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INTRODUCTION

This report clearly introduces the analysis and design procedures for the structure given as a term project in CE490-Introduction to Earthquake Engineering course in Civil Engineering Department of Middle East Technical University.

PROJECT INFORMATION

This structure is a 6 story police headquarters located at Tokat city center having considerable risk of earthquake and the building rests on ZC type soil.

ARCHITECTURAL PLAN

Architectural plan of the structure is provided at the end of this report. There are 6 stories and an entrance floor in this project. It is also important to note that there is no basement floor in the building and all floors have the same architectural plans.

STRUCTURAL PLAN

Structural plans are provided at the end of this report.

LATERAL LOAD RESISTING SYSTEM

Lateral load resisting system is selected as moment frame system with high ductility level and it is considered that all infilled and curtain walls do not contact with the structural framing system. The structural framing system consists of the following three components;

- Slab Thickness: 15cm
- Beams: 30x60 cm
- Columns: 40x60 cm

It is important to note that there is no shear wall in this structure.

DESIGN CRITERIA

This part of the report will introduce the analysis and drawing tools, design codes, material properties, loads acting on structure and load combinations.

ANALYSIS AND SHOP DRAWING TOOLS

The followings tools are utilized to complete this project.

- SAP2000 v19.2.2
- Autodesk AutoCAD 2017

DESIGN CODES

Design of this structure is performed in accordance with the following specifications.

- Turkish Earthquake Specification for Buildings-TBDY 2017
- Requirements for design and construction of Reinforced Concrete Structures-TS500
- Design Loads for Buildings-TS498

MATERIAL PROPERTIES

This is a reinforced concrete structure as it is stated at the beginning of this report; therefore, the following materials are used in the design of the building.

Table 1: Concrete Properties

Property	Value	Units
Unit Weight	25	kN/m^3
Modulus of Elasticity (E)	32000	MPa
Shear Modulus (G)	12800	MPa
Poisson's Ratio	0.20	-
Coefficient of Thermal Expansion (α)	10^{-5}	/ ºC

Table 2: Reinforcing Steel Properties

Property	**Value**	**Units**
Modulus of Elasticity Es	200000	MPa
Shear Modulus Gs	81000	MPa
Poisson's Ratio	0.30	-
Coefficient of Thermal Expansion	10^{-5}	/ ºC

LOADS ACTING ON THE STRUCTURE

DEAD LOAD

Self-weight of the structure is automatically calculated by SAP2000 software.

LIVE LOAD

Live load acting on the structure is selected as 5 kN/m^2 in accordance with the Section-12 and Table-7 in TS498. Furthermore, live load application at different floor levels are also determined under the guidance of Section-13(Live Load Reduction) and Table-8.

Table 3: Effective Live Load at Floor Levels

Floors	Live Load Reduction Factor (β)	Live Load kN/m^2	Effective Live Load at Floor Levels kN/m^2
1	1,00	5,00	5,00
2	1,00	5,00	5,00
3	1,00	5,00	5,00
4	0,95	5,00	4,75
5	0,88	5,00	4,40
6	0,80	5,00	4,00
Roof Level	1,00	1,50	1,50

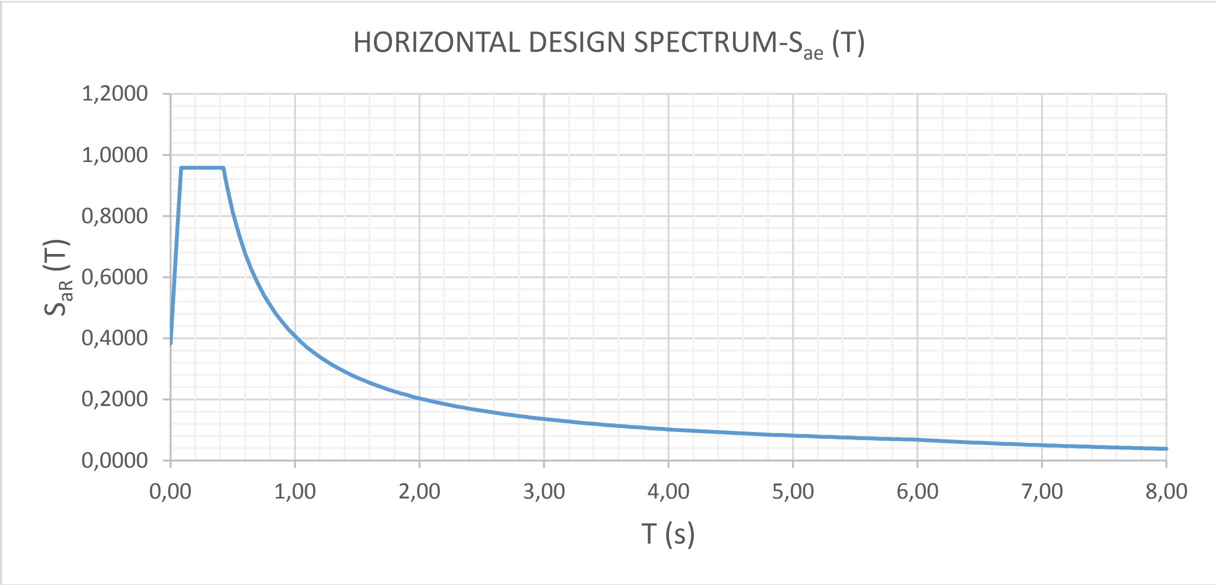
SNOW LOAD

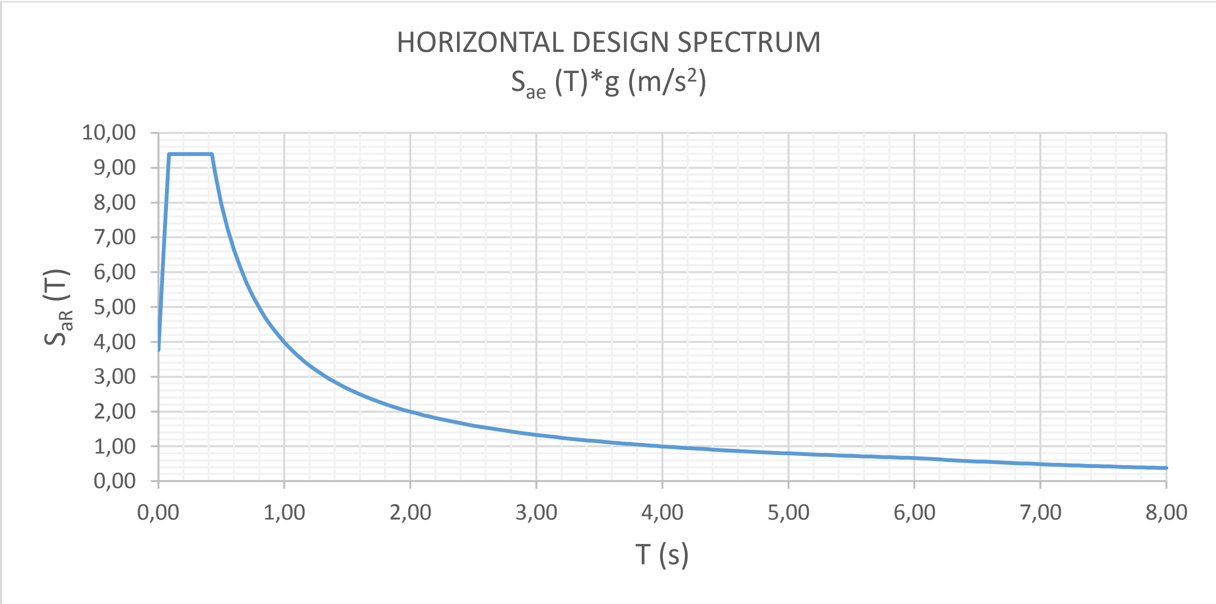
Snow load acting on the structure roof is selected as 0,80 kN/m^2 in accordance with the Section-8 and Table-4 in TS498.

SEISMIC LOAD

Seismic load calculations are performed under the guidance of TBDY 2018 both in equivalent lateral load procedure and mode superposition procedure, and they are introduced in the upcoming Design Base Shear section.

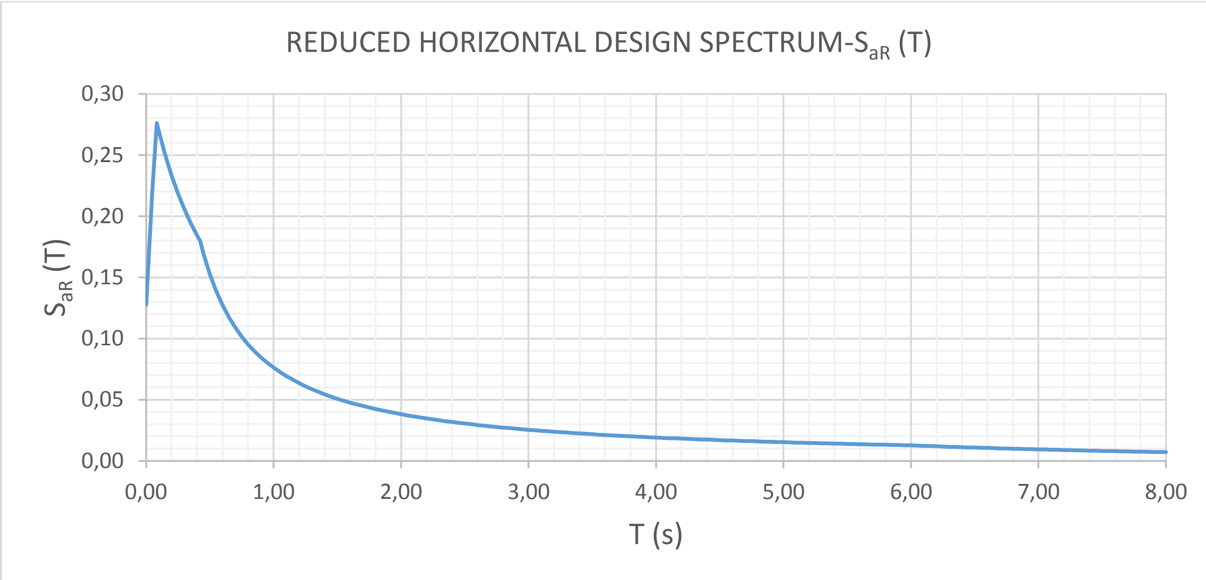
HORIZONTAL DESIGN SPECTRUM

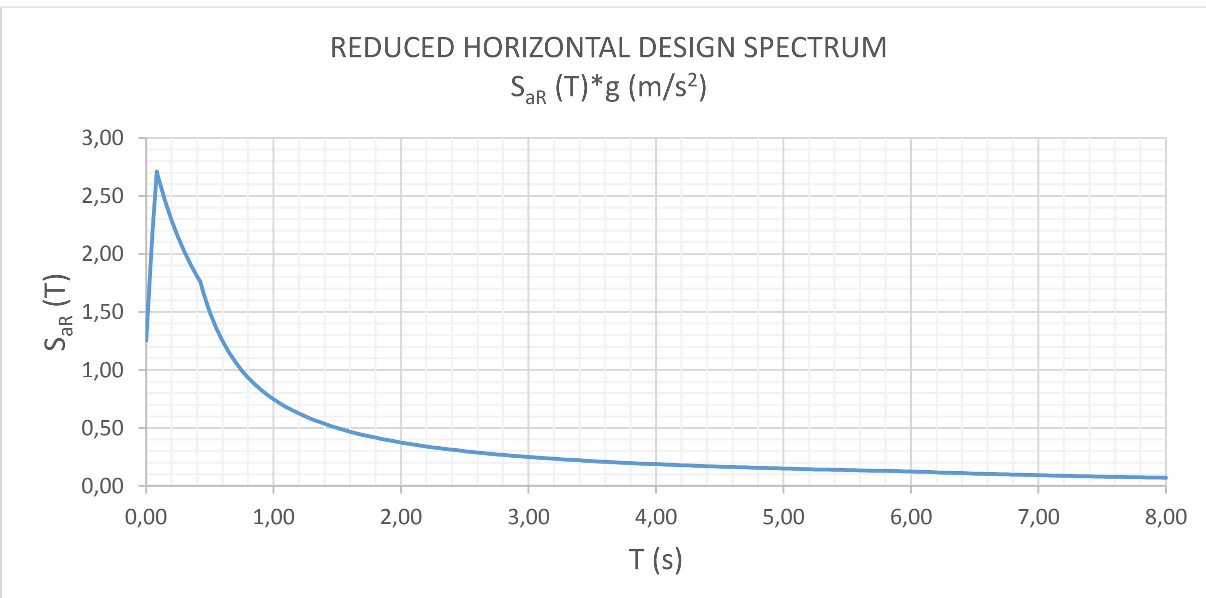




The above two plots demonstrate the horizontal design spectrum for the building resting on ZC soil type in Tokat city center. The first one taken from AFAD Webpage and the other plots are obtained by performing some calculations which are demonstrated in given MS Excel File.

REDUCED HORIZONTAL DESIGN SPECTRUM





The above two plots demonstrate the reduced horizontal design spectrum for the building resting on ZC soil type in Tokat city center.

VERTICAL DESIGN SPECTRUM

Application of vertical design spectrum is not necessary for this building owing to the reasons listed below in accordance with the Section-4.4.3.1 in TDBY 2018.

- There is no column resting on a beam.
- There is no span length larger than 20 m.
- There are no cantilevers larger than 5 m.

LOAD COMBINATIONS

The following load combinations are considered in the design of this structure.

- 1,4G + 1,6Q
- 1,0G + 1,0Q ± 1,0EX ± 0,3EY
- 1,0G + 1,0Q ± 1,0EY ± 0,3EX
- 1,0G + 1,0Q + 0,2S ± 1,0EX ± 0,3EY
- 1,0G + 1,0Q + 0,2S ± 1,0EY ± 0,3EX
- 0,9G ± 1,0EX ± 0,3EY
- 0,9G ± 1,0EY ± 0,3EX

ANALYSIS RESULTS

This part of the report provides the information about the analysis results of the structure.

STRUCTURAL MODEL

Structural model is prepared by using SAP2000 v19.2.2 and slabs, beams and columns are all modelled to achieve the actual building behavior. Centerline modelling is utilized to connect frame elements and 1,2x1,2 m and 1,2x1,0 m meshes are provided in slabs. Moreover, all gravity loads are applied to slabs in kN per meter squares.

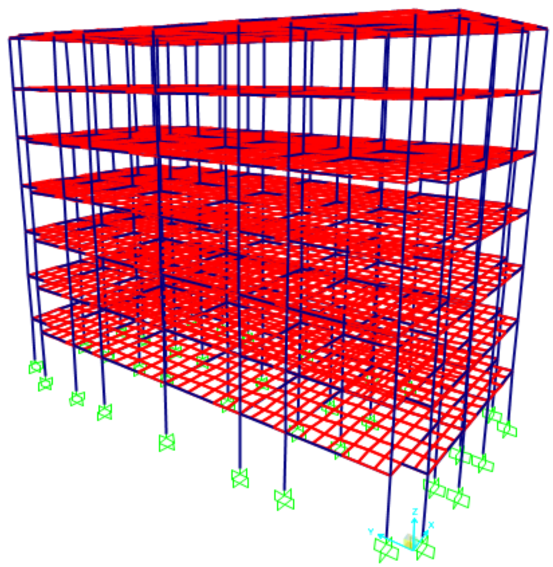


Figure 1: 3D View of Structural Model

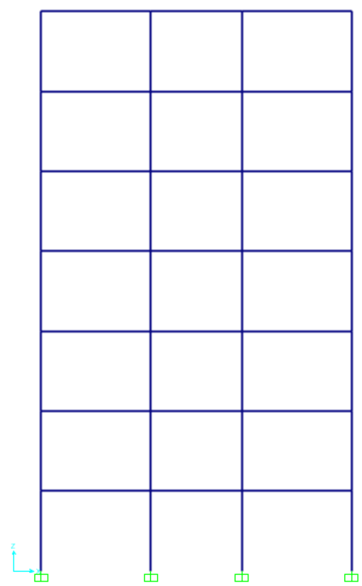


Figure 2: Structural Section View in Short Direction

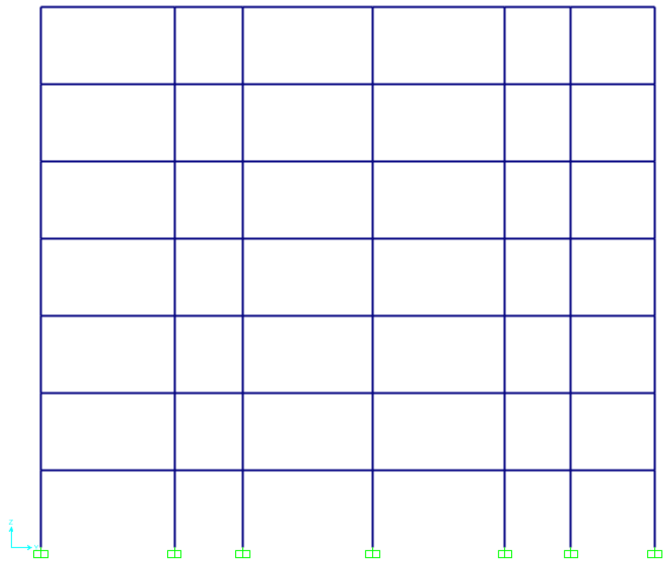


Figure 3: Structural Section View in Long Direction

EIGENVALUE ANALYSIS

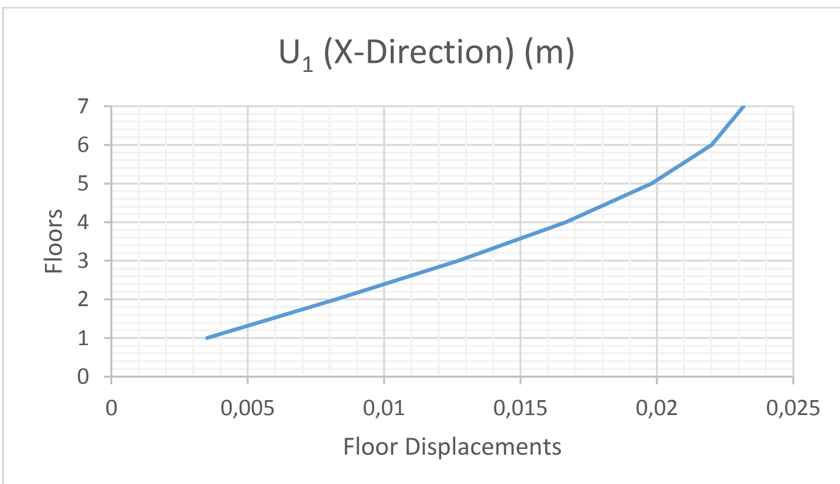
Eigenvalue analysis is performed by utilizing SAP2000 for the first 9 modes and they are presented in tabular and graphical form. R=8 and I =1,5 is selected for this design.

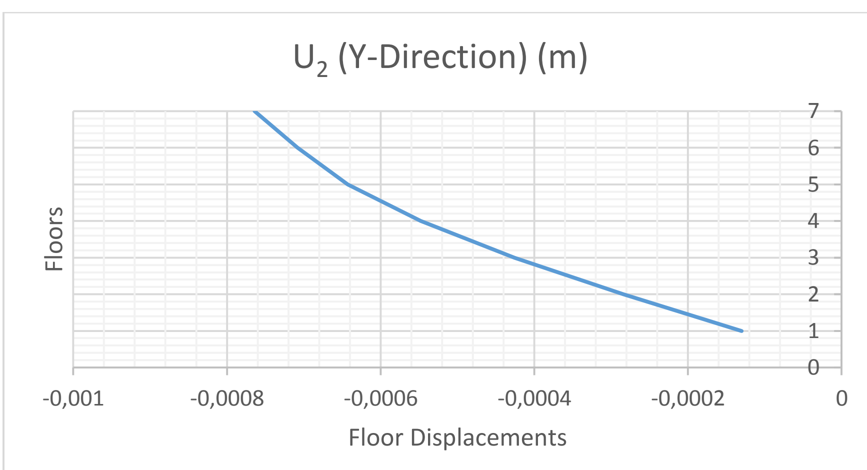
Table 4: Eigenvalue Analysis Results Taken From SAP2000

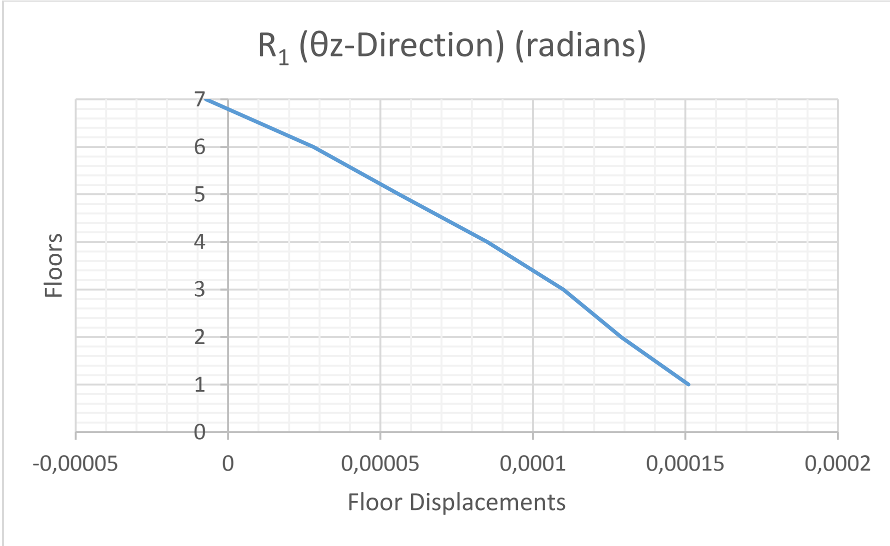
Mode Number	**Period**	**Frequency**	**CircFreq**	**Eigenvalue**
Unitless	Sec	Cyc/sec	rad/sec	rad2/sec2
1	0,839277	1,191502268	7,486429543	56,0466273
2	0,780323	1,281520846	8,052032947	64,83523458
3	0,686769	1,456094369	9,148910744	83,7025678
4	0,276438	3,617443053	22,72906504	516,6103976
5	0,254821	3,924319912	24,65722921	607,9789525
6	0,225745	4,429783772	27,83315231	774,6843677
7	0,160842	6,217289899	39,06438454	1526,02614
8	0,146176	6,841081566	42,98378318	1847,605617
9	0,131216	7,621038612	47,88439783	2292,915556

1st MODE

T (sec)	f (cyc/sec)	W_n (rad/sec)	W_n^2 (rad^2/sec^2)
0,8393	1,1915	7,4864	56,0466
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	0,003502	-0,00013	0,000151
2	0,008239	-0,000284	0,000129
3	0,012737	-0,000426	0,00011
4	0,016658	-0,000548	0,000085
5	0,019798	-0,000643	0,000056
6	0,022004	-0,000708	0,000028
7	0,023182	-0,000764	-0,000007224

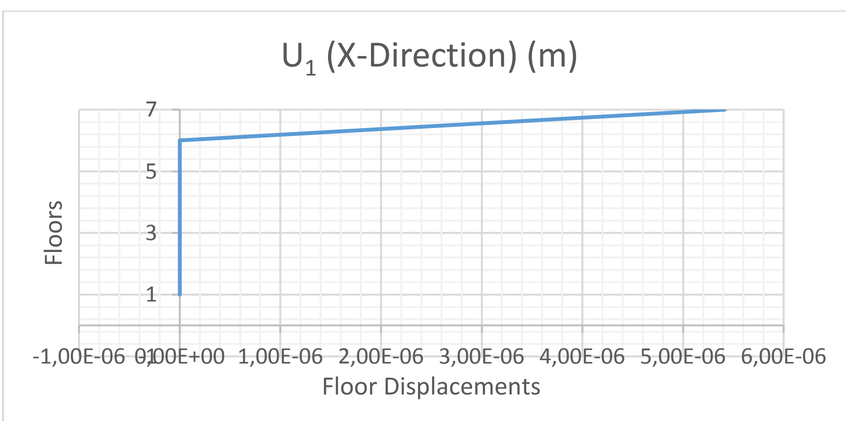


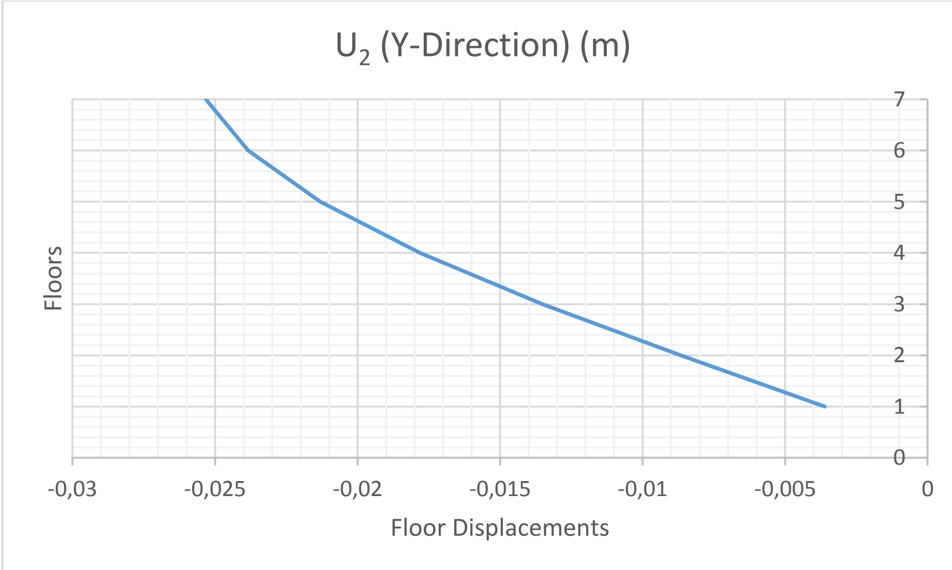


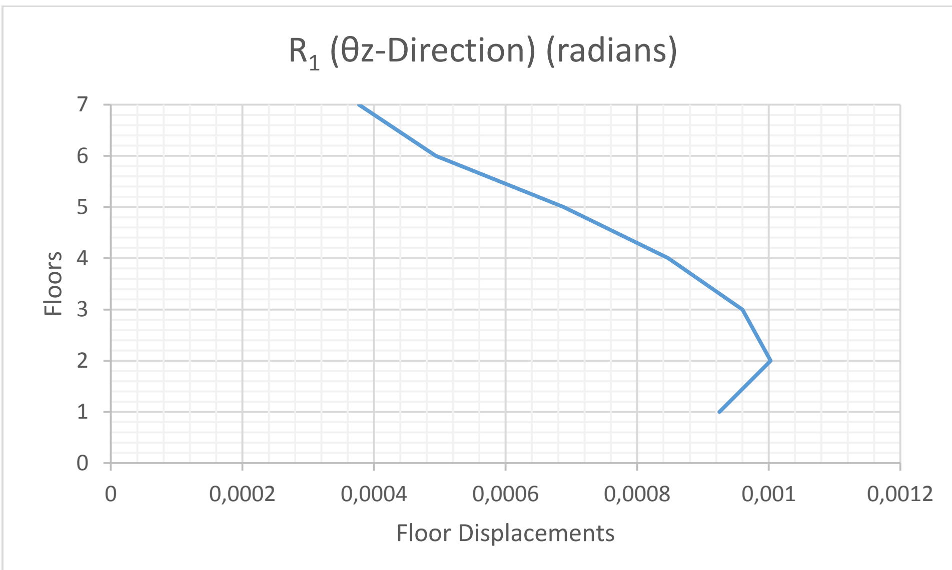


2^{nd} MODE

T (sec)	**f (cyc/sec)**	**W_n (rad/sec)**	**W_n^2 (rad^2/sec^2)**
0,7803	1,2815	8,0520	64,8352
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	-3,939E-11	-0,003611	0,000925
2	5,233E-11	-0,008647	0,001003
3	-4,397E-11	-0,01351	0,00096
4	2,695E-11	-0,017814	0,000847
5	-1,489E-11	-0,021314	0,000688
6	1,308E-11	-0,023834	0,000494
7	0,000005416	-0,025326	0,000377

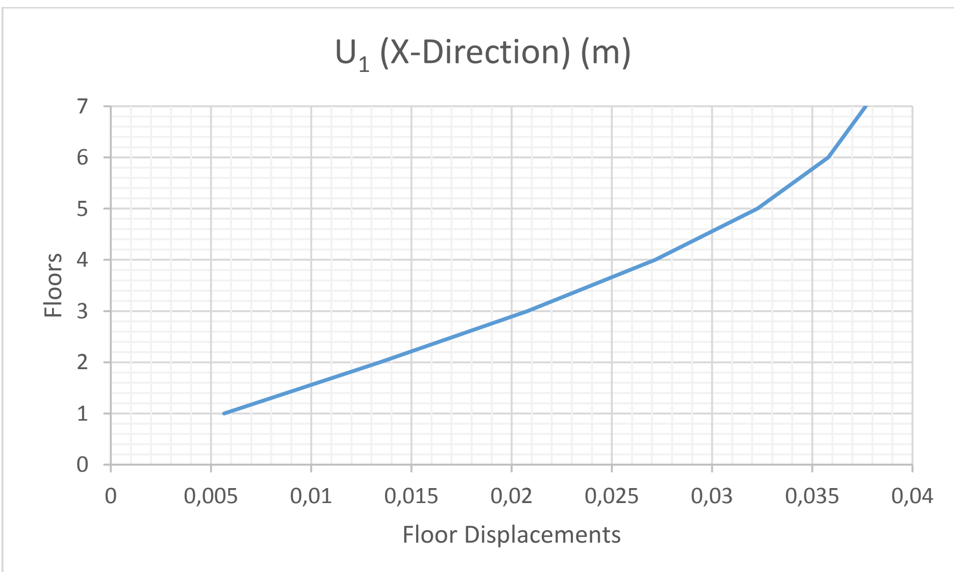


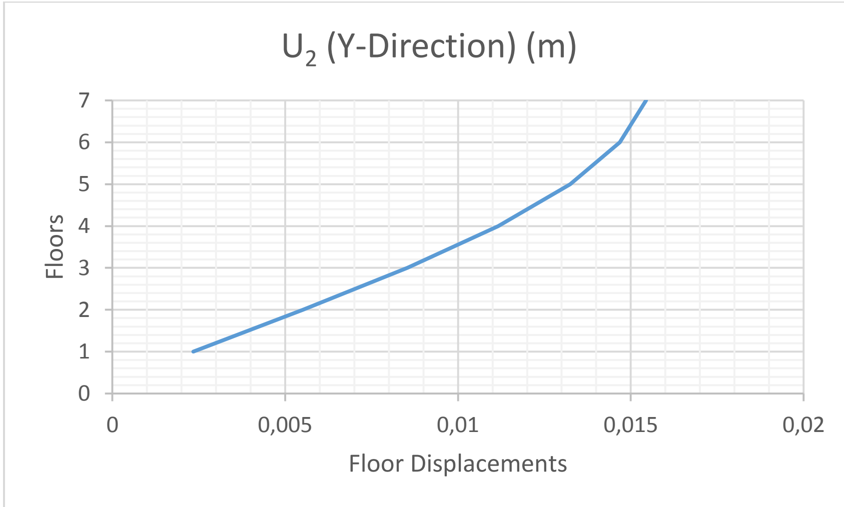

U$_2$ (Y-Direction) (m)
Floors
Floor Displacements
-0,03
-0,025
-0,02
-0,015
-0,01
-0,005
0
0
1
2
3
4
5
6
7

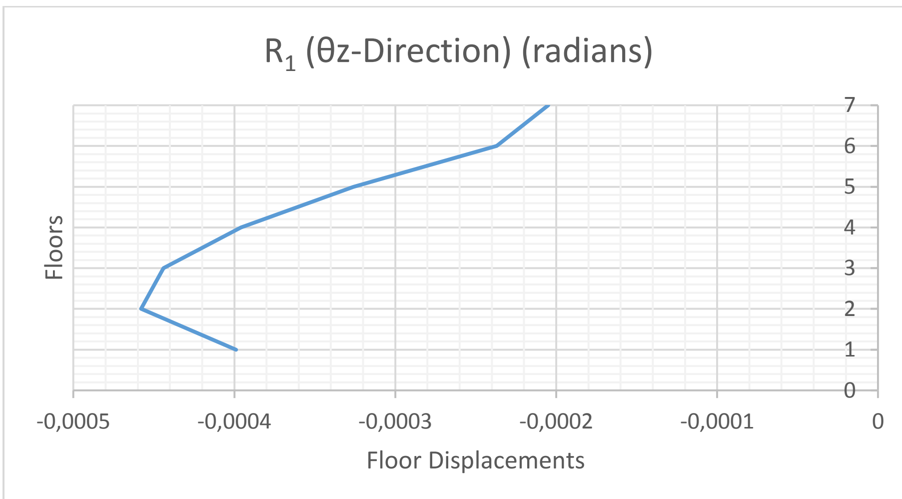

R$_1$ (θz-Direction) (radians)
Floors
Floor Displacements
0
0,0002
0,0004
0,0006
0,0008
0,001
0,0012
0
1
2
3
4
5
6
7

3^{rd} MODE

T (sec)	f (cyc/sec)	W_n (rad/sec)	W_n^2 (rad^2/sec^2)
0,6868	1,4561	9,1489	83,7026
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	0,005655	0,002336	-0,000399
2	0,013414	0,005522	-0,000458
3	0,020779	0,008541	-0,000444
4	0,027177	0,011161	-0,000396
5	0,032264	0,013241	-0,000326
6	0,035795	0,014683	-0,000237
7	0,037666	0,01544	-0,000205

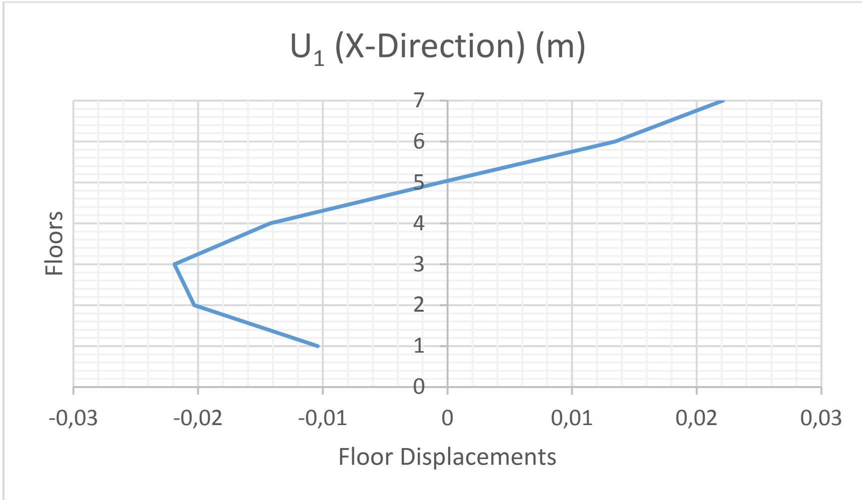


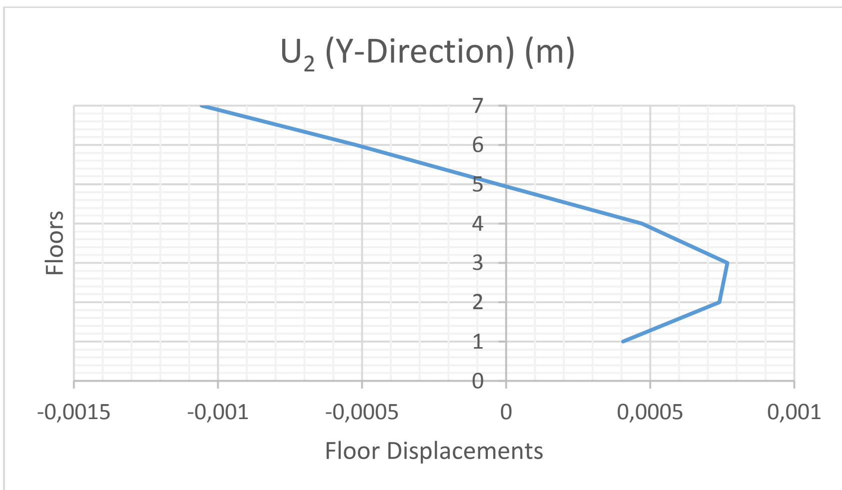


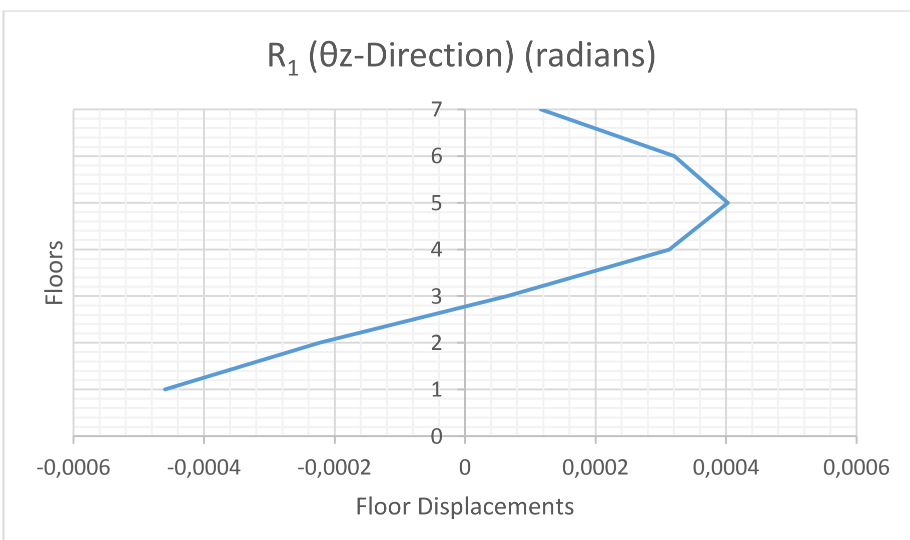


4th MODE

T (sec)	f (cyc/sec)	W_n (rad/sec)	W_n^2 (rad^2/sec^2)
0,2764	3,6174	22,7291	516,6104
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	-0,010413	0,000405	-0,00046
2	-0,020313	0,00074	-0,000223
3	-0,021907	0,000768	0,000065
4	-0,014243	0,000471	0,000313
5	-0,000555	-0,000028	0,000403
6	0,013454	-0,000521	0,000321
7	0,022109	-0,001057	0,000116

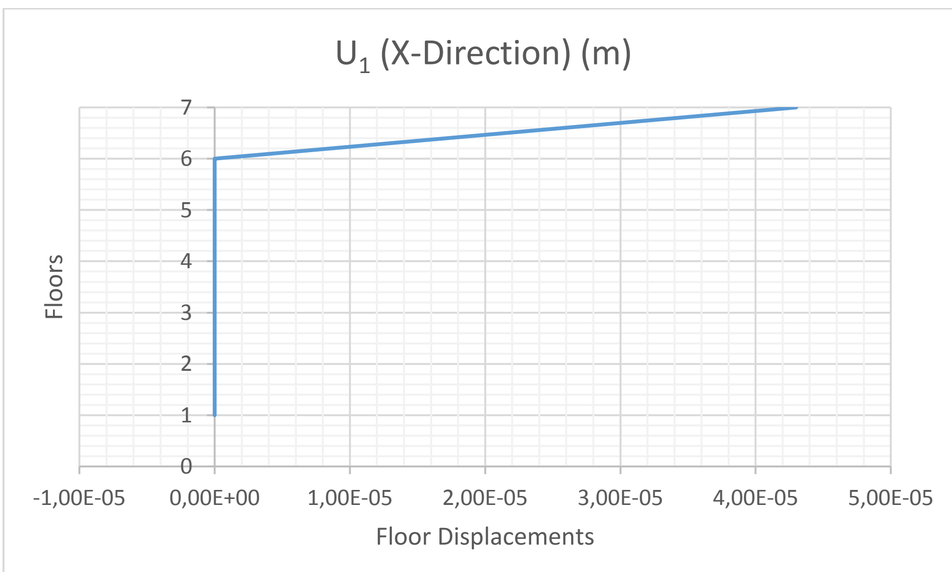

U1 (X-Direction) (m)
Floors
7
6
5
4
3
2
1
0
-0,03
-0,02
-0,01
0
0,01
0,02
0,03
Floor Displacements

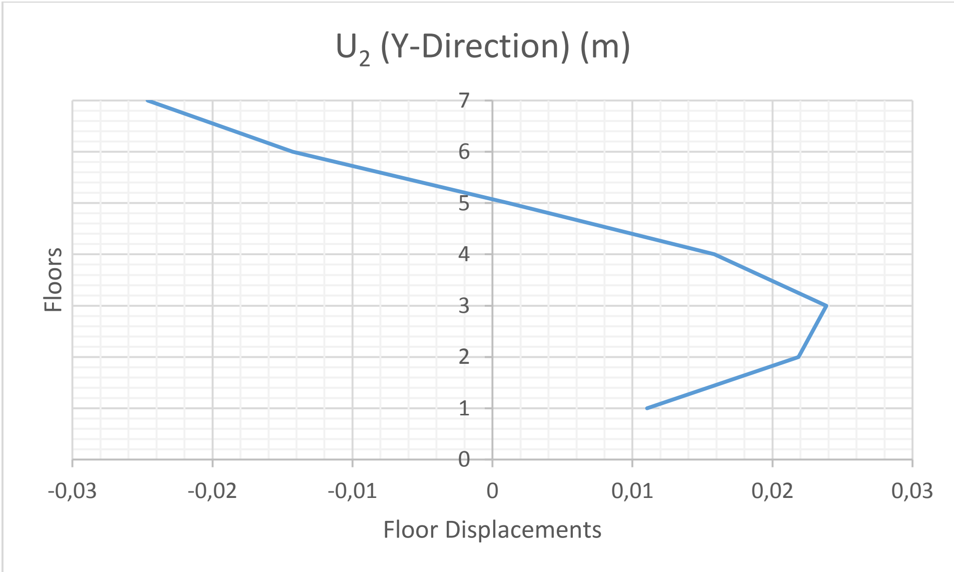

U2 (Y-Direction) (m)
Floors
7
6
5
4
3
2
1
0
-0,0015
-0,001
-0,0005
0
0,0005
0,001
Floor Displacements

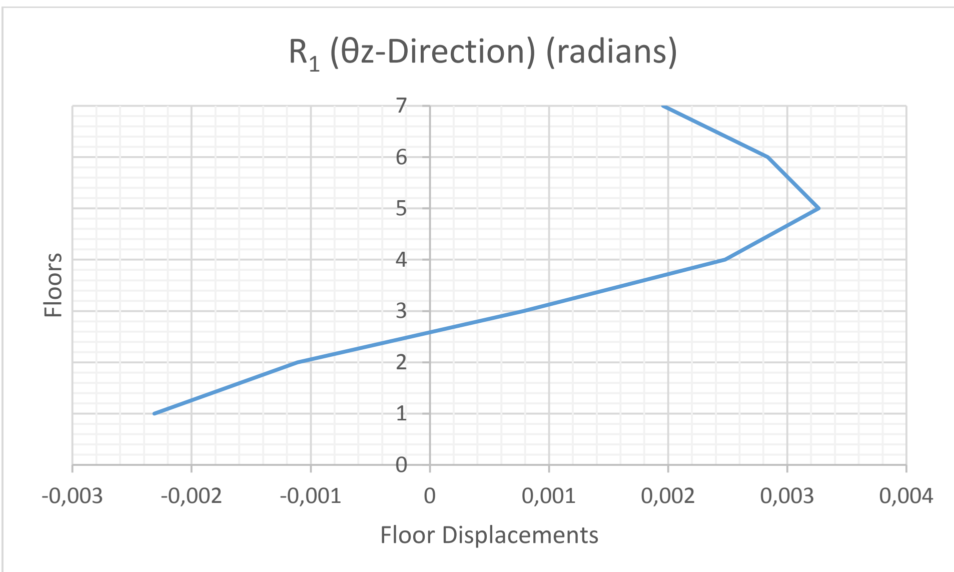

R1 (θz-Direction) (radians)
Floors
7
6
5
4
3
2
1
0
-0,0006
-0,0004
-0,0002
0
0,0002
0,0004
0,0006
Floor Displacements

5^{th} MODE

T (sec)	f (cyc/sec)	W_n (rad/sec)	W_n^2 (rad^2/sec^2)
0,2548	3,9243	24,6572	607,9790
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	-1,224E-10	0,01105	-0,002312
2	5,12E-11	0,021864	-0,001113
3	3,081E-12	0,023855	0,000784
4	9,298E-11	0,015843	0,002478
5	-2,927E-10	0,001119	0,003263
6	3,403E-10	-0,014281	0,002837
7	0,000043	-0,024638	0,001957

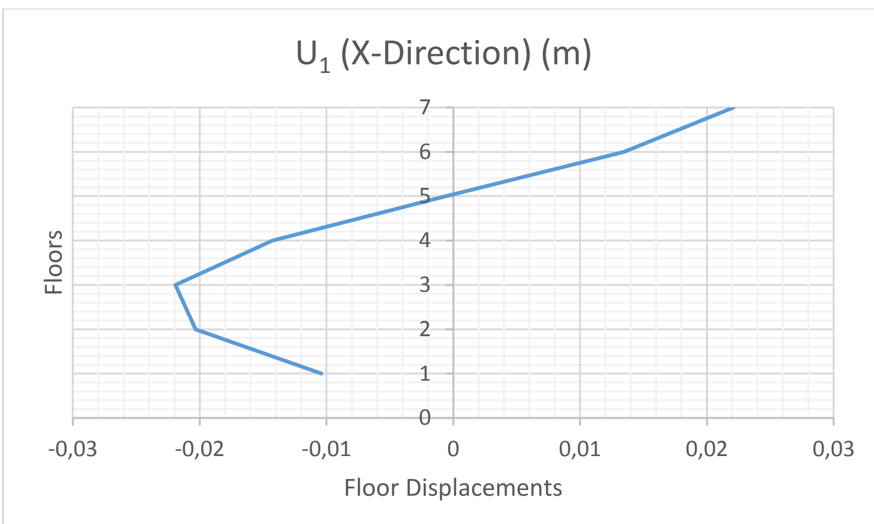


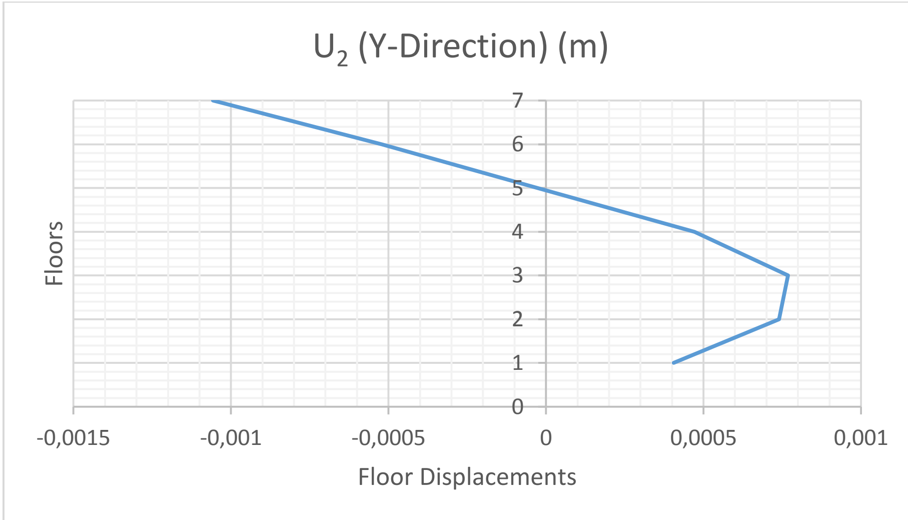

U_2 (Y-Direction) (m)
Floors
Floor Displacements
0
1
2
3
4
5
6
7
-0,03
-0,02
-0,01
0
0,01
0,02
0,03

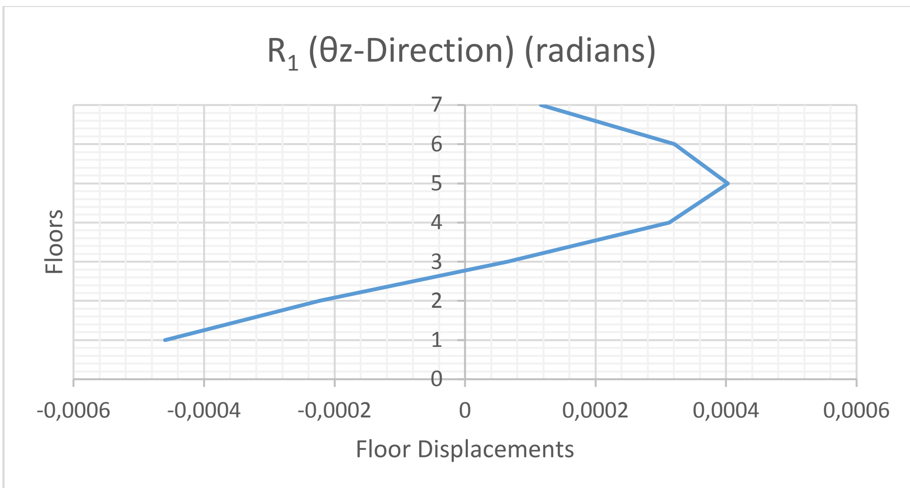

R_1 (θz-Direction) (radians)
Floors
Floor Displacements
0
1
2
3
4
5
6
7
-0,003
-0,002
-0,001
0
0,001
0,002
0,003
0,004

6th MODE

T (sec)	f (cyc/sec)	W_n (rad/sec)	W_n^2 (rad^2/sec^2)
0,2257	4,4298	27,8332	774,6844
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	0,016884	0,00695	-0,000869
2	0,033275	0,013642	-0,000385
3	0,035985	0,014715	0,000446
4	0,023323	0,009505	0,001176
5	0,000656	0,000226	0,001508
6	-0,022466	-0,00921	0,001292
7	-0,036951	-0,014999	0,000968

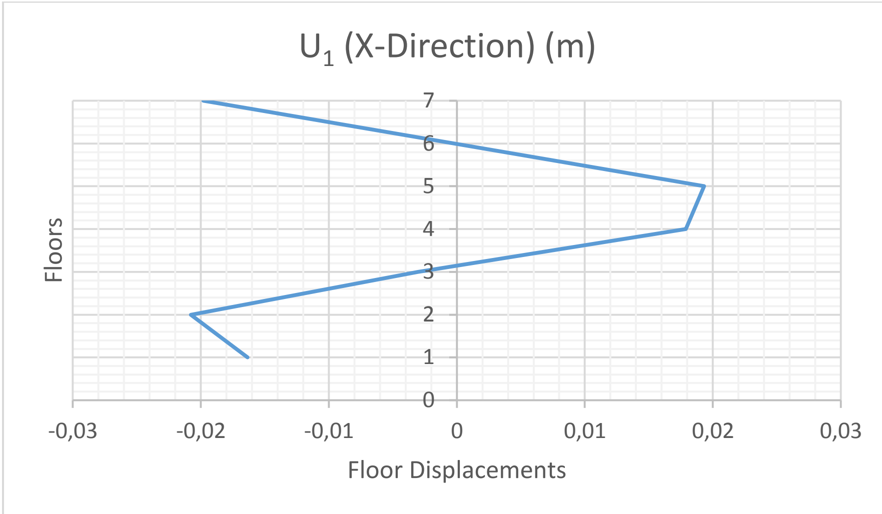


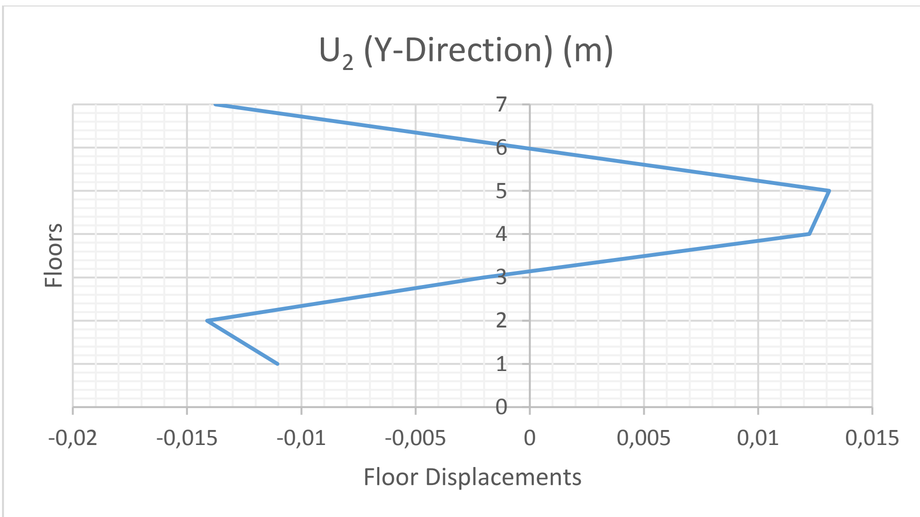


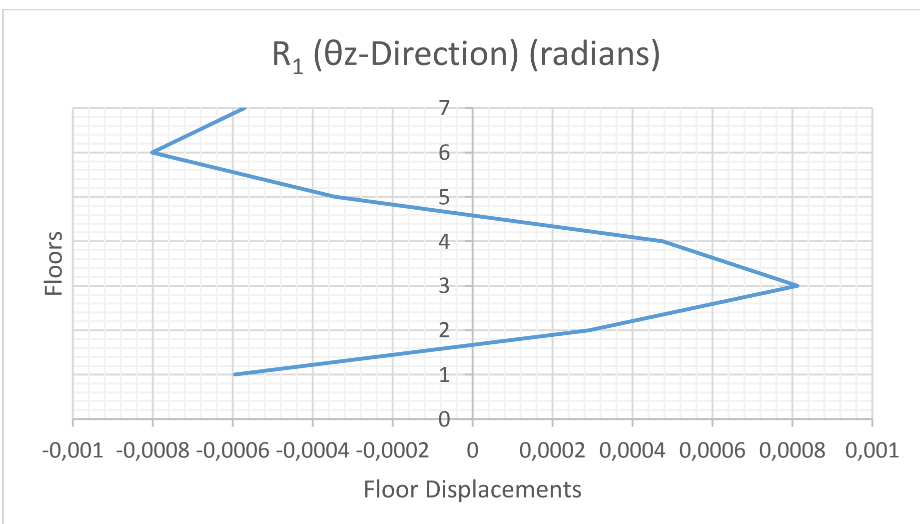


7^{th} MODE

T (sec)	**f (cyc/sec)**	**W_n (rad/sec)**	**W_n^2 (rad^2/sec^2)**
0,1608	6,2173	39,0644	1526,0261
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	-0,016346	0,000708	-0,000595
2	-0,020789	0,000822	0,000291
3	-0,003042	0,000033	0,000813
4	0,017886	-0,000819	0,000475
5	0,019314	-0,000801	-0,000345
6	-0,000173	0,000085	-0,000802
7	-0,019807	0,001656	-0,000571

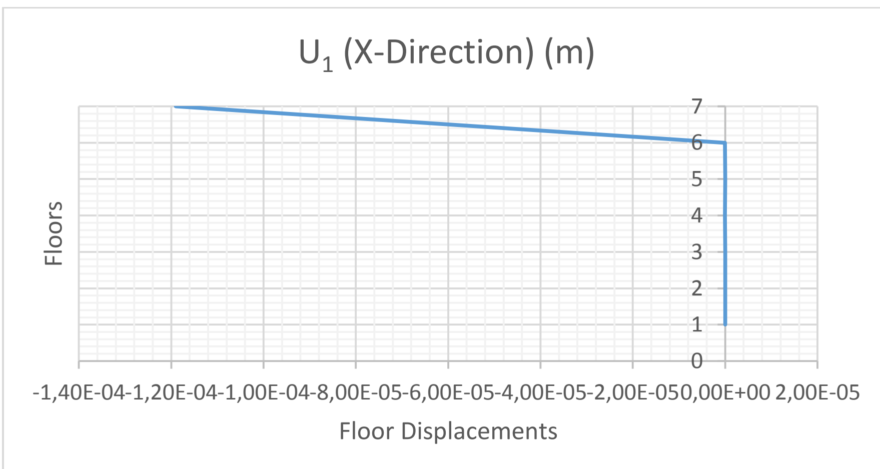

U1 (X-Direction) (m)
Floors
Floor Displacements
7
6
5
4
3
2
1
0
-0,03
-0,02
-0,01
0
0,01
0,02
0,03

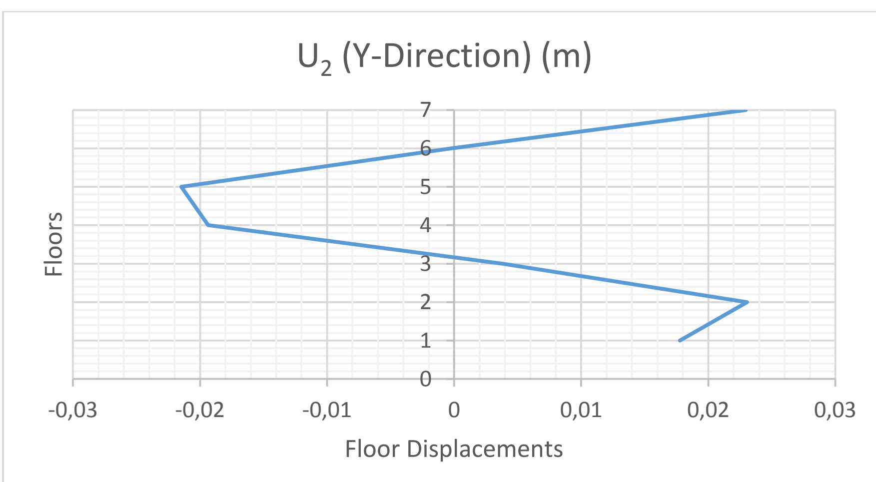

U2 (Y-Direction) (m)
Floors
Floor Displacements
7
6
5
4
3
2
1
0
-0,02
-0,015
-0,01
-0,005
0
0,005
0,01
0,015

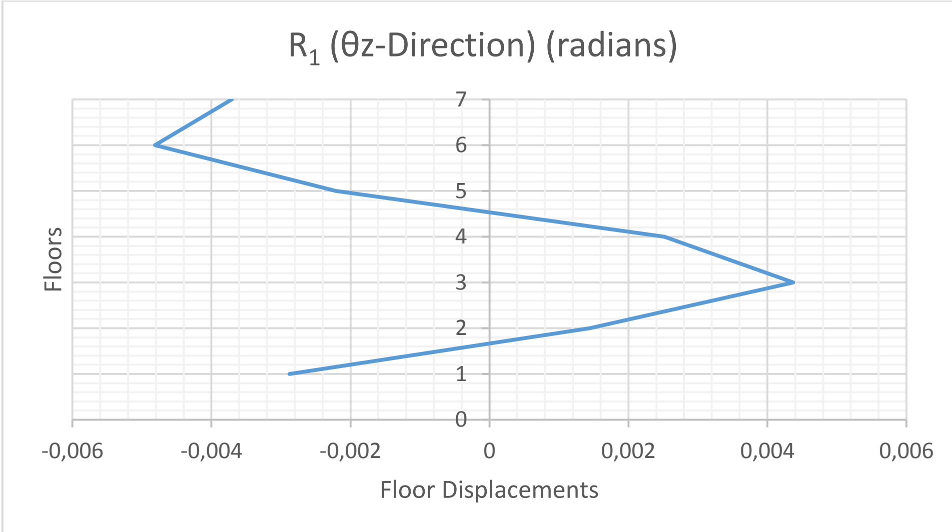

R1 (θz-Direction) (radians)
Floors
Floor Displacements
7
6
5
4
3
2
1
0
-0,001
-0,0008
-0,0006
-0,0004
-0,0002
0
0,0002
0,0004
0,0006
0,0008
0,001

8th MODE

T (sec)	f (cyc/sec)	W_n (rad/sec)	W_n^2 (rad^2/sec^2)
0,1462	6,8411	42,9838	1847,6056
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	5,881E-09	0,017764	-0,002876
2	-9,756E-09	0,023055	0,001445
3	1,727E-08	0,003861	0,004373
4	-3,107E-08	-0,019351	0,002519
5	4,276E-08	-0,021473	-0,002211
6	-3,924E-08	-0,000253	-0,004815
7	-0,000119	0,02296	-0,003706

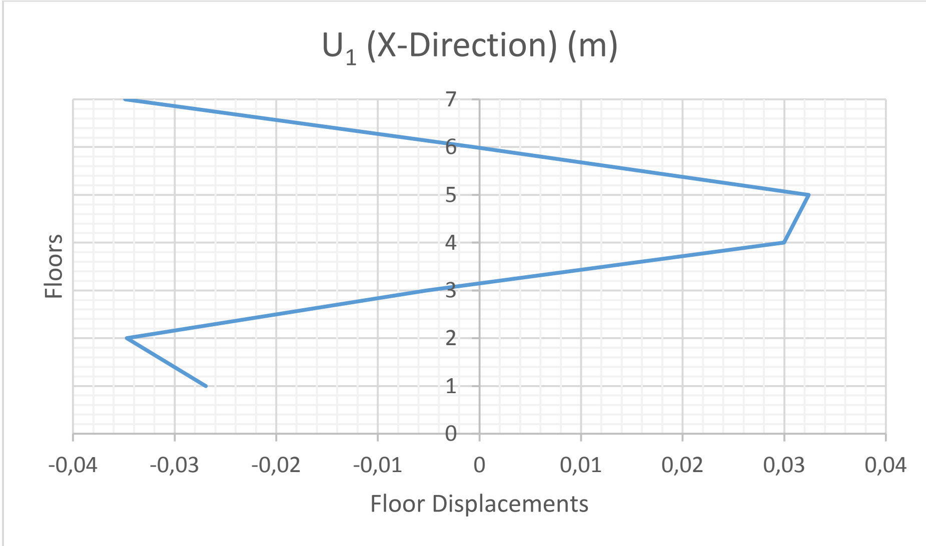


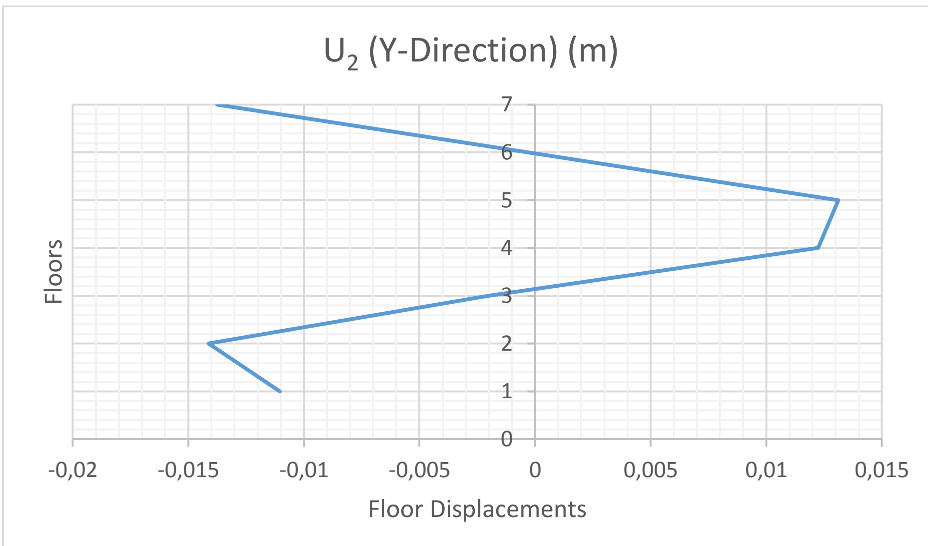


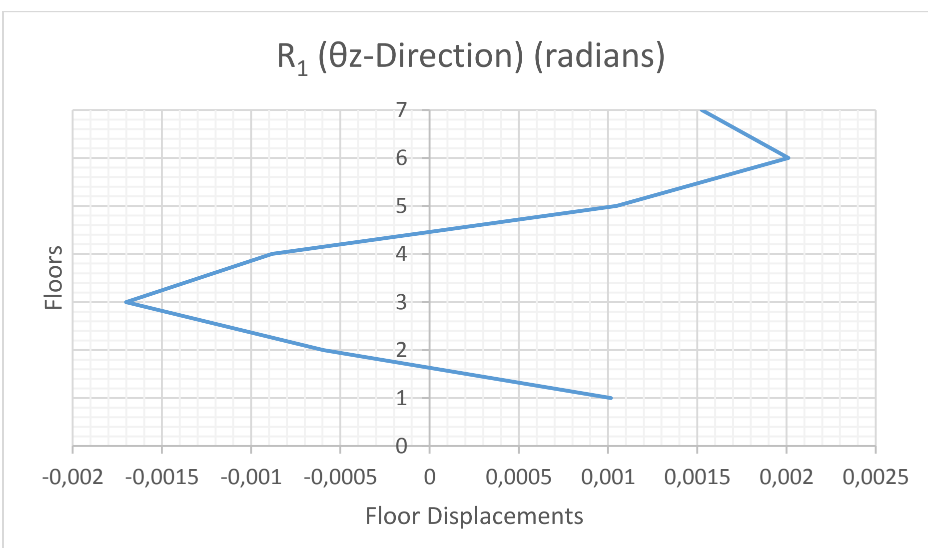


9^{th} MODE

T (sec)	**f (cyc/sec)**	**W_n (rad/sec)**	**W_n^2 (rad^2/sec^2)**
0,1312	7,6210	47,8844	2292,9156
Floors	**U_1 X-direction[m]**	**U_2 Y-Direction[m]**	**R_1 (θz-Direction) (radians)**
1	-0,026943	-0,011045	0,001016
2	-0,034712	-0,014124	-0,000596
3	-0,005207	-0,002028	-0,001701
4	0,029976	0,012244	-0,000882
5	0,032398	0,013115	0,001049
6	-0,000617	-0,00033	0,00201
7	-0,034874	-0,013757	0,001525


U_1 (X-Direction) (m)
Floors
Floor Displacements
0
1
2
3
4
5
6
7
-0,04
-0,03
-0,02
-0,01
0
0,01
0,02
0,03
0,04


U_2 (Y-Direction) (m)
Floors
Floor Displacements
0
1
2
3
4
5
6
7
-0,02
-0,015
-0,01
-0,005
0
0,005
0,01
0,015


R_1 (θz-Direction) (radians)
Floors
Floor Displacements
0
1
2
3
4
5
6
7
-0,002
-0,0015
-0,001
-0,0005
0
0,0005
0,001
0,0015
0,002
0,0025

MODAL MASSES

Mass Source (G+0.3Q);

Number of Floor= 6+ 1 (roof) = 7

Number of columns at a floor = 36

Slab thickness = 0.15 m

Slab area =504.76 m^2

Slab volume = 0.15 x 504.76 = 75.714 m^3

Beam dimensions = 0.3 x 0.6 m^2

Beam cross-section area under slab = (0.6-0.15) x 0.3 = 0.135 m^2

Beam length of a roof = 268 m

Beam volume under a slab = 268 x 0.135 = 36.18 m^3

Column dimension = 0.4x0.6 = 0.24 m^2

Column Length = (3.5-0.6) = 2.90 m

Column volume = 0.24 x 2.90 = 0.696 m^3

Unit Weight of Reinforced Concrete = 25kN/m^3

Live load for floor = 5 kPa

Live load for roof = 1.5 kPa

Snow load = 0.8 kPa

DEAD LOAD = (75.714+36.18+0.696 x 36) x 25 x 7 = 23966.25 kN

LIVE LOAD = 504.76 x (5 x 6 + (1.5+0.8) x 1) = 16303.748 kN

Mass Source = (23966.25 + 0.3 x 16303.748)/9.81 = 2941.6 tons

Table 5: Modal Mass Data from SAP2000

OutputCase	CaseType	GlobalFZ	GlobalMX
Text	Text	KN	KN-m
G	LinStatic	26983,95	432939,49
Q	LinStatic	14966,134	239319,77

To calculate the required information, the following formulas are used in MATLAB. Matlab code is also given as file in mail.

- Modal Mass, $M_n = Ø_n^T * m * Ø_n$
- Modal Excitation Factor, $L_{ni} = Ø_n^T * m * l_i$
- Effective Modal Mass, $M_{ni}^* = L_{ni}^2 / M_n$
- Modal Force, $F_n = (L_{ni}/M_n) * m * Ø_n * Sa_n$

Where i = x or y

MATLAB SCRIPT;

```
clear all
close all
clc
mass=diag([87.03  87.03  2320  87.03  87.03  2320  87.03  87.03  2320  87.03  87.03  2320
87.03 87.03 2320 79.355 79.355 2116.13]);
phi_1=load('mode_1.txt'); % mode shape for 1. mode
phi_1_Transpose=transpose(phi_1);
phi_2=load('mode_2.txt'); % mode shape for 2. mode
phi_2_Transpose=transpose(phi_2);
phi_3=load('mode_3.txt'); % mode shape for 3. mode
phi_3_Transpose=transpose(phi_3);
l_x=load('l_x.txt'); % length factor (1 0 0 1 0 0...)
l_y=load('l_y.txt'); % length factor (0 1 0 0 1 0...)

M_1= phi_1_Transpose*mass*phi_1; % modal mass for 1. mode
M_2= phi_2_Transpose*mass*phi_2; % modal mass for 2. mode
M_3= phi_3_Transpose*mass*phi_3; % modal mass for 3. mode
fprintf('modal masses %f\n', M_1,M_2,M_3)

L_1= phi_1_Transpose*mass*l_y; % modal excitation factor for 1. mode
L_2= phi_2_Transpose*mass*l_x; % modal excitation factor for 2. mode
L_3= phi_3_Transpose*mass*l_x; % modal excitation factor for 3. mode (lx or ly)
fprintf('modal excitation factors %f\n', L_1,L_2,L_3)

M_eff_1= (L_1)^2/M_1;  % effective modal mass for 1. mode
M_eff_2= (L_2)^2/M_2;  % effective modal mass for 2. mode
M_eff_3= (L_3)^2/M_3;  % effective modal mass for 3. mode
fprintf('effective modal masses %f\n', M_eff_1,M_eff_2,M_eff_3)

f_1=(L_1/M_1)*mass*phi_1; % modal forces for 1. mode
f_2=(L_2/M_2)*mass*phi_2; % modal forces for 2. mode
f_3=(L_3/M_3)*mass*phi_3; % modal forces for 3. Mode
```

As a result of the MATLAB calculation made (given in above), modal mass, modal excitation, effective modal mass and modal force vectors are found as follows:

MODE NUMBER	**1**	**2**	**3**		
Modal Masses, M_n	0,9998	1	1		
Modal Excitation Factor, Ln	20,5667	-20,6154	-3,31E-14		
Effective Modal Mass, Mn*	423,0919	424,9994	1,10E-27		
MODE NUMBER	**1**	**2**	**3**	Floor Number	Direction
MODAL FORCES, fn/SaR	2,15E-14	17,8843471	1,61E-29	1	X
	17,43087786	-9,96E-14	-1,77E-29		Y
	2,65E-13	-1,17E-13	1,59E-13		θ_z
	1,92E-13	44,12058383	8,94E-29	2	X
	43,40353041	-9,35E-14	-4,53E-29		Y
	2,48E-13	-3,69E-13	3,77E-13		θ_z
	1,44E-13	68,68464845	1,81E-28	3	X
	68,01909529	-2,44E-13	-5,34E-29		Y
	1,96E-13	-5,75E-13	5,76E-13		θ_z
	2,87E-13	88,95704391	2,76E-28	4	X
	88,54012262	-1,86E-13	-1,16E-28		Y
	1,40E-13	-8,49E-13	7,39E-13		θ_z
	2,61E-13	103,4575749	2,67E-28	5	X
	103,4340701	-3,10E-13	-1,13E-28		Y
	3,18E-13	-8,20E-13	8,54E-13		θ_z
	3,03E-13	101,8952272	2,67E-28	6	X
	102,2641677	-3,04E-13	-8,37E-29		Y
	4,02E-13	-8,84E-13	8,37E-13		θ_z

MINIMUM NUMBER OF MODES TO BE CONSIDERED IN DESIGN

Table 6: Modal Mass Participations taken from SAP2000

OutputCase	StepType	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	0,839277	0,82794	0	4,763E-19	0,82794	0	4,763E-19
MODAL	Mode	2	0,780323	0	0,82222	2,428E-07	0,82794	0,82222	2,428E-07
MODAL	Mode	3	0,686769	0,0019	0	0	0,82984	0,82222	2,428E-07
MODAL	Mode	4	0,276438	0,10051	0	1,195E-16	0,93035	0,82222	2,428E-07
MODAL	Mode	5	0,254821	2,03E-19	0,1054	3,083E-07	0,93035	0,92762	5,511E-07
MODAL	Mode	6	0,225745	0,00028	5,881E-19	4,527E-17	0,93063	0,92762	5,511E-07
MODAL	Mode	7	0,160842	0,03575	1,917E-16	1,046E-14	0,96638	0,92762	5,511E-07
MODAL	Mode	8	0,146176	6,127E-17	0,03717	1,182E-07	0,96638	0,96479	6,693E-07
MODAL	Mode	9	0,131216	0,00013	5,522E-15	8,148E-13	0,9665	0,96479	6,693E-07

As it is seen from Figure-5, modal mass participation is above the 90 percent after the 4th mode and therefore minimum number of modes should be 4 for this structure.

According to TDBY 2018;

$$\sum_{n=1}^{YM} m_{txn}^{(X)} \geq 0.90 m_t \quad ; \quad \sum_{n=1}^{YM} m_{tyn}^{(Y)} \geq 0.90 m_t$$

DESIGN BASE SHEAR

EQUIVALENT LATERAL LOAD PROCEDURE

Because of time limitation, our team could not manage to perform equivalent lateral load procedure.

THE MODE SUPERPOSITION PROCEDURE

The mode superposition procedure is applied by means of SAP2000 and all the calculations and design are performed in accordance with this method.

Table 7: Design Base Shears in X and Y Directions in SAP2000

OutputCase	CaseType	StepType	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
Envelope	Combination	Max	2475,169	2644,231	61723,344	989026,9157	-154444,311	52586,192
Envelope	Combination	Min	-2475,169	-2644,231	24283,656	347033,7571	-493786,76	-52586,192

Design base shear is calculated by utilizing the SAP2000 and they are shown in figure above.

BUILDING IRREGULARITY CHECKS

This section provides the information about the irregularity checks of the structure in accordance with the TBDY-2018.

Earthquake code states that relative floor translations for the buildings having an isolation joint between structural framing system and the nonstructural systems should satisfy the following condition;

$$\Delta_i^X = u_i^X - u_{i-1}^X$$

$$\delta_i^X = \frac{R}{I} \Delta_i^X \ where\ R = 8\ and\ I = 1{,}5\ in\ this\ project.$$

$$\lambda \frac{\delta_{i,max}^{(X)}}{h_i} \leq 0{,}016\ where\ h_i = 3{,}5\ m\ in\ this\ project$$

CALCULATION OF λ

According to Section-4.9.1.4 in TDBY 2018, λ is the ratio of calculated elastic design spectrum acceleration of DD-3 earthquake in accordance with the Section-2.3.5.1 to elastic design spectrum acceleration of DD-2 earthquake for the vibration period in the direction of earthquake.

$T_1 = 0{,}84\ s\ and\ T_2 = 0{,}78\ s\ for\ the\ Y\ and\ X\ directions, respectively.$

$According\ to\ 2.3.5.1\ S_{ae}\ (T) = \frac{S_{D1}}{T}\ when\ T_B < T \leq T_L$

$T_L = 6\ s\ for\ both\ DD-2\ and\ DD-3\ earthquakes$

$T_B = 0{,}383\ for\ DD-3\ earthquake$

$T_B = 0{,}424\ for\ DD-2\ earthquake$

$S_{D1} = 0{,}156\ for\ DD-3\ earthqauke\ and\ it\ is\ taken\ from\ TBDY-2018\ website$

$S_{D1} = 0{,}406\ for\ DD-2\ earthqauke\ and\ it\ is\ taken\ from\ TBDY-2018\ website$

$$\lambda = \frac{S_{ae}\ (T_1)^{DD-3}}{S_{ae}\ (T_1)^{DD-2}} = \frac{\frac{S_{D1}{}^{DD-3}}{T_1}}{\frac{S_{D1}{}^{DD-2}}{T_1}} = \frac{S_{D1}{}^{DD-3}}{S_{D1}{}^{DD-2}} = \frac{0{,}156}{0{,}406} = 0{,}3842\ for\ T_1\ period.$$

$$\lambda = \frac{S_{ae}\ (T_2)^{DD-3}}{S_{ae}\ (T_2)^{DD-2}} = \frac{\frac{S_{D1}{}^{DD-3}}{T_2}}{\frac{S_{D1}{}^{DD-2}}{T_2}} = \frac{S_{D1}{}^{DD-3}}{S_{D1}{}^{DD-2}} = \frac{0{,}156}{0{,}406} = 0{,}3842\ for\ T_2\ period.$$

As it is shown in below two tables, building is safe with respect to relative floor translations

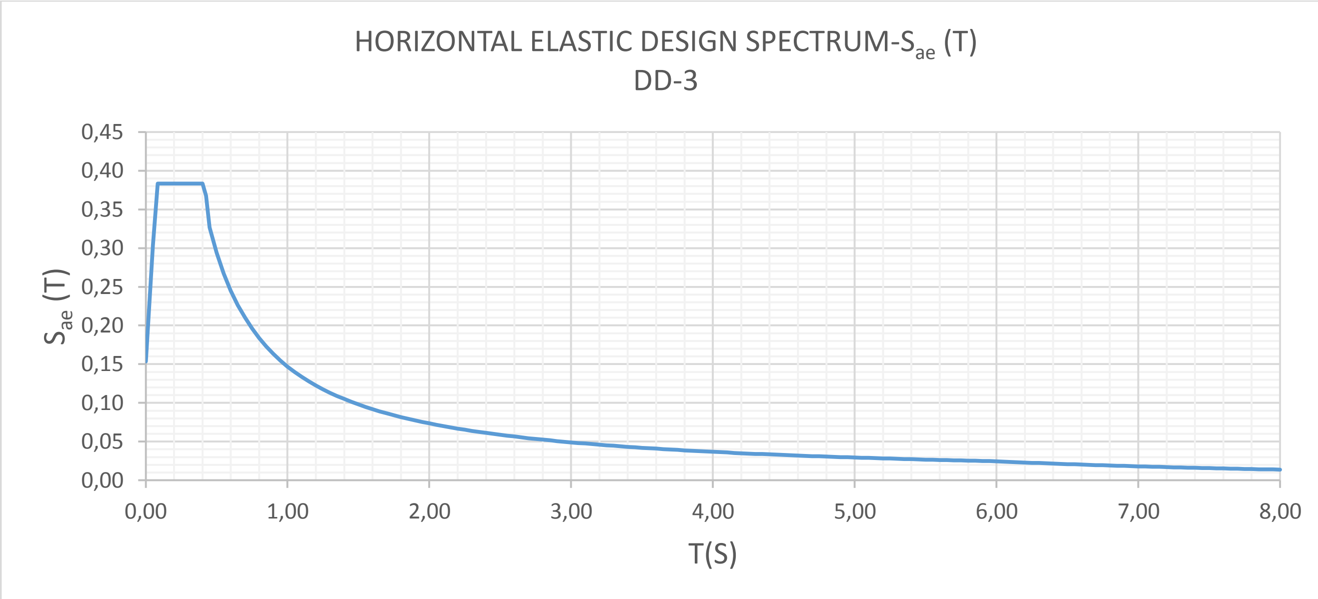


Table 8: Building Drift Check in Y-Direction Under the $1{,}0G+1{,}0Q+1{,}0E_Y+0{,}3E_X$

Y-DIRECTION (T_1=0,84 s)					
$1{,}0G+1{,}0Q+1{,}0E_X+0{,}3E_Y$					
Floors	**$u_i^{(EY)}$ (mm)**	**$u_{i-1}^{(EY)}$ (mm)**	**$\Delta_i^{(EY)}=u_i^{(EY)}-u_{i-1}^{(EY)}$ (mm)**	**$\delta_i^{(EY)}=(R/I)*\Delta_i^{(EY)}$ (mm)**	**$\lambda*\delta_{i,max}^{(EY)}/h_i\leq0{,}016$**
1st Story	3,0889	0,0000	3,08885	16,47	0,0018
2nd Story	7,2956	3,0889	4,20676	22,44	0,0025
3rd Story	11,2310	7,2956	3,93537	20,99	0,0023
4th Story	14,6523	11,2310	3,42132	18,25	0,0020
5th Story	17,4357	14,6523	2,78338	14,84	0,0016
6th Story	19,4714	17,4357	2,03571	10,86	0,0012
Roof	20,6376	19,4714	1,16623	6,22	0,0007

Table 9: Building Drift Check in X-Direction Under the $1,0G+1,0Q+1,0E_X+0,3E_Y$

EX-DIRECTION (T_2=0,78 s)					
$1,0G+1,0Q+1,0E_X+0,3E_Y$					
Floors/Displacements	$u_i^{(EX)}$	$u_{i-1}^{(EX)}$	$\Delta_i^{(EX)}=u_i^{(EX)}-u_{i-1}^{(EX)}$	$\delta_i^{(EX)}=(R/I)*\Delta_i^{(EX)}$	$\lambda*\delta_{i,max}^{(EX)}/h_i \leq 0,016$
1st Story	3,1474	0,0000	3,14743	16,79	0,0018
2nd Story	7,5631	3,1474	4,4157	23,55	0,0026
3rd Story	11,6602	7,5631	4,09706	21,85	0,0024
4th Story	15,1533	11,6602	3,49311	18,63	0,0020
5th Story	17,9249	15,1533	2,77162	14,78	0,0016
6th Story	19,8749	17,9249	1,94996	10,40	0,0011
Roof	20,9353	19,8749	1,06038	5,66	0,0006

PLAN IRREGULARITY CONDITIONS

A1-TORSIONAL IRREGULARITY

Table 10: Torsional Irregularity Check

Floors/Displacements	$(\Delta_i)_{max}$	$(\Delta_i)_{min}$	$(\Delta_i)_{ave}=1/2[(\Delta_i)_{max}+(\Delta_i)_{min}]$	$\eta_{bi}=(\Delta_i)_{max}/(\Delta_i)_{ave}$
1st Story	23,1268	20,9353	22,03105	1,05
2nd Story	21,9823	19,8749	20,9286	1,05
3rd Story	19,8455	17,9249	18,8852	1,05
4th Story	16,8025	15,1533	15,9779	1,05
5th Story	12,9629	11,6602	12,31155	1,05
6th Story	8,4505	7,5631	8,0068	1,06
Roof	3,0543	3,1474	3,10085	0,98

All η_{bi} values are smaller than 1,2 and therefore there is no torsional irregularity in building.

A2-SLAB DISCONTINUITY

The following conditions provides the safety of the slabs;

- No gaps in slab system.
- There is no sudden stiffness change in the slab.
- Stairs and elevator shafts are less than 1/3 in this building.

A3-PLAN ASPERITIES

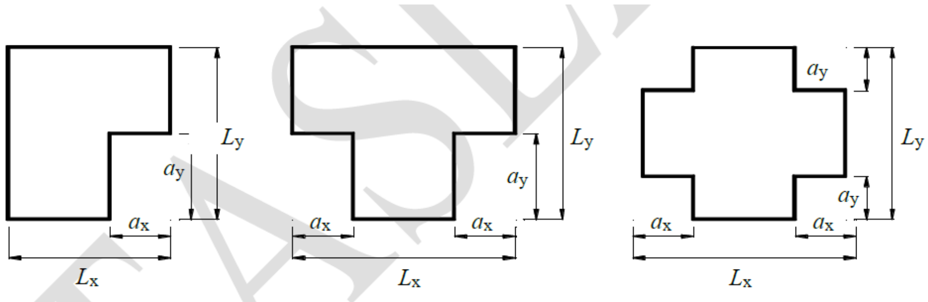


Figure 4: A3 Type Irregularity Condition

The structure designed in this report is perfectly symmetric and such irregularity does not exist in this building.

VERTICAL IRREGULARITY CONDITIONS

B1-WEAK STORY

$$\eta_{ci} = \frac{(\sum A_e)_i}{(\sum A_e)_{i+1}} \ where \sum A_e = \sum A_e + \sum A_w + 0{,}15 \sum A_k$$

This structure is perfectly symmetric with respect to the column cross sections perpendicular to earthquake direction and therefore;

$\eta_{ci} = 1 \ for \ all \ floors$. Therefore, there is no weak story in this structure.

B2-SHOFT STORY

Table 11: Soft Story Check in EY-Direction

Y-DIRECTION (T_1=0,84 s)					
1,0G+1,0Q+1,0E_Y+0,3E_X					
Floors	**u_i (EY) (mm)**	**u_{i-1} (EY) (mm)**	**Δ_i (EY)=u_i (EX)-u_{i-1} (EY) (mm)**	$\eta_{ki}=(\Delta_i/h_i)_{ave}/((\Delta_{i+1}/h_{i+1})_{ave}$	$\eta_{ki}=(\Delta_i/h_i)_{ave}/((\Delta_{i-1}/h_{i-1})_{ave}$
1st Story	3,0889	0,0000	3,08885	0,7343	-
2nd Story	7,2956	3,0889	4,20676	1,0690	1,3619
3rd Story	11,2310	7,2956	3,93537	1,1502	0,9355
4th Story	14,6523	11,2310	3,42132	1,2292	0,8694
5th Story	17,4357	14,6523	2,78338	1,3673	0,8135
6th Story	19,4714	17,4357	2,03571	1,7455	0,7314
Roof	20,6376	19,4714	1,16623	-	0,5729

Table 12:Soft Story Check in EX-Direction

EX-DIRECTION (T_2=0,78 s)					
1,0G+1,0Q+1,0E_X+0,3E_Y					
Floors	**u_i** (EX)	**u_{i-1}** (EX)	**Δ_i (EX)=u_i (EX)-u_{i-1} (EX)**	$\eta_{ki}=(\Delta_i/h_i)_{ave}/((\Delta_{i+1}/h_{i+1})_{ave}$	$\eta_{ki}=(\Delta_i/h_i)_{ave}/((\Delta_{i-1}/h_{i-1})_{ave}$
1st Story	3,1474	0,0000	3,14743	0,7128	-
2nd Story	7,5631	3,1474	4,4157	1,0778	1,4030
3rd Story	11,6602	7,5631	4,09706	1,1729	0,9278
4th Story	15,1533	11,6602	3,49311	1,2603	0,8526
5th Story	17,9249	15,1533	2,77162	1,4214	0,7935
6th Story	19,8749	17,9249	1,94996	1,8389	0,7035
Roof	20,9353	19,8749	1,06038	-	0,5438

There is no soft story mechanism in this structure as it is seen from above two table because in both tables $\boldsymbol{\eta}_{ki}$<2,0.

B3-DISCONTINUITY OF VERTICAL STRUCTURAL ELEMENTS

There is no column discontinuity in this project and the structural framing system is regular in accordance with the TBDY 2018.

FINAL DESIGN

SLAB DESIGN

Slab thickness is determined in accordance with the Section-11 in TS500 by utilizing the following equation.

$$h \geq \frac{l_{sn}}{15 + \frac{20}{m}}\left(1 - \frac{\alpha_s}{4}\right) \text{ and } h \geq 80 \text{ mm}$$

<u>15 cm slab thickness</u> is determined for this structure and detailed calculations are shown in attachments at the end of this report.

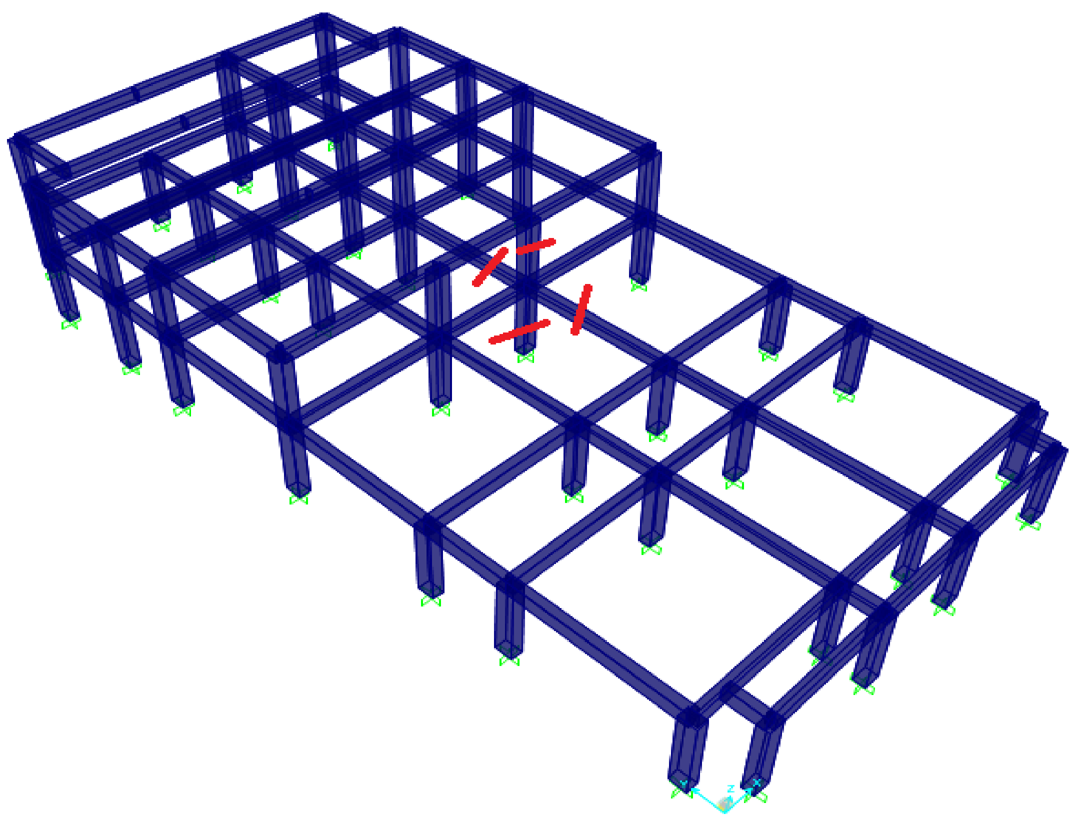


Figure 5: 3D View of Designed Beams and Columns

This figure above is to demonstrate the beams and columns which are designed for this project and they are called B01 and B02 for beams and C01 and C02 for columns.

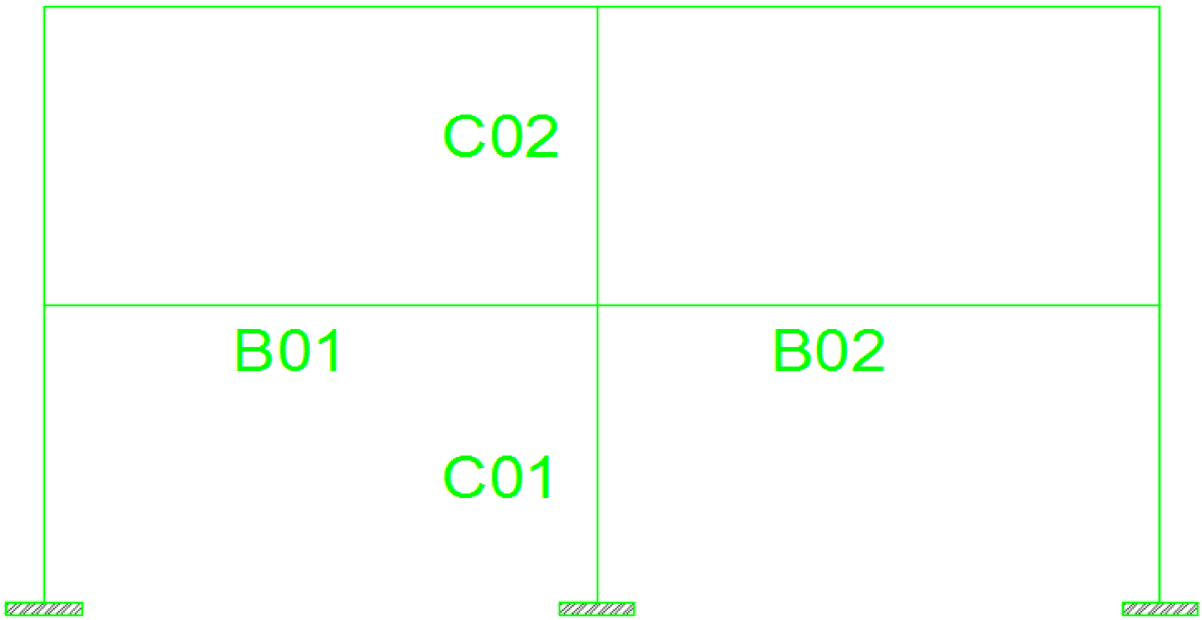


Figure 6: Section Views of the Designed Beams and Columns

It is noted that B01 and B02 are on the left and right side on Figure-5

BEAM DESIGN

Two adjacent beams are designed having a dimension of 300x600 mm. Same concrete class and reinforcing steel are utilized; therefore, minimum tension steel ratio can be calculated as the following;

$$\rho_{min} = 0,8 * \frac{f_{ctd}}{f_{yd}} = 0,8 * \frac{1,9}{1,5 * \frac{420}{1,15}} = 0,00277$$

Moreover, dimensions of the beams are also same; thus, minimum tension steel area can be calculated as the following;

$$A_{s,min} = \rho_{min} * b_w * d = 0,00277 * 300 * (600 - 25) = 477,825\ mm^2$$

According to TS500 Section 7.3,

$$A_{sl} = 0,001 * b_w * d = 0,001 * 300 * (600 - 25) = 172,5\ mm^2$$

$$\boldsymbol{Use\ 4\emptyset 10\ (314\ mm^2)\ for\ the\ mid - depth\ of\ the\ section.}$$

FLEXURAL DESIGN

$A_s = \frac{M_d}{f_{yd}*0,875*d} = \frac{84,19*10^6}{365*0,875*575} = 458,45\ mm^2, A_{s,min} > A_s\ use\ A_{s,min}\ for\ the\ span.$

$\boldsymbol{Use\ 3\emptyset 14\ (462\ mm^2)\ for\ the\ span\ of\ B01\ beam}.$

$A_s = \frac{M_d}{f_{yd}*0,875*d} = \frac{85,58*10^6}{365*0,875*575} = 460,57\ mm^2, A_{s,min} > A_s\ use\ A_{s,min}\ for\ the\ span.$

$\boldsymbol{Use\ 3\emptyset 14\ (462\ mm^2)\ for\ the\ span\ of\ B02\ beam}$

$\boldsymbol{Also\ use\ 2\emptyset 14\ (308\ mm^2)\ hanger\ bars\ in\ both\ beams}$

$A_s = \frac{M_d}{f_{yd}*0,875*d} = \frac{179,62*10^6}{365*0,875*575} = 978,11\ mm^2$

$A_s > A_{s,min}\ use\ A_s\ for\ the\ left\ support\ of\ 1st\ beam$

$\boldsymbol{2\emptyset 14\ (308\ mm^2)\ already\ exist\ and\ use\ \ 5\emptyset 14\ (770)\ mm^2\ for\ the\ left\ support\ of\ B01\ beam}$

$A_s = \frac{M_d}{f_{yd}*0,875*d} = \frac{157,46*10^6}{365*0,875*575} = 857,44\ mm^2$

$A_s > A_{s,min}\ use\ A_s\ for\ the\ right\ support\ of\ 1st\ beam$

$\boldsymbol{2\emptyset 14\ (308\ mm^2)\ already\ exist\ and\ use\ \ 4\emptyset 14\ (616)\ mm^2\ for\ the\ right\ support\ of\ B01\ beam}$

$A_s = \frac{M_d}{f_{yd}*0,875*d} = \frac{154,17*10^6}{365*0,875*575} = 859,52\ mm^2$

$A_s > A_{s,min}\ use\ A_s\ for\ the\ left\ support\ of\ 2nd\ beam$

$\boldsymbol{2\emptyset 14\ (308\ mm^2)\ already\ exist\ and\ 4\emptyset 14\ (616)\ mm^2\ comes\ from\ right\ support\ of\ 1st\ beam}$

$A_s = \frac{M_d}{f_{yd}*0,875*d} = \frac{179,1*10^6}{365*0,875*575} = 975,3\ mm^2$

$A_{s,min} > A_s\ use\ A_{s,min}\ for\ the\ right\ support\ of\ 2nd\ beam$

$\boldsymbol{2\emptyset 14\ (308\ mm^2)\ already\ exist\ and\ use\ 5\emptyset 14\ (770)\ mm^2\ for\ the\ rigth\ support\ of\ B02\ beam}$

Clear spacing between two bars should be greater than 20 mm, bar diameter and 4/3 of nominal aggregate dimension according to TS500-Section 7.3.

$$l_b = \left(0{,}12 * \frac{f_{yd}}{f_{ctd}} * \emptyset\right) \geq 20\emptyset \ accoridng\ to\ TS500\ Section\ 9.1.2.1$$

$$l_b \geq 50\emptyset \ accoridng\ to\ TSC2007\ Section\ 3.4.3.1\ (c)$$

$$l_b = \left(0{,}12 * \frac{365}{1{,}267} * 14\right) = 484\ mm \geq 20 * 14 = 280\ mm$$

$$l_b \geq 50\emptyset = 50 * 14 = 700\ mm\ and\ l_b\ should\ be\ 70\ cm$$

It is important to note that ¼ of the larger tension steel used at the supports for a beam should be continues along the beam in accordance with the TSC2007 Section 3.4.3.1 (a). In the design, 2ϕ14 are selected as hanger bars that are continues along the beam 1 and 2.

SHEAR DESIGN

According to TS500 Section 8.1;

$$s_{max} \leq \begin{Bmatrix} \frac{d}{4} = \frac{600-25}{4} = 143.75\ mm \\ 8\emptyset_l = 8 * 14 = 112\ mm \\ 150\ mm \end{Bmatrix}$$

$$where\ d\ is\ the\ effective\ depth\ and\ \emptyset_l\ is\ the\ smallest\ longitudinal\ bar.$$

Above condition should be applied to near the supports (region having the length of two times of effective depth.

$$s_{max} \leq \frac{d}{2} = \frac{600-25}{2} = 287{,}5\ mm\ should\ be\ applied\ at\ the\ span.$$

Use $\emptyset 8$ ($50\ mm^2$) for the shear reinforcement for the B01 beam

$$\frac{A_{sw}}{s} \geq 0{,}3 * \frac{f_{ctd}}{f_{ywd}} * d \rightarrow \frac{50*2}{s} \geq 0{,}3 * \frac{\frac{1{,}9}{1{,}5}}{365} * (250) \leftrightarrow \boldsymbol{s \leq 384.21\ mm}$$

According to TSC2007, plastic moment of the column is the 1.4 times of its moment capacity.

$$M_{pi} \cong 1{,}4 * M_{ri} = 1{,}4 * 179{,}62 = 251{,}47\ kN - m$$

$$M_{pj} \cong 1{,}4 * M_{rj} = 1{,}4 * 157{,}46 = 220{,}44\ kN - m$$

$$V_{dy}\ is\ the\ shear\ force\ under\ 1.0G + 1.0Q\ load\ combination$$

$$V_{dy} = 79\ kN\ taken\ from\ SAP2000$$

$$V_e = V_{dy} \pm \frac{(M_{pi} + M_{pj})}{l_n} = 79 \pm \frac{251{,}47 + 220{,}44}{3.2 - 0.6} = 260{,}5\ kN$$

$$V_e \geq V_d\ and\ V_e = V_c + V_w$$

$$V_{cr} = 0{,}65 * f_{ctd} * b_w * d * \left(1 + \gamma \frac{N_d}{A_c}\right), \gamma = 0\ (TS500\ Section\ 8.1.3)$$

$$V_{cr} = 0{,}65 * f_{ctd} * b_w * d\ and\ V_c = 0{,}8 * V_{cr}$$

$$V_{cr} = \frac{0{,}65 * \frac{1{,}9}{1{,}5} * 250 * (600 - 25)}{1000} = 118{,}35\ kN\ and\ V_c = 0{,}8 * 118{,}35 = 94{,}68\ kN$$

$$V_{max} = 0{,}22 * f_{cd} * b_w * d = \frac{0{,}22 * \frac{30}{1{,}5} * 250 * (600 - 25)}{1000} = 632{,}5\ kN \geq\ V_e = 260{,}5\ kN$$

$$V_w = V_e - V_c = 260{,}5 - 94{,}68 = 165{,}82\ kN$$

$$V_w = \frac{A_{sw}}{s} f_{ywd} d \rightarrow s = \frac{A_{sw} * f_{ywd} * d}{V_w} = \frac{2 * 50 * 365 * (600 - 25)}{165{,}82 * 1000} = 126{,}6\ mm$$

Use ∅8/10 cm close to the supports and use ∅8/20 cm at span.

Use ∅8 ($50\ mm^2$) for the shear reinforcement for the B02 beam

Drawings provided at the end.

COLUMN DESIGN

Column design is done by using interaction diagrams. To do that, minimum reinforcement is put in the column section by using SAP2000, and interaction diagrams of the columns are obtained from SAP2000. Diagrams are plotted in MS Excel to provide better understanding. Furthermore, loads coming for different load combinations are also exported from SAP2000 and axial loads (P) and moments (M2 and M3) are also plotted on the corresponding interaction diagrams.

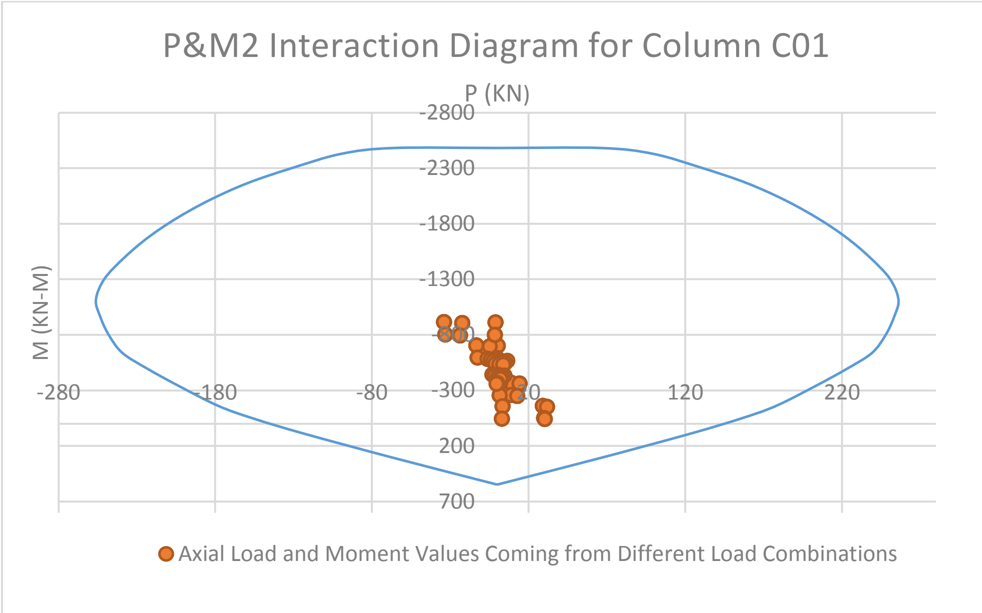
According to TS 500 Section 7.4;

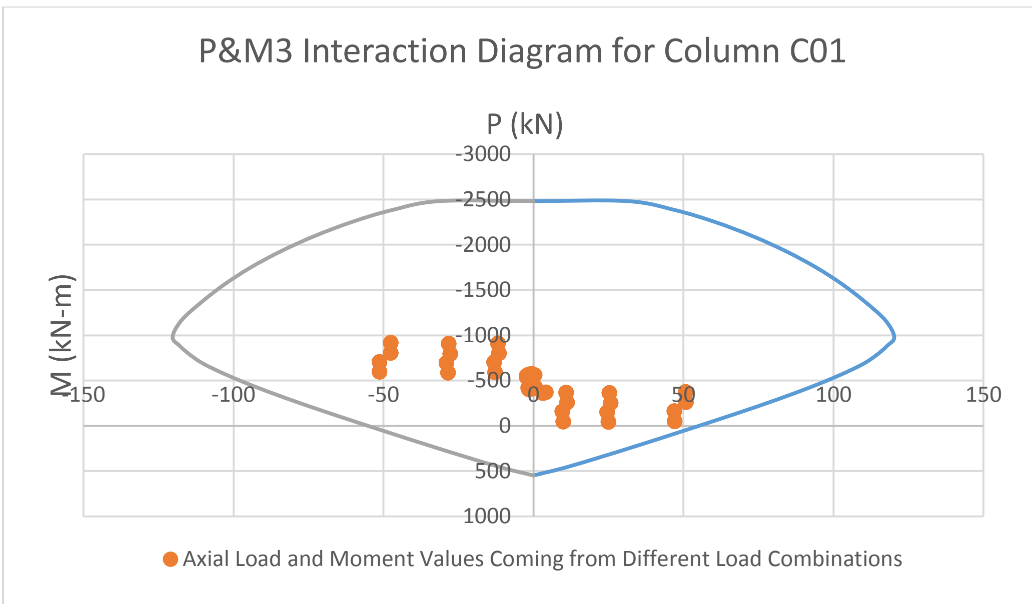
$$\rho_t = \frac{A_{st}}{A_c} \geq 0{,}01\ and\ thus A_{st} = 250 * 600 * 0{,}01 = 1500\ mm^2$$

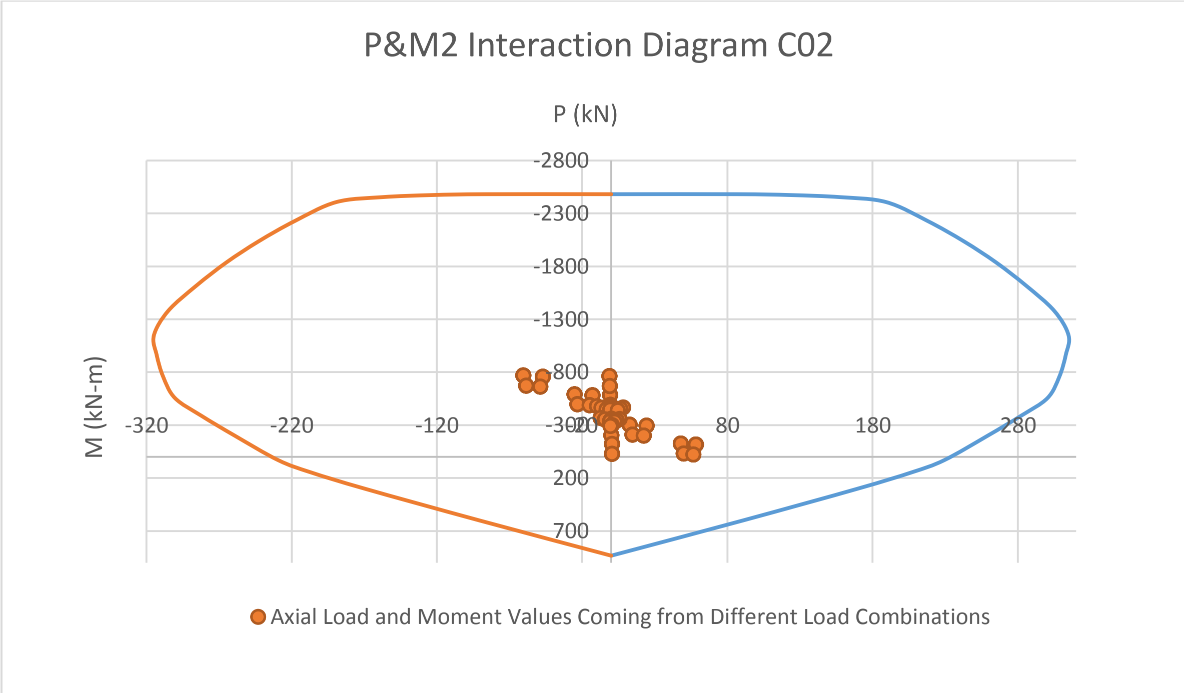
$$N_d \leq 0{,}9 * f_{cd} * A_c = 0{,}9 * \frac{30}{1{,}5} * 250 * \frac{600}{1000} = 2700\ kN\ since\ N_d = 1000\ kN$$

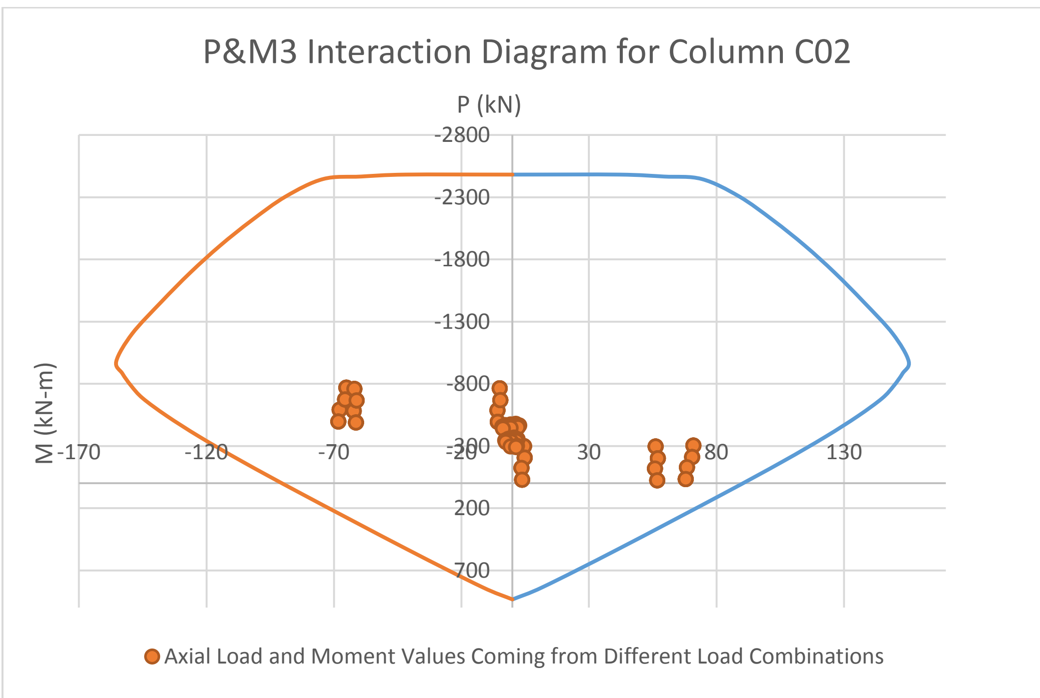
$8\emptyset 16\ (1608\ mm^2)\ is\ put\ in\ columns\ to\ start\ with\ the\ iteration.$

FLEXURAL DESIGN









In accordance with the interaction diagrams shown above, minimum reinforcement (8Ø16) is sufficient for columns C01 and C02

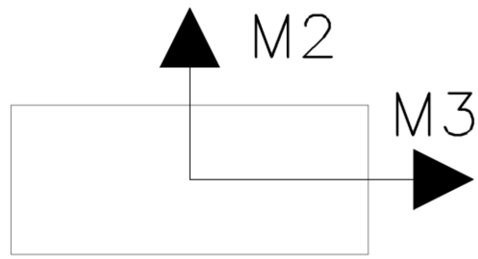


Figure 7: Column Orientation and Moment M2 and M3 Directions

CHECK FOR STRONG COLUMN-WEAK BEAM CONDITION

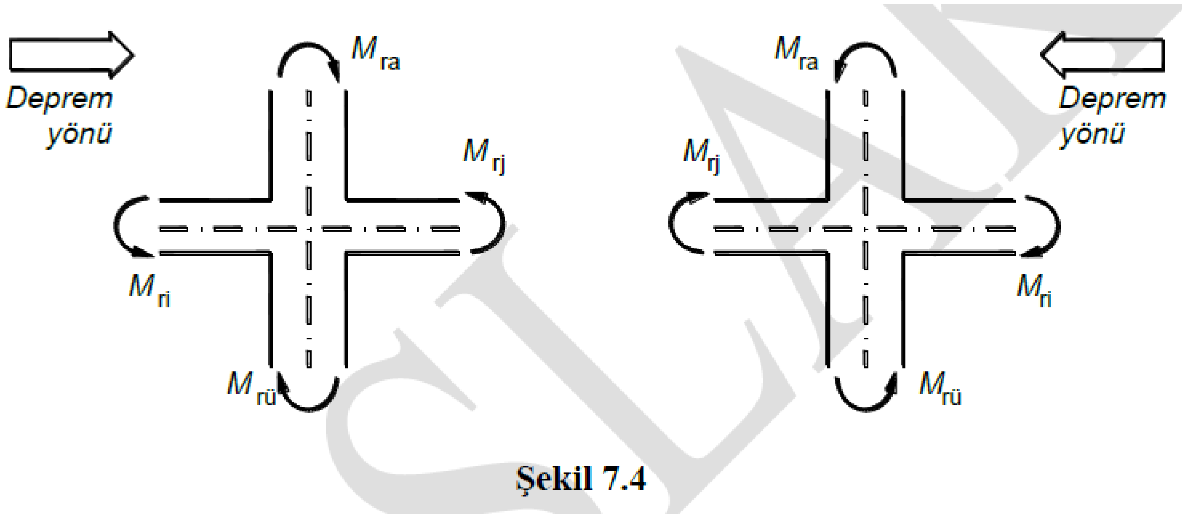


Figure 8: Moment Directions for Strong Column Weak Beam

Joint A

$$(M_{ra} + M_{rü}) \geq 1{,}2 * (M_{ri} + M_{rj})$$

$$M_{ra} = 340{,}7\ kN - m\ for\ this\ section\ according\ to\ N_d = 2735{,}72\ kN\ for\ Column\ C01$$

$$M_{rü} = 310{,}6\ kN - m\ for\ this\ section\ according\ to\ N_d = 3228{,}79\ kN\ for\ Column\ C02$$

$$M_{ri} = (3 * 113 + 2 * 154) * \frac{420}{1{,}15} * 0{,}875 * (600 - 25) = 118{,}8\ kN - m$$

$$M_{rj} = \left(\frac{0{,}85 * 103{,}75 * 300 * 0{,}85 * 20}{1000}\right) * \left(300 - \frac{0{,}85 * 103{,}75}{2}\right) + \frac{154 * 365 * 3}{1000} * 0275 + \frac{154 * 365 * 7}{1000} * 275 = 269{,}67\ kN - m$$

$$340{,}7 + 310{,}6 = 651{,}3 \geq 647{,}2 = 1{,}2 * 269{,}67\ \ TSC2007\ Section\ 3.3.5.1\ is\ OK$$

SHEAR DESIGN

At joint A;

$$V_e = \frac{(M_a + M_ü)}{l_n}$$

$$M_ü + M_a = \sum M_p = M_{pi} + M_{pj} = 1{,}4 * M_{ri} + 1{,}4 * M_{rj} = 1{,}4 * 319{,}8 * 2 = 639{,}6\ kN - m$$

According to TBDY 2018, plastic moment of the column is the 1.4 times of its moment capacity.

$$V_e = \frac{(639{,}6)}{3{,}5 - 0{,}6} = 220{,}6\ kN$$

$$N_d = 3228{,}8\ kN \geq 0{,}05 * A_c * f_{ck} = 0{,}05 * 250 * 600 * \frac{30}{1000} = 225\ kN, so\ V_c \neq 0$$

$$V_{cr} = 0{,}65 * f_{ctd} * b_w * d * \left(1 + \gamma \frac{N_d}{A_c}\right)$$

$$\gamma = 0{,}07\ owing\ to\ axial\ compression\ (TS500\ Section\ 8.1.3)$$

$$V_c = 0{,}8 * V_{cr} = 0{,}8 * 0{,}65 * \frac{1{,}9}{1{,}5} * 250 * (600 - 25) * \left(1 + 0{,}07 * \frac{3228{,}8}{250*600}\right) = 94{,}83\ kN$$

$$V_w = V_e - V_c = 220{,}6 - 94{,}83 = 125{,}77\ kN$$

$$\frac{A_{sw}}{s} = \frac{V_w}{f_{ywd}*d} = \frac{63{,}68}{\frac{420}{1{,}15}*(600-25)} = 0{,}6$$

Use Φ12 with 2 legs;

$$s = \frac{2*79}{0{,}6} = 263{,}3\ mm$$

$$N_d = 3228{,}8\ kN > 0{,}20 * A_c * f_{ck} = 0{,}2 * 250 * 600 * \frac{25}{1000} = 750\ kN$$

Since $N_d \geq 0{,}20 * A_c * f_{ck}$;

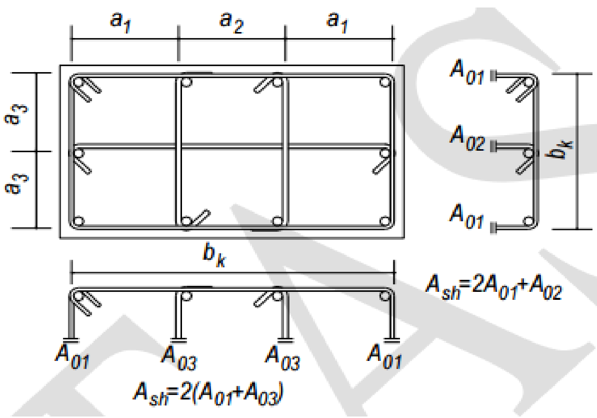


Figure 9: Transverse Reinforcement Area According to TDBY 2018

For 60 cm direction;

$$A_{sh} \geq 0{,}30 * s * b_k * \left[\left(\frac{A_c}{A_{ck}}\right) - 1\right] * \left(\frac{f_{ck}}{f_{ywk}}\right) \; and \; A_{sh} \geq 0{,}075 * s * b_k * \left(\frac{f_{ck}}{f_{ywk}}\right)$$

$$A_{sh} \geq 0{,}3 * 100 * 200 * \left[\left(\frac{250*600}{200*500}\right) - 1\right] * \left(\frac{30}{420}\right) = 214{,}3 \; mm^2$$

$$A_{sh} \geq 0{,}075 * 100 * 200 * \left(\frac{30}{420}\right) = 107{,}14 \; mm^2$$

$A_{sh} = 2 * \left(\pi * \frac{12^2}{4}\right) = 226{,}2 \; mm^2 \; mm^2$ **(2 legs Φ12 is ok)**

For 40 cm direction;

$$A_{sh} \geq 0{,}30 * s * b_k * \left[\left(\frac{A_c}{A_{ck}}\right) - 1\right] * \left(\frac{f_{ck}}{f_{ywk}}\right) \; and \; A_{sh} \geq 0{,}075 * s * b_k * \left(\frac{f_{ck}}{f_{ywk}}\right)$$

$$A_{sh} \geq 0{,}3 * 100 * 500 * \left[\left(\frac{400*600}{200*500}\right) - 1\right] * \left(\frac{30}{420}\right) = 535{,}7 \; mm^2$$

$$A_{sh} \geq 0{,}075 * 100 * 500 * \left(\frac{30}{420}\right) = 267{,}85 \; mm^2$$

$A_{sh} = 2 * \left(\pi * \frac{12^2}{4}\right) = 226{,}2 \; mm^2 \; mm^2$ **(2 legs Φ12 is not ok, use 2Φ14 crossties)**

$$\left\{\begin{array}{c} s_0 \leq 200 \; mm \\ s_0 \leq \frac{b_{min}}{2} = \frac{250}{2} = 125 \; mm \end{array}\right\}$$

$$Thus \; select \; s_0 = 100 \; mm \; at \; the \; middle \; zone \; of \; columns$$

use Φ12/10 cm at the column confinement region and Φ12/10 cm at the column central zone.

BEAM TO COLUMN CONNECTION

Joint A is designed as beam to column connection and it is not a surrounded connection because there are 4 beams connected to this column.

$$V_e = 1{,}25 * f_{yk} * (A_{s1} + A_{s2}) - V_{col}$$

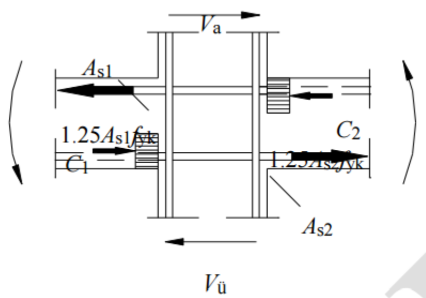


Figure 10: Beam Column Design Principle taken from TDBY 2018

In this design, A_{s1} and A_{s2} are the tension steels provided for the 1st and 2nd beams, respectively.

A_{s1}= 3Φ14+4Φ14=1078 mm^2

A_{s2}= 3Φ14=462 mm^2

V_{col}=106,5 KN is obtained from analysis

$$V_e = 1{,}25 * 420 * (1078 + 462) - 106{,}5 = 702\ kN$$

$$V_e \leq 0{,}6 * b_j * h * f_{cd}\ where\ b_j = 250\ mm(not\ surrounded\ by\ 4\ sides)\ and\ h = 600\ mm$$

$$V_e = 702 \leq 0{,}6 * 250 * 600 * \frac{30}{1{,}5} = 1800\ kN \rightarrow OK$$

REINFORCEMENT FOR THE CONNECTION

h=600 mm

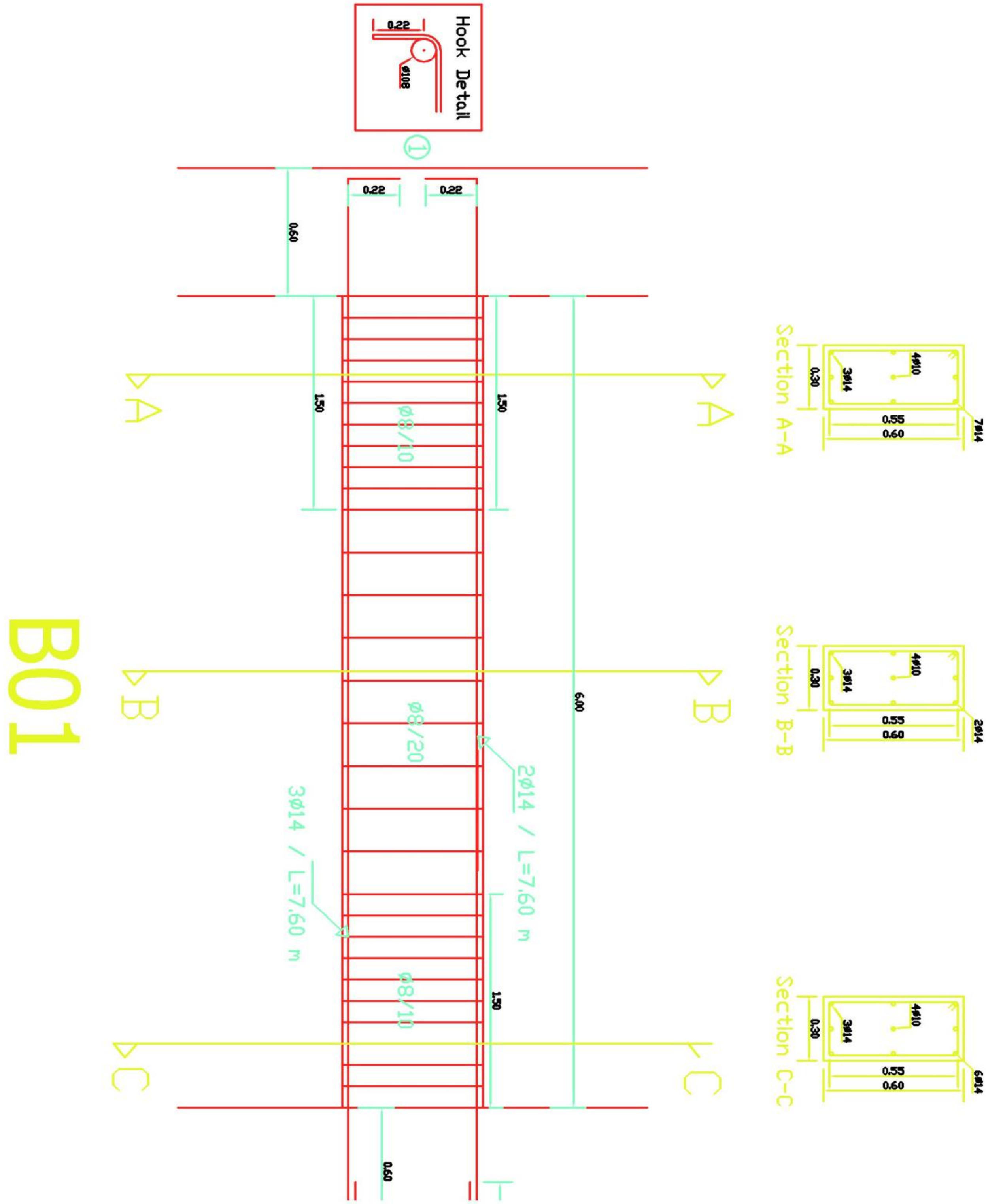
0.6 *6=3,6 stirrups minimum

Max spacing, 10 cm and Beam depth, 60 cm

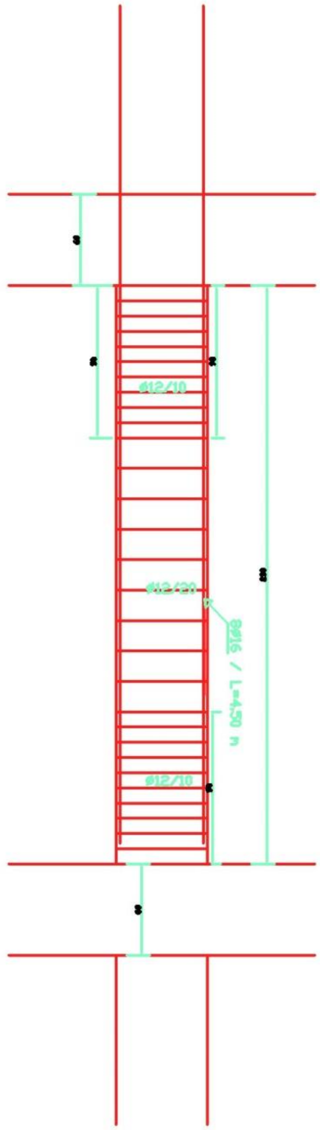
Use Φ10/10 stirrups at connections

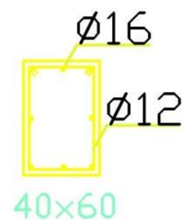
Drawings for the columns provided at the end.

ATTACHMENTS



C01





C01