Chapter Four

Analysis and Design Calculations

4.1 Introduction:

Twentystoreysbuilding is selected to be designed according to international codes using manual calculations, Etabs and Safe programmes. Therefore three codes are used to design this building, BS8110, ACI 2005, EC2- 1992.And the results were tabled in the remaining section.

The building sections and material properties are as follows:

- Slab thickness = 250mm.

Columns: i) Internal columns dimension:

- Fromstorey20to storey16 = 500 mm * 500mm.
- From storey15to storey 11 = 600 mm *600 mm.
- Fromstorey10to storey6 = 700 mm *700 mm.
- Fromstorey5to storey 1 = 850 mm *850 mm.

ii) edge and corner columns dimension:

- Fromstorey20to storey16 = 400 mm *400 mm.
- From storey15to storey11 = 450 mm * 450 mm.
- Fromstorey10to storey6 =500 mm *500 mm.
- From storey5to storey1 = 550 mm * 550 mm.

- The raft foundations depth=1300mm.
- **The material properties are:**
  - Characteristic concrete cube strength f_{cu}= 25 N/mm^2.
  - Characteristic strength of reinforcementf_y=460 N/mm^2
  - Characteristic strength of link reinforcementf_{yv}=250 N/mm^2.
  - Concrete density=24kN/m^3.
  - Max aggregate size =20 mm
  - Wind speed=116.2 mile/hr

4.2 Manual calculations:

Reference **BS8110-1997**	**Calculation**	**Output**
     Table (3.3) Table (3.4)	1-Flat slab design: Frame 2/A-G Column strip: b=2500mm middle strip: b=2500mm Durability and fire resistance Nominal cover for mild condition of exposure=25mm Fire resistance for 250mm slab with 25mm cover 2hr>1 hr= L=7-2/3(0.5)= 6.7m d=250-25-12-6=207mm loading weight of slab=0.25*7*5*24=210KN finishing and partition=5*7*5=175KN total D.L= 385KN imposed load=2.5*7*5= 87.5 KN Design load=1.4*385+1.6*87.5=679KN Ultimate design load per unit area=19.4 kN/m^2 Moment and reinforcement d=250-25-16-8=201mm	 C=25mm 1hr fire resistance

Table(3-12) Table(3-18) Table(3-18) Table(3-12)	Outer support M=-0.04*679*6.7=181.97 KN.m Middle strip: M=0.25*181.97=45.49 KN.m $\frac{m}{f_{cu}bd^2} = \frac{45.49*10^6}{2500*201^2*25} = 0.018$ La.d= 190.95mm $A_s = \frac{45.49*10^6}{0.95*460*190.95} = 545.15mm^2 < A_{smin}$ Use T16@500mm $< 3d$ column strip: M=0.75*181.97=136.48 KN.m $\frac{m}{f_{cu}bd^2} = \frac{136.4*10^6}{2500*201^2*25} = 0.054$ La.d= 188.11mm $A_s = \frac{136.48*10^6}{0.95*460*188.11} = 1660mm^2 > A_{smin}$ Use T16@250mm $< 3d$ Shear U=3*400=1200 KN V=0.46*679=312.34mm $V = \frac{312.34}{1200*201} = 1.3 < 0.8\sqrt{f_{cu}}$ **Near middle of end span:**	 Top steel A_s=1828mm^2 A_{smin}=812.5 mm^2 Top steel A_s=2010mm^2 A_{smin}=812.5 mm^2 Shear ok

Table(3-12)	M=-0.075*679*6.7=341.20 KN.m	
	Middle strip:	
Table(3-18)	M=0.45*341.20=153.54 KN.m	
	$\frac{m}{f_{cu}bd^2} = \frac{153.54 * 10^6}{2500 * 207^2 * 25} = 0.057$	
	La.d= 192.93mm	
	$A_s = \frac{153.54 * 10^6}{0.95 * 460 * 192.93} = 1821mm^2 > A_{smin}$	Bottom steel A_s=1883mm² A_{smin}=812.5 mm²
	Use T12@150mm< 3d	
	column strip:	
Table(3-18)	M=0.55*341.20=187.66 KN.m	
	$\frac{m}{f_{cu}bd^2} = \frac{187.66 * 10^6}{2500 * 207^2 * 25} = 0.07$	
	La.d= 189.40 mm	
	$A_s = \frac{187.66 * 10^6}{0.95 * 460 * 189.40} = 2267mm^2 > A_{smin}$	Bottom steel A_s=2354mm² A_{smin}=812.5 mm²
	Use T12@120mm< 3d	
	Deflection:	
Table(3-9)	Basic span/effect depth ratio	
	$\frac{m}{bd^2} = \frac{153.54 * 10^6}{2500 * 207^2} = 1.43$	
	$f_s = \frac{2}{3} * 460 * \frac{1821}{1883} = 297$	
Equation (8)	m.f = $0.55 + \frac{477-297}{120\{0.9+1.43\}} = 1.19$	
Equation (7)	limiting(L/d) =1.43*26*1.19=31	

	Actual $=\frac{5000}{207}=24<$ limiting o.k	
	First interior support	
Table(3-12)	M= - 0.086*679*6.7=391.24 KN.m	Deflection ok
	Middle strip:	
Table(3-18)	M=0.25*391.24=97.81 KN.m	
	$\frac{m}{f_{cu}bd^2}=\frac{97.81*10^6}{2500*201^2*25}=0.039$	
	La.d= 191.94mm	
	$A_s=\frac{97.81*10^6}{0.95*460*191.94}=1166mm^2 > A_{smin}$	Top steel A_s=1256mm^2 A_{smin}=812.5 mm^2
	Use T16@400mm$< 3d$	
	column strip:	
Table(3-18)	M=0.75*391.24=293.43 KN.m	
	$\frac{m}{f_{cu}bd^2}=\frac{293.43*10^6}{2500*201^2*25}=0.116$	
	La.d= 170.38mm	
	$A_s=\frac{293.43*10^6}{0.95*460*170.38}=3940mm^2 > A_{smin}$	Top steel A_s=4020mm^2 A_{smin}=812.5 mm^2
	Use T16@125mm$< 3d$	
	Shear	
	U=4*500 = 2000mm	
Table(3-12)	V=0. 6*679=407. 4mm	
	$V=\frac{407.4}{2000*201}=1.01<0.8\sqrt{f_{cu}}$	Shear ok

Table(3-12) Table(3-18) Table(3-18) Table(3-12) Table(3-18) Table(3-18)	middle interior span: M=-0.063*679*6.7=286.61 KN.m Middle strip: M=0.45*286.61=128.97 KN.m $\frac{m}{f_{cu}bd^2} = \frac{128.97 * 10^6}{2500 * 207^2 * 25} = 0.048$ La.d= 195.26mm $A_s = \frac{128.97 * 10^6}{0.95 * 460 * 195.26} = 1511mm^2 > A_{smin}$ Use T12@150mm$< 3d$ **column strip:** M=0.55*286.61=157.63 KN.m $\frac{m}{f_{cu}bd^2} = \frac{157.63 * 10^6}{2500 * 207^2 * 25} = 0.059$ La.d= 192.44 mm $A_s = \frac{157.63 * 10^6}{0.95 * 460 * 192.44} = 1874mm^2 > A_{smin}$ Use T12@150mm$< 3d$ **interior support** M= - 0.063*679*6.7=286.61 KN.m Middle strip: M=0.25*286.61=71.65 KN.m $\frac{m}{f_{cu}bd^2} = \frac{71.65 * 10^6}{2500 * 201^2 * 25} = 0.028$ La.d= 190.95mm $A_s = \frac{71.65 * 10^6}{0.95 * 460 * 190.95} = 859mm^2 > A_{smin}$ Use T16@500mm$< 3d$ column strip: M=0.75*286.61=214.95 KN.m $\frac{m}{f_{cu}bd^2} = \frac{214.95 * 10^6}{2500 * 201^2 * 25} = 0.085$ la.d= 179.74mm	Bottom steel A_s=1883mm^2 A_{smin}=812.5mm^2 Bottom steel A_s=1883mm^2 A_{smin}=812.5mm^2 Top steel A_s=1005mm^2 A_{smin}=812.5mm^2

<table>
<tr>
<td>Table(3-12)

Table(3-12)

Table(3-18)</td>
<td>$A_s = \frac{214.95 * 10^6}{0.95 * 460 * 179.74} = 2737mm^2 > A_{smin}$
Use T16@170mm $< 3d$
Shear
U=4*500 = 2000mm
V=0. 5*679=339.5mm
$V = \frac{339.5}{2000*201} = 0.84 < 0.8\sqrt{fcu}$
Frame 1/A-G
Column strip: b=1250mm
middle strip: b=1250mm
L=7-2/3(0.5)= 6.7m
d=250-25-12-6=207mm
loading
weight of slab=0.25*7*2.5*24=105KN
finishing and partition=5*7*2.5=87.5KN
total D.L= 192.5KN
imposed load=2.5*7*2.5= 43.75 KN
Design
load=1.4*192.5+1.6*43.75=339.5KN
Ultimate design load per unit
area=10.65kN/m^2
Moment and reinforcement
d=250-25-16-8=201mm
Outer support
M=-0.04*339.5*6.7=91 KN.m
Middle strip:
M=0.25*91=22.75 KN.m
$\frac{m}{f_{cu}bd^2} = \frac{22.75 * 10^6}{1250 * 201^2 * 25} = 0.018$
La.d= 190.95mm
$A_s = \frac{22.75*10^6}{0.95*460*190.95} = 273mm^2 < A_{smin}$

Use T16@400mm $< 3d$

column strip:
M=0.75*91= 68.25 KN.m</td>
<td>Top steel
A_s=2955mm^2
A_{smin}=812.5mm^2

Shear ok

Top steel
A_s=628mm^2
A_{smin}=406.25mm^2</td>
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	$\frac{m}{f_{cu}bd^2} = \frac{68.25 * 10^6}{1250 * 201^2 * 25} = 0.054$ La.d= 188.11mm $A_s = \frac{68.25 * 10^6}{0.95 * 460 * 188.11} = 831mm^2 > A_{smin}$ Use T16@250mm$< 3d$	Top steel A_s=1005mm^2 A_{smin}=406.25mm^2
Table(3-12)	Shear U=2*400=800mm V=0.46*339.5=156.17mm $V = \frac{156.17}{800 * 201} = 0.97 < 0.8\sqrt{f_{cu}}$	Shear ok
Table(3-12)	Near middle of end span: M= 0.075*339.5*6.7=170.6 KN.m	
Table(3-18)	Middle strip: M=0.45*170.6=76.77 KN.m $\frac{m}{f_{cu}bd^2} = \frac{76.77 * 10^6}{1250 * 207^2 * 25} = 0.057$ La.d= 192.85mm $A_s = \frac{76.77 * 10^6}{0.95 * 460 * 192.85} = 911mm^2 > A_{smin}$ Use T12@150mm$< 3d$	Bottom steel A_s=942mm^2 A_{smin}=406.25mm^2
Table(3-18)	**column strip:** M=0.55*170.6=93.83 KN.m $\frac{m}{f_{cu}bd^2} = \frac{93.83 * 10^6}{1250 * 207^2 * 25} = 0.07$ La.d= 189.40 mm $A_s = \frac{93.83 * 10^6}{0.95 * 460 * 189.40} = 1134mm^2 > A_{smin}$ Use T12@120mm$< 3d$	Bottom steel A_s=1177mm^2 A_{smin}=406.25mm^2
Table(3-12)	First interior support M= - 0.086*339.5*6.7=195.62 KN.	
Table(3-18)	**Middle strip:** M=0.25*195.62=48.91 KN.m $\frac{m}{f_{cu}bd^2} = \frac{48.91 * 10^6}{1250 * 201^2 * 25} = 0.039$	

	La.d= 191.94mm $A_s = \frac{48.91*10^6}{0.95*460*191.94} = 583mm^2 > A_{smin}$ Use T16@400mm$< 3d$	Top steel A_s=628mm^2 A_{smin}=406.25mm^2
Table(3-18)	**column strip:** M=0.75*195.62=146.72 KN.m $\frac{m}{f_{cu}bd^2} = \frac{146.72*10^6}{1250*201^2*25} = 0.116$ La.d= 170.85mm $A_s = \frac{146.72*10^6}{0.95*460*= 170.85} = 1965mm^2 > A_{smin}$ Use T16@125mm$< 3d$	Top steel A_s=2010mm^2 A_{smin}=406.25mm^2
Table(3-12)	Shear U=3*400 = 1200mm V=0. 6*339.5 = 203.7mm $V = \frac{203.7*10^6}{1200*201} = 0.84 < 0.8\sqrt{f_{cu}}$	Shear ok
Table(3-12)	middle interior span: M=-0.063*339.5*6.7=143.30 KN.m	
Table(3-18)	**Middle strip:** M=0.45*143.30=64.49 KN.m $\frac{m}{f_{cu}bd^2} = \frac{64.49 \ *10^6}{1250*207^2*25} = 0.048$ La.d= 195.26mm $A_s = \frac{64.49*10^6}{0.95*460*195.26} = 757mm^2 > A_{smin}$ Use T12@170mm$< 3d$	Bottom steel A_s=831mm^2 A_{smin}=406.25mm^2
Table(3-18)	**column strip:** M=0.55*143.30=78.82 KN.m $\frac{m}{f_{cu}bd^2} = \frac{78.82 \ *10^6}{1250*207^2*25} = 0.059$ La.d= 192.44 mm $A_s = \frac{78.82*10^6}{0.95*460*192.44} = 937mm^2 > A_{smin}$	Bottom steel A_s=1087mm^2

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<td>Table(3-12)

Table(3-18)

Table(3-18)

Table(3-12)</td>
<td>Use T12@130mm$< 3d$

interior support:
M= - 0.063*339.5*6.7=143.30 KN.m
Middle strip:
M=0.25*143.30=35.83KN.m
$\frac{m}{f_{cu}bd^2} = \frac{35.83*10^6}{1250*201^2*25} = 0.028$
La.d= 190.95mm
$A_s = \frac{35.83*10^6}{0.95*460*190.95} = 430mm^2 > A_{smin}$
Use T16@400mm$< 3d$
column strip:
M=0.75*143.30=107.48 KN.m
$\frac{m}{f_{cu}bd^2} = \frac{107.48*10^6}{1250*201^2*25} = 0.08$
la.d= 179.74mm
$A_s = \frac{107.48*10^6}{0.95*460*179.74} = 1368mm^2 > A_{smin}$
Use T16@170mm$< 3d$
Shear
U=3*400 = 1200mm
V=0. 5*339.5=169.75mm
$V = \frac{169.75}{1200*201} = 0.7 < 0.8\sqrt{f_{cu}}$
Frame B/1-4
Column strip: b=2500mm
middle strip: b=4500mm
L=5-2/3(0.5)= 4.7m
d=250-25-6=219mm
loading
As frame 2/A-H
Moment and reinforcement
d=250-25-8=217mm
Outer support</td>
<td>A_{smin}=406.25mm^2

Top steel
A_s=628mm^2
A_{smin}=406.25mm^2

Top steel
A_s=1478mm^2
A_{smin}=406.25mm^2

Shear ok</td>
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Table(3-12) able(3-18)	M=-0.04*679*4.7=127.65 KN.m Middle strip: M=0.25*127.65 =31.91 KN.m $\frac{m}{f_{cu}bd^2} = \frac{31.91*10^6}{4500*217^2*25} = 0.006$ La.d= 206.15mm $A_s = \frac{31.91*10^6}{0.95*460*206.15} = 354.21mm^2 < A_{smin}$ Use T16@500mm$< 3d$	Top steel A_s=1809mm^2 A_{smin}=1463mm^2
Table(3-18)	column strip: M=0.75*127.65= 95.74 KN.m $\frac{m}{f_{cu}bd^2} = \frac{= 95.74*10^6}{2500*217^2*25} = 0.033$ La.d= 206.15mm $A_s = \frac{95.74*10^6}{0.95*460*206.15} = 1063mm^2 > A_{smin}$ Use T16@400 mm$< 3d$	Top steel A_s=1256mm^2 A_{smin}=812.5mm^2
Table(3-12) Table(3-18)	Near middle of end span: M=-0.075*679*4.7=239.35 KN.m Middle strip: M=0.45*239.35=107.71 KN.m $\frac{m}{f_{cu}bd^2} = \frac{107.71*10^6}{4500*219^2*25} = 0.036$ La.d= 208.05mm $A_s = \frac{107.71*10^6}{0.95*460*208.05} = 1185mm^2 < A_{smin}$ Use T12@ 340 mm$< 3d$	bottom steel A_s=1496mm^2 A_{smin}=1463mm^2
Table(3-18)	column strip: M=0.55*239.35=131.64 KN.m $\frac{m}{f_{cu}bd^2} = \frac{131.64*10^6}{2500*219^2*25} = 0.044$ La.d= 207.74mm $A_s = \frac{131.64*10^6}{0.95*460*207.74} = 1450mm^2 > A_{smin}$	bottom steel A_s=1487mm^2

Table(3-12) Table(3-18) Table(3-18) Table(3-12) Table(3-18)	Use T12@190mm$< 3d$ First interior support M= - 0.086*679*4.7=274.45 KN.m Middle strip: M=0.25*274.45= 68.61 KN.m $\frac{m}{f_{cu}bd^2} = \frac{68.61 * 10^6}{4500 * 217^2 * 25} = 0.013$ La.d= 206.15mm $A_s = \frac{68.61 * 10^6}{0.95 * 460 * 206.15} = 762mm^2$ $< A_{smin}$ Use T16@500 mm$< 3d$ column strip: M=0.75*274.45 = 205.84 KN.m $\frac{m}{f_{cu}bd^2} = \frac{205.84 * 10^6}{2500 * 217^2 * 25} = 0.07$ La.d= 191.52mm $A_s = \frac{205.84 * 10^6}{0.95 * 460 * 191.52} = 2495mm^2$ $> A_{smin}$ Use T16@190mm$< 3d$ middle interior span: M=-0.063*679*4.7=201.05 KN.m Middle strip: M=0.45*201.05 = 90.47KN.m $\frac{m}{f_{cu}bd^2} = \frac{90.47 * 10^6}{4500 * 219^2 * 25} = 0.017$ La.d= 208.05mm $A_s = \frac{90.47*10^6}{0.95*460*208.05} = 995mm^2 < A_{smin}$ Use T12@340 mm$< 3d$ column strip: M=0.55*201.05=110.58 KN.m $\frac{m}{f_{cu}bd^2} = \frac{110.58 * 10^6}{2500 * 219^2 * 25} = 0.037$ La.d= 208.05 mm	A_{smin}=812.5mm^2 Top steel A_s=1809mm^2 A_{smin}=1463mm^2 Top steel A_s=2645mm^2 A_{smin}=812.5mm^2 Bottom steel A_s=1496mm^2 A_{smin}=1463mm^2

	$A_s = \frac{110.58*10^6}{0.95*460*208.05} = 1216mm^2 > A_{smin}$	Bottom A_s=1469mm^2 A_{smin}=812.5mm^2
	Use T12@200mm$< 3d$	
	interior support	
Table(3-12)	M= - 0.063*679*4.7=201.05 KN.m	
	Middle strip:	
Table(3-18)	M=0.25*201.05 =50.26 KN.m	
	$\frac{m}{f_{cu}bd^2} = \frac{50.26*10^6}{4500*217^2*25} = 0.009$	
	La.d = 206.15mm	
	$A_s = \frac{50.26*10^6}{0.95*460*206.15} = 558mm^2 < A_{smin}$	Top steel A_s=1809mm^2 A_{smin}=1463mm^2
	Use T16@500mm$< 3d$	
	column strip:	
Table(3-18)	M=0.75*201.05 =150.79 KN.m	
	$\frac{m}{f_{cu}bd^2} = \frac{150.79*10^6}{2500*217^2*25} = 0.051$	
	la.d= 203.85mm	
	$A_s = \frac{150.79*10^6}{0.95*460*203.85} = 1693mm^2 > A_{smin}$	Top steel A_s=2010mm^2 A_{smin}=812.5mm^2
	Use T16@250mm$< 3d$	
	Frame A/1- 4	
	Column strip: b=1250mm	
	middle strip: b=2250mm	
	L=5-2/3(0.5)= 4.7m	
	d=250-25-6=219mm	
	loading	
	As frame 1/A-G	
	Moment and reinforcement	
	d=250-25-8=217mm	
	Outer support	
Table(3-12)	M=-0.04*339.5*4.7= 63.83 KN.m	
	Middle strip:	
Table(3-18)	M=0.25*63.83=15.96 KN.m	

	$\frac{m}{f_{cu}bd^2} = \frac{15.96*10^6}{2250*217^2*25} = 0.006$ La.d= 206.15mm $A_s = \frac{15.96*10^6}{0.95*460*206.15} = 177mm^2 < A_{smin}$ Use T16@500mm$< 3d$	Top steel A_s=905mm^2 A_{smin}=731.25mm^2
Table(3-18)	column strip: M=0.75*63.83= 47.87 KN.m $\frac{m}{f_{cu}bd^2} = \frac{47.87*10^6}{1250*217^2*25} = 0.033$ La.d= 206.15mm $A_s = \frac{47.87*10^6}{0.95*460*206.15} = 532mm^2 > A_{smin}$ Use T16@400mm$< 3d$	Top steel A_s=628mm^2 A_{smin}=406.25mm^2
Table(3-12)	Near middle of end span: M=-0.075*339.5*4.7=119.67 KN.m	
Table(3-18)	Middle strip: M=0.45*119.67 =53.85KN.m $\frac{m}{f_{cu}bd^2} = \frac{53.85*10^6}{2250*219^2*25} = 0.02$ La.d= 208.05mm $A_s = \frac{53.85*10^6}{0.95*460*208.05} = 592mm^2 < A_{smin}$ Use T12@300mm$< 3d$	Bottom steel A_s=848mm^2 A_{smin}=731.25mm^2
Table(3-18)	column strip: M=0.55*119.67 =65.82 KN.m $\frac{m}{f_{cu}bd^2} = \frac{65.82*10^6}{1250*219^2*25} = 0.044$ La.d= 207.7 mm $A_s = \frac{65.82*10^6}{0.95*460*207.7} = 725mm^2 > A_{smin}$ Use T12@170mm$< 3d$	Bottom steel A_s=831mm^2 A_{smin}=406.25mm^2
Table(3-12)	First interior support M= - 0.086*339.5*4.7=137.23 KN.m	
Table(3-18)	Middle strip: M=0.25*137.23=34.31 KN.m $\frac{m}{f_{cu}bd^2} = \frac{34.31*10^6}{2250*217^2*25} = 0.013$	

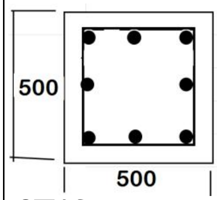
Table(3-18)	La.d= 206.15mm $A_s = \frac{34.31 * 10^6}{0.95 * 460 * 206.15} = 381mm^2 < A_{smin}$ Use T16@500mm$< 3d$ column strip: M=0.75*137.23=102.92KN.m $\frac{m}{f_{cu}bd^2} = \frac{102.92 * 10^6}{1250 * 217^2 * 25} = 0.07$ La.d= 199mm $A_s = \frac{102.92*10^6}{0.95*460*199} = 1183mm^2 > A_{smin}$ Use T16@200 mm$< 3d$	Top steel A_s=905mm^2 A_{smin}=731.25mm^2 Top steel A_s=1256mm^2 A_{smin}=406.25mm^2
Table(3-12) Table(3-18)	middle interior span: M=-0.063*339.5*4.7=100.53 KN.m Middle strip: M=0.45*100.53= 45.24KN.m $\frac{m}{f_{cu}bd^2} = \frac{45.24 * 10^6}{2250 * 219^2 * 25} = 0.017$	
Table(3-18)	La.d= 208.05mm $A_s = \frac{45.24 * 10^6}{0.95 * 460 * 208.05} = 498mm^2 < A_{smin}$ Use T12@300mm$< 3d$ column strip: M=0.55*100.53=55.29KN.m $\frac{m}{f_{cu}bd^2} = \frac{55.29 * 10^6}{1250 * 219^2 * 25} = 0.037$ La.d= 208.05 mm $A_s = \frac{55.29 * 10^6}{0.95 * 460 * 208.05} = 608mm^2 > A_{smin}$ Use T12@200mm$< 3d$	Bottom steel A_s=848mm^2 A_{smin}=731.25mm^2 Bottom steel A_s=706mm^2 A_{smin}=406.25mm^2
Table(3-12) Table(3-18)	interior support M= - 0.063*339.5*4.7=100.53 KN.m Middle strip: M=0.25*100.53=25.13KN.m	

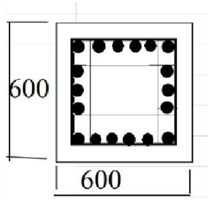
Table(3-18)	$\frac{m}{f_{cu}bd^2} = \frac{25.13*10^6}{2250*217^2*25} = 0.009$ La.d= 206.15mm $A_s = \frac{25.13*10^6}{0.95*460*206.15} = 279mm^2 < A_{smin}$ Use T16@500mm$< 3d$ column strip: M=0.75*100.53=75.40 KN.m $\frac{m}{f_{cu}bd^2} = \frac{75.40*10^6}{1250*217^2*25} = 0.051$ la.d= 203.85mm $A_s = \frac{75.40*10^6}{0.95*460*203.85} = 847mm^2 > A_{smin}$ Use T16@250mm$< 3d$ Punching shear Internal column d=(201+217)/ 2=209 1-at face of column side perimeter =500mm u =4*500=2000mm v=679-19.4(0.5^2)=674.15KN $V = \frac{1.15*674.15*10^3}{2000*209} = 1.85\, N/mm^2 < 0.8\sqrt{f_{cu}}$ 2-first critical perimeter@1.5d from load area side perimeter =2*1.5*209+500=1127mm u =4*1127=4508 mm v=679-19.4(1.127^2)=654.38KN $V = \frac{1.15*654.38*10^3}{4508*209} = 0.8\, N/mm^2$	Top steel A_s=905mm^2 A_{smin}=731.25mm^2 Top steel A_s=1005mm^2 A_{smin}=406.25mm^2 Punching Shear ok
Table(3-8)	$\frac{100*A_S}{b_v*d} = \frac{100*4020}{2500*209} = 0.77$ $v_c = 0.68 < v$ Use shear reinforcement $A_{Sv} = \frac{0.4*4508*209}{0.95*250} = 1587$	

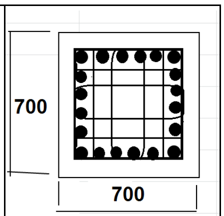
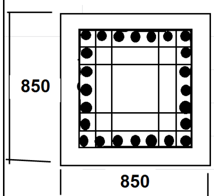
Table(3-8)	Number of links size 8mm=$\frac{1587}{101}=16$ Distributed @two perimeter@140mm and280mm $u_1=4(140*2+500)=3120mm$ $u_2=4(280*2+500)=4240$ Number of links@ $u_1=\frac{3120*16}{3120+4240}=7$ Number of links@ $u_2=16-7=9$ Spacing=$\frac{4240}{2*16}+3120=230<1.5d$ edge column 1-at face of column side perimeter =400mm u =3*400=1200mm v=339.5-19.4(0.4^2)=336.4KN $V=\frac{1.4*336.4*10^3}{1200*209}=1.88\,N/mm^2<0.8\sqrt{f_{cu}}$ 2-first critical perimeter@1.5d from load area side perimeter =1.5*209+400=713.5mm and the other=2*1.5*209+400=1027 u =2*713.5+1027=2454 mm v=339.5-19.4(0.714*1.027)=325kN $V=\frac{1.4*325*10^3}{2454*209}=0.8\,9N/mm^2$ $\frac{100*A_S}{b_v*d}=\frac{100*2010}{1250*209}=0.69$ $v_c=0.68<v$ Use shear reinforcement $A_{Sv}=\frac{0.4*2454*209}{0.95*250}=864$ Number of links size 8mm=$\frac{864}{101}=9$ Distributed @twoperimeter@140mm and280mm	

	$u_1 = 140*2+400+2(140+400)$ $= 1760$ $u_2 = 280*2+400+2(280+400)$ $= 2320$ Number of links@ $u_1 = \frac{1760*9}{1760+2320} = 4$ Number of links@ $u_2 = 9-4 = 5$ Spacing=$\frac{1760+2320}{2*9} = 227 < 1.5d$	

Reference BS8110-1997	Calculation	Output
	2-Design of column: (1)Design of internal column: Fcu=35N/mm^2 Fy = 460N/mm^2 fyv = 250N/mm^2 Column height =3.4m **Design of column from storey20 to storey16:** Assume: b=500m, h= 500mm bar diameter=16mm **loading:** area carried by column for one storey=35m^2 **dead load:** self-weight of slab=0.25*24=6 finishing= 1 minimum partition =4 total =11 total dead load=35*11=385kN Self-weight of column=0.5*0.5*24*3.4=20.4 kN total dead load=405.4 kN imposed loadoffice building=2.5*35=87.5 kN reduction50%=0.5*87.5=43.8 kN	

3.8.4.4	design load=1.4gk+1.6qk =1.4*405.4+1.6*43.8=638 kN N=638*5=3188.2 kN Short brace column supporting an approximately symr arrange of beams $N = 0.35 f_{cu} A_c + 0.7 f_y A_{sc}$ 3188200=0.35*35*500*500+0.7*460A_{sc} A_{sc}= 390mm^2 Number of bar =8bars Use 8T16(A_{sc}=1608mm^2) **Design ofcolumn from storey16 to storey11:** Assume: b=600m, h= 600mm bar diameter=25mm **dead load:** dead load from slab==385kN Self-weight of column=0.6*0.6*24*3.4=24.888 kN total dead load=409.89kN imposed loadoffice building=43.8 kN design load=1.4gk+1.6qk =1.4*409.89+1.6*43.8=643.92 kN N=643.92*10=7076 kN	 **8T16** A_{sc}=1608 mm^2

3.8.4.4	Short brace column supporting an approximately symmetrical arrange of beams $N = 0.35 f_{cu} A_c + 0.7 f_y A_{sc}$ 7076=0.35*35*600*600+0.7*460A_{sc} A_{sc}= 8280mm^2 Number of bar =17say18bars Use18T25(A_{sc}=8831mm^2) **Design ofcolumn from storey10 to storey6:** Assume: b=700m, h= 700mm bar diameter=32mm **loading:** **dead load:** dead load from slab =385kN Self weight of column=0.7*0.7*24*3.4=29.92 kN total dead load=414.92kN imposed loadoffice building= 43.8 kN design load=1.4gk+1.6qk =1.4*414.92+1.6*43.8=650.97 kN N=650.97*15=10751 kN	 600 600 18T25 A_{sc}=8831 mm^2
3.8.4.4	Short brace column supporting an approximately symr arrange of beams	

3.8.4.4	$N = 0.35 f_{cu} A_c + 0.7 f_y A_{sc}$ 10751000=0.35*35*700*700+0.7*460A_{sc} A_{sc}= 14747mm^2 Number of bar =20 Use 20T32(A_{sc}=16076.8mm^2) **Design ofcolumn from storey5 to storey1:** Assume: b=850m, h= 850mm bar diameter=32mm **loading:** dead load from slab =385kN Self-weight of column=0.85*0.85*24*3.4=58.956 kN total dead load=443.96kN imposed loadoffice building= 43.8 kN design load=1.4gk+1.6qk =1.4*443.96+1.6*43.8= 691.58 kN N=691.58 *20=14559 kN Short brace column supporting an approximately Symmetrical arrange of beams $N = 0.35 f_{cu} A_c + 0.7 f_y A_{sc}$ 14559000=0.35*35*850*850+0.7*460A_{sc} A_{sc}= 17727mm^2 Number of bar =24bars	 20T32 A_{sc}=16076.8 mm^2  24T32 A_{sc}=19292.16 mm^2

Use 24T32(A_{sc}=19292.16mm^2)

(2) Edge column:

Design ofcolumn from storey20 to storey16:

Assume:

b=400m, h= 400mm

bar diameter=16mm

loading:

area carried by column for one storey=17.5m^2

dead load:

self-weight of slab=0.25*24=6

finishing= 1kN/m^2

minimum partition =4kN/m^2

total =11kN/m^2

total dead load=17.5*11=192.5kN

Self-weight of column=0.4*0.4*24*3.4=13.056 kN

total dead load=205.56kN

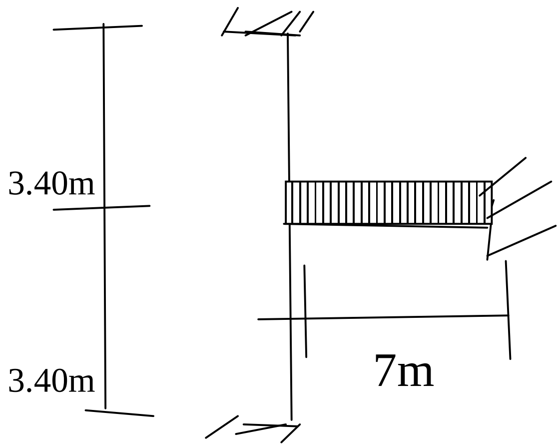
imposed loadoffice building=2.5*17.5=43.75 kN

reduction50%=0.5*43.75=21.875 kN

design load=1.4gk+1.6qk

=1.4*205.56+1.6*21.875= 322.78 kN

N=322.78 *5=613.9 kN



W = $\frac{192.5*1.4+1.6*21.875}{7} = 48.5$ kN/m

Member stiffness:

$k_{beam} = \frac{I}{2L} = \frac{5000*(250)^3}{2*12*7000} = 465.03*10^{-6}$

$k_{column} = \frac{I}{L} = \frac{400*(400)^3}{12*3400} = 627.45*10^{-6}$

Distribution factor:

$D_{beam} = \frac{k_{beam}}{k_{beam}+2*k_{column}} =$

$$\frac{465.03*10^{-6}}{465.03*10^{-6}+2*627.45*10^{-6}} = 0.2704$$

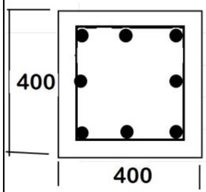
$D_{column} = \frac{k_{column}}{k_{beam}+2*k_{column}} =$

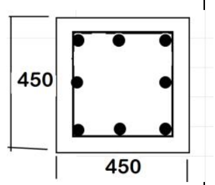
$$\frac{627.45*10^{-6}}{465.03*10^{-6}+2*627.45*10^{-6}} = 0.3648$$

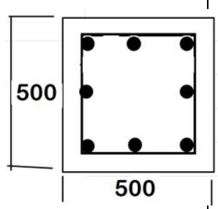
Fixed end moment:

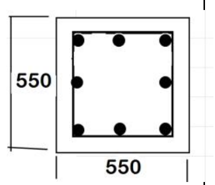
M= $\frac{wl^2}{12} = \frac{48.5*7^2}{12} = 198.04$ kNm

M_{col}= 0.2704*198.04=72.248 kNm

Part 3 Chart No28	$\frac{N}{bh} = \frac{613.9 * 10^3}{400 * 400} = 3.84$ $\frac{M}{bh^2} = \frac{198.04 * 10^6}{400 * 400^2} = 3.09$ Use charts Fcu=35 N/mm^2 Fy = 460 N/mm^2 $\frac{d}{h}$= 0.85 $\frac{100A_{sc}}{bh} = 0.4$ A_{sc} = 640mm^2 Number of bars=3.2 say 8bars Use 8T16 (A_{sc}=1608mm^2) **Design ofcolumn from storey15 to storey11:** Assume: b=450m, h= 450mm bar diameter=16mm **loading:** dead load from slab =192.5kN Self weight of column=0.45*0.45*24*3.4=14.79 kN total dead load=207.29kN imposed loadoffice building= 21.875 kN design load=1.4gk+1.6qk =1.4*207.29+1.6*21.875= 325.21 kN	 8T16 A_{sc}=1608 mm^2

Part 3 Chart No28	N=325.21 *10= 3252.1 kN M= $\frac{wl^2}{12}=\frac{72.248*7^2}{12}=295$ kNm M_{col}= 72.248 kNm $\frac{N}{bh}=\frac{3252.1*10^3}{450*450}=16.064$ $\frac{M}{bh^2}=\frac{295*10^6}{450*450^2}=0.7928$ Use charts Fcu=35 N/mm^2 Fy = 460 N/mm^2 $\frac{d}{h}$= 0.85 $\frac{100A_{sc}}{bh}=0.4$ $A_{sc}=810mm^2$ Number of bars=4.03 say 8bars Use 8T16 (A_{sc}=1608mm^2) **Design ofcolumn from storey10 to storey6:** Assume: b=500m, h= 500mm bar diameter=20mm **loading:** dead load from slab =192.5kN imposed loadoffice building = 21.875 kN	 8T16 A_{sc}=1608 mm^2

Part 3 Chart No28	design load=1.4gk+1.6qk =1.4*209.16+1.6*21.875= 327.82 kN N=327.82 *15= 4917.4 kN M_{col}=72.248 kNm $\frac{N}{bh} = \frac{4917.4 * 10^3}{500 * 500} = 16.064$ $\frac{M}{bh^2} = \frac{295 * 10^6}{500 * 500^2} = 0.7928$ Use charts Fcu=35 N/mm^2 Fy = 460 N/mm^2 $\frac{d}{h}$= 0.85 $\frac{100A_{sc}}{bh} = 0.4$ $A_{sc} = 810mm^2$ Number of bars=4.03 say 8bars Use 8T20 (A_{sc}=2512mm^2) **Design of column from storey5 to storey1:** Assume: b=550m, h= 550mm bar diameter=20mm **loading:** dead load from slab =192.5kN Self-weight of column=0.55*0.55*24*3.4=18.666 kN total dead load=211.17kN	 8T20 A_{sc}=2512 mm^2

Part 3 Chart No28	imposed loadoffice building =21.875 kN design load=1.4gk+1.6qk =1.4*211.17+1.6*21.875= 330.63 kN N=327.82 *20= 6612.6 kN M_{col}=72.248 kNm $\frac{N}{bh} = \frac{4917.4 * 10^3}{550 * 550} = 16.064$ $\frac{M}{bh^2} = \frac{295 * 10^6}{550 * 550^2} = 0.7928$ Use charts Fcu=35 N/mm^2 Fy = 460 N/mm^2 $\frac{d}{h}$= 0.85 $\frac{100A_{sc}}{bh} = 0.4$ $A_{sc} = 810mm^2$ Number of bars=4.03 say 8bars Use 8T20 (A_{sc}=2512mm^2) **Shear:** Size of link = $\frac{1}{4}$ diameter of largest compression bar and not less than 6 mm =$\frac{1}{4}$(32)=8mm :. Size of link= 8 mm Maximum spacing between link =12 times smallest compression bar= 12*16=192mm Use R8@175mm	 8T20 A_{sc}=2512 mm^2

Reference **BS8110-1997**	**Calculation**	**Output**
	3-Design of raft foundation: Fcu=25 N/mm^2 Fy = 460 N/mm^2 fyv = 250 N/mm^2 B.C =250 Raft thickness =1300mm cover =50 Width of panel=5000 mm Span =7000 mm x-direction=30 y-direction=21 B=850mm,B= raft foundation width H=850mm,H= raft foundation length Bar dia=20mm **Loading** **(a) serviceability limit state** **Dead load:** Number of internal columns=10 400.096*10*20= 80019 kN Number of edge columns=14 211.166*14*20= 59126 kN Number of corner columns= 4	

	120.95*4*20= 9676 kN Total dead load of columns=148822 kN Self weight of concrete wall(length54.5m)=1236.1 kN Self weight of raft =19656 kN Total dead load=169714 kN Live load Number of internal columns=10 43.8*10*20= 8760kN Number of edge columns=14 21.875*14*20= 6125 kN Number of corner columns= 4 10.94*4*20= 875.2 kN Total live load=15760 kN Total load= 1.0D.L+1.0L.L =185474 kN Require base area=741.9 kN Provide base area=33.5*24.5= 820.75 kN Design load=1.4gk+1.6qk=1.4*169714+1.6*15760=233567 kN Design load per unit area= 284.58 kN **Moment and reinforcement** d=1300-50-10=1240mm L=7-2/3(0.96)= 6.36m At mid span:	
Table (3-12)	M=-0.075*9960.3*6.36=4751 KN.m	
	Middle strip:	
Table (3-18)	M=0.45*4751 =2138 KN.m	

	$\frac{m}{f_{cu}bd^2} = \frac{2138 \ast 10^6}{2500 \ast 1240^2 \ast 25} = 0.0222$	
	La.d= 1178mm	
	$A_s = \frac{2138 \ast 10^6}{0.95 \ast 460 \ast 1178} = 4153mm^2 > A_{smin}$	Bottom steel
	Use T20@150mm$< 3d$	A_s=5233mm^2
	column strip:	A_{smin}=812.5 mm^2
Table (3-18)	M=0.55*4751 =2613 KN.m	
	$\frac{m}{f_{cu}bd^2} = \frac{2613 \ast 10^6}{2500 \ast 1240^2 \ast 25} = 0.0272$	
	La.d= 1130mm	
	$A_s = \frac{2613 \ast 10^6}{0.95 \ast 460 \ast 1130} = 5292mm^2 > A_{smin}$	Bottom steel
	Use T20@140mm$< 3d$	A_s =5607mm^2
	At support	A_{smin}=812.5 mm^2
Table (3-12)	M= - 0.086*9960.3*6.36=5448KN.m	
	Middle strip:	
Table (3-18)	M=0.25*5448=1362 KN.m	
	$\frac{m}{f_{cu}bd^2} = \frac{1362 \ast 10^6}{2500 \ast 1240^2 \ast 25} = 0.0141$	
	La.d= 1178mm	Top steel
	$A_s = \frac{1362 \ast 10^6}{0.95 \ast 460 \ast 1178} = 2646mm^2 > A_{smin}$	A_s =3140mm^2
	Use T20@250mm$< 3d$	A_{smin}=812.5 mm^2
	column strip:	
Table (3-18)	M=0.75*5448=4086 KN.m	

	$$\frac{m}{f_{cu}bd^2} = \frac{4086*10^6}{2500*1240^2*25} = 0.043$$	
	La.d= 1178mm	Top steel
	$$A_s = \frac{4086*10^6}{0.95*460*1178} = 7936mm^2 > A_{smin}$$	A_s =8177mm^2 A_{smin}=812.5 mm^2
	Use T25@150mm$< 3d$	
	Shear	
	U=4*850 = 3400mm	
Table (3-12)	V=0. 6*9960.3=5976.18mm	
	$$V = \frac{5976.18*10^3}{3400*1240} = 1.42 < 0.8\sqrt{f_{cu}}$$	Shear OK
	Frame B/1-5	
	Column strip: b=2500mm	
	middle strip: b=4500mm	
	L=5-2/3(0.96)= 4.36m	
	d=250-25-6=219mm	
	loading	
	As frame 2/A-H	
	Moment and reinforcement	
	d=1300-50-20-10=1220mm	
	At mid span:	
Table (3-12)	M=-0.075*9960.3*4.36=3257 KN.m	
	Middle strip:	
Table (3-18)	M=0.45*3257=1465.66 KN.m	
	$$\frac{m}{f_{cu}bd^2} = \frac{1465.66*10^6}{4500*1220^2*25} = 0.009$$	
	La.d= 1159mm	

	$A_s = \frac{1465.66*10^6}{0.95*460*1159} = 2894mm^2 > A_{smin}$ Use T20@ 475 mm $< 3d$ column strip:	Bottom steel A_s =2975mm^2 A_{smin}=1463 mm^2
Table (3-18)	M=0.55*3257=1791.35 KN.m $\frac{m}{f_{cu}bd^2} = \frac{1791.35*10^6}{2500*1220^2*25} = 0.0193$ La.d= 1159mm $A_s = \frac{1791.35 \ *10^6}{0.95*460*1159} = 3537mm^2 > A_{smin}$ Use T20@200mm$< 3d$ At support	Bottom steel A_s =3925mm^2 A_{smin}=812.5 mm^2
Table (3-12)	M= - 0.086*9960.3*4.36=3734.71 KN.m Middle strip:	
Table (3-18)	M=0.25*3734.71= 933.68KN.m $\frac{m}{f_{cu}bd^2} = \frac{933.68*10^6}{4500*1220^2*25} = 0.006$ La.d= 1159mm $A_s = \frac{933.68*10^6}{0.95*460*1159} = 1843mm^2 > A_{smin}$ Use T16@475 mm$< 3d$ column strip:	Top steel A_s =1904mm^2 A_{smin}=1463 mm^2
Table (3-18)	M=0.75*3734.71 = 2801 KN.m $\frac{m}{f_{cu}bd^2} = \frac{2801*10^6}{2500*1220^2*25} = 0.03$ La.d= 1159 mm	

Table (3-8)	$A_s = \frac{2801*10^6}{0.95*460*1159} = 5530mm^2 > A_{smin}$ Use T20@140mm$< 3d$ Punching shear Internal column d=(1240+1220)/ 2=1230 1-at face of column side perimeter =850mm u =4*850=3400mm v=14559-415.97(0.85^2)=14258.46KN $V = \frac{1.15*14258.46*10^3}{3400*1230} = 3.92\,N/mm^2 < 0.8\sqrt{f_{cu}}$ 2-first critical perimeter@1.5d from load area side perimeter =2*1.5*1230+850=4540mm u =4*4540=18160mm v=14559-415.97 (4.54^2)=5985KN $V = \frac{1.15*5985*10^3}{18160*1230} = 0.31\,N/mm^2$ $\frac{100*A_S}{b_v*d} = \frac{100*3110}{2500*1230} = 0.101$ $v_c = 0.36 > v$ Edge column: 1-at face of column side perimeter =550mm u =3*550=1650mm v=6612.6-377.86(0.55^2)=6498.3KN $V = \frac{1.4*6498.3*10^3}{1650*1230} = 4.48\,N/mm^2 < 0.8\sqrt{f_{cu}}$ 2-first critical perimeter@1.5d from load area side perimeter =2*1.5*1230+550=4240mm and the other=1.5*1230+500=2345mm u =3*4240=12720mm	Top steel A_s =5607mm^2 A_{smin}=812.5 mm^2 Punching Shear OK

Table (3-8)	v=6612.6-377.86 $(4.24*2.345)$=2855.61KN $$V=\frac{1.4*2855.61*10^3}{12720*1230}=0.26\,N/mm^2$$ $v_c=0.36<v$	

	Design of staircase:	
	Slope length=$\sqrt{(1.7^2 + (3^2})$ =3.45	
	Try 125mm thickness of waist	
	Consider 1m length of stair	
	Loading:	
	Weight of waist +steps	
	(0.125*3.45*1+0.3*(1.7)/2)*24=16.47 kN	
	Live load=2.5*3*1= 7.5 kN	
	Design load= 1.4*16.47+1.6*7.5= 35 kN	
	$$M = \frac{wl}{8} = \frac{35*3}{8} = 13.13$$	
	d=125-25-6=94	
	K =$\frac{M}{bf_{cu}d^2}$ = $\frac{13.13*10^6}{1000*94^2*25}$ = 0.06< 0.156	
	Z= 87.2	
	$$A_s = \frac{13.13*10^6}{0.95*460*87.2} = 344mm^2$$	$A_s = 452\ mm^2$
	Use R12@250mm c/c	
	Transverse distribution steel:	
	$$= \frac{0.13\ bh}{100} = \frac{0.13*\ 1000*125}{100} = 162.5mm^2$$	
	Use R10 @250mm c/c	$A_s = 314\ mm^2$
	Check deflection:	
Table (3-9)	$$\frac{m}{bd^2} = \frac{13.13*10^6}{1000*94^2} = 1.49$$	
Equation	$$f_s = \frac{2}{3}*460*\frac{344}{452} = 233.39$$	

(8) Equation (7)	$m.f = 0.55+\frac{477-233.39}{120\{0.9+1.49\}} = 1.4$ limiting =20*1.4*1.15=32.2 Actual $=\frac{3000}{94}$=31.91 o.k	Deflection O.K

CP3 part2 1972	Frame analysis: wind speed= $\frac{116.2*8*1000}{5*60*60}$ =51.64m/s $\boldsymbol{S_1} = \boldsymbol{S_3} = 1$ Design speed v_s = =V.$\boldsymbol{S_1}.\boldsymbol{S_2}.\boldsymbol{S_3}$ $cp_e = 0.8$ $cp_i = -0.25$ $\boldsymbol{c_f} = (0.8 + 0.25) = 1.05$	

Table (4.1) Wind load analysis

NO	H	S_2	q=0.613$v_s{}^2$	F=c_fqA_e
20	68	1.165	2.219	19.82
19	64.6	1.16	2.199	39.25
18	61.2	1.15	2.162	38.57
17	57.8	1.15	2.162	38.57
16	54.4	1.146	2.147	38.27
15	51	1.143	2.136	38.08
14	47.6	1.138	2.17	37.79
13	44.2	1.131	2.091	37.30
12	40.8	1.124	2.065	36.81
11	37.4	1.116	1.036	36.33
10	34	1.105	1.996	35.65
9	30.6	1.095	1.96	34.97
8	27.2	1.084	1.921	34.29
7	23.8	1.074	1.886	33.61
6	20.4	1.063	1.847	32.93
5	17	1.045	1.785	31.86
4	13.6	1.024	1.714	30.6
3	10.2	1.03	1.734	30.99
2	6.8	0.928	1.408	25.16
1	3.4	0.843	1.162	20.69

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Earthquake forces

Total design base shear

$V=\frac{ZIC}{R_w} * W$

$Z=0.12$,$I=1$,$R_w = 6$

$C=\frac{1.25S}{T^{\frac{2}{3}}}$,$S=1$,$T=C_t(h_n)^{\frac{3}{4}}$,$C=0.049$

Vertical distribution of base shear :

Force shear in each floor

$F(n)= (V-F_t)*\frac{W_n\,h_n}{\sum_{i=1}^{n} W_i\,h_i}$

Table (4.2) Earth quake forces in each floor

Story No	h_n (m)	T (Sec)	C	W(kN)	V (kN)	F_t (kN)	$W_i\,h_i$	$\frac{W_i\,h_i}{W_i\,h_i}$	F(n)
20	68	1.16	1.132	8703.16	4063	330	591815	0.0936	349.381
19	64.6	1.16	1.132	8703.16	4063	329.9	562224	0.0889	331.981
18	61.2	1.16	1.132	8703.16	4063	329.9	532633	0.0842	314.509
17	57.8	1.16	1.132	8703.16	4063	329.9	503043	0.0796	297.36
16	54.4	1.16	1.132	8703.16	4063	329.9	473452	0.0749	279.563
15	51	1.16	1.132	8703.16	4063	329.9	451623	0.0714	266.674
14	47.6	1.16	1.132	8855.35	4063	329.9	421515	0.0667	248.896
13	44.2	1.16	1.132	8855.35	4063	329.9	391406	0.0619	231.117
12	40.8	1.16	1.132	8855.35	4063	329.9	361298	0.0571	213.339
11	37.4	1.16	1.132	8855.35	4063	329.9	331190	0.0524	195.561
10	34	1.16	1.132	8855.35	4063	329.9	307061	0.0486	181.313
9	30.6	1.16	1.132	9031.2	4063	329.9	276355	0.0437	163.182
8	27.2	1.16	1.132	9031.2	4063	329.9	245649	0.0389	145.05
7	23.8	1.16	1.132	9031.2	4063	329.9	214943	0.0340	126.919
6	20.4	1.16	1.132	9031.2	4063	329.9	184236	0.0291	108.788
5	17	1.16	1.132	9031.2	4063	329.9	185062	0.0250	93.3324
4	13.6	1.16	1.132	9297.78	4063	329.9	126450	0.0200	74.6659
3	10.2	1.16	1.132	9297.78	4063	329.9	94837.4	0.0150	55.9995
2	6.8	1.16	1.132	9297.78	4063	329.9	63224.9	0.0100	37.333
1	3.4	1.16	1.132	9297.78	4063	329.9	31612.5	0.0050	18.6665
				179437.5			6322629		

Shear wall design:

Unsymmetrical layout of shear walls:

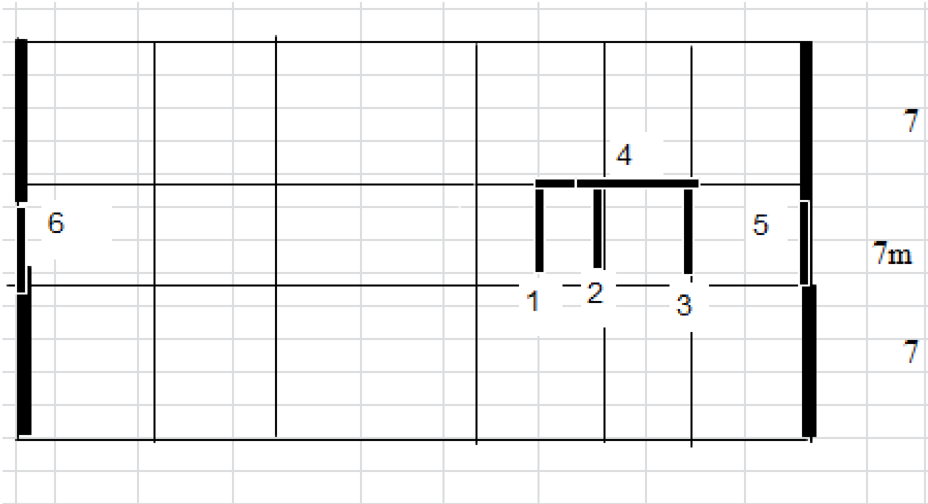


Figure (4.1) Unsymmetrical layout of shear walls

K_x:

$K_1 = K_2 = K_3 = 0.3 * 2.5^3 = 4.688$

$K_4 = \mathbf{0}$

$K_5 = K_6 = 0.3 * 21^3 = 2778$

$\sum k = (4.688 * 2 + 2778 * 2) = 5565.376$

$K_y: -$

$K_4 = 0.3 * 5^3 = 37.5$

$\bar{x} = \frac{4.688 * 0 + 4.688 * 2.5 + 4.688 * 5 + 2778 * 5 + 2.77}{5565.376}$

$= 14.970$

$P_i = P_d + P_r$

$M_t x \frac{K_i r_i}{\sum(r_i{}^2 K_i)} \pm Fx \frac{K_x}{\sum K_x}$

The torsional moment(M_t)is:

F=349.4 KN

$M_t = 349.4(15 - 14.970) = 6.988$

Table (4.3) The torsional moment (m_t)

wall	K_x	K_y	r	K_r	K_r^2	p_d	p_r	p_i
1	4.69	0	4.98	23.36	116.31	0.294147	0.00013	0.29427763
2	4.69	0	7.48	35.08	262.41	0.294147	0.0002	0.29434312
3	4.69	0	9.98	46.81	467.13	0.294147	-0.0003	0.29388576
4	0	37.5	0	0	0	0	0	0
5	2778	0	15.02	41730	626786	174.2493	0.23307	174.482352
6	2778	0	14.98	41619	623452	174.2493	-0.2325	174.016827
	5571	**37.5**			**1251083**	**349.4**	**7E-04**	**349.38**

Table (4.4) The distribution of loads in each wall

Storey no	Load in wall(5,6)	Load in wall(1,2,3)	Load in wall(4)	Load in wall(5,6)
20	349.38	174.482566	0.29428	0.293886
19	331.98	165.792959	0.27962	0.27925
18	314.51	157.067014	0.26491	0.264553
17	297.04	148.341068	0.25019	0.249855
16	279.56	139.615123	0.23547	0.235158
15	266.67	133.178005	0.22461	0.224316
14	248.9	124.299471	0.20964	0.209361
13	231.12	115.420937	0.19467	0.194407
12	213.34	106.542404	0.17969	0.179453
11	195.56	97.66387	0.16472	0.164498
10	181.31	90.5484402	0.15272	0.152513
9	163.18	81.4935962	0.13745	0.137262
8	145.05	72.4387522	0.122117	0.122011
7	126.92	63.3839082	0.1069	0.106759
6	108.79	54.3290641	0.09163	0.091508
5	93.332	46.6106097	0.07861	0.078508
4	74.666	37.2884878	0.06289	0.062806
3	55.999	27.9663658	0.04717	0.047105
2	37.333	18.6442439	0.03144	0.031403
1	18.666	9.32212194	0.01572	0.015702

Table (4.5) The distribution of torsional moment in each wall

Height(m)	Wall load	(5,6) mt	Wall load	(1,2,3) mt	Wall load	(4) mt
68	174.483	11864.8145	0.2943	20.0109	0.293886	19.98426
64.6	165.793	10710.2251	0.2796	18.0636	0.27925	18.03955
61.2	157.067	9612.50123	0.2649	16.2122	0.264553	16.19062
57.8	148.341	8574.11375	0.2502	14.4609	0.249855	14.44163
54.4	139.615	7595.0627	0.2355	12.8096	0.235158	12.79259
51	133.18	6792.07823	0.2246	11.4554	0.224316	11.4401
47.6	124.299	5916.65482	0.2096	9.97888	0.209361	9.965596
44.2	115.421	5101.60543	0.1947	8.60424	0.194407	8.592784
40.8	106.542	4346.93007	0.1797	7.33143	0.179453	7.321662
37.4	97.6639	3652.62874	0.1647	6.16043	0.164498	6.15223
34	90.5484	3078.64697	0.1527	5.19237	0.152513	5.185456
30.6	81.4936	2493.70404	0.1374	4.20582	0.137262	4.200219
27.2	72.4388	1970.33406	0.1222	3.32312	0.122011	3.318692
23.8	63.3839	1508.53701	0.1069	2.54426	0.106759	2.540873
20.4	54.3291	1108.31291	0.0916	1.86925	0.091508	1.866764
17	46.6106	792.380365	0.0786	1.33641	0.078508	1.33463
13.6	37.2885	507.123433	0.0629	0.8553	0.062806	0.854163
10.2	27.9664	285.256931	0.0472	0.48111	0.047105	0.480467
6.8	18.6442	126.780858	0.0314	0.21383	0.031403	0.213541
3.4	9.32212	31.6952146	0.0157	0.05346	0.015702	0.053385
		86069.3864		**145.163**		**144.9692**

Design wall(5,6):

M=86069.3864kN.m , N=0.3*21*68*24= 4536 KN

$$\frac{N}{bh} = 0.72$$

$$\frac{M}{bh^2} = 0.650562256$$

Use chart

Use charts

Fcu=35 N/mm^2

Fy = 460 N/mm^2

$\frac{d}{h}$= 0.85

Bar diameter 20

$$\frac{100A_{sc}}{bh} = 1.2$$

A_{sc} = 75600mm^2

Use T20@150mm c/c (A_{sc}=87920mm^2)

Design wall(4):

M=144.9692kN.m , N=0.3*5*68*24= 1260 KN

$$\frac{N}{bh} = 1.68$$

$$\frac{M}{bh^2} = 0.077316906$$

	Use charts Fcu=35 N/mm^2 Fy = 460 N/mm^2 $\frac{d}{h}$= 0.85 Bar diameter 16 $\frac{100A_{sc}}{bh}$ = 1.2 A_{sc} = 9000mm^2 Use T16@200mm c/c , (A_{sc}=10084mm^2)	

4.3 Design by Etabs and safeprogrammes:

4.3.1 Flat slab design:

1- BS8110-1997code:

Output design for Etabsprogramme for BSI -8110 1997 from figures below:

Design result for flat slab:

For span 5m:

Column strip =2500mm, middle strip =4500mm

Maximum top reinforcement for column strip =2631mm^2

UseT16@190mm c/c$< 3d$ (As=2645mm^2)

Maximum bottom reinforcement for column strip =1059mm^2

$< A_{smin}(1463mm^2)$

UseT12@190mm c/c$< 3d$ (As=1487mm^2)

Maximum top reinforcement for middle strip =714mm^2

$< A_{smin}(1463mm^2)$

UseT16@500mm c/c$< 3d$ (As =1809mm^2)

Maximum bottom reinforcement for middle strip =1244mm^2

$< A_{smin}(1463mm^2)$

UseT12@300mm c/c$< 3d$ (As=1695mm^2)

For span 7m:

Column strip =2500mm, middle strip =2500mm

Maximum top reinforcement for column strip =3699mm^2

$> A_{smin}$

UseT16@130mm c/c$< 3d$ (As =3865mm^2)

Maximum bottom reinforcement for column strip =2033mm^2

$> A_{smin}$

UseT12@130mm c/c$< 3d$ (As =2173mm^2)

Maximum top reinforcement for middle strip =1789mm^2

$> A_{smin}$

UseT16@275mm c/c$< 3d$ (As =1809mm^2)

Maximum bottom reinforcement for middle strip =1852mm^2

$> A_{smin}$

UseT12@150mm c/c$< 3d$ (As =1883mm^2)

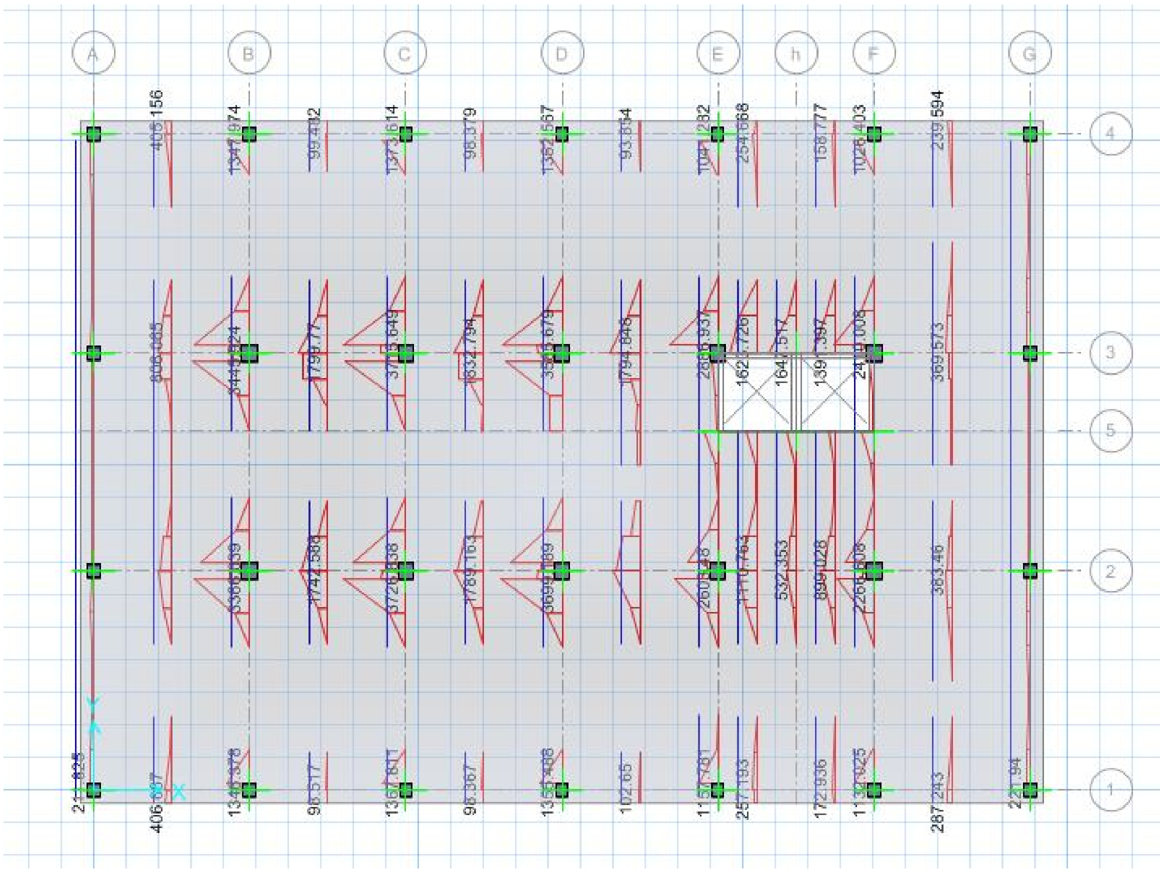


Figure (4.2): Flat slab top reinforcement for span 5m for BS-8110-1997code

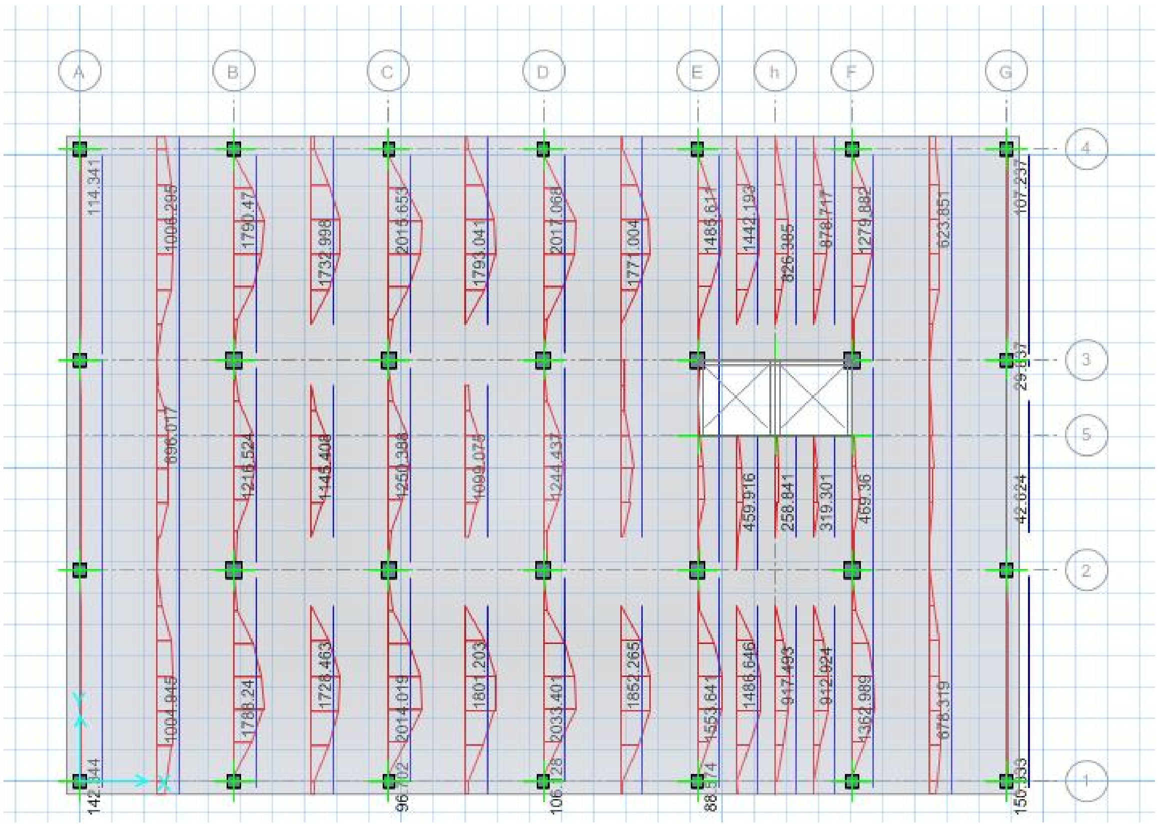


Figure (4.3): Flat slab bottom reinforcement for span 5m for BS-8110-1997code

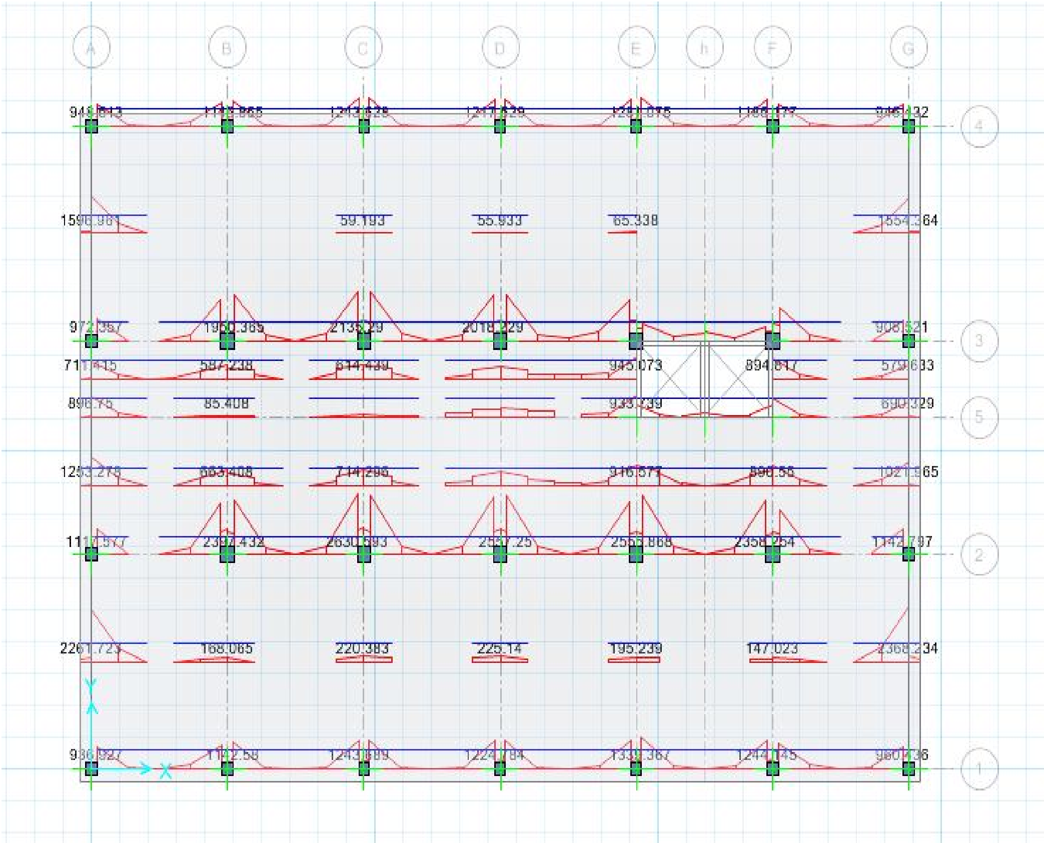


Figure (4.4): Flat slab top reinforcement for span 7m for BS-8110-1997code

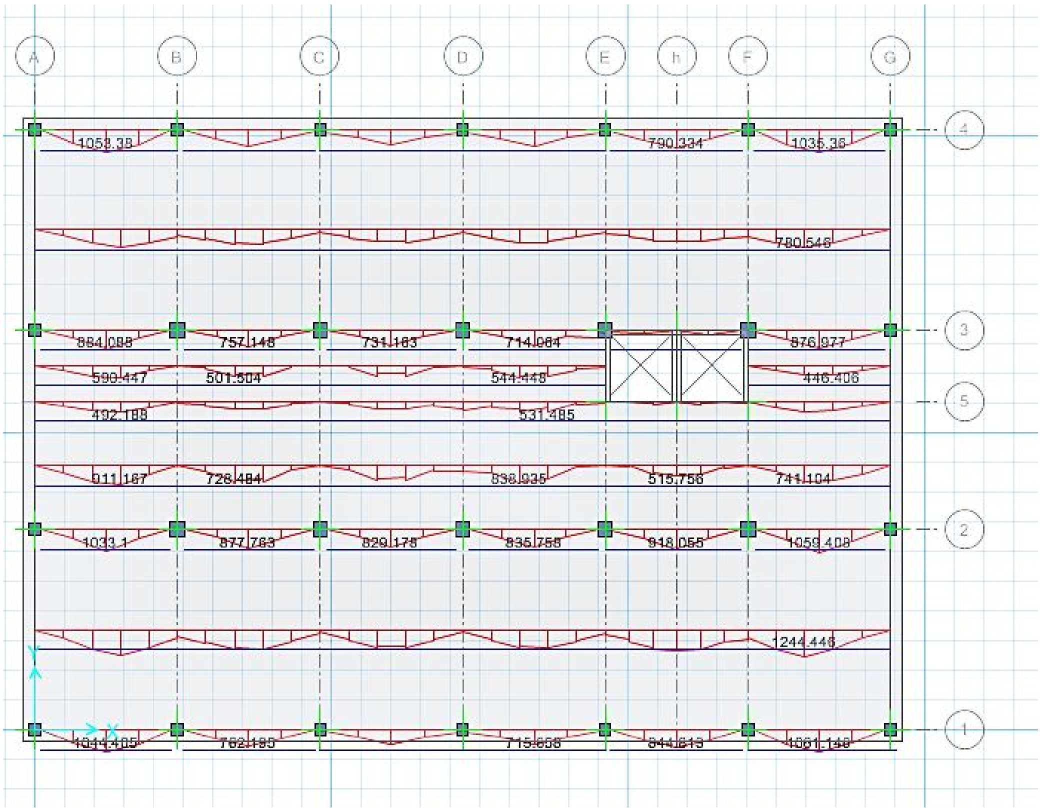


Figure (4.5): Flat slab bottom reinforcement for span 7m for BS-8110-1997code

2- ACI-2005 code:

Output design for Etabsprogramme for ACI -2005 from figures below:

Result of design for flat slab:

For span 5m:

Column strip =2500mm, middle strip =4500mm

Maximum top reinforcement for column strip= 2666mm^2

$> A_{smin}$

UseT16@180mm c/c$< 3d$ (As =2792mm^2)

Maximum bottom reinforcement for column strip in =1075mm^2

$> A_{smin}$

UseT12@250mm c/c$< 3d$ (As =1130mm^2)

Maximum top reinforcement for middle strip =724mm^2

$< A_{smin}(1463mm^2)$

UseT16@500mm c/c$< 3d$ (As =1809mm^2)

Maximum bottom reinforcement for middle strip =1261mm^2

$> A_{smin}$

UseT12@400mm c/c$< 3d$ (As =1271mm^2)

For span 7m:

Column strip =2500mm, middle strip =2500mm

Maximum top reinforcement for column strip =3777mm^2

$> A_{smin}$

UseT16@130mm c/c$< 3d$ (As =3865mm^2)

Maximum bottom reinforcement for column strip =2061mm^2

$> A_{smin}$

UseT12@130mm c/c$< 3d$ (As =2173mm^2)

Maximum top reinforcement for middle strip =1813mm^2

$> A_{smin}$

UseT16@270mm c/c$< 3d$ (As =1861mm^2)

Maximum bottom reinforcement for middle strip =1877mm^2

$> A_{smin}$

UseT12@150mm c/c$< 3d$ (As =1883mm^2)

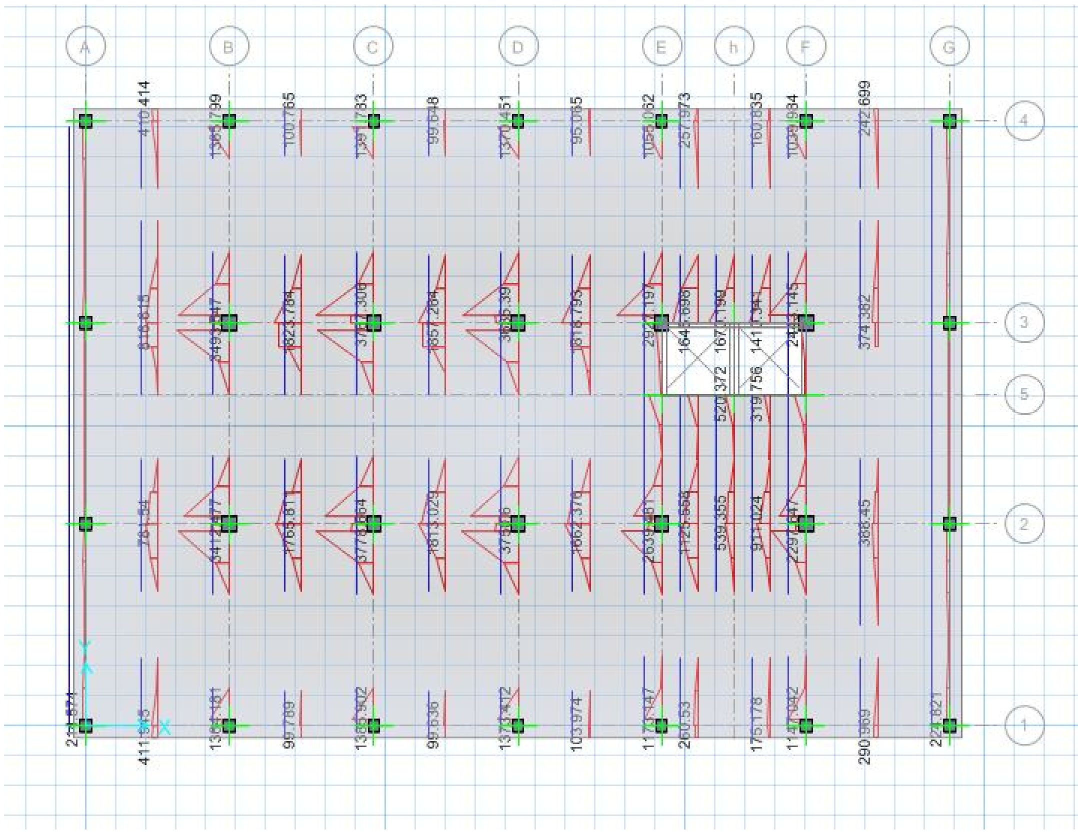


Figure (4.6): Flat slab top reinforcement for span 5m for ACI 2005 code:

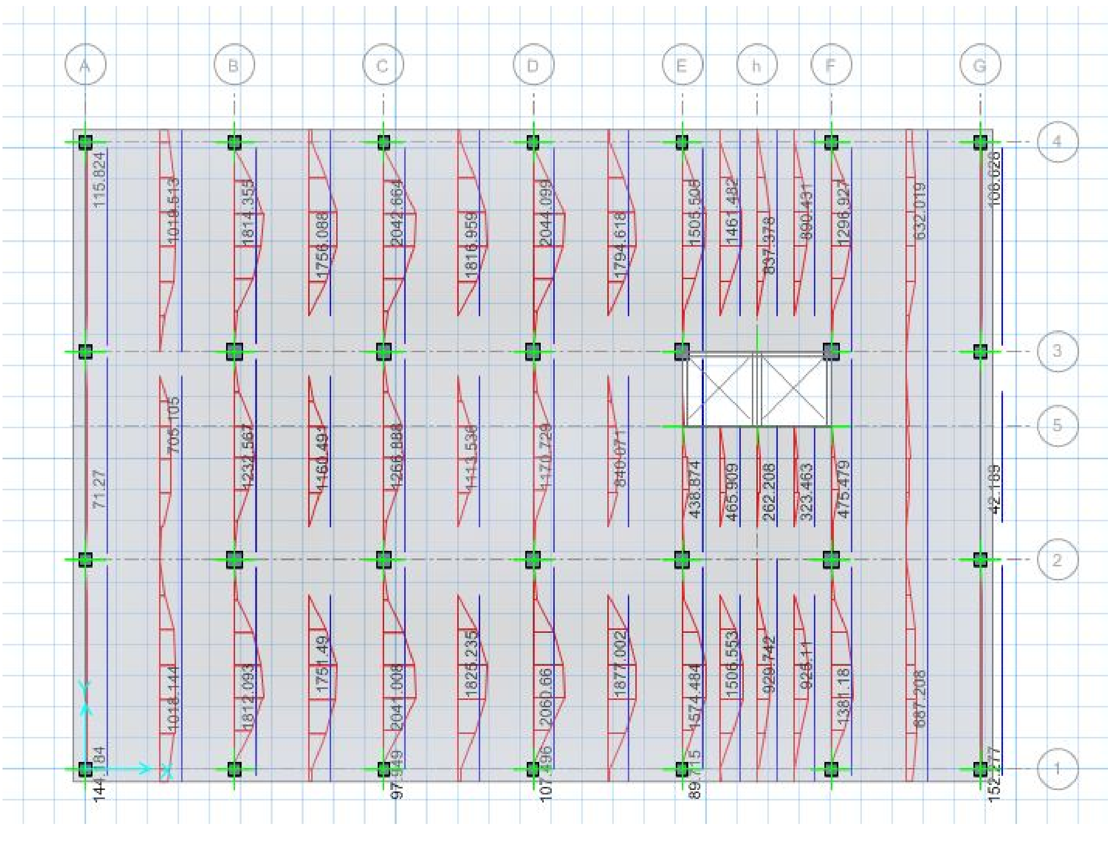


Figure (4.7): Flat slab bottom reinforcement for span 5m for ACI 2005 code:

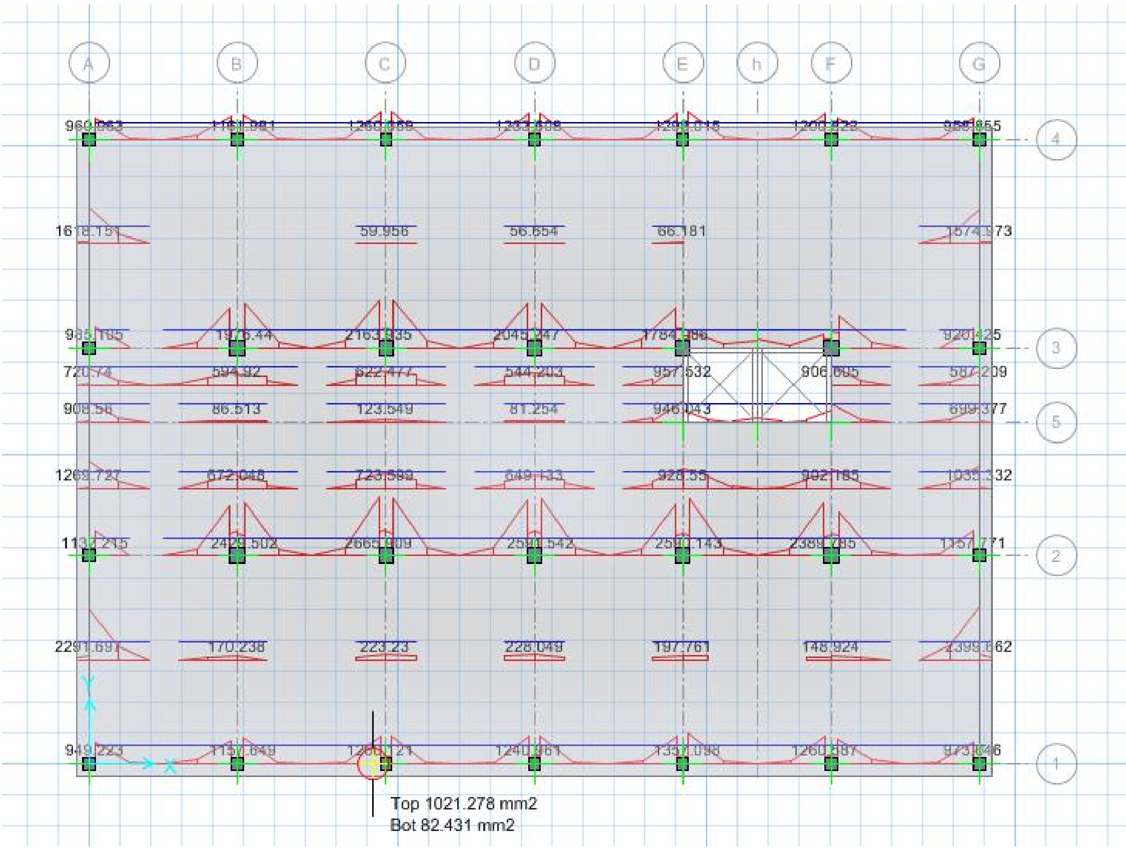


Figure (4.8): Flat slab top reinforcement for span 7m for ACI 2005 code

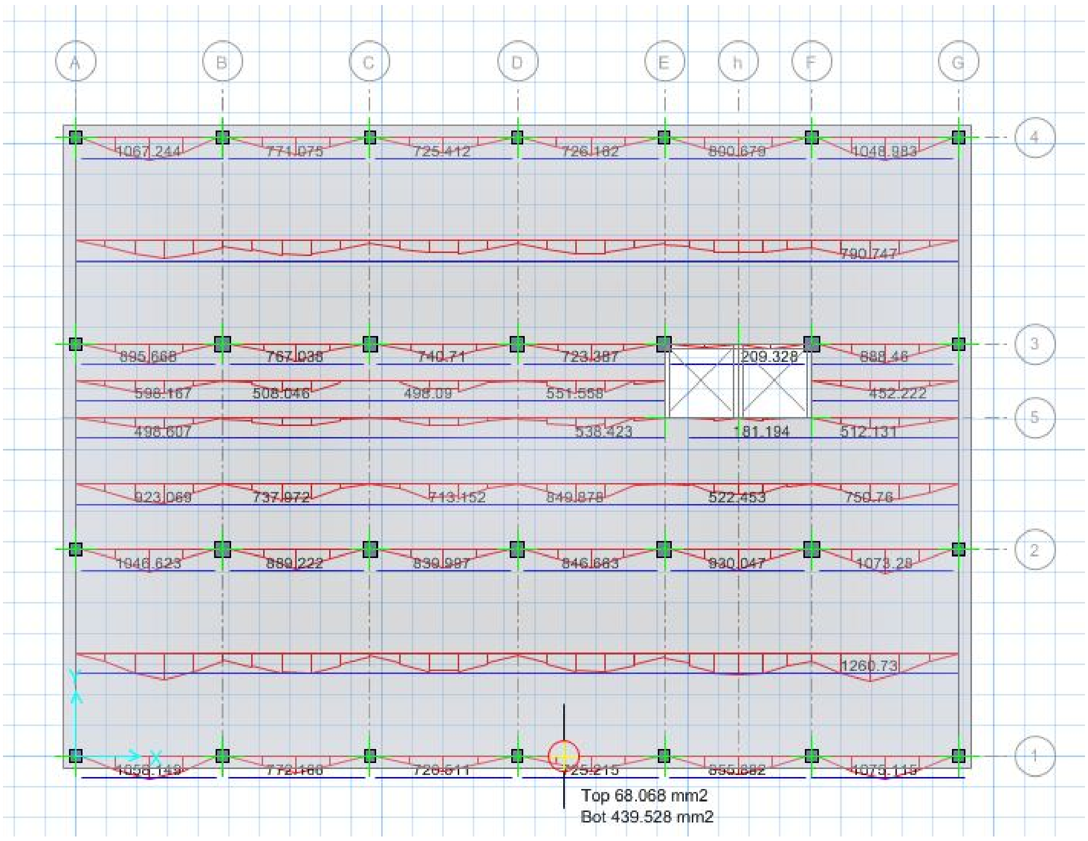


Figure (4.9): Flat slab bottom reinforcement for span 7m for ACI 2005 code:

3-EC2-1992:

Output design for Etabsprogramme for EU-1992 from figures blew:

Result of design for flat slab:

For span 5m:

Column strip =2500mm, middle strip =4500mm

Maximum top reinforcement for column strip =2621mm^2

$> A_{smin}$

UseT16@190mm c/c$< 3d$ (As =2645mm^2)

Maximum bottom reinforcement for column strip =1055mm^2

$< A_{smin}(1463mm^2)$

UseT12@190mm c/c$< 3d$ (As=1487mm^2)

Maximum top reinforcement for middle strip =710mm^2

$< A_{smin}(1463mm^2)$

UseT16@500mm c/c$< 3d$ (As =1809mm^2)

Maximum bottom reinforcement for middle strip =1237mm^2

$< A_{smin}(1463mm^2)$

UseT12@300mm c/c$< 3d$ (As=1695mm^2)

For span 7m:

Column strip =2500mm, middle strip =2500mm

Maximum top reinforcement for column strip =3700

$> A_{smin}$

UseT16@130mm c/c$< 3d$ (As =3865mm^2)

Maximum bottom reinforcement for column strip =2033mm^2

$> A_{smin}$

UseT12@130mm c/c$< 3d$ (As =2173mm^2)

Maximum top reinforcement for middle strip =1781mm^2

$> A_{smin}$

UseT16@280mm c/c$< 3d$ (As =1795mm^2)

Maximum bottom reinforcement for middle strip =1845mm^2

$> A_{smin}$

UseT12@150mm c/c$< 3d$ (As =1883mm^2)

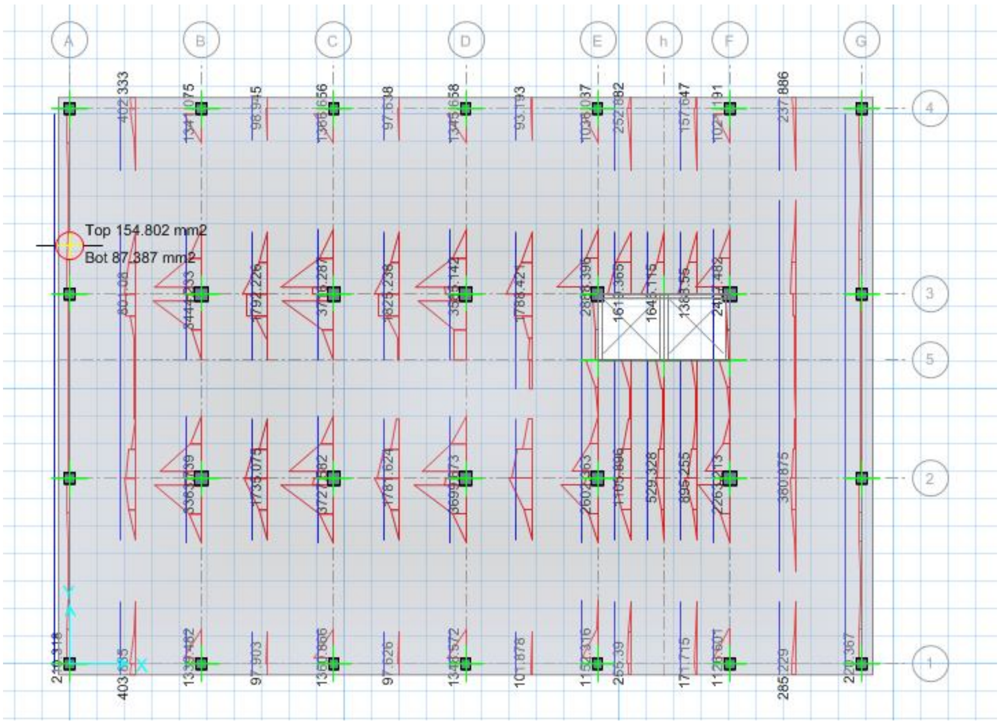


Figure (4.10): Flat slab top reinforcement for span 5m EC2-1992 code:

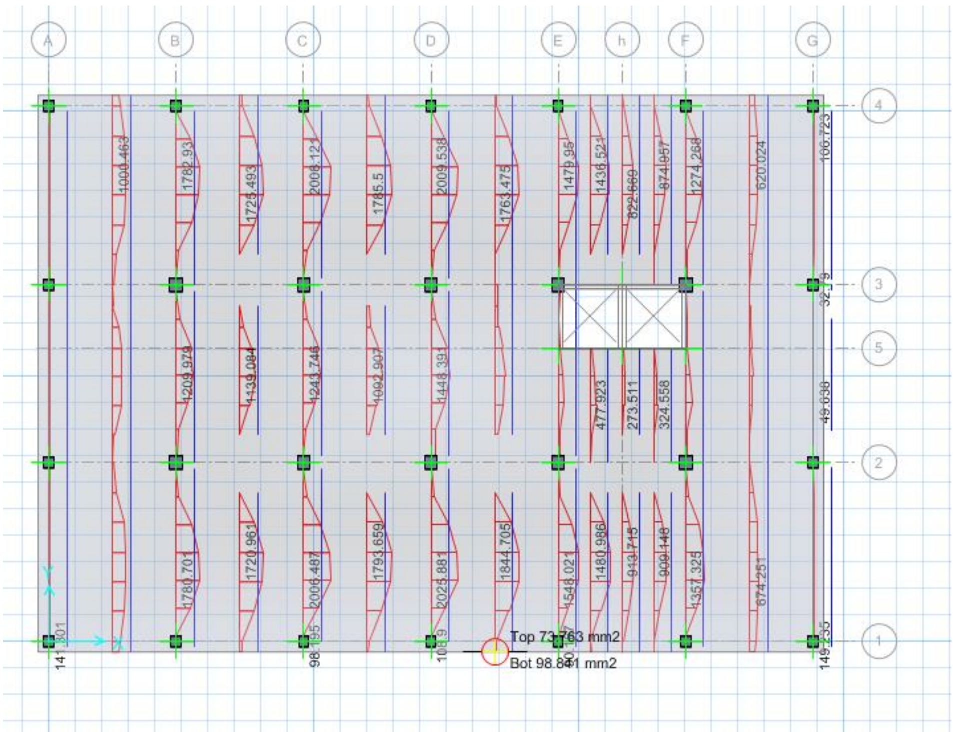


Figure (4.11): Flat slab bottom reinforcement for span 5m EC2-1992code:

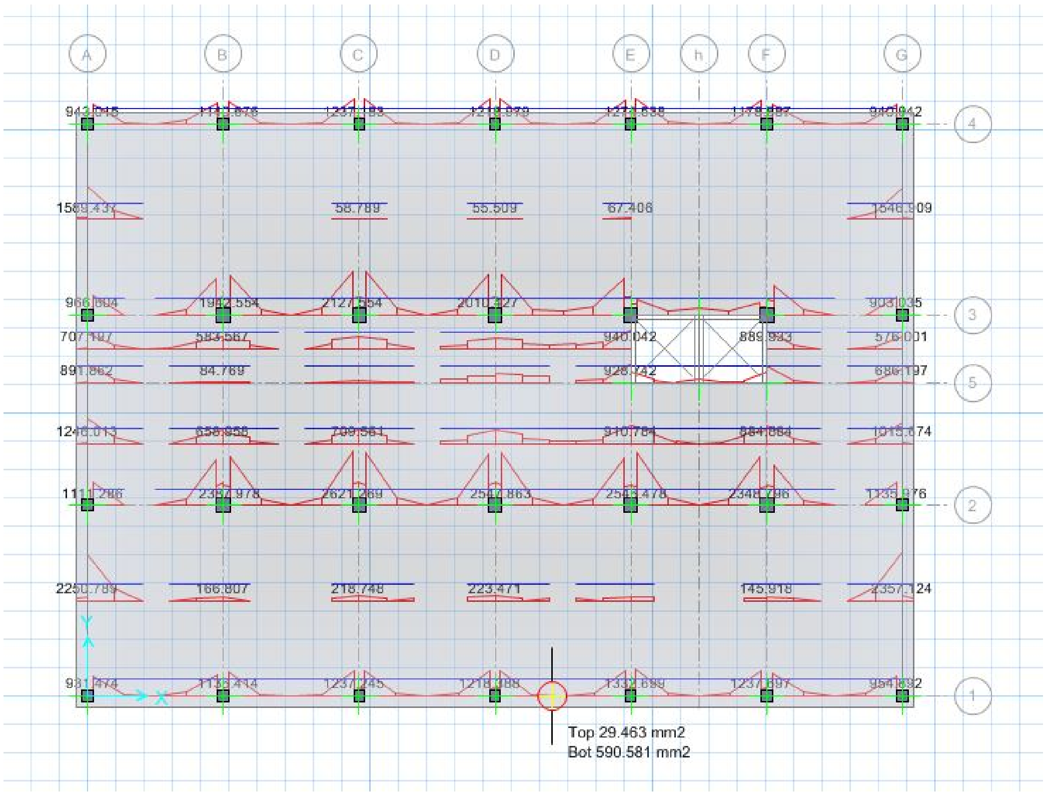


Figure (4.12): Flat slab top reinforcement for span 7m EC2-1992code:

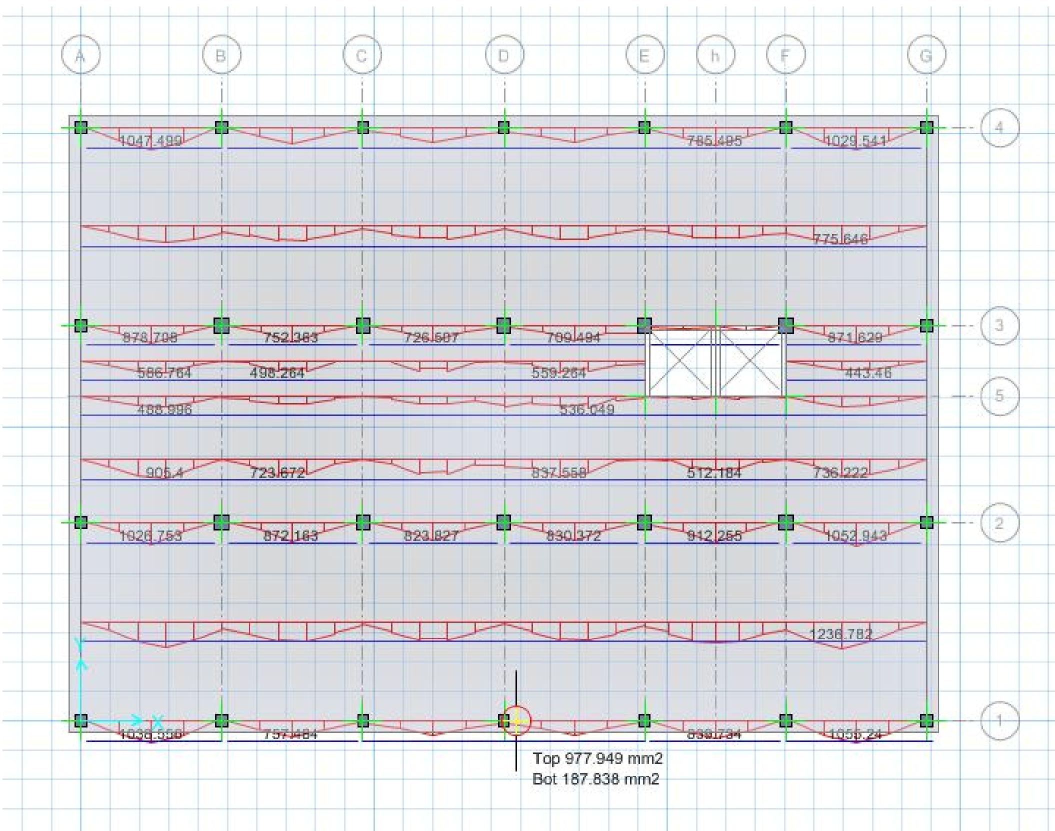


Figure (4.13): Flat slab Bottom reinforcement for span 7m EC2-1992code:

4.3.2 Column design:

1- BS8110-1997code:

Output design for Etabsprogramme for BSI -8110 1997:

Internal columns:

From storey 20 to storey 16:

Use 8$\emptyset 25mm$

From storey 15 to storey 11:

Use 18$\emptyset 25mm$

From storey 10 to storey 6:

Use 18$\emptyset 32mm$

From storey 5 to storey 1:

Use 22$\emptyset 32mm$

Output design for Etabsprogramme for BSI -8110 1997:

Edge and Corner columns:

From storey 20 to storey 16:

Use4$\emptyset 16mm$

From storey 15 to storey 11:

Use 6$\emptyset 16mm$

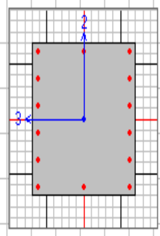
From storey 10 to storey 6:

Use 6$\emptyset 20mm$

From storey 5 to storey 1:

Use 6$\emptyset 20mm$

Units KN-m



BS8110 97 COLUMN SECTION DESIGN Type: Sway Frame Units: KN-m (Summary)

Level : STORY1	L=3.400		
Element : C2	B=0.850	D=0.850	dc=0.046
Section ID : C.85	E=24821128.40	fcu=35000.000	Lt.Wt. Fac.=1.000
Combo ID : DCON2	fy=460000.000	fyv=250000.000	
Station Loc : 0.000	RLLF=0.400		

Gamma(Concrete): 1.500
Gamma(Steel) : 1.050

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3

Rebar Area	Design N	Design M2	Design M3	Minimum M2	Minimum M3
0.013	14938.252	-62.458	-298.765	298.765	298.765

AXIAL FORCE & BIAXIAL MOMENT FACTORS

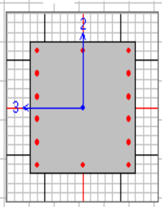
	Mi Moment	Madd Moment	Beta Factor	L Length
Major Bending(M3)	-6.306	-101.580	1.000	3.400
Minor Bending(M2)	5.904	-101.580	1.000	3.400

SHEAR DESIGN FOR V2,V3

	Rebar Asv/s	Shear V	Shear Vc/GammaM	Shear Vs/GammaM
Major Shear(V2)	0.001	0.135	633.023	273.462
Minor Shear(V3)	0.001	6.672	2705.719	273.462

Figure (4.14): BS8110-1997 internal column from storey1 to storey5

Units KN-m



BS8110 97 COLUMN SECTION DESIGN Type: Sway Frame Units: KN-m (Summary)

Level : STORY6	L=3.400		
Element : C2	B=0.700	D=0.700	dc=0.046
Section ID : C.7	E=24821128.40	fcu=35000.000	Lt.Wt. Fac.=1.000
Combo ID : DCON2	fy=460000.000	fyv=250000.000	
Station Loc : 0.000	RLLF=0.400		

Gamma(Concrete): 1.500
Gamma(Steel) : 1.050

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3

Rebar Area	Design N	Design M2	Design M3	Minimum M2	Minimum M3
0.012	10933.963	-98.316	-218.679	218.679	218.679

AXIAL FORCE & BIAXIAL MOMENT FACTORS

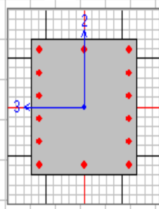
	Mi Moment	Madd Moment	Beta Factor	L Length
Major Bending(M3)	-4.627	-90.283	1.000	3.400
Minor Bending(M2)	9.456	-90.283	1.000	3.400

SHEAR DESIGN FOR V2,V3

	Rebar Asv/s	Shear V	Shear Vc/GammaM	Shear Vs/GammaM
Major Shear(V2)	0.001	5.446	1993.360	183.204
Minor Shear(V3)	0.001	12.953	1993.360	183.204

Figure (4.15): BS8110-1997 internal column from storey6 to storey10

Units KN-m



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BS8110 97 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY11          L=3.400
Element      : C9               B=0.600        D=0.600        dc=0.046
Section ID   : C.6              E=24821128.40  fcu=35000.000  Lt.Wt. Fac.=1.000
Combo ID     : DCON2            fy=460000.000  fyv=250000.000
Station Loc  : 0.000            RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.050

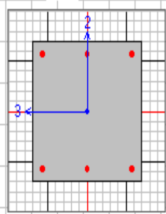
AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3
                Rebar     Design     Design     Design    Minimum    Minimum
                 Area          N         M2         M3         M2         M3
                0.007   7254.587     80.386   -145.092    145.092    145.092

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                           Mi       Madd       Beta          L
                       Moment     Moment     Factor     Length
   Major Bending(M3)   11.080    -69.886      1.000      3.400
   Minor Bending(M2)   -9.257     69.886      1.000      3.400

SHEAR DESIGN FOR V2,V3
                        Rebar      Shear      Shear      Shear
                        Asv/s          V  Vc/GammaM  Vs/GammaM
   Major Shear(V2)      0.001     15.946   1328.965    133.032
   Minor Shear(V3)      0.001     12.689   1328.965    133.032
```

Figure (4.16): BS8110-1997 internal column from storey11 to storey15

Units KN-m



```
BS8110 97 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY16          L=3.400
Element      : C2               B=0.500        D=0.500        dc=0.046
Section ID   : C.5              E=24821128.40  fcu=35000.000  Lt.Wt. Fac.=1.000
Combo ID     : DCON26           fy=460000.000  fyv=250000.000
Station Loc  : 3.400            RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.050

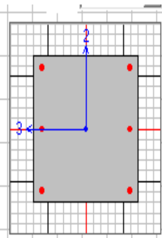
AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3
                Rebar     Design     Design     Design    Minimum    Minimum
                 Area          N         M2         M3         M2         M3
                0.001   1980.505     49.173     39.610     39.610     39.610

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                           Mi       Madd       Beta          L
                       Moment     Moment     Factor     Length
   Major Bending(M3)    6.577     22.895      1.000      3.400
   Minor Bending(M2)   10.511     22.895      1.000      3.400

SHEAR DESIGN FOR V2,V3
                        Rebar      Shear      Shear      Shear
                        Asv/s          V  Vc/GammaM  Vs/GammaM
   Major Shear(V2)   8.400E-04     9.172    428.395     90.860
   Minor Shear(V3)   8.400E-04    14.513    425.118     90.860
```

Figure (4.17): BS8110-1997 internal column from storey16 to storey20

BS8110 97 COLUMN SECTION DESIGN Type: Sway Frame Units: KN-m (Summary)



Level : STORY1 L=3.400
Element : C23 B=0.550 D=0.550 dc=0.046
Section ID : C.55 E=24821128.40 fcu=35000.000 Lt.Wt. Fac.=1.000
Combo ID : DCON2 fy=460000.000 fyv=250000.000
Station Loc : 3.400 RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel) : 1.050

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3

Rebar Area	Design N	Design M2	Design M3	Minimum M2	Minimum M3
0.011	7566.210	170.962	-151.324	151.324	151.324

AXIAL FORCE & BIAXIAL MOMENT FACTORS

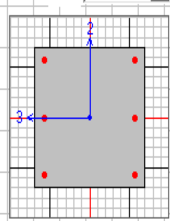
	Mi Moment	Madd Moment	Beta Factor	L Length
Major Bending(M3)	-1.178	-79.514	1.000	3.400
Minor Bending(M2)	37.464	79.514	1.000	3.400

SHEAR DESIGN FOR V2,V3

	Rebar Asv/s	Shear V	Shear Vc/GammaM	Shear Vs/GammaM
Major Shear(V2)	9.240E-04	0.150	672.604	110.946
Minor Shear(V3)	9.240E-04	39.694	1312.731	110.946

Figure (4.18): BS8110-1997 edge and corner column from storey1 to storey5

BS8110 97 COLUMN SECTION DESIGN Type: Sway Frame Units: KN-m (Summary)



Level : STORY6 L=3.400
Element : C24 B=0.500 D=0.500 dc=0.046
Section ID : C.500 E=24821128.40 fcu=35000.000 Lt.Wt. Fac.=1.000
Combo ID : DCON2 fy=460000.000 fyv=250000.000
Station Loc : 3.400 RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel) : 1.050

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3

Rebar Area	Design N	Design M2	Design M3	Minimum M2	Minimum M3
0.007	5435.745	156.864	108.715	108.715	108.715

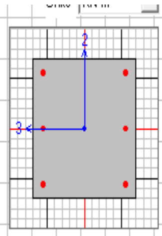
AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Mi Moment	Madd Moment	Beta Factor	L Length
Major Bending(M3)	5.436	62.837	1.000	3.400
Minor Bending(M2)	37.611	62.837	1.000	3.400

SHEAR DESIGN FOR V2,V3

	Rebar Asv/s	Shear V	Shear Vc/GammaM	Shear Vs/GammaM
Major Shear(V2)	8.400E-04	7.944	1017.425	90.860
Minor Shear(V3)	8.400E-04	53.263	1017.425	90.860

Figure (4.19): BS8110-1997 edge and corner column from storey6 to storey10



```
BS8110 97 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY11           L=3.400
Element      : C13               B=0.450         D=0.450         dc=0.046
Section ID   : C.45              E=24821128.40   fcu=35000.000   Lt.Wt. Fac.=1.000
Combo ID     : DCON2             fy=460000.000   fyv=250000.000
Station Loc  : 3.400             RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.050

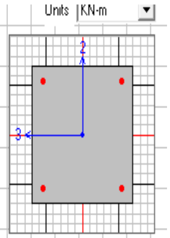
AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3
                 Rebar     Design     Design     Design    Minimum    Minimum
                  Area          N         M2         M3         M2         M3
                 0.004   3568.028   -131.363     71.361     71.361     71.361

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                               Mi       Madd       Beta          L
                           Moment     Moment     Factor     Length
   Major Bending(M3)        8.492     45.829      1.000      3.400
   Minor Bending(M2)      -34.213    -45.829      1.000      3.400

SHEAR DESIGN FOR V2,V3
                            Rebar      Shear      Shear      Shear
                            Asv/s          V Vc/GammaM Vs/GammaM
   Major Shear(V2)      7.560E-04     12.124    677.216     72.774
   Minor Shear(V3)      7.560E-04     48.295    670.987     72.774
```

Figure (4.20): BS8110-1997 edge and corner column from storey11 to storey15

Units KN-m



```
BS8110 97 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY16           L=3.400
Element      : C13               B=0.400         D=0.400         dc=0.046
Section ID   : C.4               E=24821128.40   fcu=35000.000   Lt.Wt. Fac.=1.000
Combo ID     : DCON20            fy=460000.000   fyv=250000.000
Station Loc  : 3.400             RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.050

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR N, M2, M3
                 Rebar     Design     Design     Design    Minimum    Minimum
                  Area          N         M2         M3         M2         M3
                 0.001   1401.443    -85.296    123.152     28.029     28.029

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                               Mi       Madd       Beta          L
                           Moment     Moment     Factor     Length
   Major Bending(M3)       41.160     20.251      1.000      3.400
   Minor Bending(M2)      -26.018    -20.251      1.000      3.400

SHEAR DESIGN FOR V2,V3
                            Rebar      Shear      Shear      Shear
                            Asv/s          V Vc/GammaM Vs/GammaM
   Major Shear(V2)      6.720E-04     57.553    264.399     56.688
   Minor Shear(V3)      6.720E-04     36.634    265.711     56.688
```

Figure (4.21)BS8110-1997 edge and corner column from storey16 to storey20

2- ACI-2005 code:

Output design for Etabsprogramme for ACI -2005:

Internal columns:

From storey 20 to storey 16:

Use 10 $\emptyset 25mm$

From storey 15 to storey 11:

Use 18$\emptyset 25mm$

From storey 10 to storey 6:

Use 18$\emptyset 32mm$

From storey 5 to storey 1:

Use 22$\emptyset 32mm$

Output design for Etabsprogramme for ACI -2005:

Edge and Corner columns:

From storey 20 to storey 16:

Use4$\emptyset 16mm$

From storey 15 to storey 11:

Use 6$\emptyset 16mm$

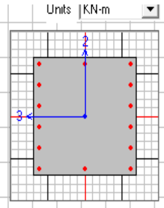
From storey 10 to storey 6:

Use 6$\emptyset 20mm$

From storey 5 to storey 1:

Use 6$\emptyset 20mm$

ACI 318-05/IBC 2003 COLUMN SECTION DESIGN Type: Sway Special Units: KN-m (Summary)



Level : STORY1	L=3.400		
Element : C4	B=0.850	D=0.850	dc=0.046
Section ID : C.85	E=24821128.40	fc=35000.000	Lt.Wt. Fac.=1.000
Combo ID : DCON16	fy=460000.000	fys=250000.000	
Station Loc : 0.000	RLLF=0.400		

Phi(Compression-Spiral): 0.700 Overstrength Factor: 1.25
Phi(Compression-Tied): 0.650
Phi(Tension Controlled): 0.900
Phi(Shear): 0.750
Phi(Seismic Shear): 0.600
Phi(Joint Shear): 0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3

Rebar Area	Design Pu	Design M2	Design M3	Minimum M2	Minimum M3
0.027	16297.820	-663.973	-643.197	663.973	663.973

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Cm Factor	Delta_ns Factor	Delta_s Factor	K Factor	L Length
Major Bending(M3)	0.575	1.000	1.000	1.000	3.400
Minor Bending(M2)	0.400	1.000	1.000	1.000	3.400

SHEAR DESIGN FOR V2,V3

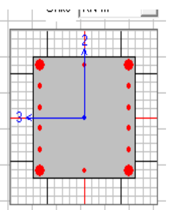
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
Major Shear(V2)	0.000	201.034	1062.263	0.000	0.000
Minor Shear(V3)	0.000	29.668	1062.263	0.000	0.000

JOINT SHEAR DESIGN

	Joint Shear Ratio	Shear VuTop	Shear VuTot	Shear phi*Vc	Joint Area
Major Shear(V2)	N/A	N/A	N/A	N/A	N/A
Minor Shear(V3)	N/A	N/A	N/A	N/A	N/A

Figure (4.22): ACI-2005 internal column from storey1 to storey 5

ACI 318-05/IBC 2003 COLUMN SECTION DESIGN Type: Sway Special Units: KN-m (Summary)



Level : STORY6	L=3.400		
Element : C9	B=0.700	D=0.700	dc=0.046
Section ID : C.7	E=24821128.40	fc=35000.000	Lt.Wt. Fac.=1.000
Combo ID : DCON15	fy=460000.000	fys=250000.000	
Station Loc : 0.000	RLLF=0.400		

Phi(Compression-Spiral): 0.700 Overstrength Factor: 1.25
Phi(Compression-Tied): 0.650
Phi(Tension Controlled): 0.900
Phi(Shear): 0.750
Phi(Seismic Shear): 0.600
Phi(Joint Shear): 0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3

Rebar Area	Design Pu	Design M2	Design M3	Minimum M2	Minimum M3
0.015	10376.615	19.373	376.049	376.049	376.049

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Cm Factor	Delta_ns Factor	Delta_s Factor	K Factor	L Length
Major Bending(M3)	0.400	1.000	1.000	1.000	3.400
Minor Bending(M2)	0.400	1.000	1.000	1.000	3.400

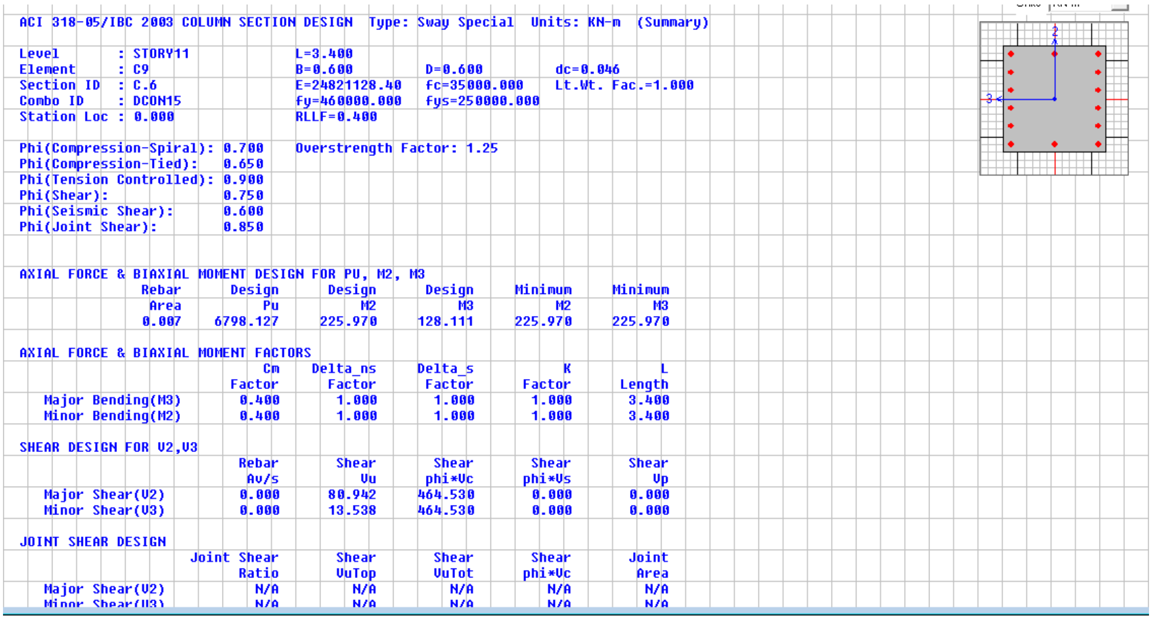
SHEAR DESIGN FOR V2,V3

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
Major Shear(V2)	0.000	80.367	684.621	0.000	0.000
Minor Shear(V3)	0.000	12.759	684.621	0.000	0.000

JOINT SHEAR DESIGN

	Joint Shear Ratio	Shear VuTop	Shear VuTot	Shear phi*Vc	Joint Area
Major Shear(V2)	N/A	N/A	N/A	N/A	N/A
Minor Shear(V3)	N/A	N/A	N/A	N/A	N/A

Figure (4.23): ACI-2005 internal column from storey6 to storey10



```
ACI 318-05/IBC 2003 COLUMN SECTION DESIGN  Type: Sway Special  Units: KN-m  (Summary)

Level       : STORY11             L=3.400
Element     : C9                  B=0.600          D=0.600          dc=0.046
Section ID  : C.6                 E=24821128.40    fc=35000.000     Lt.Wt. Fac.=1.000
Combo ID    : DCON15              fy=460000.000    fys=250000.000
Station Loc : 0.000               RLLF=0.400

Phi(Compression-Spiral): 0.700    Overstrength Factor: 1.25
Phi(Compression-Tied):   0.650
Phi(Tension Controlled): 0.900
Phi(Shear):              0.750
Phi(Seismic Shear):      0.600
Phi(Joint Shear):        0.850

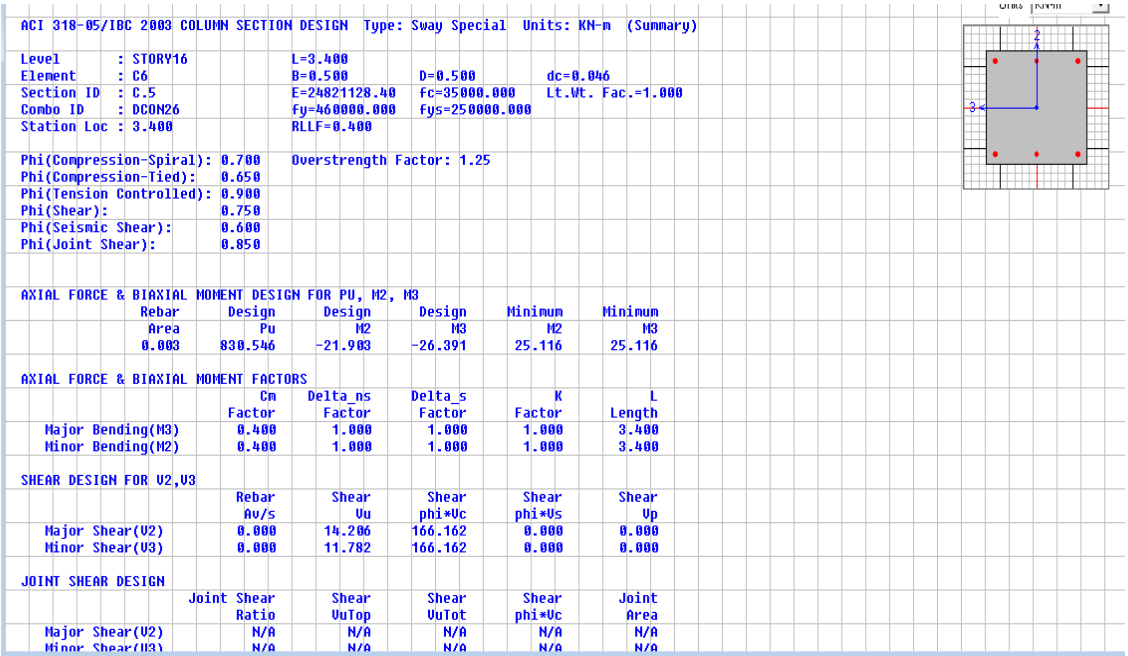
AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                Rebar     Design     Design     Design    Minimum    Minimum
                 Area         Pu         M2         M3         M2         M3
                0.007   6798.127    225.970    128.111    225.970    225.970

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                           Cm   Delta_ns   Delta_s          K          L
                       Factor     Factor    Factor     Factor     Length
   Major Bending(M3)    0.400      1.000     1.000      1.000      3.400
   Minor Bending(M2)    0.400      1.000     1.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                        Rebar      Shear     Shear      Shear      Shear
                         Av/s         Vu    phi*Vc     phi*Vs         Vp
   Major Shear(V2)      0.000     80.942   464.530      0.000      0.000
   Minor Shear(V3)      0.000     13.538   464.530      0.000      0.000

JOINT SHEAR DESIGN
                  Joint Shear      Shear     Shear      Shear      Joint
                        Ratio      VuTop     VuTot     phi*Vc       Area
   Major Shear(V2)        N/A        N/A       N/A        N/A        N/A
   Minor Shear(V3)        N/A        N/A       N/A        N/A        N/A
```

Figure (4.24): ACI-2005 internal column from storey11 to storey15



```
ACI 318-05/IBC 2003 COLUMN SECTION DESIGN  Type: Sway Special  Units: KN-m  (Summary)

Level       : STORY16             L=3.400
Element     : C6                  B=0.500          D=0.500          dc=0.046
Section ID  : C.5                 E=24821128.40    fc=35000.000     Lt.Wt. Fac.=1.000
Combo ID    : DCON26              fy=460000.000    fys=250000.000
Station Loc : 3.400               RLLF=0.400

Phi(Compression-Spiral): 0.700    Overstrength Factor: 1.25
Phi(Compression-Tied):   0.650
Phi(Tension Controlled): 0.900
Phi(Shear):              0.750
Phi(Seismic Shear):      0.600
Phi(Joint Shear):        0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                Rebar     Design     Design     Design    Minimum    Minimum
                 Area         Pu         M2         M3         M2         M3
                0.003    830.546    -21.903    -26.391     25.116     25.116

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                           Cm   Delta_ns   Delta_s          K          L
                       Factor     Factor    Factor     Factor     Length
   Major Bending(M3)    0.400      1.000     1.000      1.000      3.400
   Minor Bending(M2)    0.400      1.000     1.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                        Rebar      Shear     Shear      Shear      Shear
                         Av/s         Vu    phi*Vc     phi*Vs         Vp
   Major Shear(V2)      0.000     14.206   166.162      0.000      0.000
   Minor Shear(V3)      0.000     11.782   166.162      0.000      0.000

JOINT SHEAR DESIGN
                  Joint Shear      Shear     Shear      Shear      Joint
                        Ratio      VuTop     VuTot     phi*Vc       Area
   Major Shear(V2)        N/A        N/A       N/A        N/A        N/A
   Minor Shear(V3)        N/A        N/A       N/A        N/A        N/A
```

Figure (4.25):ACI-2005 internal column from storey16 to storey20

```
ACI 318-05/IBC 2003 COLUMN SECTION DESIGN  Type: Sway Special  Units: KN-m  (Summary)

Level        : STORY1             L=3.400
Element      : C24                B=0.550          D=0.550          dc=0.046
Section ID   : C.55               E=24821128.40    fc=35000.000     Lt.Wt. Fac.=1.000
Combo ID     : DCON18             fy=460000.000    fys=250000.000
Station Loc  : 0.000              RLLF=0.400

Phi(Compression-Spiral): 0.700    Overstrength Factor: 1.25
Phi(Compression-Tied):   0.650
Phi(Tension Controlled): 0.900
Phi(Shear):              0.750
Phi(Seismic Shear):      0.600
Phi(Joint Shear):        0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                     Rebar     Design     Design     Design    Minimum    Minimum
                      Area         Pu         M2         M3         M2         M3
                     0.012   6973.616   -221.343     -1.525    221.343    221.343

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                               Cm   Delta_ns    Delta_s          K          L
                           Factor     Factor     Factor     Factor     Length
   Major Bending(M3)        0.773      1.055      1.000      1.000      3.400
   Minor Bending(M2)        0.400      1.000      1.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                            Rebar      Shear      Shear      Shear      Shear
                             Av/s         Vu    phi*Vc     phi*Vs         Vp
   Major Shear(V2)          0.000      0.241    436.848      0.000      0.000
   Minor Shear(V3)          0.000     40.495    436.848      0.000      0.000

JOINT SHEAR DESIGN
                      Joint Shear      Shear      Shear      Shear      Joint
                            Ratio      VuTop      VuTot     phi*Vc       Area
   Major Shear(V2)            N/A        N/A        N/A        N/A        N/A
   Minor Shear(V3)            N/A        N/A        N/A        N/A        N/A
```

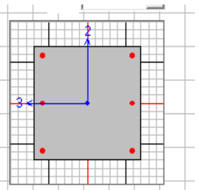


Figure (4.26): ACI-2005 edge and corner column from storey1 to storey5

```
ACI 318-05/IBC 2003 COLUMN SECTION DESIGN  Type: Sway Special  Units: KN-m  (Summary)

Level        : STORY6             L=3.400
Element      : C24                B=0.500          D=0.500          dc=0.046
Section ID   : C.500              E=24821128.40    fc=35000.000     Lt.Wt. Fac.=1.000
Combo ID     : DCON18             fy=460000.000    fys=250000.000
Station Loc  : 0.000              RLLF=0.400

Phi(Compression-Spiral): 0.700    Overstrength Factor: 1.25
Phi(Compression-Tied):   0.650
Phi(Tension Controlled): 0.900
Phi(Shear):              0.750
Phi(Seismic Shear):      0.600
Phi(Joint Shear):        0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                     Rebar     Design     Design     Design    Minimum    Minimum
                      Area         Pu         M2         M3         M2         M3
                     0.007   5175.152    -89.479   -156.497    156.497    156.497

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                               Cm   Delta_ns    Delta_s          K          L
                           Factor     Factor     Factor     Factor     Length
   Major Bending(M3)        0.400      1.000      1.000      1.000      3.400
   Minor Bending(M2)        0.400      1.000      1.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                            Rebar      Shear      Shear      Shear      Shear
                             Av/s         Vu    phi*Vc     phi*Vs         Vp
   Major Shear(V2)          0.000      6.844    334.914      0.000      0.000
   Minor Shear(V3)          0.000     53.831    334.914      0.000      0.000

JOINT SHEAR DESIGN
                      Joint Shear      Shear      Shear      Shear      Joint
                            Ratio      VuTop      VuTot     phi*Vc       Area
   Major Shear(V2)            N/A        N/A        N/A        N/A        N/A
   Minor Shear(V3)            N/A        N/A        N/A        N/A        N/A
```

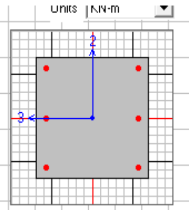


Figure (4.27): ACI-2005 edge and corner column from storey6 to storey10

```
ACI 318-05/IBC 2003 COLUMN SECTION DESIGN  Type: Sway Special  Units: KN-m  (Summary)

Level        : STORY11            L=3.400
Element      : C23                B=0.450            D=0.450            dc=0.046
Section ID   : C.45               E=24821128.40      fc=35000.000       Lt.Wt. Fac.=1.000
Combo ID     : DCON16             fy=460000.000      fys=250000.000
Station Loc  : 3.400              RLLF=0.400

Phi(Compression-Spiral): 0.700    Overstrength Factor: 1.25
Phi(Compression-Tied):   0.650
Phi(Tension Controlled): 0.900
Phi(Shear):              0.750
Phi(Seismic Shear):      0.600
Phi(Joint Shear):        0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                  Rebar      Design     Design     Design     Minimum    Minimum
                   Area          Pu         M2         M3         M2         M3
                  0.003    3496.539    100.491    110.890    100.491    100.491

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                                 Cm   Delta_ns    Delta_s          K          L
                             Factor     Factor     Factor     Factor     Length
   Major Bending(M3)          0.400      1.000      1.000      1.000      3.400
   Minor Bending(M2)          0.400      1.000      1.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                              Rebar      Shear      Shear      Shear      Shear
                               Av/s         Vu     phi*Vc     phi*Vs         Vp
   Major Shear(V2)            0.000     63.189    241.542      0.000      0.000
   Minor Shear(V3)            0.000     48.154    241.542      0.000      0.000

JOINT SHEAR DESIGN
                        Joint Shear      Shear      Shear      Shear      Joint
                              Ratio      VuTop      VuTot     phi*Vc       Area
   Major Shear(V2)              N/A        N/A        N/A        N/A        N/A
   Minor Shear(V3)              N/A        N/A        N/A        N/A        N/A
```

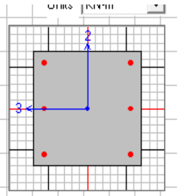


Figure (4.28): ACI-2005 edge and corner column from storey11 to storey15

```
ACI 318-05/IBC 2003 COLUMN SECTION DESIGN  Type: Sway Special  Units: KN-m  (Summary)

Level        : STORY16            L=3.400
Element      : C23                B=0.400            D=0.400            dc=0.046
Section ID   : C.4                E=24821128.40      fc=35000.000       Lt.Wt. Fac.=1.000
Combo ID     : DCON26             fy=460000.000      fys=250000.000
Station Loc  : 3.400              RLLF=0.400

Phi(Compression-Spiral): 0.700    Overstrength Factor: 1.25
Phi(Compression-Tied):   0.650
Phi(Tension Controlled): 0.900
Phi(Shear):              0.750
Phi(Seismic Shear):      0.600
Phi(Joint Shear):        0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                  Rebar      Design     Design     Design     Minimum    Minimum
                   Area          Pu         M2         M3         M2         M3
                  0.002     773.956     44.642     21.083     21.083     21.083

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                                 Cm   Delta_ns    Delta_s          K          L
                             Factor     Factor     Factor     Factor     Length
   Major Bending(M3)          0.400      1.000      1.000      1.000      3.400
   Minor Bending(M2)          0.400      1.000      1.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                              Rebar      Shear      Shear      Shear      Shear
                               Av/s         Vu     phi*Vc     phi*Vs         Vp
   Major Shear(V2)            0.000      2.182    112.848      0.000      0.000
   Minor Shear(V3)            0.000     25.192    112.848      0.000      0.000

JOINT SHEAR DESIGN
                        Joint Shear      Shear      Shear      Shear      Joint
                              Ratio      VuTop      VuTot     phi*Vc       Area
   Major Shear(V2)              N/A        N/A        N/A        N/A        N/A
   Minor Shear(V3)              N/A        N/A        N/A        N/A        N/A
```

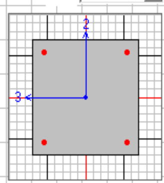


Figure (4.29): ACI-2005 edge and corner column from storey16to storey20

3- EC2-1992:

Output design for Etabsprogramme for EC2-1992:

Internal columns:

From storey 20 to storey 16:

Use 8Ø25mm

From storey 15 to storey 11:

Use 18Ø25mm

From storey 10 to storey 6:

Use 18Ø32mm

From storey 5 to storey 1:

Use 22Ø32mm

Output design for Etabsprogramme forEC2-1992:

Edge and Corner columns:

From storey 20 to storey 16:

Use4Ø16mm

From storey 15 to storey 11:

Use 6Ø16mm

From storey 10 to storey 6:

Use 6Ø20mm

From storey 5 to storey 1:

Use 6Ø20mm

```
EUROCODE 2-1992 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY1            L=3.400
Element      : C9                B=0.850          D=0.850         dc=0.046
Section ID   : C.85              E=24821128.40    fck,cyl=35000.  Lt.Wt. Fac.=1.000
Combo ID     : DCON2             fyk=460000.000   fywk=250000.00
Station Loc  : 0.000             RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.150

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NSd, MSd2, MSd3
                Rebar     Design      Design     Design    Minimum    Minimum
                 Area        NSd        MSd2       MSd3         M2         M3
                0.010  14468.128       5.832   -614.895    614.895    614.895

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                              Mi        Madd         Ma       Beta          L
                          Moment      Moment     Moment     Factor     Length
   Major Bending(M3)       0.000       0.000      0.000      1.000      3.400
   Minor Bending(M2)       0.000       0.000      0.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                           Rebar       Shear      Shear      Shear
                           Asw/s         VSd  Vc/GammaC  Vs/GammaS
   Major Shear(V2)         0.000       0.338   1351.699      0.000
   Minor Shear(V3)         0.000       6.624   1351.699      0.000
```

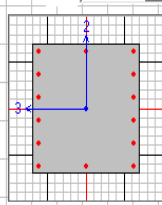


Figure (4.30): EC2-1992 internal column from storey1 to storey5

```
EUROCODE 2-1992 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY6            L=3.400
Element      : C9                B=0.700          D=0.700         dc=0.046
Section ID   : C.7               E=24821128.40    fck,cyl=35000.  Lt.Wt. Fac.=1.000
Combo ID     : DCON2             fyk=460000.000   fywk=250000.00
Station Loc  : 0.000             RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.150

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NSd, MSd2, MSd3
                Rebar     Design      Design     Design    Minimum    Minimum
                 Area        NSd        MSd2       MSd3         M2         M3
                0.009  10622.614      18.749   -371.791    371.791    371.791

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                              Mi        Madd         Ma       Beta          L
                          Moment      Moment     Moment     Factor     Length
   Major Bending(M3)       0.000       0.000      0.000      1.000      3.400
   Minor Bending(M2)       0.000       0.000      0.000      1.000      3.400

SHEAR DESIGN FOR V2,V3
                           Rebar       Shear      Shear      Shear
                           Asw/s         VSd  Vc/GammaC  Vs/GammaS
   Major Shear(V2)         0.000       7.717    723.758      0.000
   Minor Shear(V3)         0.000      12.316    723.758      0.000
```

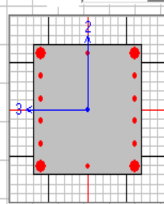
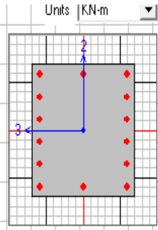


Figure (4.31): EC2-1992 internal column from storey6 to storey10

EUROCODE 2-1992 COLUMN SECTION DESIGN Type: Sway Frame Units: KN-m (Summary)

Level	: STORY11	L=3.400		
Element	: C9	B=0.600	D=0.600	dc=0.046
Section ID	: C.6	E=24821128.40	fck,cyl=35000.	Lt.Wt. Fac.=1.000
Combo ID	: DCON2	fyk=460000.000	fywk=250000.00	
Station Loc	: 0.000	RLLF=0.400		

Gamma(Concrete): 1.500
Gamma(Steel) : 1.150



AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NSd, MSd2, MSd3

Rebar Area	Design NSd	Design MSd2	Design MSd3	Minimum M2	Minimum M3
0.004	6957.421	19.177	-208.723	208.723	208.723

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Mi Moment	Madd Moment	Ma Moment	Beta Factor	L Length
Major Bending(M3)	0.000	0.000	0.000	1.000	3.400
Minor Bending(M2)	0.000	0.000	0.000	1.000	3.400

SHEAR DESIGN FOR V2,V3

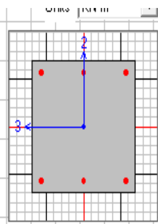
	Rebar Asw/s	Shear VSd	Shear Vc/GammaC	Shear Vs/GammaS
Major Shear(V2)	0.000	15.286	637.188	0.000
Minor Shear(V3)	0.000	12.167	637.188	0.000

Figure (4.32): EC2-1992 internal column from storey11 to storey15

EUROCODE 2-1992 COLUMN SECTION DESIGN Type: Sway Frame Units: KN-m (Summary)

Level	: STORY16	L=3.400		
Element	: C9	B=0.500	D=0.500	dc=0.046
Section ID	: C.5	E=24821128.40	fck,cyl=35000.	Lt.Wt. Fac.=1.000
Combo ID	: DCON2	fyk=460000.000	fywk=250000.00	
Station Loc	: 0.000	RLLF=0.400		

Gamma(Concrete): 1.500
Gamma(Steel) : 1.150



AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NSd, MSd2, MSd3

Rebar Area	Design NSd	Design MSd2	Design MSd3	Minimum M2	Minimum M3
0.001	3435.273	15.936	-85.882	85.882	85.882

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Mi Moment	Madd Moment	Ma Moment	Beta Factor	L Length
Major Bending(M3)	0.000	0.000	0.000	1.000	3.400
Minor Bending(M2)	0.000	0.000	0.000	1.000	3.400

SHEAR DESIGN FOR V2,V3

	Rebar Asw/s	Shear VSd	Shear Vc/GammaC	Shear Vs/GammaS
Major Shear(V2)	0.000	19.611	579.923	0.000
Minor Shear(V3)	0.000	10.152	579.923	0.000

Figure(4.33):EC2-1992 internal column from storey16 to storey20

```
EUROCODE 2-1992 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY1            L=3.400
Element      : C23               B=0.550         D=0.550          dc=0.046
Section ID   : C.55              E=24821128.40   fck,cyl=35000.   Lt.Wt. Fac.=1.000
Combo ID     : DCON2             fyk=460000.000  fywk=250000.00
Station Loc  : 0.000             RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.150

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NSd, MSd2, MSd3
                Rebar      Design      Design      Design     Minimum     Minimum
                 Area         NSd        MSd2        MSd3          M2          M3
                0.008    7290.147     -41.717    -200.479     200.479     200.479

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                              Mi        Madd          Ma        Beta           L
                          Moment      Moment      Moment      Factor      Length
   Major Bending(M3)       0.000       0.000       0.000       1.000       3.400
   Minor Bending(M2)       0.000       0.000       0.000       1.000       3.400

SHEAR DESIGN FOR V2,V3
                           Rebar       Shear       Shear       Shear
                           Asw/s         VSd   Vc/GammaC   Vs/GammaS
   Major Shear(V2)         0.000       0.144     312.448       0.000
   Minor Shear(V3)         0.000      38.057     312.448       0.000
```

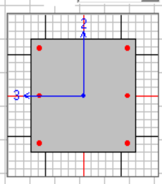


Figure (4.34): EC2-1992 edge and corner column from storey1 to storey5

```
EUROCODE 2-1992 COLUMN SECTION DESIGN  Type: Sway Frame  Units: KN-m  (Summary)

Level        : STORY6            L=3.400
Element      : C23               B=0.500         D=0.500          dc=0.046
Section ID   : C.500             E=24821128.40   fck,cyl=35000.   Lt.Wt. Fac.=1.000
Combo ID     : DCON2             fyk=460000.000  fywk=250000.00
Station Loc  : 0.000             RLLF=0.400

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.150

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NSd, MSd2, MSd3
                Rebar      Design      Design      Design     Minimum     Minimum
                 Area         NSd        MSd2        MSd3          M2          M3
                0.005    5444.604     -84.210    -136.115     136.115     136.115

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                              Mi        Madd          Ma        Beta           L
                          Moment      Moment      Moment      Factor      Length
   Major Bending(M3)       0.000       0.000       0.000       1.000       3.400
   Minor Bending(M2)       0.000       0.000       0.000       1.000       3.400

SHEAR DESIGN FOR V2,V3
                           Rebar       Shear       Shear       Shear
                           Asw/s         VSd   Vc/GammaC   Vs/GammaS
   Major Shear(V2)         0.000       1.982     334.512       0.000
   Minor Shear(V3)         0.000      51.520     334.512       0.000
```

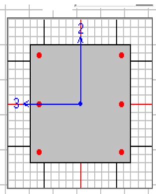
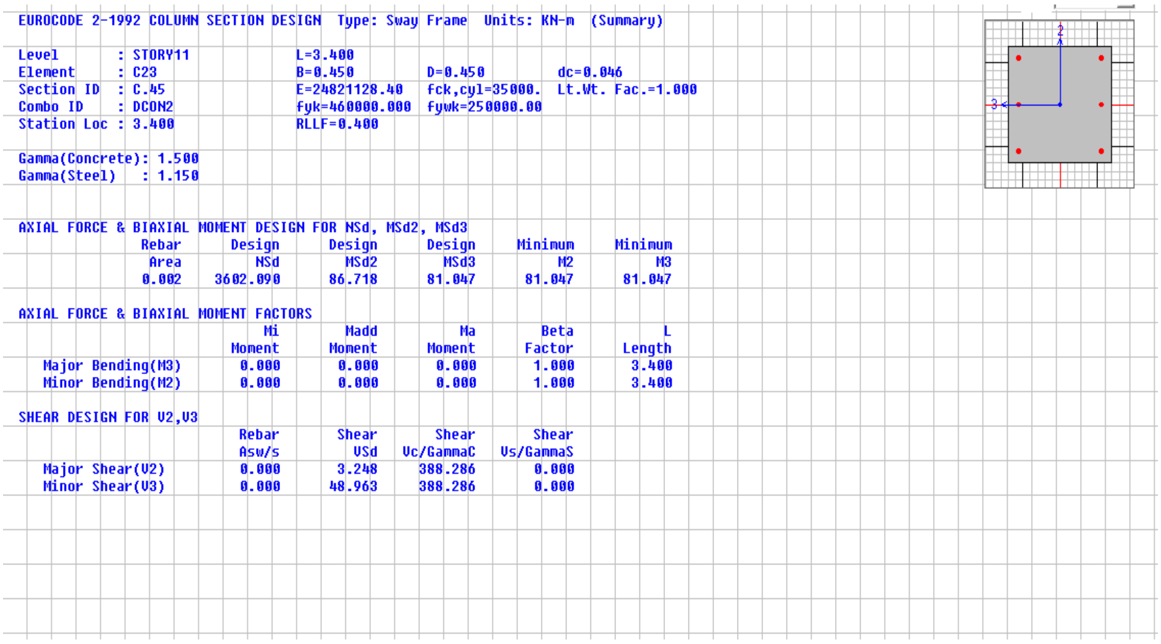
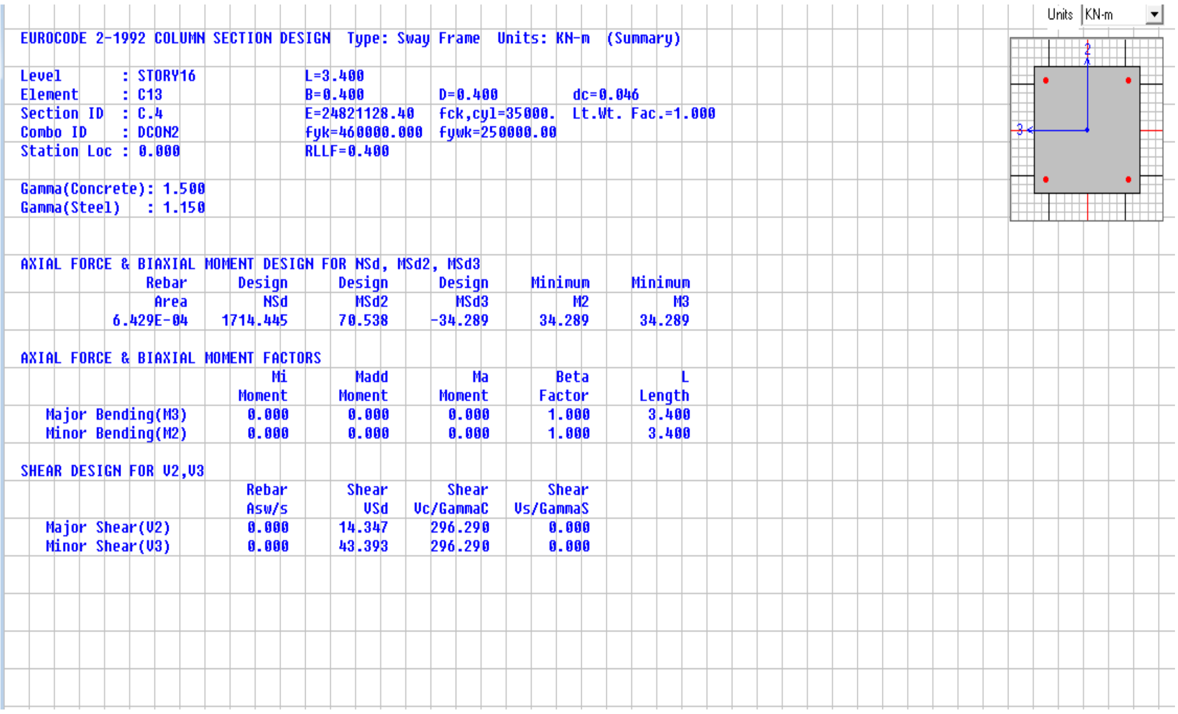


Figure (4.35): EC2-1992 edge and corner column from storey6 to storey10



Figure(4.36):EC2-1992 edge and corner column from storey11 to storey15



Figure(4.37):EC2-1992 edge and corner column from storey16 to storey20

4.3.3 Shear wall design:

1- BS8110-1997code:

Output design for Etabsprogramme for BSI -8110 1997:

Shear wall design:

Use $\emptyset 16mm$ @ 15cm c/c

Use $\emptyset 12mm$ @15cmc/c

Story ID: STORY1 Pier ID: P4 X Loc: 22.5 Y Loc: 14 Units: KN-m

Flexural Check for P-M2-M3 (RLLF = 0.400)

Station Location	D/C Ratio	Flexural Combo	P	M2	M3
Top	0.550	DWAL20	11651.431	3.844	-9249.404
Bottom	0.965	DWAL20	11822.791	-4.451	-24607.565

Shear Design For V2

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	1644.911	DWAL23	6822.806	8505.233	4543.379	2590.047	1953.332
Bot Leg 1	1627.632	DWAL23	6945.206	23952.721	4543.379	2610.566	1932.813

Figure (4.38): BS-8110-1997 shear wall

2- ACI-2005 code:

Output design for Etabsprogramme for ACI -2005:

Shear wall design:

Use $\emptyset 12mm@20cm\ c/c$

Use $\emptyset 12mm@20cm\ c/c$

Story ID: STORY1 Pier ID: P6 X Loc: 30 Y Loc: 10.5 Units: KN-m

Flexural Check for P-M2-M3 (RLLF = 0.400)

Station Location	D/C Ratio	Flexural Combo	Pu	M2u	M3u
Top	0.523	DWAL15	50091.412	251.350	17825.894
Bottom	0.531	DWAL15	50883.095	-922.228	19949.340

Shear Design

Station Location	Rebar mm^2/m	Shear Combo	Pu	Mu	Vu	Capacity phi Vc	Capacity phi Vn
Top Leg 1	750.000	DWAL18	39262.983	-130593.260	-3452.429	3138.711	5501.211
Bot Leg 1	750.000	DWAL18	40054.667	-142331.520	-3452.429	3138.711	5501.211

Boundary Element Check

Station Location	B-Zone Length	B-Zone Combo	Pu	Mu	Vu	Pu/Po
Top Leg 1	4.648	DWAL15	50091.412	17825.894	624.543	0.2926
Bot Leg 1	4.696	DWAL15	50883.095	19949.340	624.543	0.2973

Figure (4.39): AC1-2005- shear wall

3- EC2-1992:

Output design for Etabsprogramme for EC2-1992:

Shear wall design:

Use $\emptyset 12mm@15cm\ c/c$

Use $\emptyset 12mm@15cm\ c/c$

Story ID: STORY1 Pier ID: P5 X Loc: 0 Y Loc: 10.5 Units: KN-m

Flexural Design for P-M2-M3 (RLLF = 0.400)

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3	Pier Ag
Top	0.0025	0.0027	DWAL26	26269.244	-175.409	-102722.610	6.300
Bottom	0.0025	0.0027	DWAL26	26783.324	95.116	-113174.189	6.300

Shear Design For V2

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	505.263	DWAL22	36761.973	103426.194	3075.411	9069.041	2520.000
Bot Leg 1	505.263	DWAL22	37481.685	113882.591	3075.411	9152.601	2520.000

Figure (4.40): EC2-1992- shear wall

4.3.4 Raft foundation design:

1- BS8110-1997code:

Output design for Etabsprogramme for BSI -8110 1997 from figures blew:

Design result for raft foundation:

For span 5m:

Column strip =2500mm, middle strip =4500mm

Maximum top reinforcement for column strip =6704mm^2

UseT20@100mm c/c$< 3d$ (As=7850mm^2)

Maximum bottom reinforcement for column strip =6254mm^2

$> A_{smin}$

UseT20@125mm c/c$< 3d$ (As=6280mm^2)

Maximum top reinforcement for middle strip =2911mm^2

$> A_{smin}$

UseT16@300mm c/c$< 3d$ (As =3015mm^2)

Maximum bottom reinforcement for middle strip =4617mm^2

$> A_{smin}$

UseT20@300mm c/c$< 3d$ (As=4710mm^2)

For span 7m:

Column strip =2500mm, middle strip =2500mm

Maximum top reinforcement for column strip =7943mm^2

$> A_{smin}$

UseT25@150mm c/c$< 3d$ (As =8183mm^2)

Maximum bottom reinforcement for column strip =5849mm^2

$> A_{smin}$

UseT20@130mm c/c$< 3d$ (As =6038mm^2)

Maximum top reinforcement for middle strip =2616mm^2

$> A_{smin}$

UseT16@190mm c/c$< 3d$ (As =2645mm^2)

Maximum bottom reinforcement for middle strip =4918mm^2

$> A_{smin}$

UseT20@150mm

c/c$< 3d$ (As =5233mm^2)

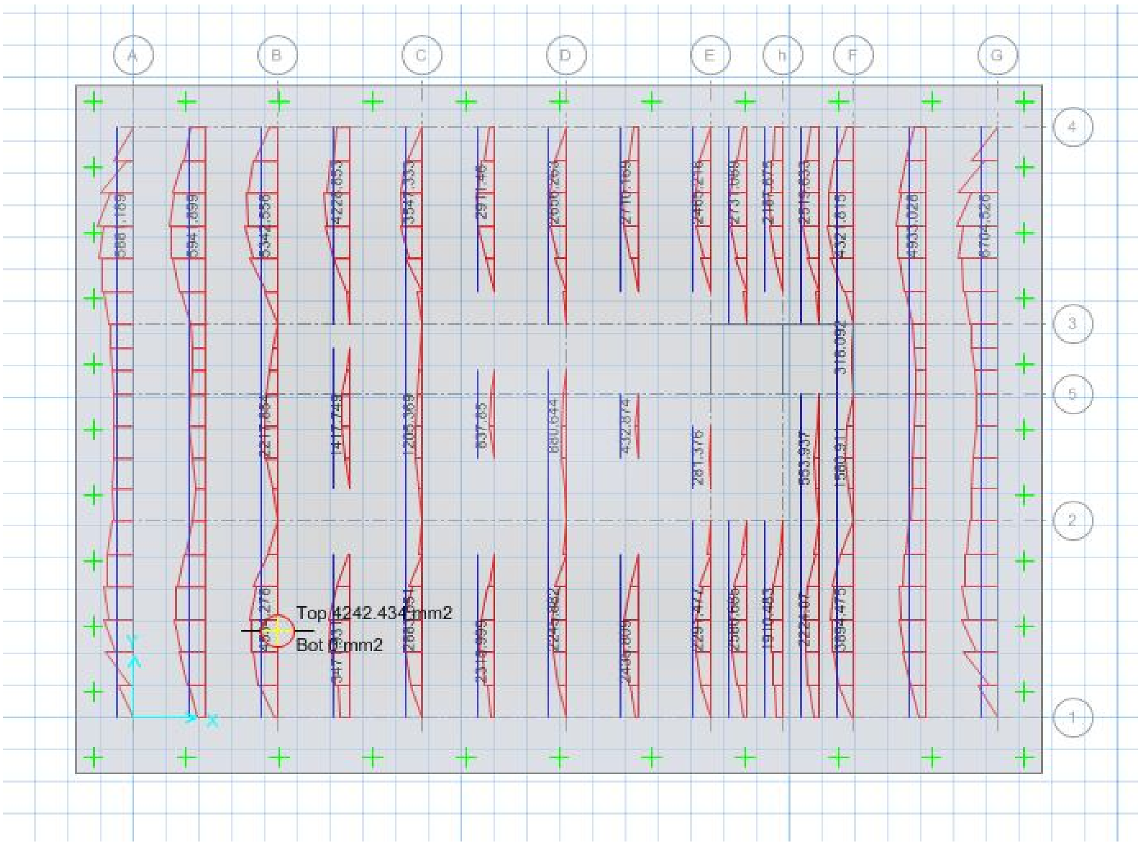


Figure (4.41): Raft foundation top reinforcement for span 5m for BS8110- code

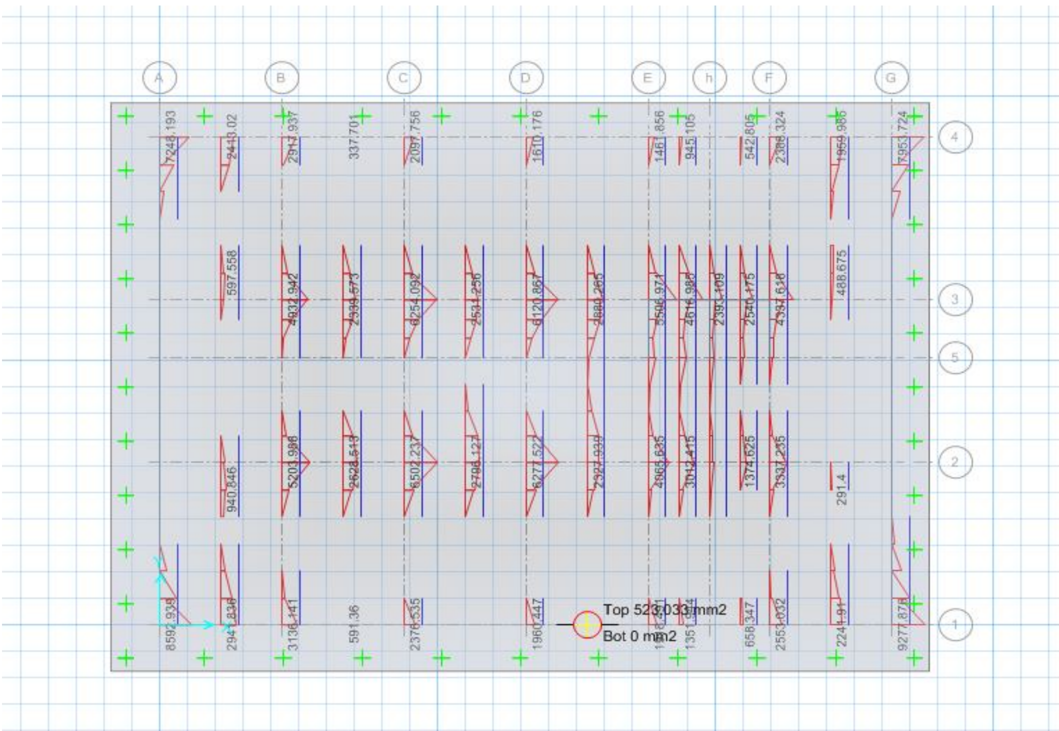


Figure (4.42): Raft foundation bottom reinforcement for span 5m for BS8110- code

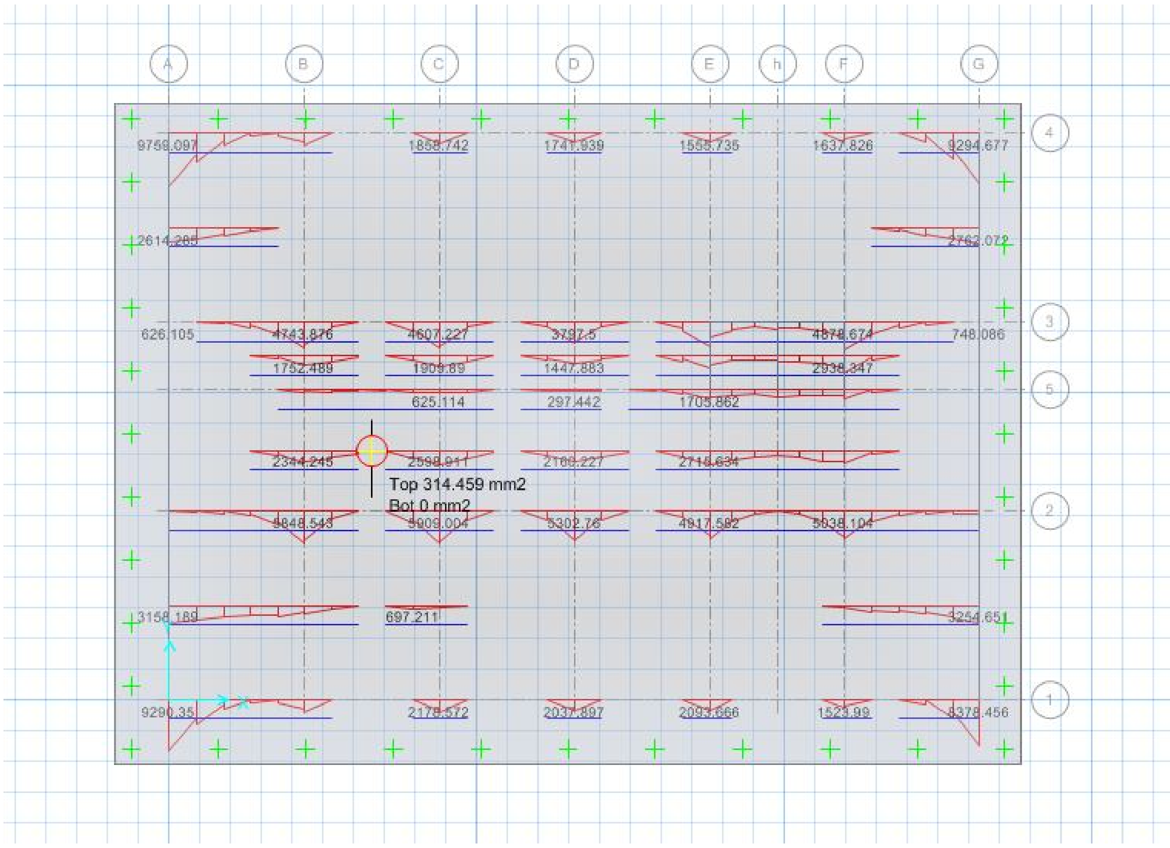


Figure (4.43): Raft foundation bottom reinforcement for span 7m for BS8110- code

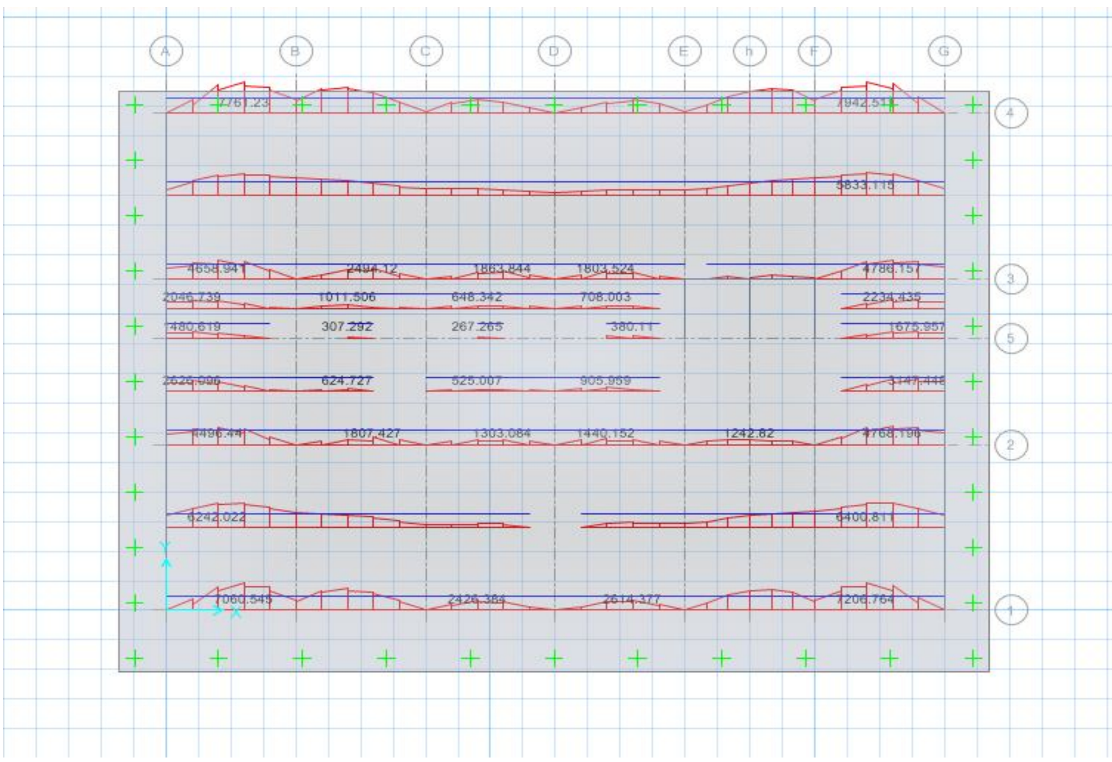


Figure (4.44): Raft foundation top reinforcement for span 7m for BS8110- code

2- ACI-2005 code:

Output design for Etabsprogramme for ACI -2005 fromfigure blew:

Result of design for raft foundation:

For span 5m:

Column strip =2500mm, middle strip =4500mm

Maximum top reinforcement for column strip= 7014mm^2

$> A_{smin}$

UseT20@100mm c/c$< 3d$ (As =7850mm^2)

Maximum bottom reinforcement for column strip in =6800mm^2

$> A_{smin}$

UseT20@110mm c/c$< 3d$ (As =7136mm^2)

Maximum top reinforcement for middle strip =2303mm^2

$> A_{smin}$

UseT16@350mm c/c$< 3d$ (As =2584mm^2)

Maximum bottom reinforcement for middle strip =4826mm^2

$> A_{smin}$

UseT20@250mm c/c$< 3d$ (As =5652mm^2)

For span 7m:

Column strip =2500mm, middle strip =2500mm

Maximum top reinforcement for column strip =8213mm^2

$> A_{smin}$

UseT25@140mm c/c$< 3d$ (As =8768mm^2)

Maximum bottom reinforcement for column strip =6162mm^2

$> A_{smin}$

UseT20@125mm c/c$< 3d$ (As =6280mm^2)

Maximum top reinforcement for middle strip =1720mm^2

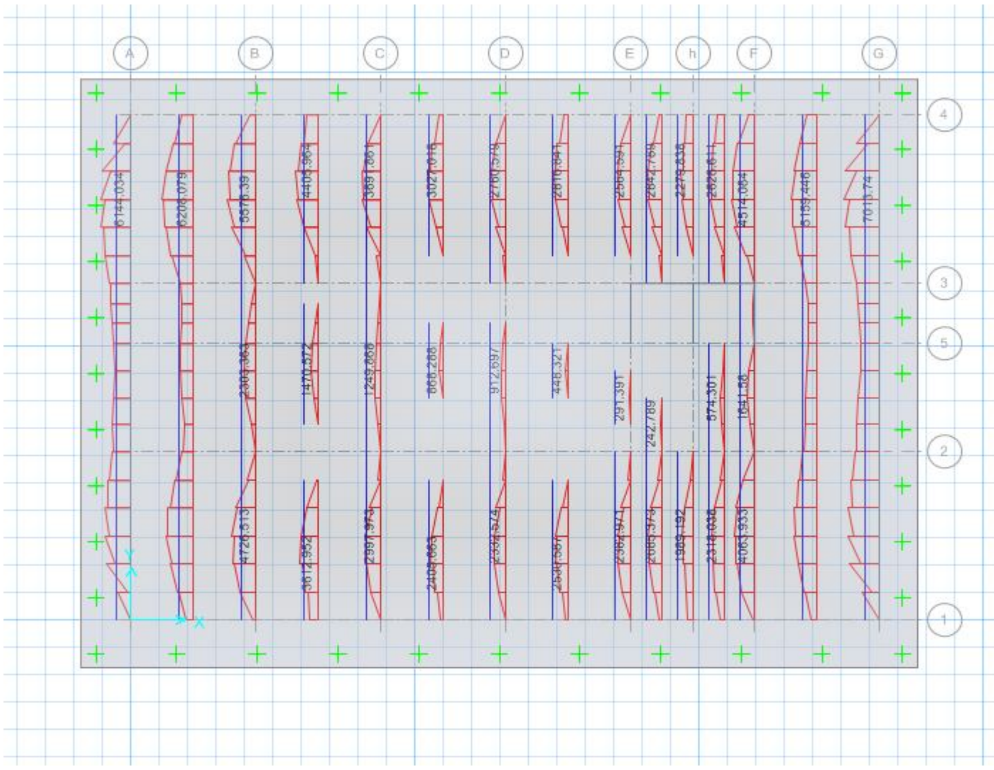
$> A_{smin}$

UseT16@250mm c/c$< 3d$ (As =2010mm^2)

Maximum bottom reinforcement for middle strip =5523mm^2

$> A_{smin}$

UseT20@140mm c/c$< 3d$ (As =5607mm^2



Figure(4.45):Raft foundation top reinforcement for span 5m for AC1 2005 code

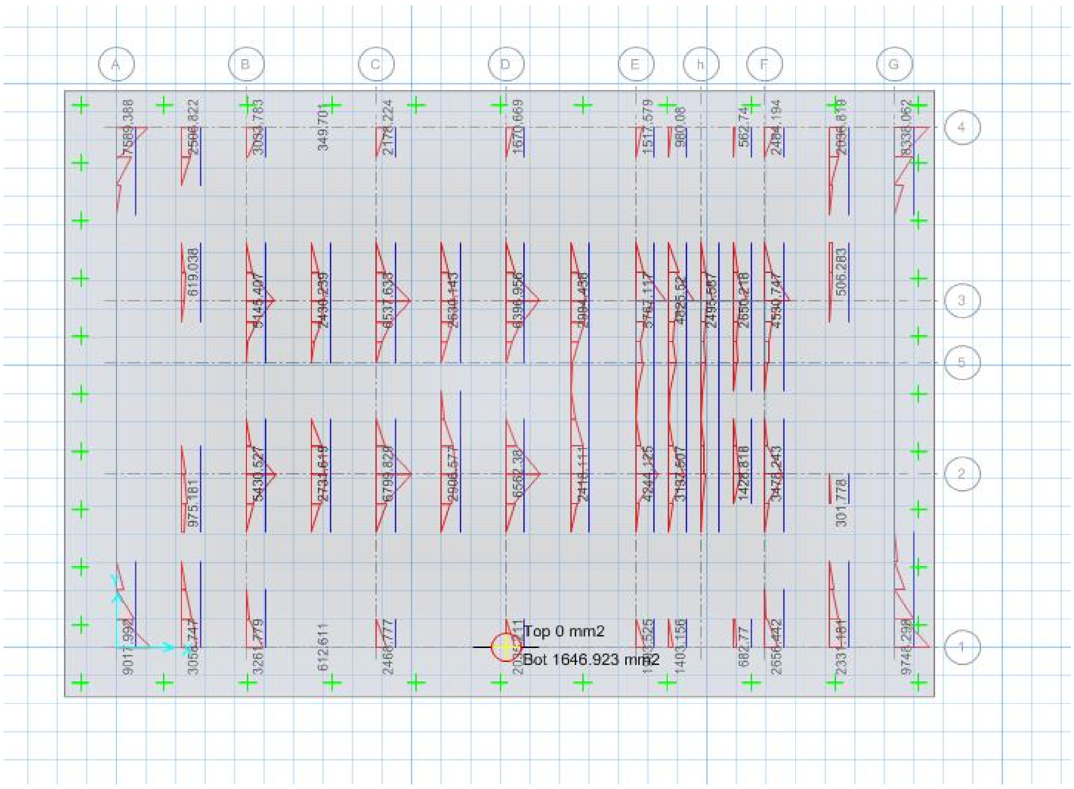


Figure (4.46): Raft foundation bottom reinforcement for span 5m for AC1 2005 code

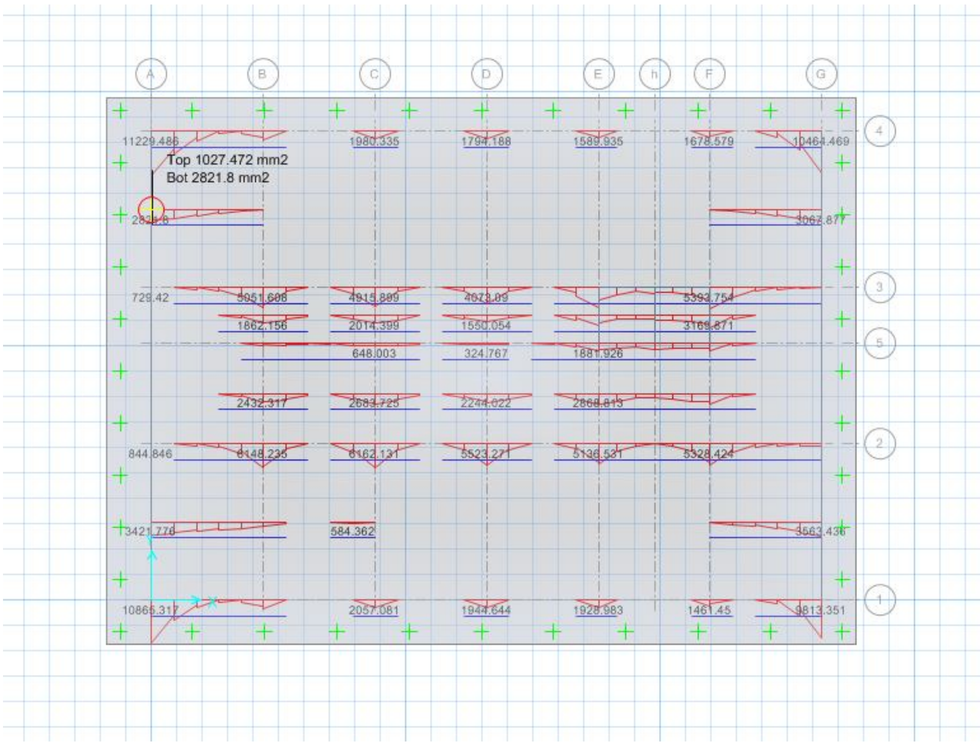


Figure (4.47): Raft foundation bottom reinforcement for span 7m for AC1 2005 code

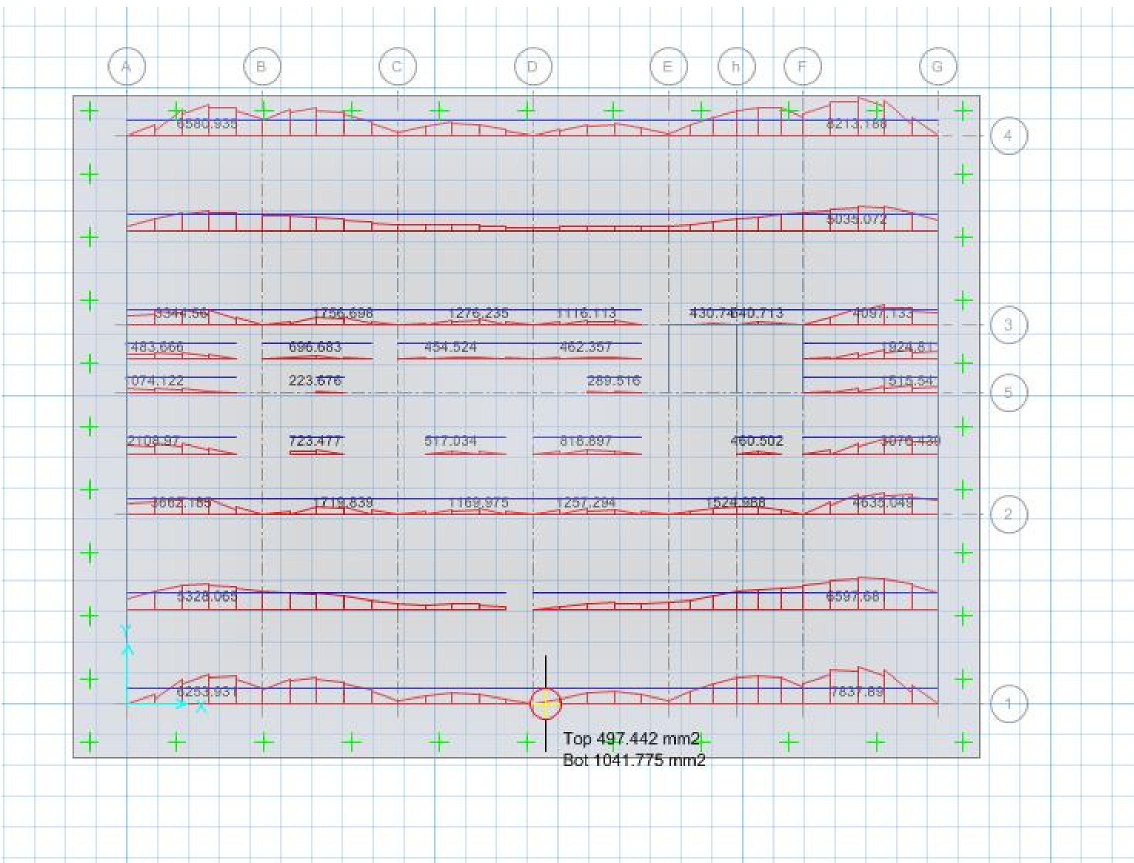


Figure (4.48): Raft foundation top reinforcement for span 7m for AC1 2005 code

3- EC2-1992:

Output design for Etabsprogramme for EC2-1992 from figures below:

Result of design for raft foundation:

For span 5m:

Column strip =2500mm, middle strip =4500mm

Maximum top reinforcement for column strip =6954mm^2 $> A_{smin}$

UseT20@110mm c/c$< 3d$ (As =7136mm^2)

Maximum bottom reinforcement for column strip =6744mm^2

$> A_{smin}$

UseT20@110mm c/c$< 3d$ (As=7136mm^2)

Maximum top reinforcement for middle strip =2807mm^2

$> A_{smin}$

UseT16@300mm c/c$< 3d$ (As =3015mm^2)

Maximum bottom reinforcement for middle strip =4786mm^2

$< A_{smin}(1463mm^2)$

UseT20@290mm c/c$< 3d$ (As=4872mm^2)

For span 7m:

Column strip =2500mm, middle strip =2500mm

Maximum top reinforcement for column strip =7786mm^2

$> A_{smin}$

UseT25@150mm c/c$< 3d$ (As =8183mm^2)

Maximum bottom reinforcement for column strip =6107mm^2

$> A_{smin}$

UseT20@125mm c/c$< 3d$ (As =6280mm^2)

Maximum top reinforcement for middle strip =3067mm^2

$> A_{smin}$

UseT16@150mm c/c$< 3d$ (As =3350mm^2)

Maximum bottom reinforcement for middle strip =5492mm^2

$> A_{smin}$

UseT20@140mm c/c$< 3d$ (As =5607mm^2)

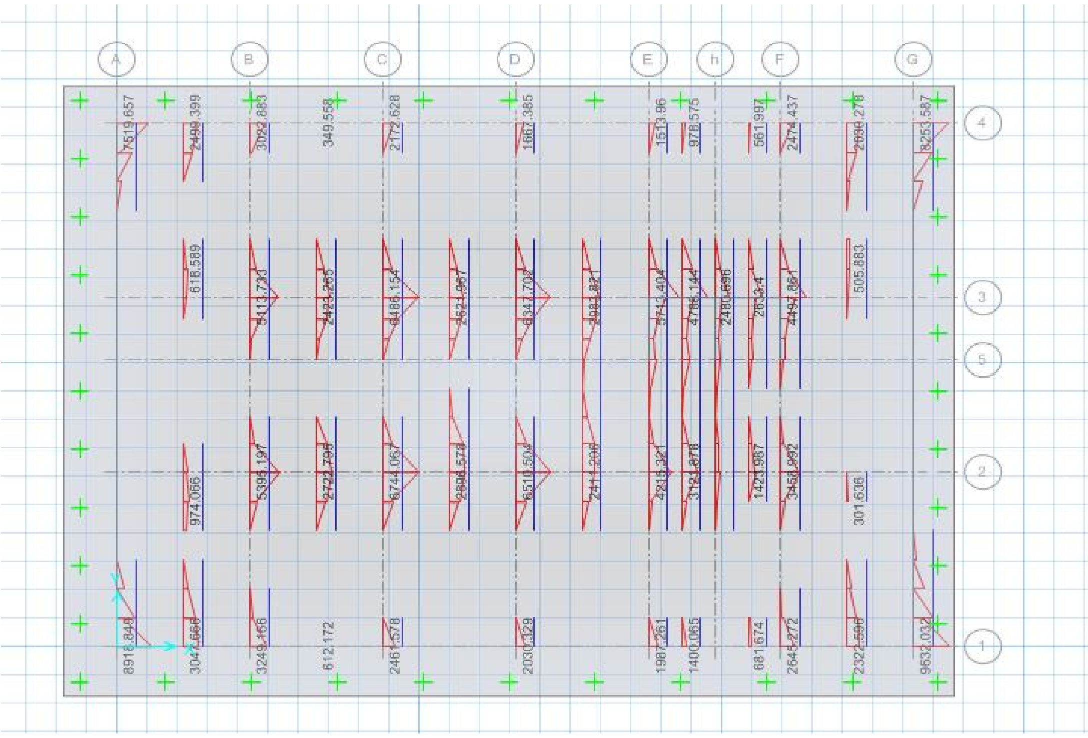
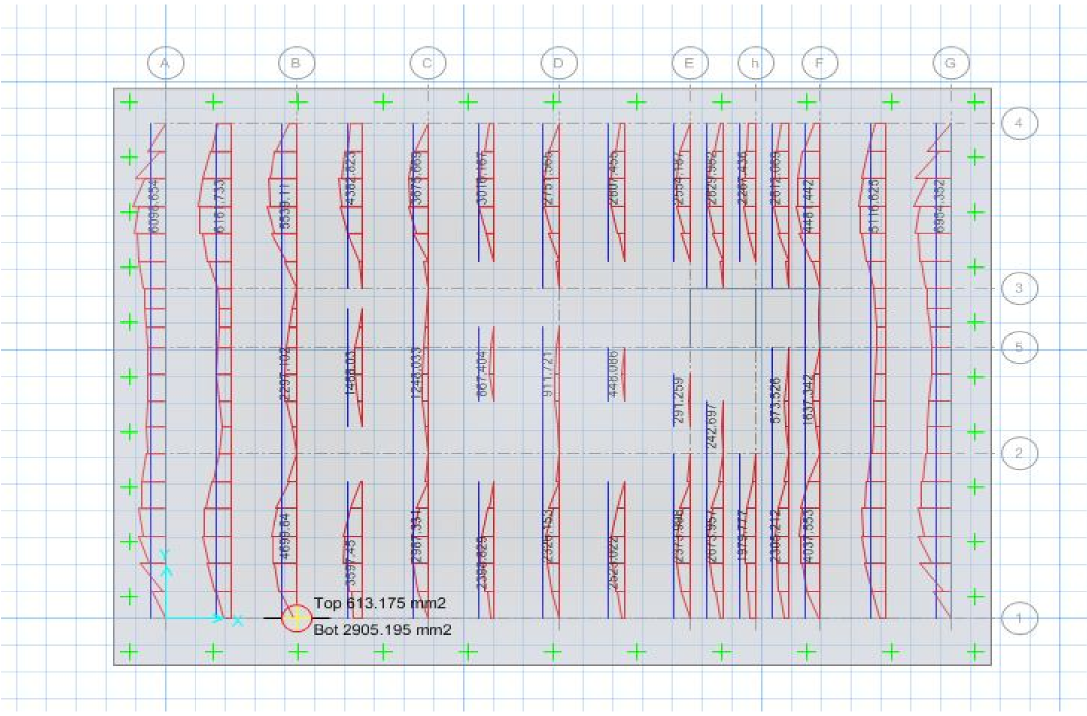


Figure (4.49): Raft foundation bottom reinforcement for span 5m for EC2-1992 code



Figure(4.50):Raft foundation top reinforcement for span 5m for EC2-1992 code

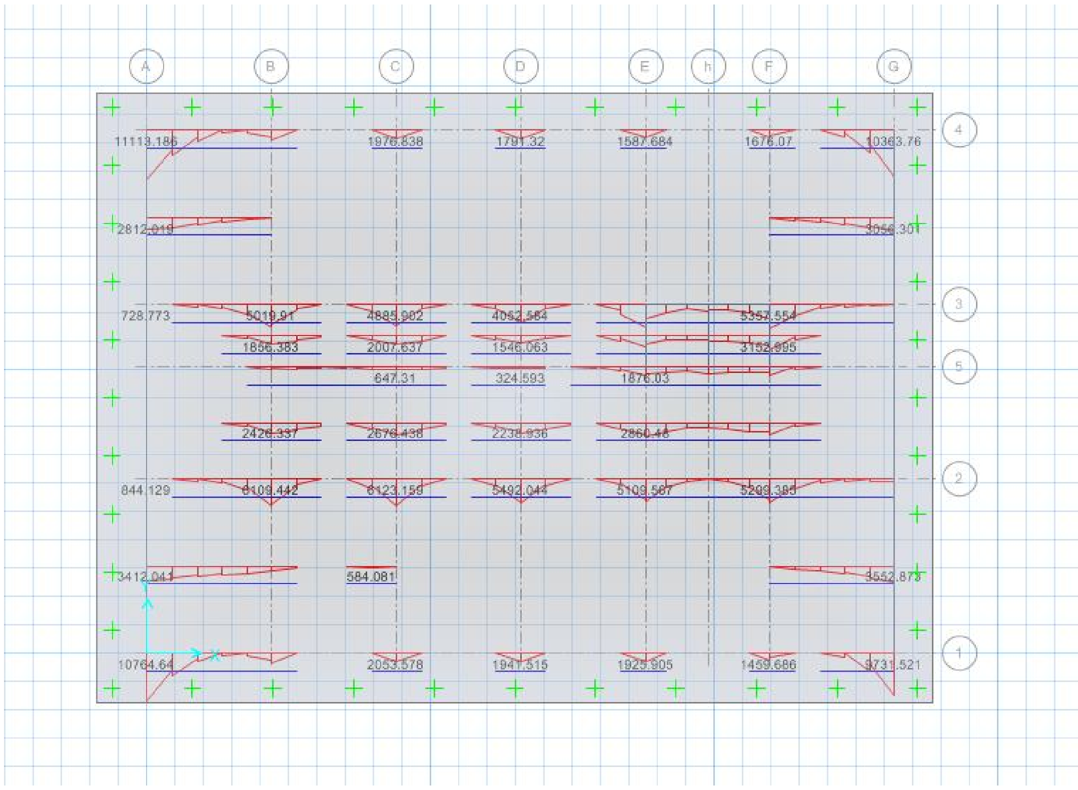
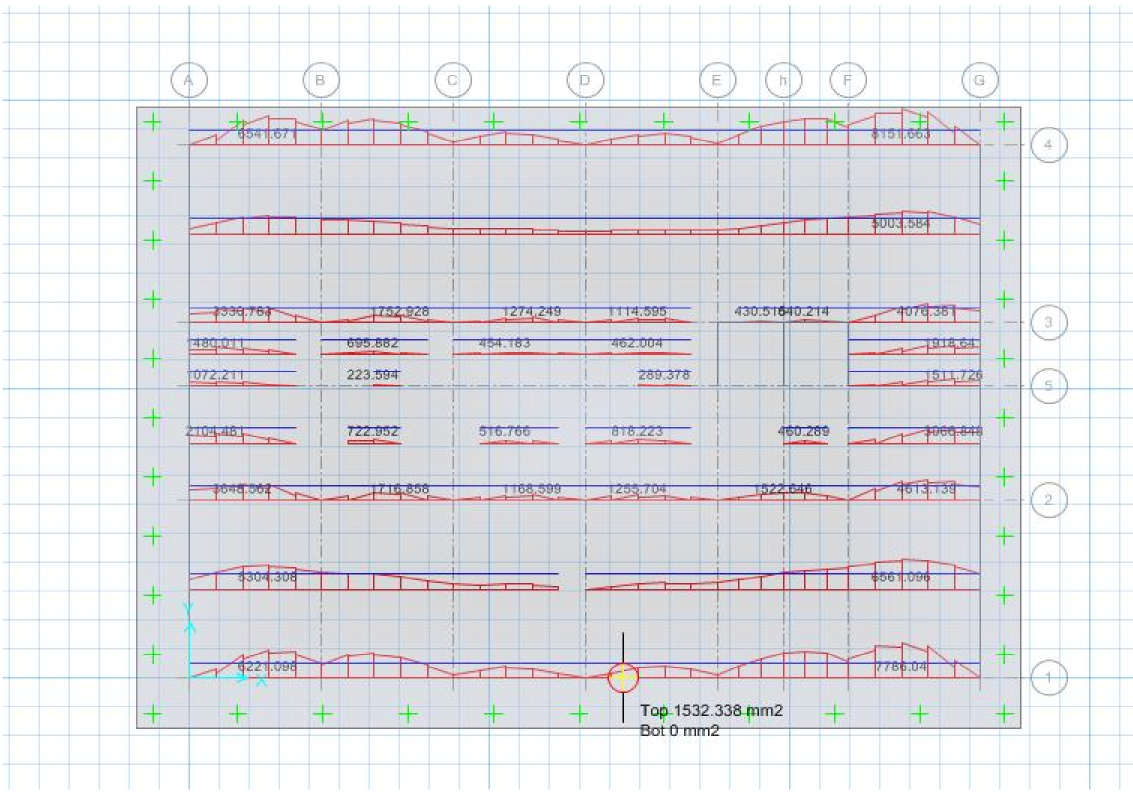


Figure (4.51): Raft foundation bottom reinforcement for span 7m for EC2-1992 code



Figure(4.52):Raft foundation top reinforcement for span 7m for EC2-1992 code

4.5Discussions of results:

From the results obtained it was observed that:

1. The area of steel in the short span for flat slab(at bottom) are equal for both column strip and middle by using three codes BS8110-1997 code ,EC2-1992 code and AC1-2005 code.

2. The area of steel for flat slab by using AC1-2005 is greater than the area of steel for flat slab by using manual calculations(BS81101997code) about 3% ,and greater than the area of steel by using both codes BSI8110-1997 and EC2-1992 codes about 1%

3. The area of steel for columns by using AC1-2005 is greater than the area of steel for columns by using manual calculations(BS81101997code) about 0.2% ,and greater than the area of steel by using both codes BSI8110-1997 and EC2-1992 codes about 2%.

4. The area of steel for raft foundation by using EC2-1992 code is greater than the area of steel in manual calculations(BS81101997code) about 29% ,and greater than the area of steel by using BSI8110-1997 code about 5% and greater than the area of steel by using AC1-2005 code about percentage 3%.

5.The sections of columns, the thickness of slab, the thickness of shear wall and the thickness of raft foundation are equal for all storeys in the manual design and design by Etabs and Safe programmes.

6.As for the shear wall design it was found that the area of steel is sizeable in the manual design(BS8110-1997)than the area of steel which resulting from design by the Etabsprogramme for the all codes by 33%.