2
The steel will be arranged into two layers
Steel for one layer = 1400 / 2 = 700mm^2
Choose Ø14

$$spacing = \frac{154}{700}\times 1000 = 220mm$$

Provide Ø14@ 220mm

Elevation from 2.1m to 17.5 m

Maximum Moment (M) = 23584.58KN.m. at elevation 2.1m

Weight above elevation 2.1 (W) = 5728.52KN.

Eccentricity (e) = M / W = 23584.58 / 5728.52 = 4.1 m

$$\sigma = \frac{W}{A} \pm \frac{M \times c}{I} = \frac{5728.52}{7.54} \pm \frac{23584.58 \times 2.3}{15.42} = 759.75 \pm 3517.8$$

$$\sigma_{max} = 4277.55 KN/m^2 = 4.28 MPa < 0.45 \times f_c' \rightarrow it\ is\ ok.$$

$$\sigma_{min} = -2758.1 KN/m^2 = -2.76 MPa \text{ "Tension stress"}$$

The tensile stress is slightly greater than the allowable tensile strength of concrete
And the minimum reinforcement will be ok.
Area of steel /m = 0.0015 A_g = 0.0015 x (1000 x 600) = 900 mm^2
Choose Ø14

$$\text{spacing} = \frac{154}{900} \times 1000 = 171.11 mm$$

Provide Ø14@ 150mm at the center of the section

Horizontal reinforcement:

$$\text{Applied shear stress} = \frac{V}{A} = \frac{1329.65}{7.54} = 176.3 KN/m^2 = 0.176 MPa$$

This is smaller than the shear stress of concrete.
So minimum horizontal reinforcement will be provided
Area of steel /m = 0.002 A_g = 0.002 x (1000 x 600) = 1200 mm^2
Choose Ø16

$$\text{spacing} = \frac{200}{1200} \times 1000 = 166.6 mm$$

Provide Ø16@ 160mm

Note:

Use these horizontal Reinforcements for all other sections.

Elevation from 19m to 23.5 m

Maximum Moment (M) = 5311.64KN.m. at elevation 19m

Weight above elevation (W) = 2176KN.

Eccentricity (e) = M / W = 5311.64 / 2176 = 2.44 m

$$\sigma = \frac{\mathrm{W}}{\mathrm{A}} \pm \frac{M \times c}{I} = \frac{2176}{7.54} \pm \frac{5311.64 \times 2.3}{15.42} = 288.6 \pm 792.27$$

$$\sigma_{\max} = 1080.9 KN/m^2 = 1.1 MPa < 0.45 \times f_c^{'} \rightarrow it\ is\ ok.$$

$$\sigma_{\min} = -503.67 KN/m^2 = -0.504 MPa \text{ "}Tension\ stress\text{"}$$

There is a Tension stress and it is less than the allowable tensile strength of concrete
So minimum reinforcement will be provided.
Area of steel /m = 0.0015 A_g = 0.0015 x (1000 x 600) = 900 mm^2
Choose Ø14

$$\text{spacing} = \frac{154}{900} \times 1000 = 171.11 mm$$

Provide Ø14@ 150mm at the center of the section

Elevation from 17.5m to 19 m

Maximum Moment (M) = 6463.57KN.m. at elevation 17.5m

Weight above elevation 17.5 (W) = 2328.72KN.

Eccentricity (e) = M / W = 6463.57 / 2328.72 = 2.8 m

$$\sigma = \frac{\mathrm{W}}{\mathrm{A}} \pm \frac{M \times c}{I} = \frac{2328.72}{3.5} \pm \frac{6463.57 \times 2}{6} = 665.35 \pm 2154.52$$

$$\sigma_{\max} = 2819.87 KN/m^2 = 2.82 MPa < 0.45 \times f_c^{'} \rightarrow it\ is\ ok.$$

$$\sigma_{\min} = -1489.2 KN/m^2 = -1.49 MPa \quad \text{"}Tension\ stress\text{"}$$

There is a Tension stress and it is less than the allowable tensile strength of concrete
So minimum reinforcement will be provided.
Area of steel /m = 0.0015 A_g = 0.0015 x (1000 x 300) = 450 mm^2
Choose Ø12

$$\text{spacing} = \frac{113}{450} \times 1000 = 251.3 mm$$

Provide Ø12@ 250mm at the center of the section

Elevation from 23.5m to 25.5 m

Maximum Moment (M) = 2589.52KN.m. at elevation 23.5m

Weight above elevation 23.5 (W) = 1174.98KN.

Eccentricity (e) = M / W = 2589.52 / 1174.98 = 2.2 m

$$\sigma = \frac{\mathrm{W}}{\mathrm{A}} \pm \frac{M \times c}{I} = \frac{1174.98}{3.5} \pm \frac{2589.52 \times 2}{6} = 335.7 \pm 863.2$$

$$\sigma_{\max} = 1198.87 KN / m^2 = 1.2 MPa < 0.45 \times f_c^{'} \rightarrow it\ is\ ok.$$

$$\sigma_{\min} = -527.47 KN / m^2 = -0.53 MPa \quad \text{"}Tension\ stress\text{"}$$

Provide Ø12@ 250mm at the center of the section

Note:
For the vertical reinforcement: It is obvious that tension stress is small and the concrete can take it, so minimum reinforcement of 0.0015A_g will be provided for all sections.

Reinforcement Details:

Section No.	Dimension		Vertical Reinforcement	Horizontal Reinforcement
	Outer Diameter (m)	inner Diameter (m)		
1	4.8	3.4	2 layerØ14@250mm	2layerØ14@ 220mm
2	4.6	3.4	1layer Ø14@150mm	Ø16@ 160mm
3	4.0	3.4	1layer Ø12@250mm	Ø16@ 160mm
4	3.0	2.4	1layer Ø12@250mm	Ø16@ 160mm
5	3.0	2.4	1layer Ø12@250mm	Ø16@ 160mm
6	3.0	2.4	1layer Ø12@250mm	Ø16@ 160mm
7	3.0	1.8	1layer Ø14@150mm	Ø16@ 160mm

Table 7.2.4
Minaret sections Reinforcement details

This is a sample of the Reinforcement of the section (section No.1)

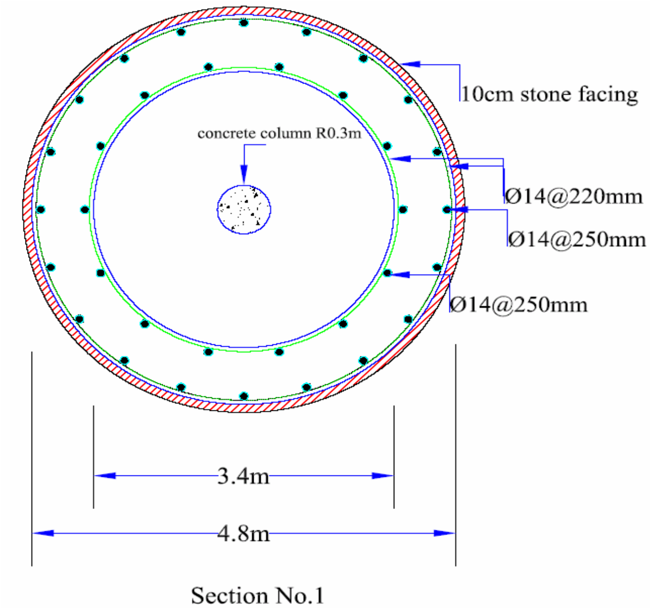

Figure 7.2.11
Reinforcement of section No.1

7.3 Design of footing for minaret

The footing maybe subjected to moment from all direction, so the footing will be designed to resist this moment by choosing square footing
We assumed the footing dimension to be 10m x 10m x 1m

Loads:
The normal force = 10230.8KN
Bending moment = 30510.16 KN.m

Check stress:

$$e = \frac{M}{P} = \frac{30510.16}{10230.8} = 2.98m$$

$$\frac{L}{6} = \frac{10}{6} = 1.67m$$

Because e > L/ 6, so tension stress will be occurred

$$\sigma = \frac{P}{A} \pm \frac{Mc}{I} = \frac{10230.8}{100} \pm \frac{30510.16 \times 5}{833.33}$$

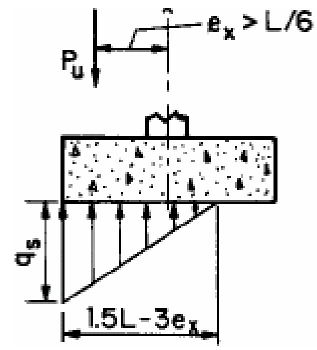

Figure 7.3.1
Pressure distribution

$$\sigma_1 = 285.7 KN/m^2$$

$$\sigma_2 = -80.75 KN/m^2$$

When $e > L/6$, the length of the triangular distribution decreases to $1.5L$ _ $3e$ and the maximum pressure rises to:

$$q_{\max} = \frac{2P}{1.5B(L-2e)} = \frac{2 \times 10230.8}{1.5 \times 10 \times (10 - 2 \times 2.98)} = 337.65 KN/m^2 < 400 KN/m^2 \rightarrow Ok.$$

As in figure 7.2.10

Put because we still have negative stress under the footing of the minaret and we should find any way to reduce it to zero or +ve value and we have two ways as follow:

1- Increase the size (Dimensions) of the footing: and this way isn't economic
2- Use the pile foundation instead of the single footing and this is economic way and the design of the pile foundation for the Minaret as shown in Appendix B