MIDDLE EAST TECHNICAL UNIVERSITY
DEPARTMENT OF CIVIL ENGINEERING

CE 490 - INTRODUCTION TO EARTHQUAKE ENGINEERING

Take Home Exam

Instructor

Prof. Dr. Uğurhan AKYÜZ

Prepared by

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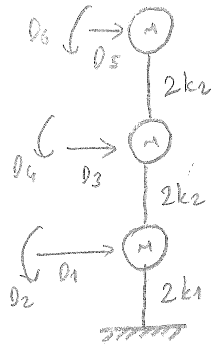
TABLE OF CONTENT

2) Equivalent Lateral Load Analysis

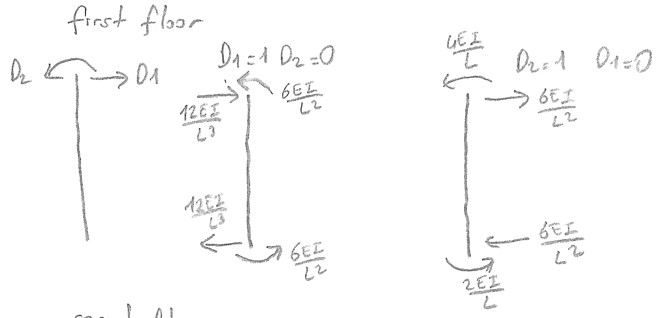
$E = 25000\ MPa$

$I_{column} = \frac{1}{12} * 400 * 400^3 = 2.133 * 10^9\ mm^4$

$EI = 5.3325 * 10^{13}\ N.mm^2 = 53325\ kN.m^2$

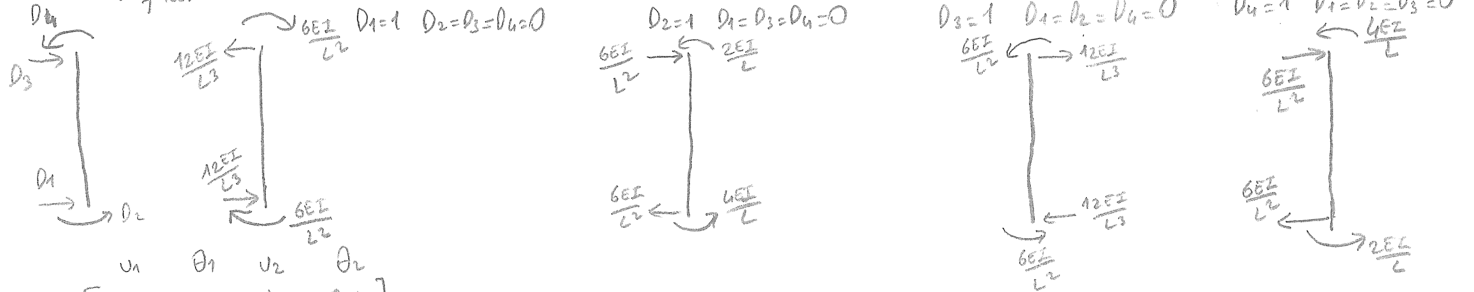


first floor



$$
\begin{array}{cc}
 & \begin{array}{cc} u_1 & \theta_1 \end{array} \\
EI & \begin{bmatrix} 3/16 & 3/8 \\ 3/8 & 1 \end{bmatrix}
\end{array}
\begin{array}{c} u_1 \\ \theta_1 \end{array}
$$

second floor



$$
EI
\begin{array}{c}
\begin{array}{cccc} u_1 & \theta_1 & u_2 & \theta_2 \end{array} \\
\begin{bmatrix}
4/3 & -2/3 & -4/3 & -2/3 \\
-2/3 & 4/3 & 2/3 & 2/3 \\
-4/3 & 2/3 & 4/3 & 2/3 \\
-2/3 & 2/3 & 2/3 & 4/3
\end{bmatrix}
\end{array}
\begin{array}{c} u_1 \\ \theta_1 \\ u_2 \\ \theta_2 \end{array}
$$

third floor

$$
EI
\begin{array}{c}
\begin{array}{cccc} u_2 & \theta_2 & u_3 & \theta_3 \end{array} \\
\begin{bmatrix}
4/3 & -2/3 & -4/3 & -2/3 \\
-2/3 & 4/3 & 2/3 & 2/3 \\
-4/3 & 2/3 & 4/3 & 2/3 \\
-2/3 & 2/3 & 2/3 & 4/3
\end{bmatrix}
\end{array}
\begin{array}{c} u_2 \\ \theta_2 \\ u_3 \\ \theta_3 \end{array}
$$

$$
EI
\begin{array}{c}
\begin{array}{cccccc} u_1 & \theta_1 & u_2 & \theta_2 & u_3 & \theta_3 \end{array} \\
\begin{bmatrix}
31/144 & -7/24 & -4/3 & -2/3 & 0 & 0 \\
-7/24 & 7/3 & 2/3 & 2/3 & 0 & 0 \\
-4/3 & 2/3 & 8/3 & 0 & -4/3 & -2/3 \\
-2/3 & 2/3 & 0 & 8/3 & 2/3 & 2/3 \\
0 & 0 & -4/3 & 2/3 & 4/3 & 2/3 \\
0 & 0 & -2/3 & 2/3 & 2/3 & 4/3
\end{bmatrix}
\end{array}
\begin{array}{c} u_1 \\ \theta_1 \\ u_2 \\ \theta_2 \\ u_3 \\ \theta_3 \end{array}
$$

$$
EI
\begin{array}{c}
\begin{array}{ccc|ccc} u_1 & u_2 & u_3 & \theta_1 & \theta_2 & \theta_3 \end{array} \\
\left[\begin{array}{ccc|ccc}
31/144 & -4/3 & 0 & -7/24 & -2/3 & 0 \\
-4/3 & 8/3 & -4/3 & 2/3 & 0 & -2/3 \\
0 & -4/3 & 4/3 & 0 & 2/3 & 2/3 \\
\hline
-7/24 & 2/3 & 0 & 7/3 & 2/3 & 0 \\
-2/3 & 0 & 2/3 & 2/3 & 8/3 & 2/3 \\
0 & -2/3 & 2/3 & 0 & 2/3 & 4/3
\end{array}\right]
\end{array}
\begin{array}{c} u_1 \\ u_2 \\ u_3 \\ \theta_1 \\ \theta_2 \\ \theta_3 \end{array}
$$

$$
\left[\begin{array}{c|c} k_{dd} & k_{ds} \\ \hline k_{sd} & k_{ss} \end{array}\right]
$$

$k_{sd} u_d + k_{ss} u_s = 0$

$f_d = \underbrace{(k_{dd} - k_{ds} . k_{ss}^{-1} . k_{sd})}_{k_d \text{ : condensed stiffness matrix}} u_d$

$$
k_d = EI \begin{bmatrix}
0.4366 & -0.3222 & 0.0907 \\
-0.3222 & 0.3555 & -0.1333 \\
0.0907 & -0.1333 & 0.0593
\end{bmatrix}
$$

$$
k_d = \begin{bmatrix}
23281.7 & -17181.3 & 4836.6 \\
-17181.3 & 18957.0 & -7108.2 \\
4836.6 & -7108.2 & 3162.2
\end{bmatrix}
$$

$$
m = \frac{(G + 0.3Q) * L}{g} = \frac{(10 + 0.3 * 15) * 8}{9.81} = 11.82\ \frac{kN.s^2}{m}
$$

$$
M = \begin{bmatrix}
11.82 & 0 & 0 \\
0 & 11.82 & 0 \\
0 & 0 & 11.82
\end{bmatrix} \text{ton}
$$

$$k = \begin{bmatrix} 46563.4 & -34362.6 & 9673.2 \\ -34362.6 & 37914 & -14216.4 \\ 9673.2 & -14216.4 & 6324.4 \end{bmatrix}$$

$$\left| -w_n^2 M + k \right| = 0$$

$$(1.7654 * 10^9 - 938522.9\lambda + 139.71\lambda^2)$$

$$\begin{vmatrix} 46563.4 - 11.82\lambda & -34362.6 & 9673.2 \\ -34362.6 & 37914 - 11.82\lambda & -14216.4 \\ 9673.2 & -14216.4 & 6324.4 - 11.82\lambda \end{vmatrix} = 0$$

$$\begin{matrix} 46563.4 - 11.82\lambda & -34362.6 & 9673.2 \\ -34362.6 & 37914 - 11.82\lambda & -14216.4 \end{matrix}$$

$$\Big[(46563.4 - 11.82\lambda)(37914 - 11.82\lambda)(+6324.4 - 11.82\lambda) + (-34362.6 * -14216.4 * 9673.2) + (9673.2 * -34362.6 * -14216.4)\Big] - \Big[(9673.2 * 9673.2 * (37914 - 11.82\lambda) + -14216.4 * -14216.4 * (46563.4 - 11.82\lambda) + (+6324.4 - 11.82\lambda) * -34362.6 * -34362.6\Big] = 0$$

$$-1651.4\lambda^3 + 1.2684 * 10^7 \lambda^2 - 9.73 * 10^9 \lambda + 1.833 * 10^{11} = 0$$

$\lambda_1 = 20.039$ $\qquad w_1 = 4.476$ rad/s

$\lambda_2 = 6819.202$ $\qquad w_2 = 82.578$ rad/s

$\lambda_3 = 841.51$ $\qquad w_3 = 29.009$ rad/s

$T_1 = 1.40$ s

$T_2 = 0.076$ s

$T_3 = 0.217$ s

$$\begin{bmatrix} 46362.54 & -34362.6 & 9673.2 \\ -34362.6 & 37677.14 & -14216.4 \\ 9673.2 & -14216.4 & 6087.54 \end{bmatrix} \begin{Bmatrix} \phi_{11} \\ \phi_{12} \\ \phi_{13} \end{Bmatrix} = \underline{0}$$

$$\phi_3 = \begin{Bmatrix} 3.29491 \\ -2.98407 \\ 1.00000 \end{Bmatrix}$$

$$\begin{bmatrix} -34039.57 & -34362.6 & 9673.2 \\ -34362.6 & -42688.97 & -14216.4 \\ 9673.2 & -14216.4 & -74755.91 \end{bmatrix} \begin{Bmatrix} \phi_{21} \\ \phi_{22} \\ \phi_{23} \end{Bmatrix} = \underline{0}$$

$$\phi_1 = \begin{Bmatrix} 0.21970 \\ 0.57770 \\ 1.00000 \end{Bmatrix}$$

$$\begin{bmatrix} 36616.75 & -34362.6 & 9673.2 \\ -34362.6 & 27967.35 & -14216.4 \\ 9673.2 & -14216.4 & -3622.25 \end{bmatrix} \begin{Bmatrix} \phi_{21} \\ \phi_{22} \\ \phi_{23} \end{Bmatrix} = \underline{0}$$

$$\phi_2 = \begin{Bmatrix} -1.39187 \\ -1.20175 \\ 1.00000 \end{Bmatrix}$$

$$M_n = \phi_n^T m \phi_n$$

$$M_3 = \underset{1\times3}{\{3.29491 \quad -2.98407 \quad 1.00000\}} \underset{3\times3}{\begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix}} \underset{3\times1}{\begin{Bmatrix} 3.29491 \\ -2.98407 \\ 1.00000 \end{Bmatrix}} = 245.33627 \text{ tons}$$

$$M_1 = \{0.21970 \quad 0.57770 \quad 1.00000\} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix} \begin{Bmatrix} 0.21970 \\ 0.57770 \\ 1.00000 \end{Bmatrix} = 16.33530 \text{ tons}$$

$$M_2 = \{-1.39187 \quad -1.20175 \quad 1.00000\} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix} \begin{Bmatrix} -1.39187 \\ -1.20175 \\ 1.00000 \end{Bmatrix} = 51.78939 \text{ tons}$$

$$L_n = \varnothing' m l$$

$$L_3 = \{3.23491 \quad -2.98407 \quad 1.00000\}_{1\times3} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix}_{3\times3} \begin{Bmatrix} 1 \\ 1 \\ 1 \end{Bmatrix}_{3\times1} = 15.43613 \text{ tons}$$

$$L_1 = \{0.21970 \quad 0.57770 \quad 1.00000\} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \\ 1 \end{Bmatrix} = 21.24523 \text{ tons}$$

$$L_2 = \{-1.33187 \quad -1.20175 \quad 1.00000\} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \\ 1 \end{Bmatrix} = -18.83653 \text{ tons}$$

base shear

According to Turkish Seismic Code, reduction factor;

$$R(T) = \frac{R}{I} \qquad T > T_B$$

$$R(T) = D + \left(\frac{R}{I} - D\right) \cdot \frac{T}{T_B} \qquad T \leq T_B$$

$T = 1.40s$, $T_B = 0.506s$, $R = 8$, $I = 1$, so

$$R(T) = \frac{8}{1} = 8 \qquad S_{ae} = 5.23854 \text{ m/s}^2 \qquad S_{aR}(T) = \frac{S_{ae}(T)}{R(T)}$$

$$S_{aR,1} = \frac{5.23854}{8} = 0.654818 \text{ m/s}^2$$

$$V_b =$$

$$V_b = \frac{(21.24523)^2}{16.33530} * 0.654818 = 18.033343 \text{ kN}$$

modal force

$$f_{j1} = V_{b1} \frac{m_j \varnothing_{j1}}{L_1}$$

$$f_{11} = 18.033343 * \frac{11.82 * 0.21970}{21.24523} = 2.21153 \text{ kN}$$

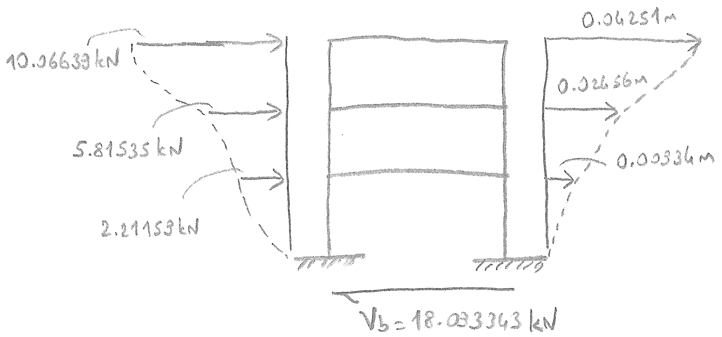
$$f_{21} = 18.033343 * \frac{11.82 * 0.57770}{21.24523} = 5.81535 \text{ kN}$$

$$f_{31} = 18.033343 * \frac{11.82 * 1.00000}{21.24523} = 10.06633 \text{ kN}$$

displacement

$$u_{n,max} = \vec{\varnothing}_n \cdot q_{n,max} = \vec{\varnothing}_n \frac{L_n}{M_n} \cdot \frac{PS_a}{w_n^2}$$

$$u_{1,max} = \begin{Bmatrix} 0.21970 \\ 0.57770 \\ 1.00000 \end{Bmatrix} \cdot \frac{21.24523}{16.33530} \cdot \frac{0.654818}{4.4762} = \begin{Bmatrix} 0.00934 \\ 0.02456 \\ 0.04251 \end{Bmatrix} m$$



$$M_b = (10.06633 * 10) + (5.81535 * 7) + (2.21153 * 4)$$

$$M_b = 150.21771 \text{ kN.m}$$

3) Response Spectrum Analysis

First four steps are done.

$$f_n = \frac{L_n}{M_n} (m \varnothing_n) S_{aR,n}$$

first mode
$$f_1 = \frac{21.24523}{16.33530} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix}_{3\times3} \begin{Bmatrix} 0.21970 \\ 0.57770 \\ 1.00000 \end{Bmatrix}_{3\times1} 0.654818 = \begin{Bmatrix} 2.21153 \\ 5.81535 \\ 10.06633 \end{Bmatrix} \text{kN}$$

second mode
$$f_2 = \frac{-18.83653}{51.78933} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix} \begin{Bmatrix} -1.33187 \\ -1.20175 \\ 1.00000 \end{Bmatrix} 3.26054 = \begin{Bmatrix} 13.51044 \\ 16.84545 \\ -14.01743 \end{Bmatrix} \text{kN}$$

third mode
$$f_3 = \frac{15.43613}{245.33627} \begin{bmatrix} 11.82 & 0 & 0 \\ 0 & 11.82 & 0 \\ 0 & 0 & 11.82 \end{bmatrix} \begin{Bmatrix} 3.23491 \\ -2.98407 \\ 1.00000 \end{Bmatrix} 2.82227 = \begin{Bmatrix} 6.34000 \\ -6.28523 \\ 2.10628 \end{Bmatrix} \text{kN}$$

$$u_{n1} = \begin{Bmatrix} 0.0095 \\ 0.0243 \\ 0.0431 \\ -0.0043 \\ -0.0058 \\ -0.0062 \end{Bmatrix} \quad u_{n2} = \begin{Bmatrix} 0.0020 \\ 0.0017 \\ -0.0015 \\ -0.0004 \\ 0.0006 \\ 0.0012 \end{Bmatrix} \quad u_{n3} = \begin{Bmatrix} 0.8585*10^{-4} \\ -0.7853*10^{-4} \\ 0.2513*10^{-4} \\ 0.2611*10^{-4} \\ 0.2470*10^{-4} \\ -0.6418*10^{-4} \end{Bmatrix} \longrightarrow$$

I obtained these values by using MATLAB.

$$r_{EQ} = \sqrt{r_1^2 + r_2^2 + r_3^2 \ldots + r_n^2}$$

$$U = \begin{Bmatrix} \sqrt{0.0095^2 + 0.0020^2 + (0.8585*10^{-4})^2} \\ \sqrt{0.0243^2 + 0.0017^2 + (-0.7853*10^{-4})^2} \\ \sqrt{0.0431^2 + (-0.0015^2)^2 + (0.2513*10^{-4})^2} \\ \sqrt{(-0.0043)^2 + (-0.0004)^2 + (0.2611*10^{-4})^2} \\ \sqrt{(-0.0058)^2 + (0.0006)^2 + (0.2470*10^{-4})^2} \\ \sqrt{(-0.0062)^2 + (0.0012)^2 + (-0.6418*10^{-4})^2} \end{Bmatrix} = \begin{Bmatrix} 0.00971 \\ 0.02496 \\ 0.04313 \\ 0.00432 \\ 0.00583 \\ 0.00632 \end{Bmatrix} \begin{matrix} u_1 \\ u_2 \\ u_3 \\ \theta_1 \\ \theta_2 \\ \theta_3 \end{matrix}$$

$$V_b = \begin{Bmatrix} \sqrt{2.21159^2 + 19.51044^2 + 6.96000^2} \\ \sqrt{5.81535^2 + 16.84545^2 + (-6.28529)^2} \\ \sqrt{10.06639^2 + (-14.01743)^2 + 2.10628^2} \end{Bmatrix} = \begin{Bmatrix} 20.82575 \\ 18.89689 \\ 17.38554 \end{Bmatrix} kN$$

$$M_b = (17.38554*10) + (18.89689*7) + (20.82575*4)$$

$$M_b = 389.43663 \text{ kN.m}$$

4) Comparison of V_b, M_b, story drift

Even if displacements are closely for Equivalent Earthquake Lateral Load and Response Spectrum Analysis, there is huge difference between them for both shear and moment values. For this reason, I repeated these steps in SAP2000 programming, I continued according to this.

1.) Design spectrum

My location is Yalova- Center and the local site class for this place is ZD. In the light of these information, related data was taken from the web tool of the Turkish Seismic Hazard Map. And design response spectrum curve for DD-2 (recurrence interval of 475 years) and reduced one can be seen below:

Kullanıcı Girdileri

Rapor Başlığı	Yalova Center	
Deprem Yer Hareketi Düzeyi	DD-2	50 yılda aşılma olasılığı %10 (tekrarlanma periyodu 475 yıl) olan deprem yer hareketi düzeyi
Yerel Zemin Sınıfı	ZD	Orta sıkı – sıkı kum, çakıl veya çok katı kil tabakaları
Enlem	40.656045°	
Boylam	29.283611°	

Çıktılar

$S_S = 1.481$ $S_1 = 0.393$ $S_{DS} = 1.481$ $S_{D1} = 0.749$

$PGA = 0.605$ $PGV = 43.079$

S_S : Kısa periyot harita spektral ivme katsayısı [boyutsuz]

S_1 : 1.0 saniye periyot için harita spektral ivme katsayısı [boyutsuz]

S_{DS} : Kısa periyot tasarım spektral ivme katsayısı [boyutsuz]

S_{D1} : 1.0 saniye periyot için tasarım spektral ivme katsayısı [boyutsuz]

PGA : En büyük yer ivmesi (g)

PGV : En büyük yer hızı (cm/sn)

Figure 1. The data is taken from *https://tdth.afad.gov.tr/*

Site Coefficients for Short Period

Site Class	Values of site coefficients F_s for short-period zone					
	$S_s \leq 0.25g$	$S_s=0.50g$	$S_s=0.75g$	$S_s=1.00g$	$S_s=1.25g$	$S_s \geq 1.50g$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1.0	1.0
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF	Site specific spectrum is needed					

Figure 2. Site coefficients for short period

$$R_a(T) = \frac{R}{I} \qquad T > T_B$$

$$R_a(T) = D + \left(\frac{R}{I} - D\right)\frac{T}{T_B} \qquad T \le T_B$$

$$S_{aR}(T) = \frac{S_{ae}(T)}{R_a(T)}$$

Figure 3. Formulas of earthquake response reduction factor and reduced (inelastic) acceleration response spectrum from TSC2018

Bina Taşıyıcı Sistemi	Taşıyıcı Sistem Davranış Katsayısı R	Dayanım Fazlalığı Katsayısı D	İzin Verilen Bina Yükseklik Sınıfları BYS
A. YERİNDE DÖKME BETONARME BİNA TAŞIYICI SİSTEMLERİ			
A1. Süneklik Düzeyi Yüksek Taşıyıcı Sistemler			
A11. Deprem etkilerinin tamamının moment aktaran *süneklik düzeyi yüksek* betonarme çerçevelerle karşılandığı binalar	8	3	BYS ≥ 3

Figure 4. Structural system behavior factor, R and a coefficient, D for buildings have more than three stories TSC2018

T(s)	S_{ae}(g)	S*g(m/s^2)	R	S/R
0.000	0.591	5.79771	3	1.93257
0.050	1.028	10.08468	3.494071	2.886226
0.100	1.466	14.38146	3.988142	3.606055
0.101	1.477	14.48937	3.998024	3.624133
0.150	1.477	14.48937	4.482213	3.232637
0.200	1.477	14.48937	4.976285	2.911684
0.250	1.477	14.48937	5.470356	2.648707
0.300	1.477	14.48937	5.964427	2.429298
0.350	1.477	14.48937	6.458498	2.243458
0.400	1.477	14.48937	6.952569	2.084031
0.450	1.477	14.48937	7.44664	1.945759
0.500	1.477	14.48937	7.940711	1.824694
0.506	1.477	14.48937	8	1.811171
0.550	1.360	13.3416	8	1.6677
0.600	1.247	12.23307	8	1.529134
0.650	1.151	11.29131	8	1.411414
0.700	1.068	10.47708	8	1.309635
0.750	0.997	9.78057	8	1.222571
0.800	0.935	9.17235	8	1.146544
0.850	0.880	8.6328	8	1.0791
…	…	…	…	…

Table 1. Design response spectrum values

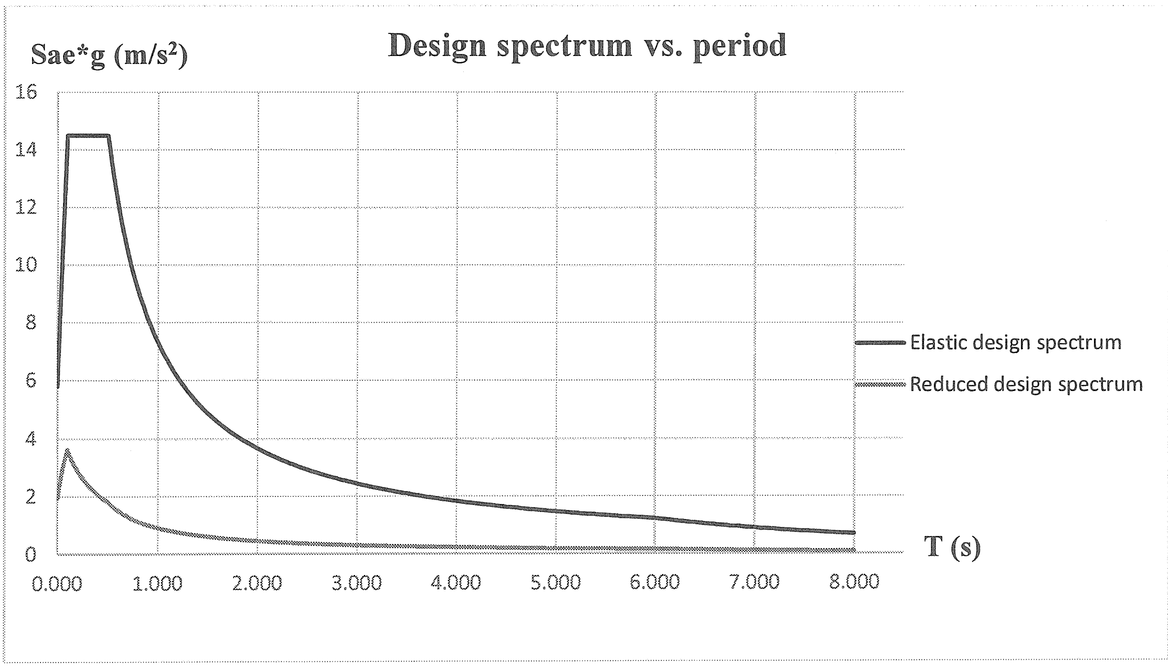


Figure 5. The graph of linear elastic and reduced (inelastic) design spectrum

To input data to the SAP2000 program, I used ones that I already calculated in hand calculations.

2.) Equivalent Lateral Load Analysis

TABLE: Modal Participating Mass Ratios						
OutputCase	**StepType**	**StepNum**	**Period**	**UX**	**UY**	**UZ**
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless
MODAL	Mode	1	0.820782	0.85709	0	0
MODAL	Mode	2	0.205198	0.12317	0	0
MODAL	Mode	3	0.08692	0.01974	0	0
MODAL	Mode	4	0.032355	0	0	0.94137
MODAL	Mode	5	0.032312	1.009E-07	0	4.538E-18
MODAL	Mode	6	0.017642	0	0	0
MODAL	Mode	7	0.017595	0	0	0
MODAL	Mode	8	0.017314	0	0	0
MODAL	Mode	9	0.011122	0	0	0.05319
MODAL	Mode	10	0.011121	5.816E-09	0	5.986E-14
MODAL	Mode	11	0.007436	1.203E-19	0	0.00544
MODAL	Mode	12	0.007435	1.718E-09	0	3.81E-13

Table 2. Modal participating mass ratios

N	H_j (m)	m_i (ton)	m_j*H_j (ton-m)	$F_{iE}^{(X)}$ (kN)	$M_o^{(X)}$ (kN-m)
1	4	11.82	47.3	7.39	29.57
2	7	11.82	82.7	12.94	90.55
3	10	11.82	118.2	18.48	184.79
		Σ =	248.2	39.70	304.91

Table 3. Total weight of the building

Equivalent Earthquake Lateral Load in x - direction		
$T_p^{(X)}$ =	0.82 s	Period
$S_{aR}(Tp^{(X)})$ =	1.119566 m/s^2	From response spectrum of DD2
m_t =	35.46 tons	Total mass
I =	1	Importance factor
S_{DS} =	1.481	From response spectrum of DD2
N =	3	The number of story
$V_{tE}^{(X)} = m_t*S_{aR}(T_p^{(X)}) \geq 0.04*m_t*I*S_{DS}*g$		
$V_{tE}^{(X)}$ =	39.7 kN $\geq$	20.6 kN OK !
$F_{iE}^{(X)} = (V_{tE}^{(X)} - \Delta F_{NE}^{(X)})*m_i*H_i/(\Sigma m_j*H_j)$		
$\Delta F_{NE}^{(X)} = 0.0075*N*V_{tE}^{(X)}$		
$\Delta F_{NE}^{(X)}$ =	0.893246 kN	
$M_o^{(X)} = \Sigma F_{iE}^{(X)}*H_i$		

Table 4. Equivalent Earthquake Lateral Load results according to TSC2018

TABLE: Joint Displacements								
Joint	OutputCase	CaseType	U1	U2	U3	R1	R2	R3
Text	Text	Text	m	m	m	Radians	Radians	Radians
13	lateral	LinStatic	0	0	0	0	0	0
14	lateral	LinStatic	0.008937	0	-0.000021	0	0.003056	0
15	lateral	LinStatic	0.018859	0	-0.000031	0	0.002905	0
16	lateral	LinStatic	0.027071	0	-0.000036	0	0.002186	0
17	lateral	LinStatic	0	0	0	0	0	0
18	lateral	LinStatic	0.008947	0	0.000021	0	0.003059	0
19	lateral	LinStatic	0.018876	0	0.000031	0	0.002907	0
20	lateral	LinStatic	0.027096	0	0.000036	0	0.002188	0

Table 5. Storey displacements after analysing of Equivalent Earthquake Lateral Load

OutputCase	CaseType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m	GlobalY m	GlobalZ m
lateral	LinStatic	-38.81	0	1.066E-14	0	-304.94	0	0	0	0

Figure 6. Base reactions after analysing of Equivalent Earthquake Lateral Load

3.) Response Spectrum Analysis

To obtain Response Spectrum Analysis, I imported the text file of reduced acceleration spectrum vs. period data of my location.

TABLE: Joint Displacements

Joint	OutputCase	CaseType	StepType	U1	U2	U3	R1	R2	R3
Text	Text	Text	Text	m	m	m	Radians	Radians	Radians
13	response spectrum	LinRespSpec	Max	0	0	0	0	0	0
14	response spectrum	LinRespSpec	Max	0.00805	0	0.000019	0	0.002738	0
15	response spectrum	LinRespSpec	Max	0.016884	0	0.000028	0	0.002654	0
16	response spectrum	LinRespSpec	Max	0.024298	0	0.000033	0	0.002067	0
17	response spectrum	LinRespSpec	Max	0	0	0	0	0	0
18	response spectrum	LinRespSpec	Max	0.00805	0	0.000019	0	0.002738	0
19	response spectrum	LinRespSpec	Max	0.016884	0	0.000028	0	0.002654	0
20	response spectrum	LinRespSpec	Max	0.024298	0	0.000033	0	0.002067	0

Table 6. Storey displacements after analysing of Response Spectrum Analysis

OutputCase	CaseType Text	StepType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m	GlobalY m	GlobalZ m
response s...	LinRespSpec	Max	36.413	0	2.25E-11	0	272.6172	0	0	0	0

Figure 7. Base reactions after analysing of Response Spectrum Analysis

4.) Comparison of values from Equivalent Earthquake Lateral Load and Response Spectrum Analysis

	Equivalent Earthquake Lateral Load	**Response Spectrum Analysis**
u_1 (m)	0.008937	0.00805
u_2 (m)	0.018859	0.016884
u_3 (m)	0.027071	0.024298
V_b (kN)	38.81	36.413
M_b(Kn.m)	304.94	272.6172
C	1.094473	1.026875

Table 7. All values obtained from Earthquake Lateral Load and Response Spectrum Analysis

C is the base shear coefficient and it was obtained by this formula:

$$C = V_b / \Sigma W$$

Results that was obtained from Earthquake Lateral Load are mostly close to others, in proportion to the results of the hand calculation. Then, choosing computer programming calculation is more reliable and preferable source for a designer.

5.) Design of beams and columns

Load combinations are used in this part are below:

- 1.4G+1.6Q
- 0.9G+E
- G+Q+E
- 0.9G-E
- G+Q-E
- Envelope (a combination that are contained all other combinations)

Column design

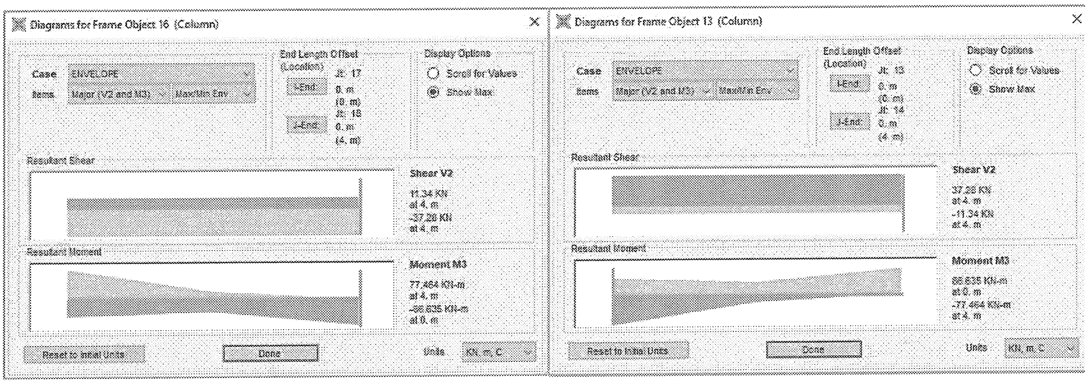


Figure 8. Shear force and moment values of the left column (left) and right one (right) at the first story

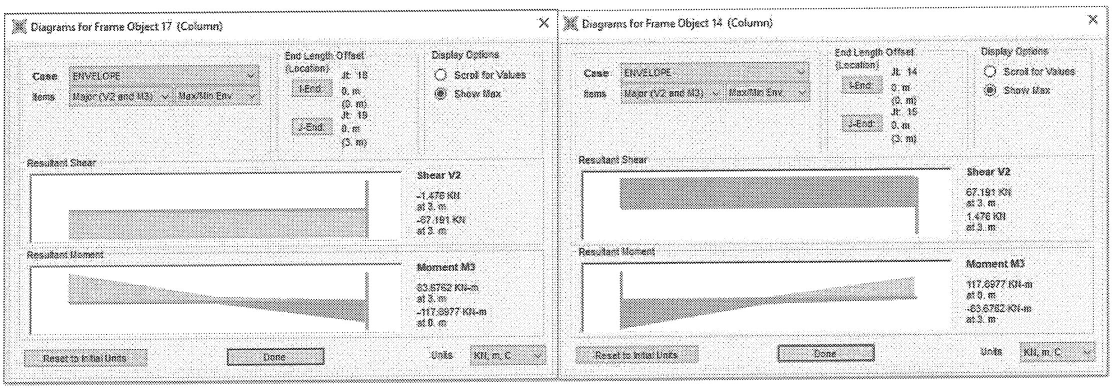


Figure 9. Shear force and moment values of the left column (left) and right one (right) at the second story

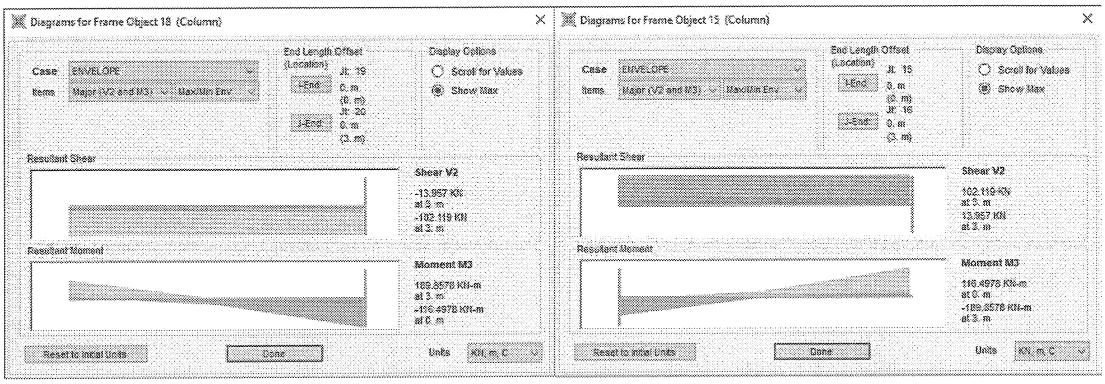


Figure 10. Shear force and moment values of the left column (left) and right one (right) at the third story

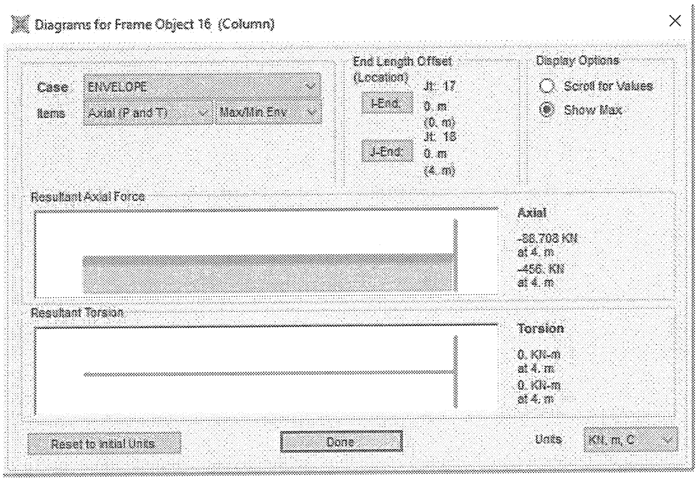


Figure 11. Axial force value of columns at the first story

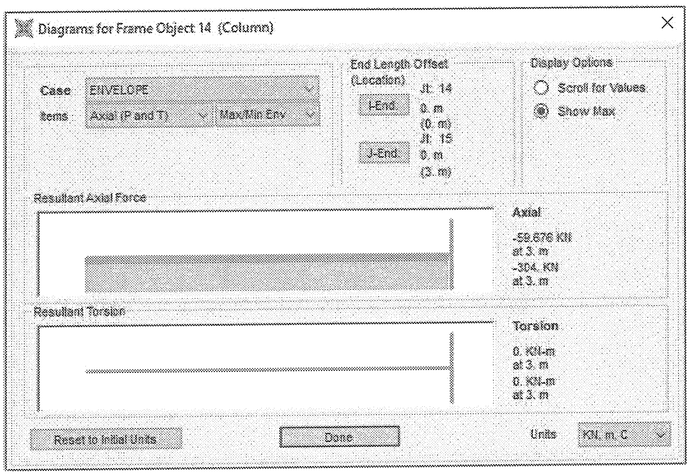


Figure 12. Axial force value of columns at the second story

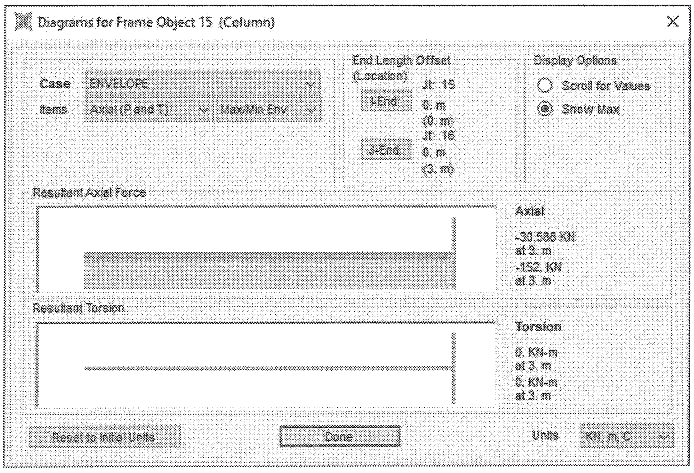


Figure 13. Axial force value of columns at the third story

- Sway check

$$\Psi = 1.5*\Delta_i*(\sum N_{di}/l_i)/V_{fi}$$

$\Psi = 1.5*0.00805*(456/4)/37.28$

$\Psi = 0.03692<0.05$ it is non-sway

$\Psi = 1.5*0.008937*(456/4)/37.28$

$\Psi = 0.04099<0.05$ it is non-sway

Therefore, second order effect is negligible.

After finding all axial forces, shear forces and moments for columns (and also beams), we need to consider just critical beam(s) and column(s). Design phase was done in this way and I assumed all beams are same, and columns are also identically.

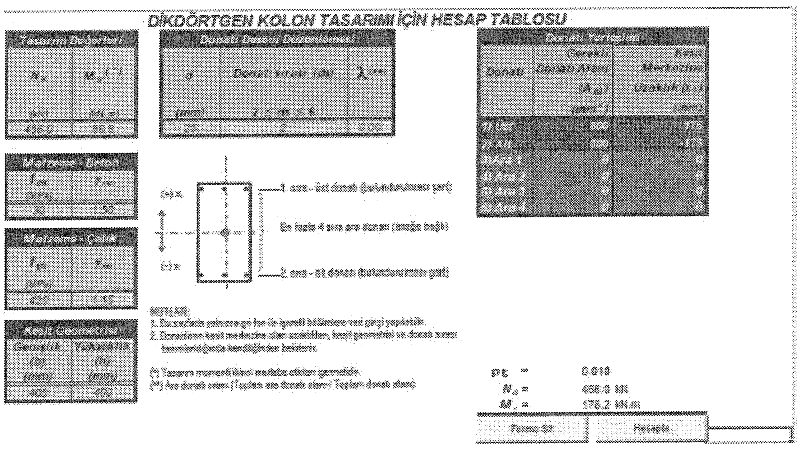


Figure 14. Reinforcement areas are obtained by using an Excel macro

A_{st}= 800+800= 1600 mm^2

Minimum cross-sectional area $A_c \geq \frac{N_{dm}}{0.4 f_{ck}}$

A_c= 800*800= 160000 mm^2

N_{dm}= 456000 N

f_{ck}= 30 MPa

160000 ≥ 456000/ (0.4*30) = 38000 OK.

Minimum and maximum reinforcement ratios $0.01 \leq \rho_t \leq 0.04$

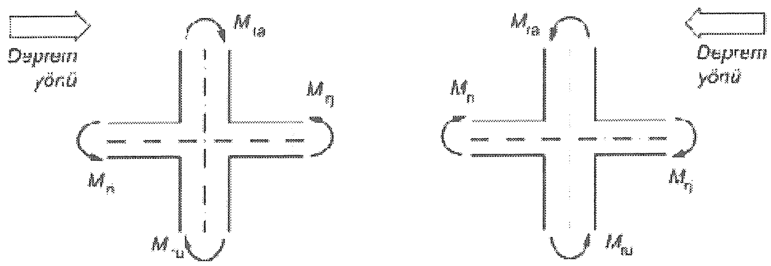
A_{st}= 800+800= 1600 mm^2

Use 8Ø18= 2036 mm^2 > A_{st} OK.

ρ_t= A_s/ A_c= 1600/160000= 0.01

0.01 ≤ ρ_t = 0.01 ≤ 0.04 OK.

$$(M_{r,bottom} + M_{r,top}) \geq 1.2(M_{ri} + M_{rj})$$



For right column on the first story;

Mr_{bottom}= 86.635 kN.m

Mr_{top}= 117.8977 kN.m

Mr_i= 108.6383 kN.m

Mr_j= 0 kN.m

(86.635+ 117.8977) ≥ 1.2*(108.6383+0) OK.

<u>Shear design:</u> $V_e = (M_{bottom} + M_{top})/l_n$

$$\left.\begin{aligned} V_e &\leq V_r \\ V_e &\leq 0.85 A_w \sqrt{f_{ck}} \end{aligned}\right\}$$ should be satisfied. If not, enlarge the section

$$V_e= (86.635+ 117.8977)/ (4-0.4) = 56.8146 \text{ kN}$$

$$V_{cr}= [0.65*(0.35*30^{1/2}/1.5) *400*400]/1000 = 132.914 \text{ kN}$$

$$V_c= 0.8* V_{cr}= 106.331 \text{ kN}$$

$$V_r= V_c+V_w> 106.331 \text{ kN so } V_e< V_r \quad \text{OK.}$$

By using Ø8, A_w= 100.531 mm^2

$$V_e\leq 0.85*100.531*30^{1/2}= 468.036 \text{ kN} \quad \text{OK.}$$

$$A_{sw}/s= [(V_d-V_c) *1000]/ (365*400) = 2.39499$$

$$s= 41.97550 \text{ mm}$$

$$\min A_{sw}/s= 0.3*f_{ctd}*b_w/f_{yd}$$

$$\min A_{sw}/s= 0.3*1.28*400/365= 0.42082$$

$$\min s= 238.892 \text{ mm}$$

Lateral reinforcement requirements:

- *Confined joint:* 40% of the column lateral reinforcement at the column confinement region should be continued into the joint region. Bar diameter cannot be smaller than 8 mm and spacing cannot be greater than 150 mm.

- *Unconfined joint:* 60% of the column lateral reinforcement at the column confinement region should be continued into the joint region. Bar diameter cannot be smaller than 8 mm and spacing cannot be greater than 100 mm.

And also s should be less than or equal to d/2 for mid-span and d/4 for support.

So, lateral reinforcement will be Ø8/200 mm for mid-span and Ø8/100 mm for support.

Beam design

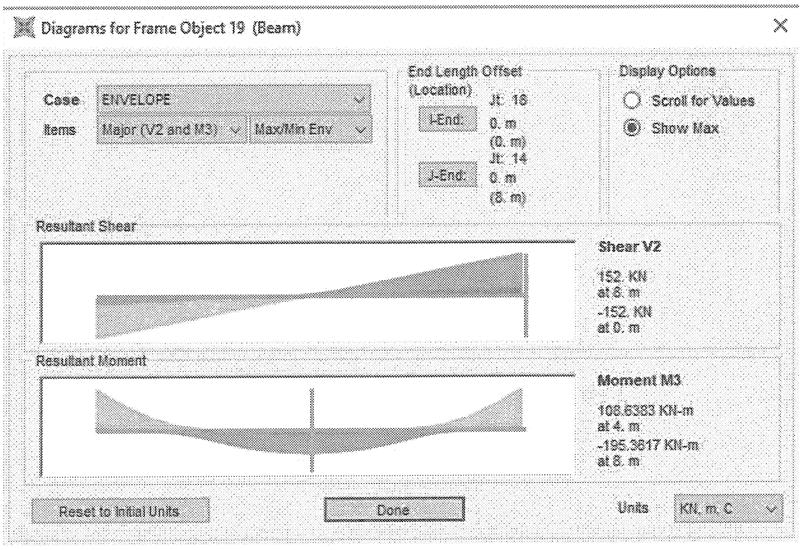


Figure 15. Shear force and moment values of beam at the first story

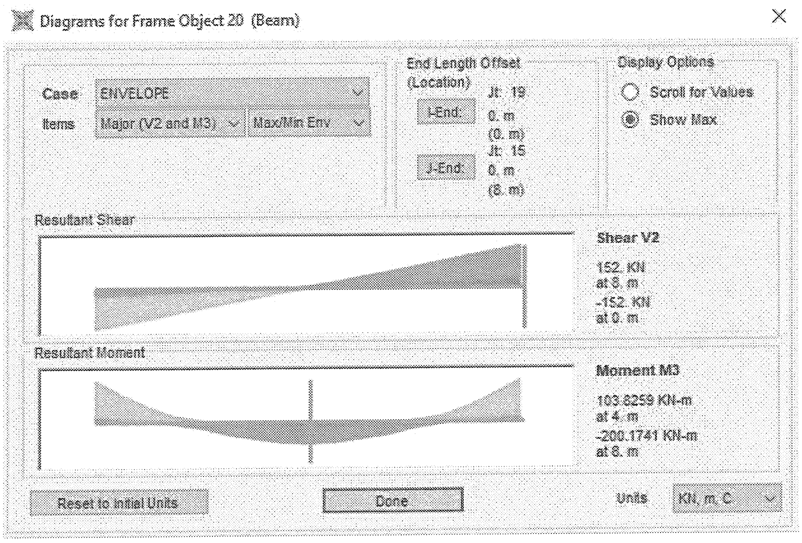


Figure 16. Shear force and moment values of beam at the second story

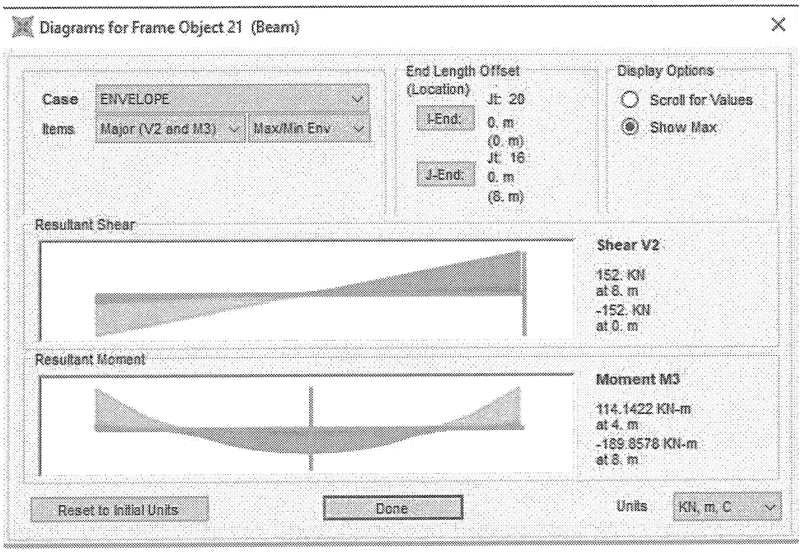


Figure 17. Shear force and moment values of beam at the third story

According to these values;

- Flexural design

For mid-span

$$M_d = +\ 108.6383 \text{ kN.m}$$

$$K_l = 247 \text{ mm}^2/\text{kN}$$

$$K = b_w * d^2 / M_d = 300*400^2/108.6383 = 442 \text{ mm}^2/\text{kN}$$

$K > K_l$ not use compression steel

$$A_{smin} = 0.8*(f_{ctd}/f_{yd}) * b_w * d = 342.0 \text{ mm}^2$$

$$A_s = M_d/(0.86*f_{yd} * d) = (108.6383*10^6)/(0.86*365*400) = 865.23 \text{ mm}^2$$

$$A_s > A_{smin}$$

Use $4\emptyset 18 = 1018 \text{ mm}^2 > A_s$ OK.

For support

$$M_d = -\ 200.1741 \text{ kN.m}$$

$$K = b_w * d^2 / M_d = 300*400^2/200.1741 = 240 \text{ mm}^2/\text{kN}$$

$K < K_l$ use compression steel

$$M_1 = b_w * d^2 / K_l = 300*400^2/(247*10^3) = 194.3320 \text{ kN.m}$$

$$M_2 = M_d - M_1 = 200.1741 - 194.3320 = 5.8421 \text{ kN.m}$$

$$A_{s1} = M_1/(0.86*f_{yd} * d) = (194.3320*10^6)/(0.86*365*400) = 1547.72 \text{ mm}^2$$

$A_{s1} > A_{smin}$

Available 2Ø18= 509 mm^2 (bent bars)

Use 2Ø16= 403 mm^2 (hanger bars)

Use 4Ø18= 1018 mm^2

ΣA_s= 1930 mm^2> A_{s1} OK.

A_{s2}= M_2/f_{yd}*(d- d′) = 5.8421*10^6/365*(400-40) = 44.46 mm^2

$A_{s2} < A_{smin}$, A_{s2}= 342.0 mm^2

Available 2Ø16= 403 mm^2 (hanger bars) > A_{s2} OK.

- Shear design

V_d =152 kN

V_d' = 136.8 kN (shear force at point d away from the column face)

$V_d > V_{cr}$

V_{cr} = 0.65*f_{ctd}*b_w*d =99.8 kn

V_c = 0.8*(0.65*1.28*300*400*1)/1000 = 79.872 kN

V_w =136.8-79.872=56.93 kN

Asw/s= (V_e-V_c)/ (f_{yd}*d)

Asw/s=(136800-79872)/ (365*400)= 0.3899 mm

For Ø8, A_{sw}= 100.531 mm^2

s= 255 mm

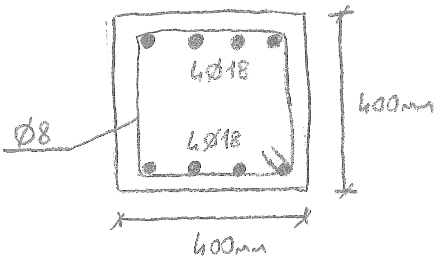
min A_{sw}/s= 0.3*f_{ctd}*b/f_{ywd}= 0.37 mm

min s= 265 mm

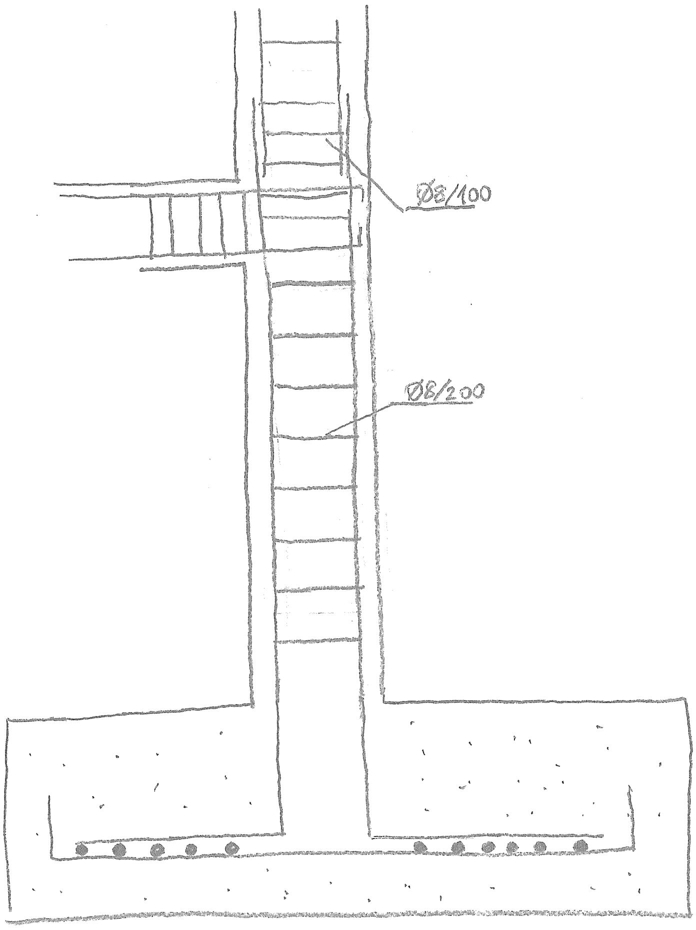
So lateral reinforcement will be Ø8/255 mm

6) Design drawings

C101

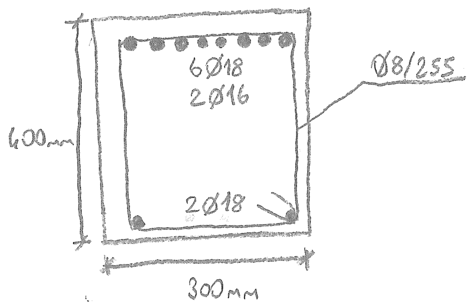


cross-section of column C101

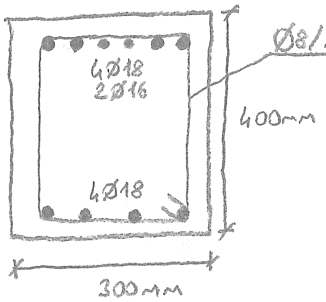


detail view of column C101

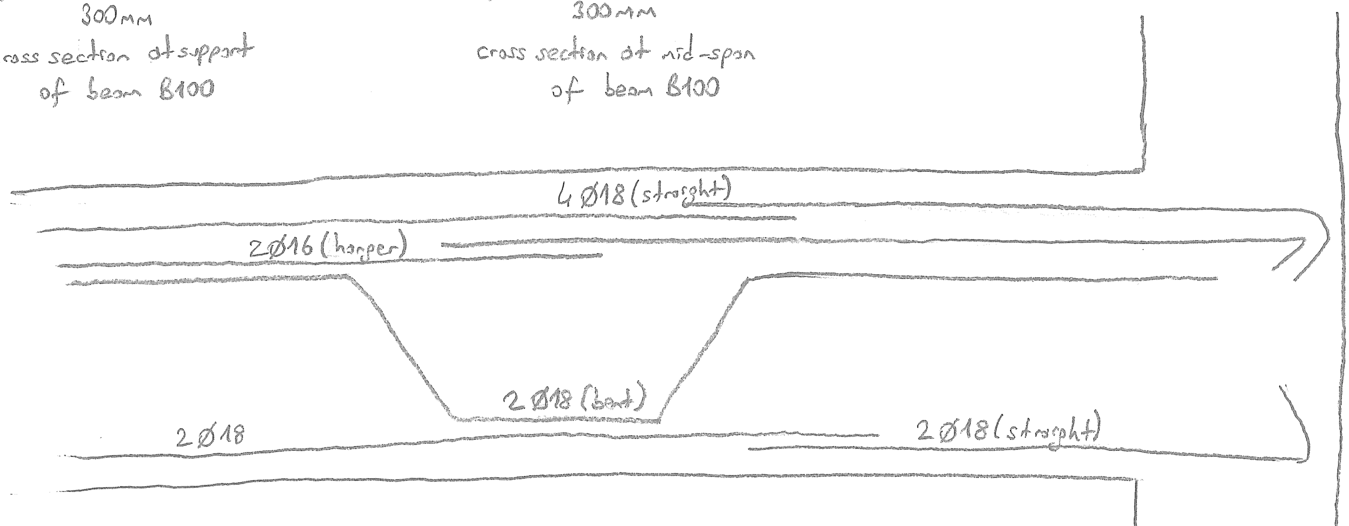
B100



cross section at support of beam B100



cross section at mid-span of beam B100



detail view of beam B100

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