ISTANBUL TECHNICAL UNIVERSITY

FACULTY OF CIVIL ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

GRADUATION DESIGN PROJECT

TOPIC:	**DESIGN STORAGE BUILDING WHICH IS MADE OF REINFORCED CONCRETE AND STEEL ELEMENTS**
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Director:	**Assist. Prof. Dr. Barış ERKUŞ**
Delivery Date:	**14.06.2017**

JUNE 2017

FOREWORD

2016-2017 Spring Semester in Structural Analysis Graduation Project, the concrete-steel storage structure was design by static, reinforced concrete and steel calculations. We would like to thank Barış Erkuş, Murat Yılmaz, Barlas Çağlayan, the Structural Analysis Group and faculty members of the Faculty of Civil Engineeering who are patinent during our studies and who do not spare us from knowledge, experience and broad engineering perspective in the realization project.

June, 2017

010120205 Ahmet Gündüz

010120231 Bora Saman

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SUMMARY

With in scope of 2016-2017 Spring Semester Engineering Design Project, the concrete-steel storage structure which is large span is dimensioning by static, reinforced concrete and steel calculations.

The designed storage structure consists of 5 openings of 6 meters and 3 openings of 6 meters. Two stories are consist of frame planes. In the third story direction parallel to the frame plane, steel roof which is 18x18 meters are desinged and reinforced concrete side which is 18x12 meters. Design steel roof is carry by cantilever columns which are height of 6 meters. The structure is located in the first degree earthquake zone as it will be built in Tuzla. The local floor class ZB and the floor safety tension is 180 kN / m^2. Concrete grade is C30, reinforcement is S420 and structural steel is ST355. By using these data; structural loads, earthqquake loads and vertical-horizontal loads are calculated.

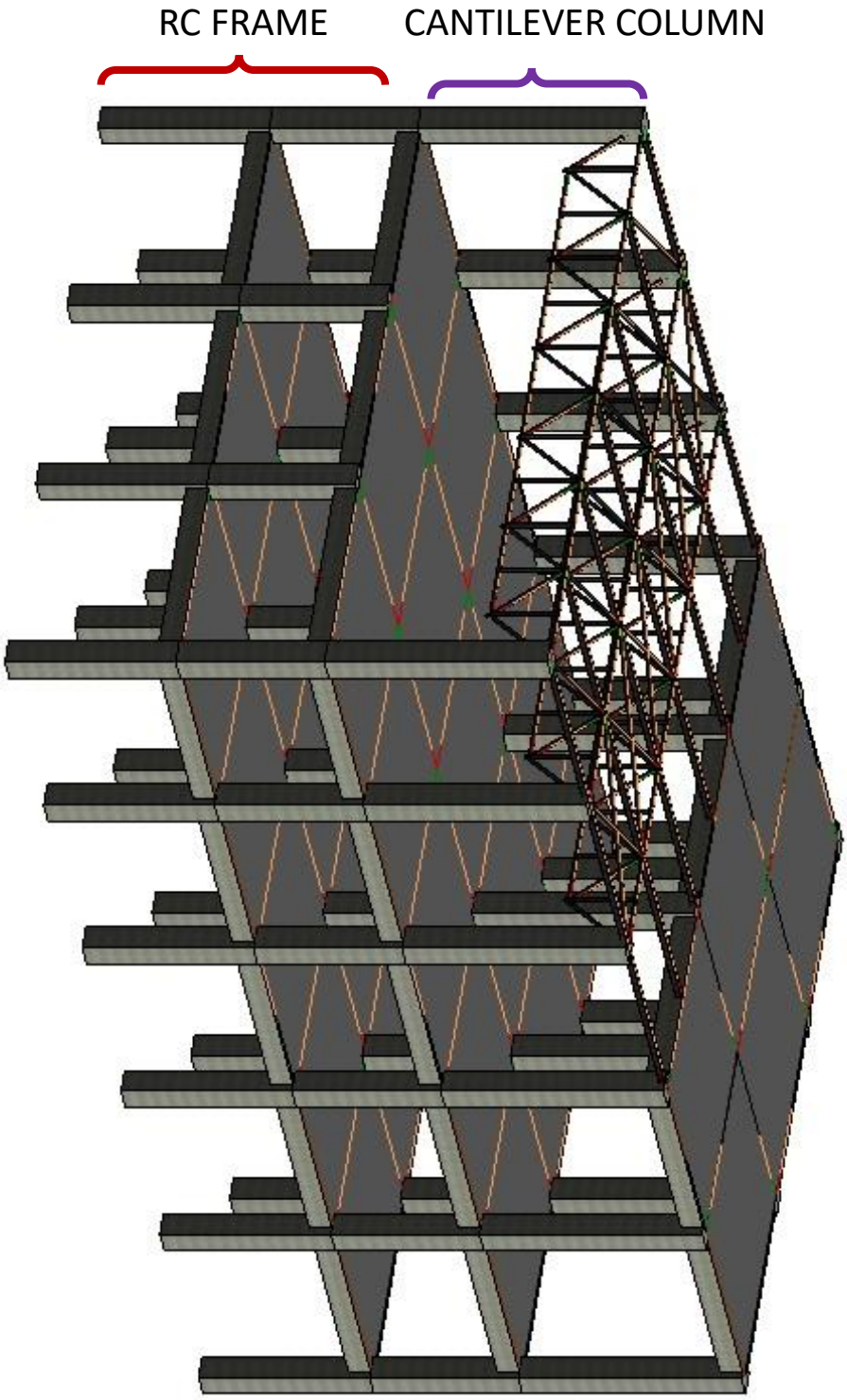
Firstly, the reinforced concrete slab system is dimensioning. In the next step, the loads affecting the frame system are calculated approximately and the column, beam and foundation are pre-dimensioned. Using equivalent seismic load method and SAP2000 program, static analysis was performed for possible combinations of load on the frame system. Based on the obtained unfavorable forces, reinforced concrete calculations of the system elements, cross-section control of steel structural elements and joint designs were made. Finally, foundation is dimensioning and are calculated reinforcement as spread footing.

CONCENTS TABLE

2. General Information

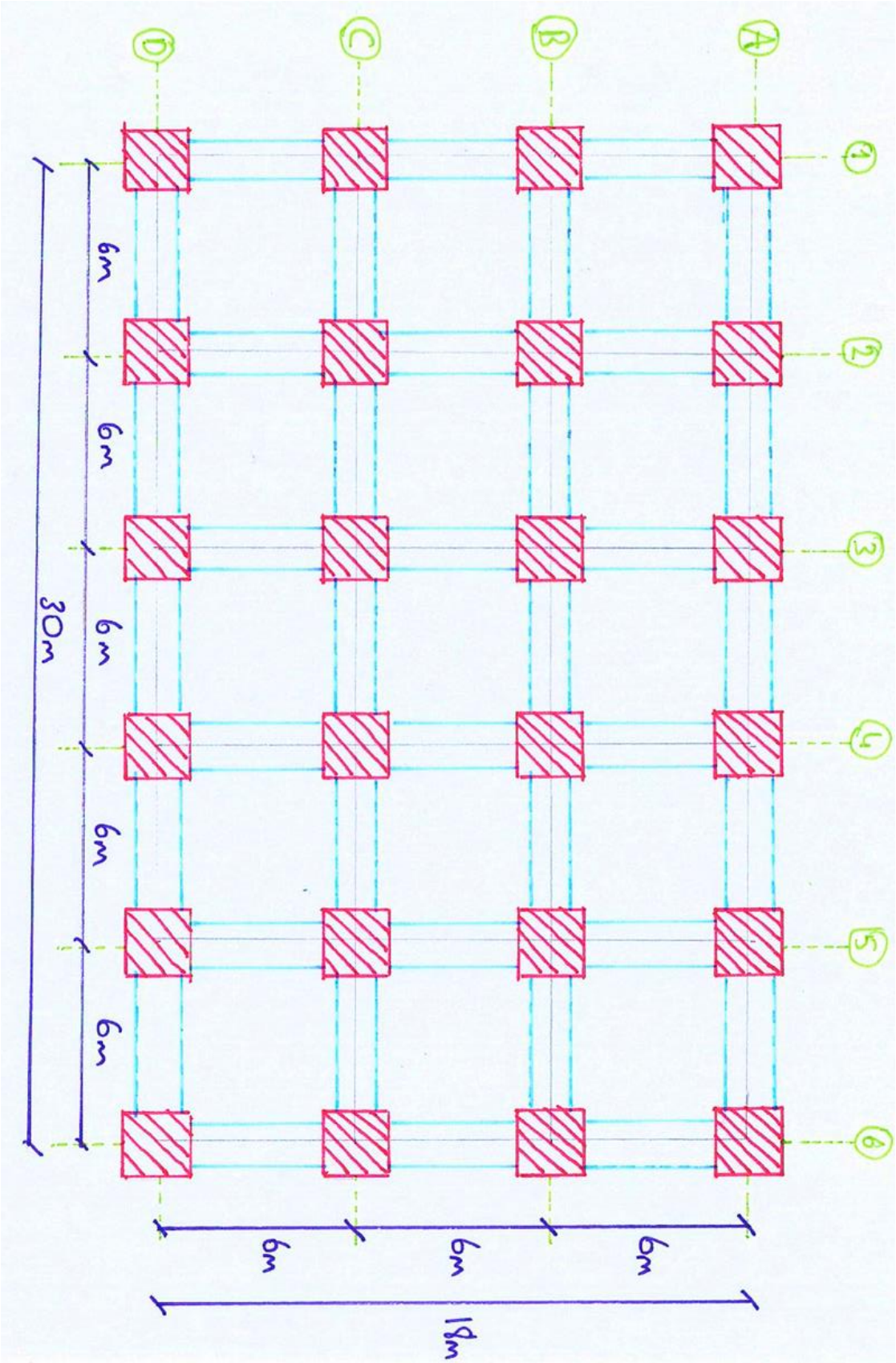
PROJECT DATA	
Building Type	Store Building
Number of Base on X Direction :	5
Number of Base on Y Direction :	3
Length	30x18m
Number of Story	3
Building Height	14.6m
Local Soil Class :	Z2
Location :	Gebze/İstanbul(40.825°,29.345°)
Allowale Soil Stress :	180 kN/m
Materials :	C30/S420 and St37

3. 3D View

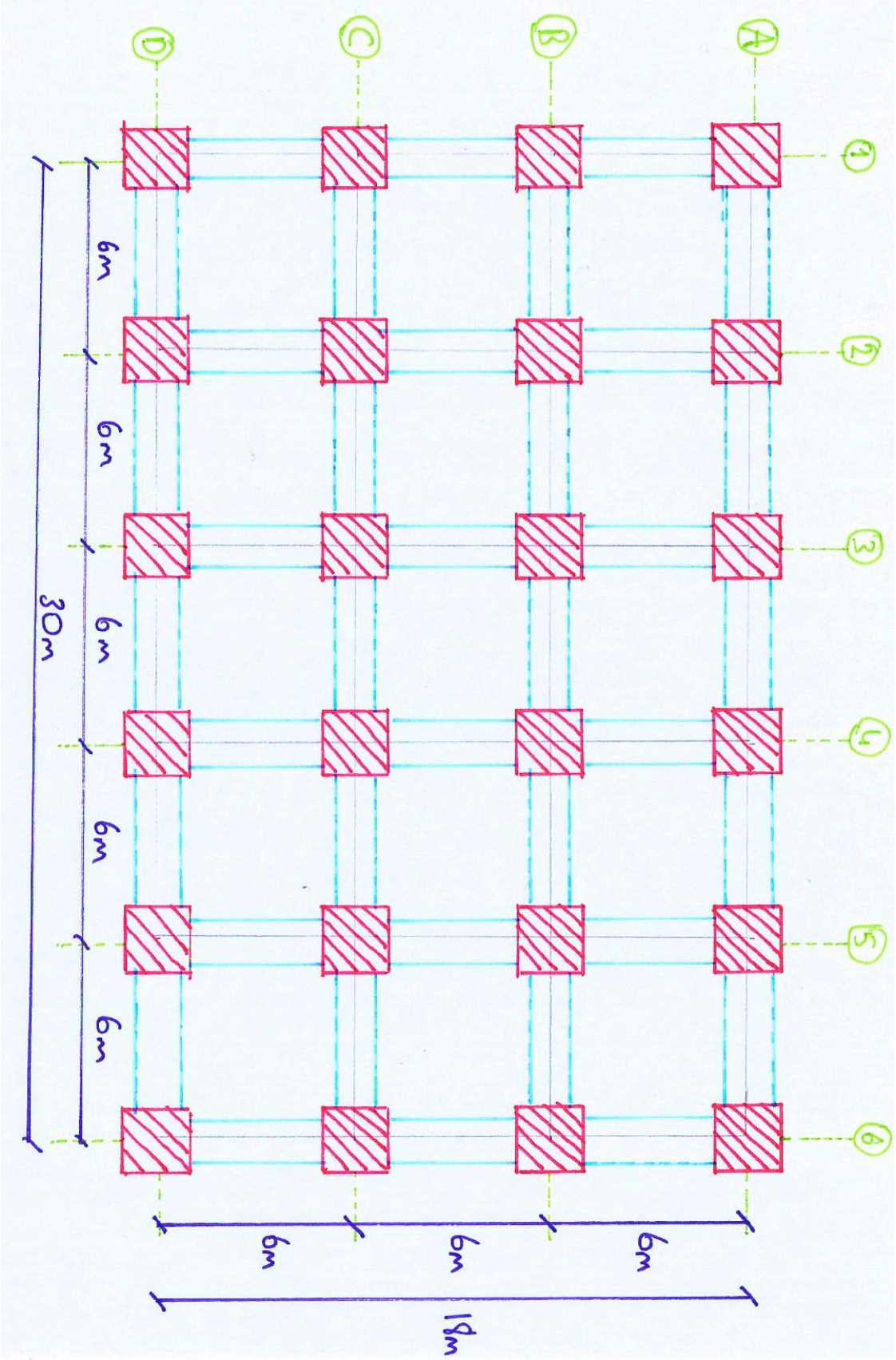


4. System Plans and Sections

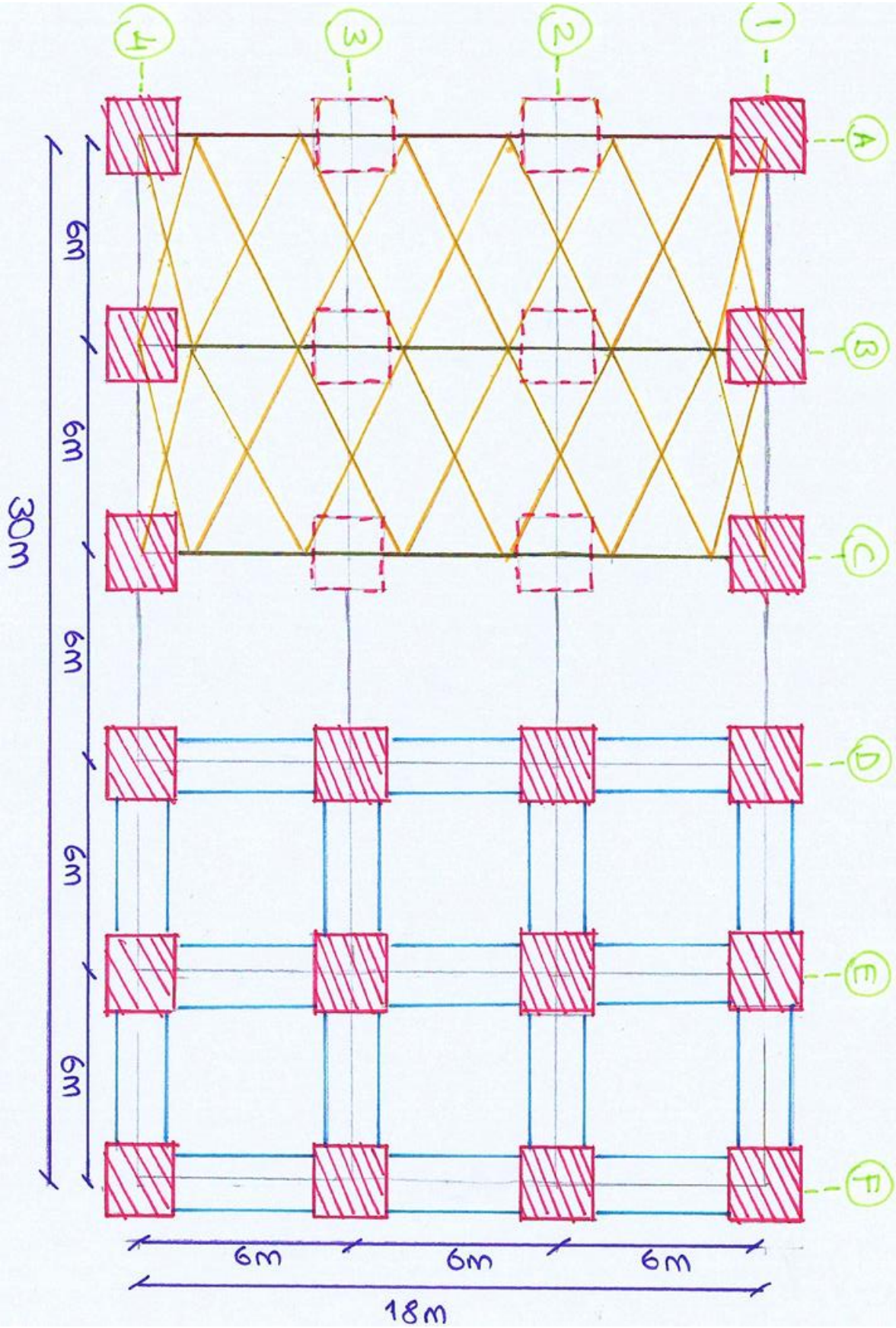
4.1. First Story Plan View



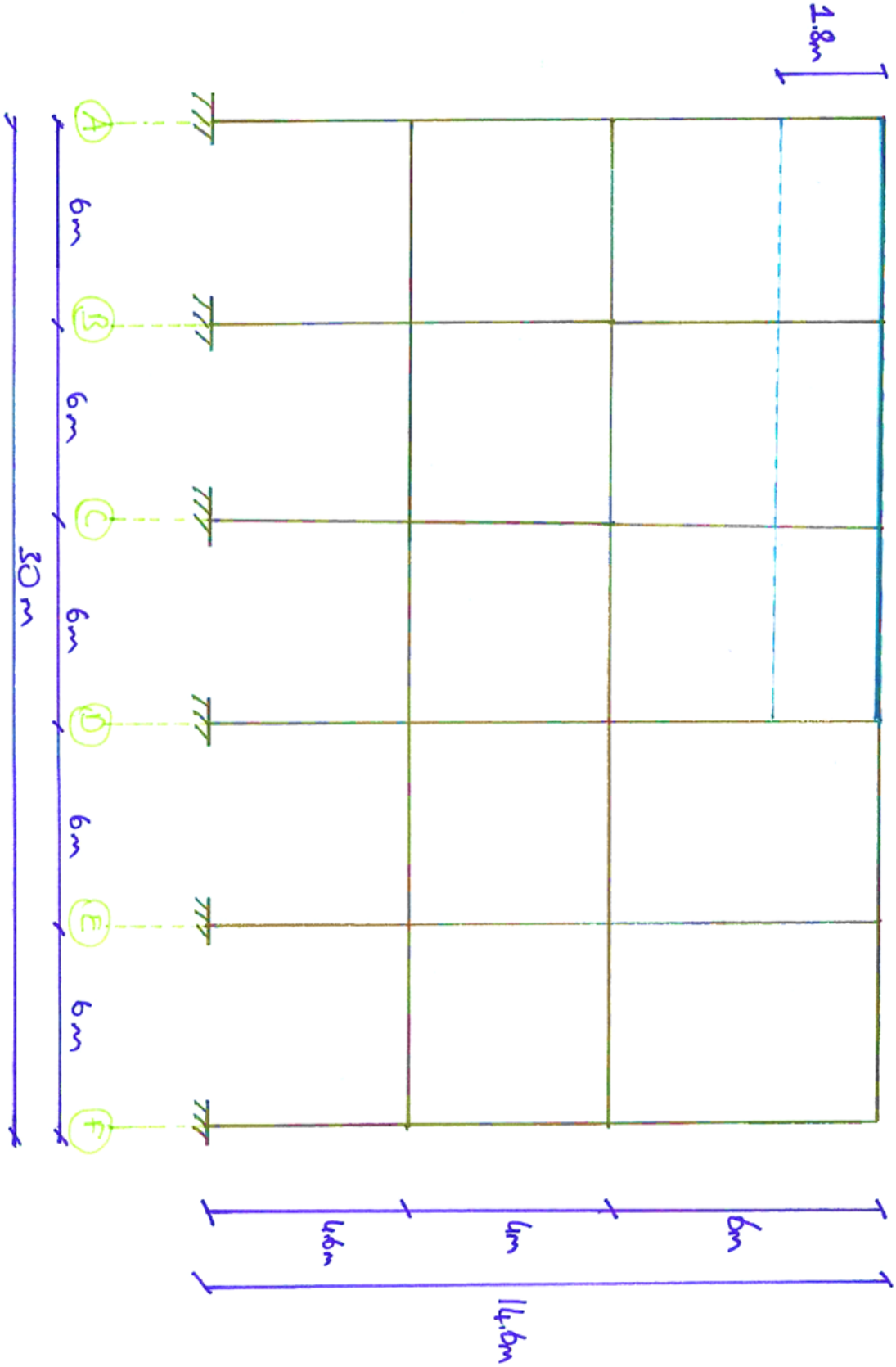
4.2. Second Story Plan View



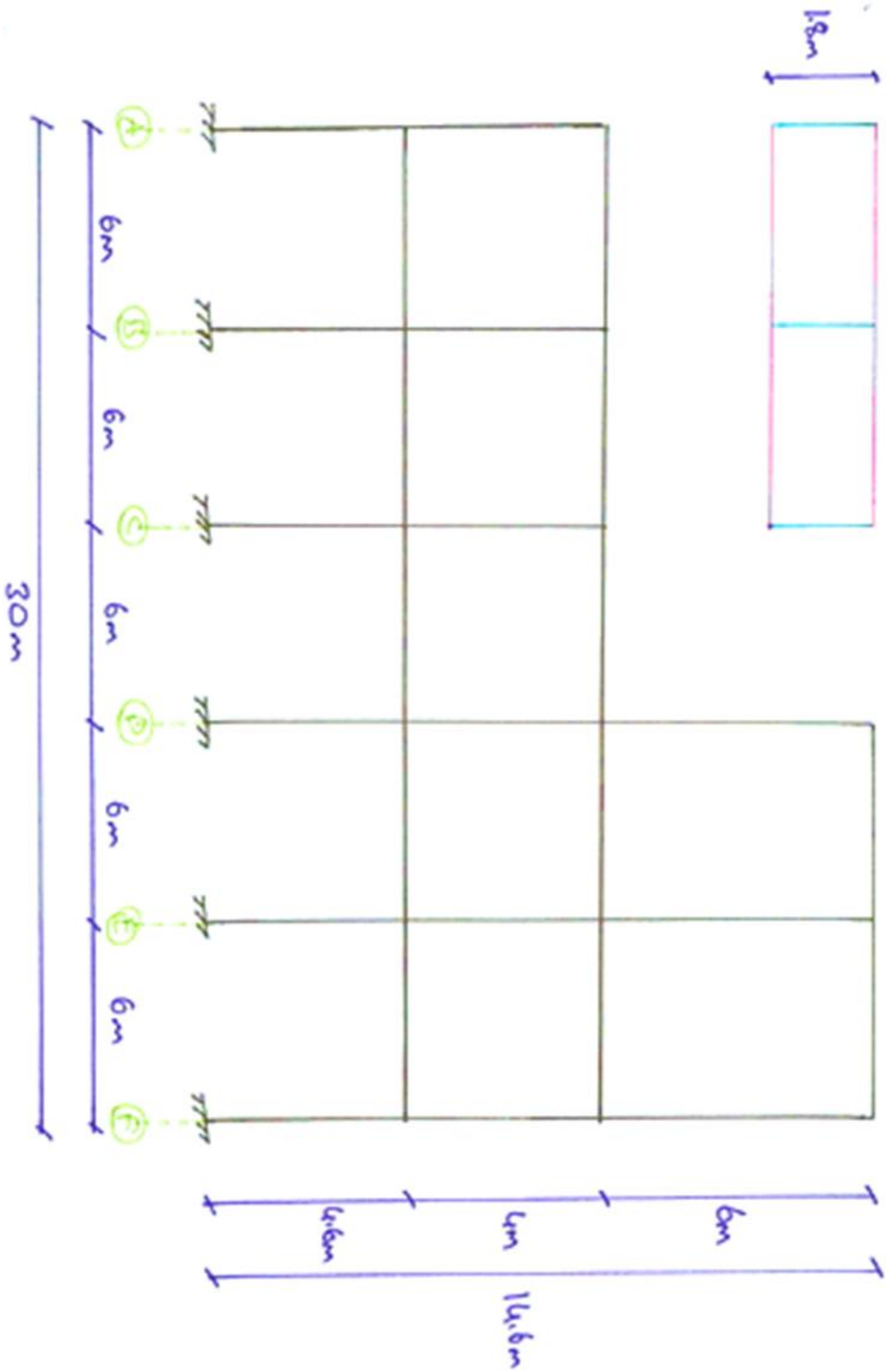
4.3. Third Story Plan View



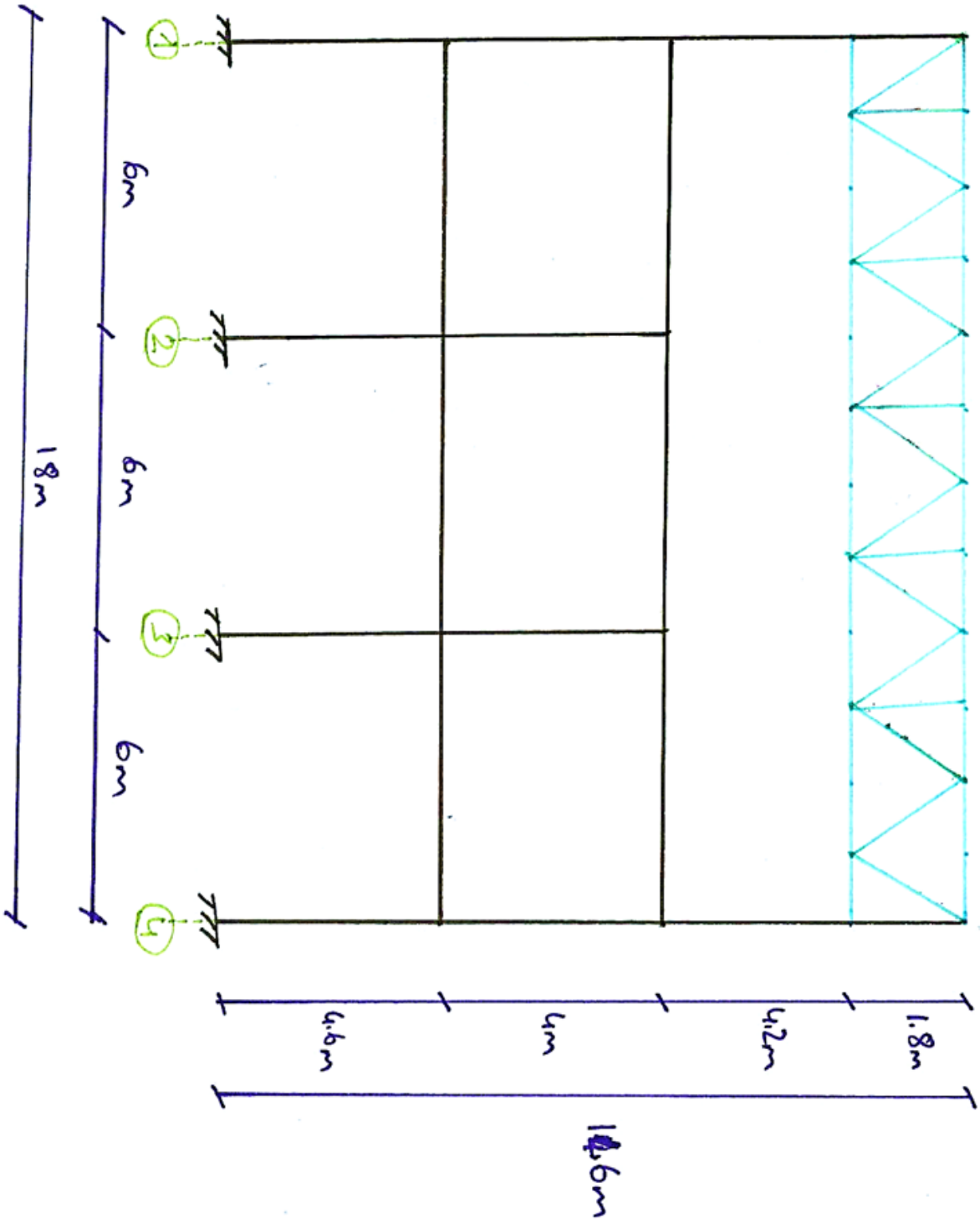
4.4. 1-1 and 4-4 Axis



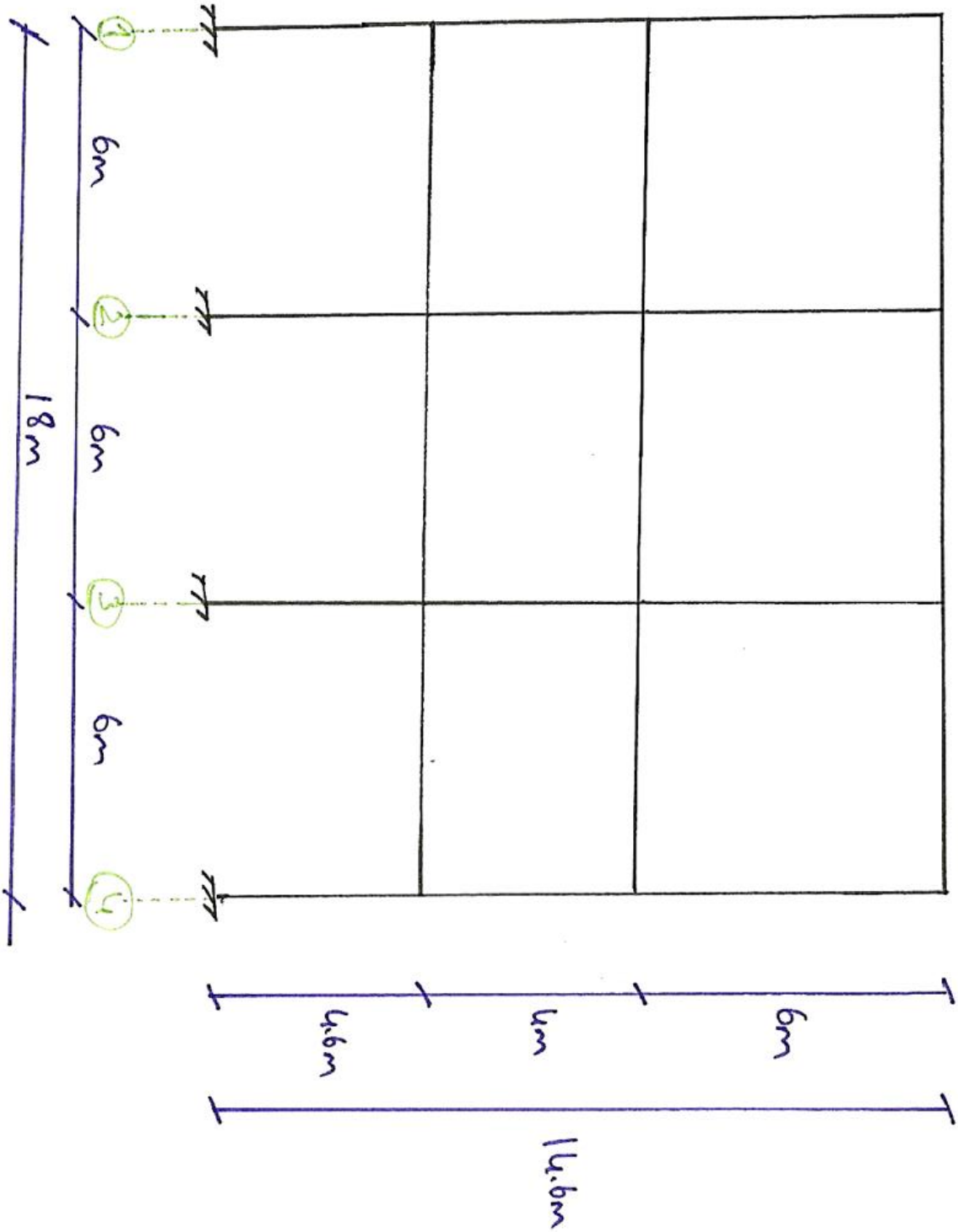
4.5. 2-2 and 3-3 Axis



4.6. A-A,B-B and C-C Axis



4.7. D-D, E-E and F-F Axis



5. Why This Structure?

- ✓ Moment Frame Section
- ✓ Without Shear Wall
- ✓ Cantilever Column
- ✓ Programming
- ✓ Combination Steel and RC

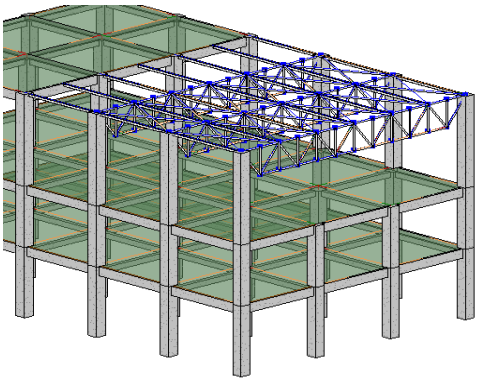
Out of Studies

- ✓ Vertical Seismic Effect
- ✓ Super Optimization
- ✓ Temperature Effect
- ✓ Long Term Delection Effect
- ✓ Soil Support Interaction (Elastic Support)

6. DESIGN BASIS

- **Frame System**

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

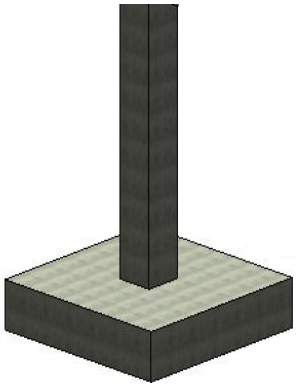


SMRF

ASCE 7-10	
R	8
C_d	5,5
Ω_d	3

- **Cantilever Column**

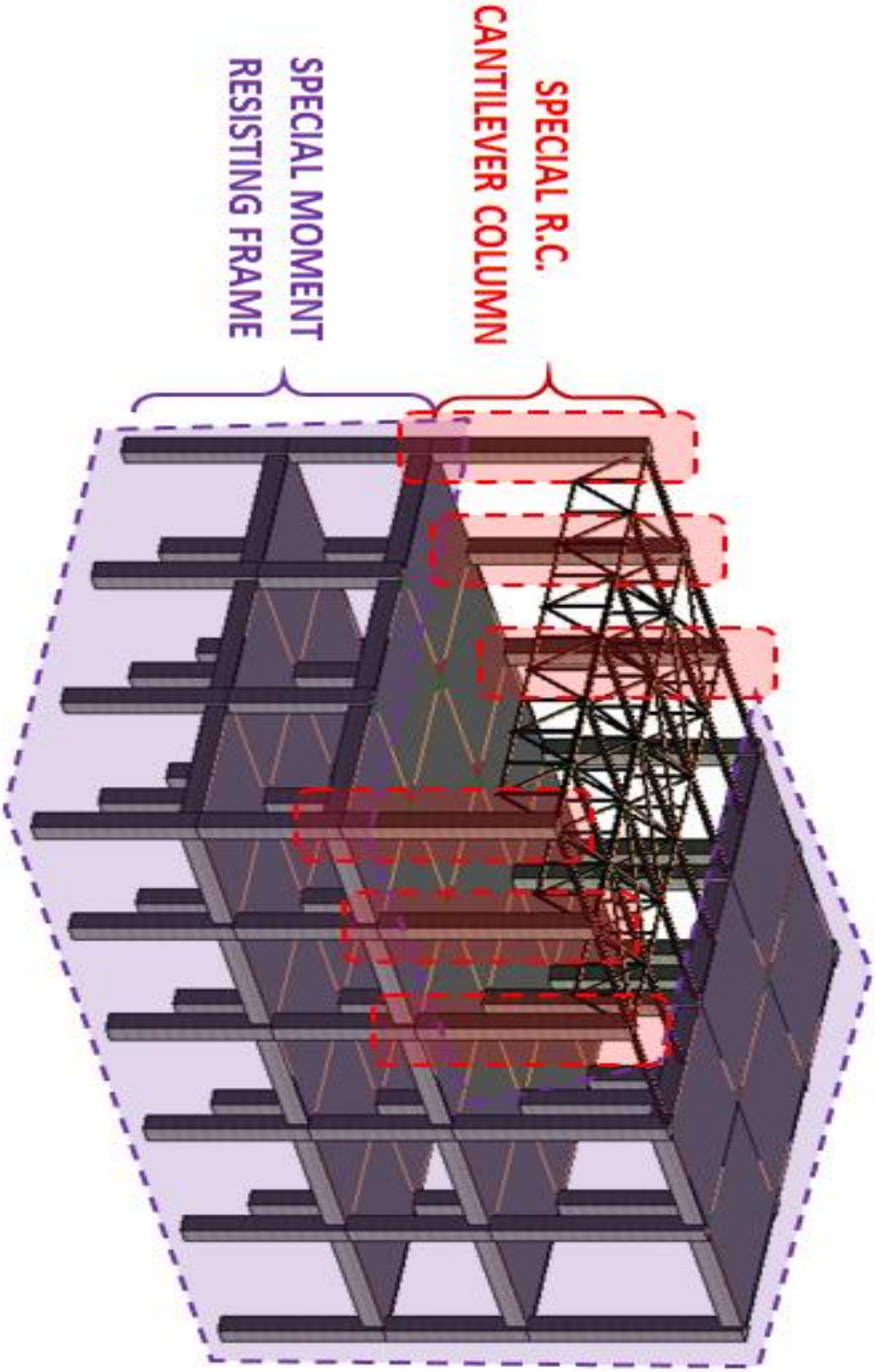
Bina Taşıyıcı Sistemi	Taşıyıcı Sistem Davranış Katsayısı *R*	Dayanım Fazlalığı Katsayısı *D*	İzin Verilen Bina Yükseklik Sınıfları BYS
A16. Deprem etkilerinin tamamının çatı düzeyindeki bağlantıları mafsallı olan ve yüksekliği 12 m'yi geçmeyen *süneklik düzeyi yüksek* betonarme kolonlar tarafından karşılandığı tek katlı binalar	3	2	–



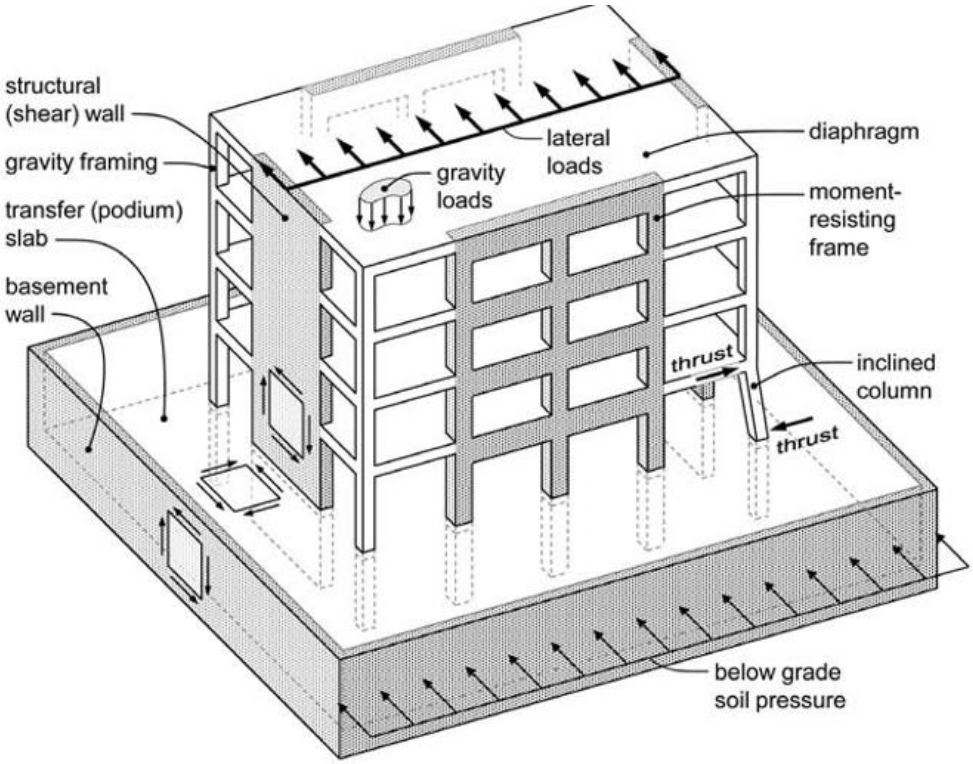
S. R. C. CANTILEVER

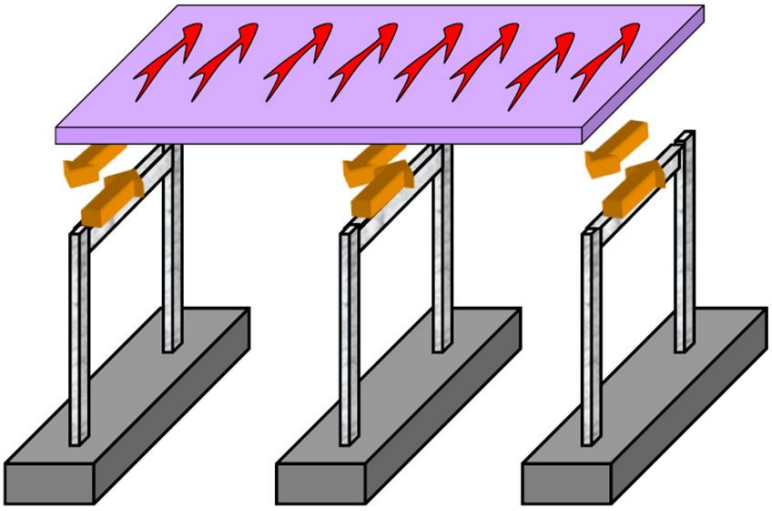
ASCE 7-10	
R	2,5
C_d	2,5
Ω_d	1,25

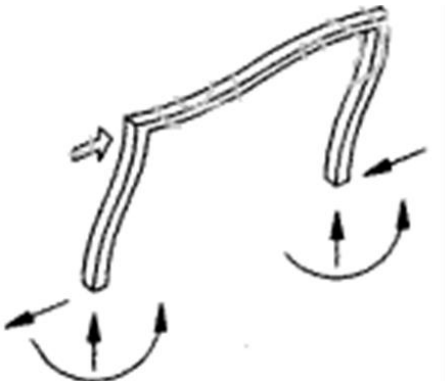
LATERAL SYSTEM



LOAD PATH

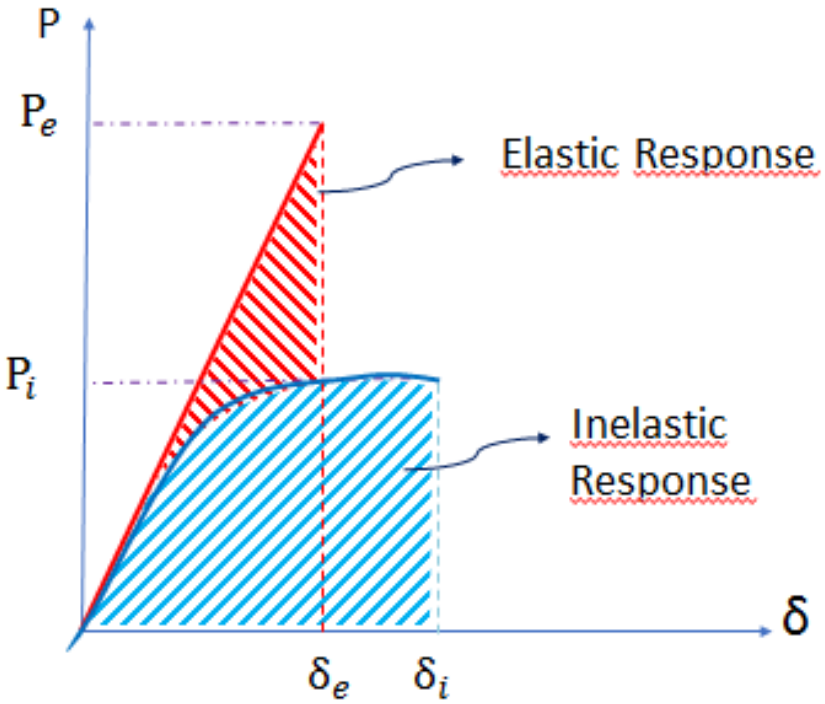

structural
(shear) wall
gravity framing
transfer (podium)
slab
basement
wall
lateral
loads
gravity
loads
diaphragm
moment-
resisting
frame
thrust
inclined
column
thrust
below grade
soil pressure



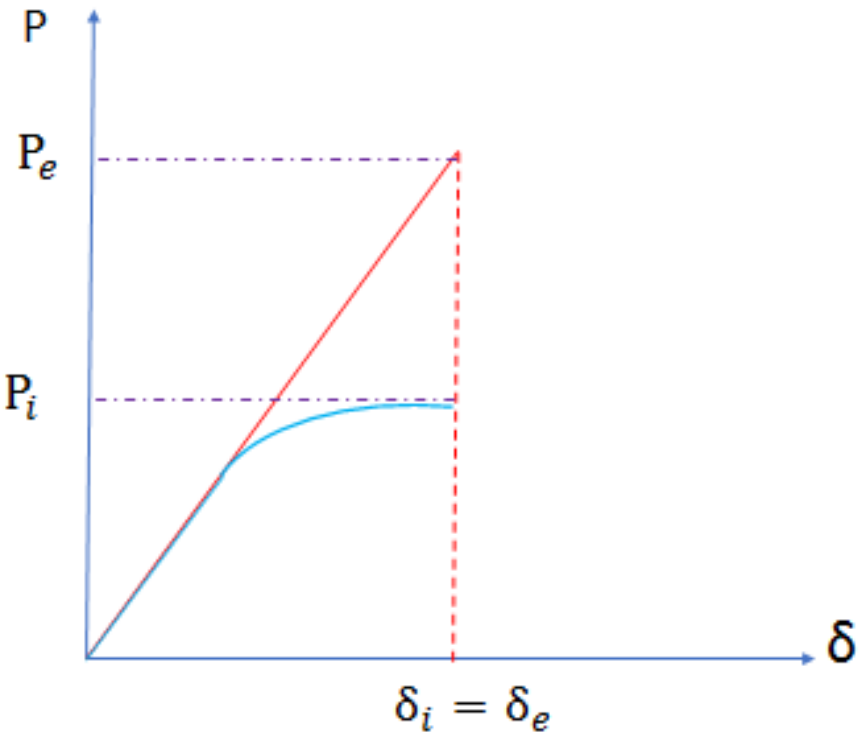


NONLINEAR BEHAVIOUR OF STRUCTURES

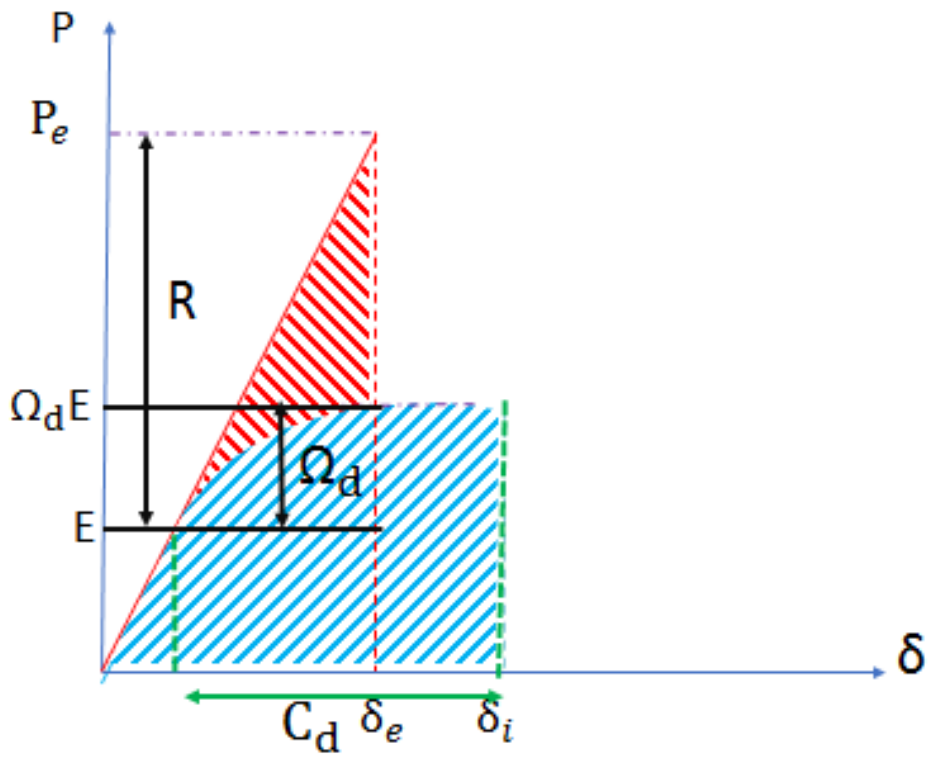
1. EQUIVALENT ENERGY



2. EQUIVALENT DISPLACEMENT



CODE PARAMETERS

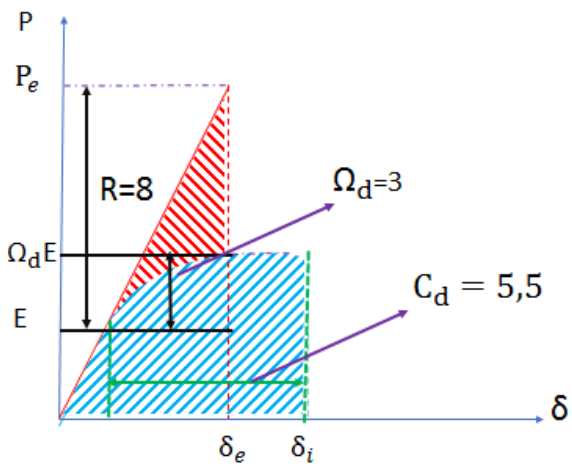


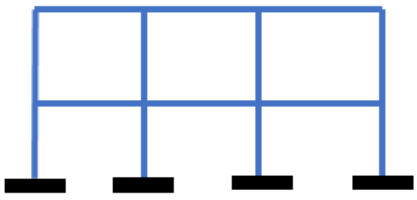
R: Seismic Modification Coefficen

Ω_d: Overstrength Factor

C_d: Deflection Amplification Factor

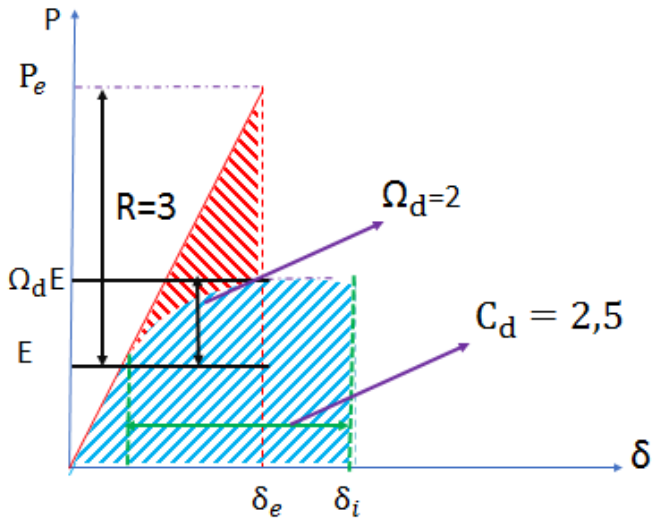
CODE PARAMETERS – SPECIAL MOMENT FRAME





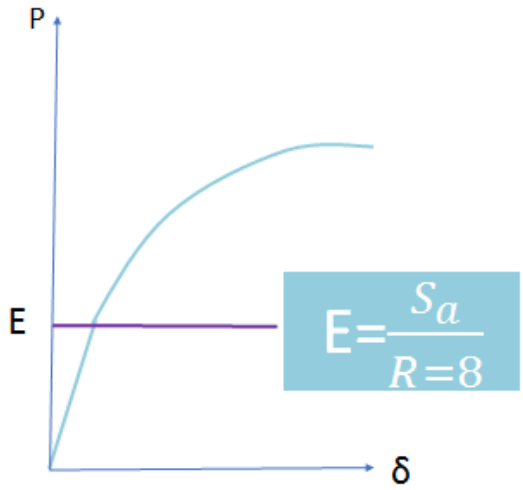
BENDING → E
SHEAR → $\Omega_e E$

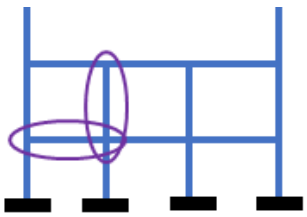
CODE PARAMETERS – CANTILEVER COLUMN



BENDING → E
SHEAR → Ω_eE

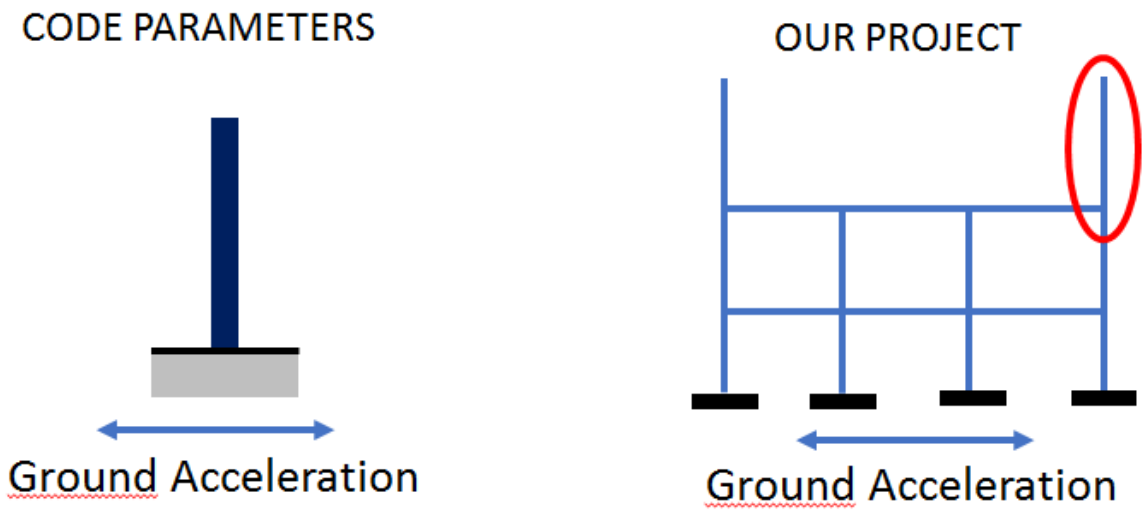
EQ PARAMETERS – SPECIAL MOMENT FRAME

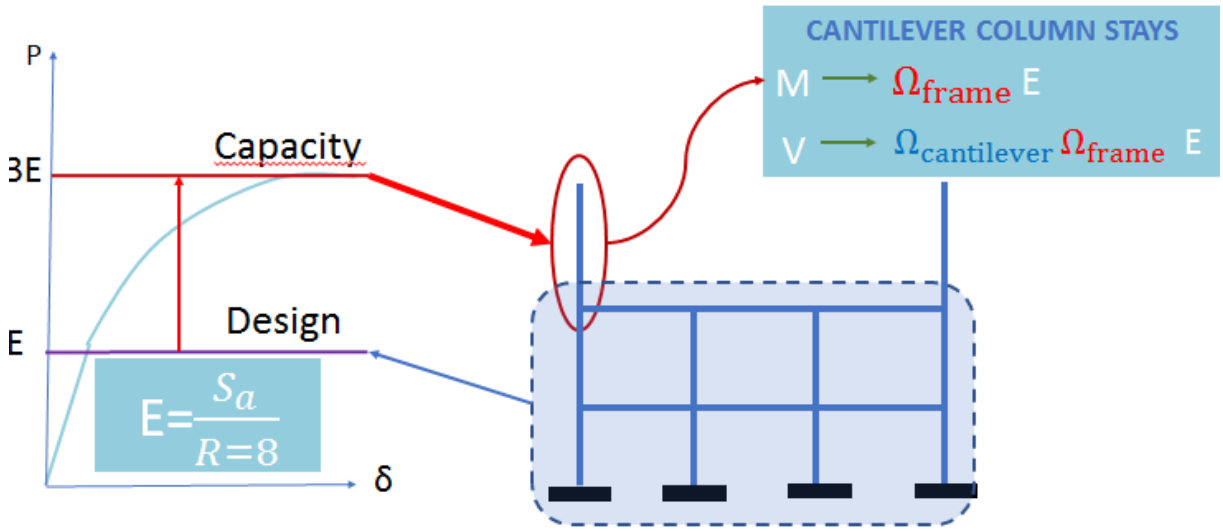


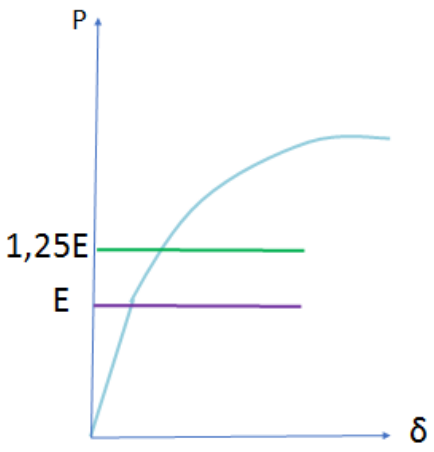


Bending (M) → **E**
Shear (V) → Ω_e **E**

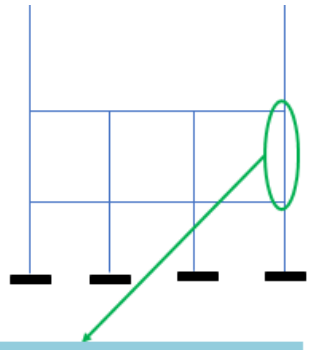
SPECİAL CASE







$$E=\frac{S_a}{R=8}$$



M ⟶ 1,25E ASCE 7-10

V ⟶ Ω_{frame} 1,25E

7. PROPERTİES OF MATERİAL

CONCRETE (from TS500)

- Foundation : C30
- Beam : C30
- Slab : C30
- Column : C30

f_{cd} = 30 f_{cd} = 20 for C30

REINFORCEMENT (from TS500)

S420 f_{yk} (yield strength) = 420Mpa

SOIL PARAMETER

- Allowable Soil Stress : $\sigma_{z.em.}$ = 180 kN/m^3
- Local Soil Class : Z2
- Group of Soil : B
- Period : T_A=0.15 sn T_B=0.40 sn
- Specific Weight : γ_{soil} = 20 kN/m^3
- Effective Ground Acceleration Coefficient : A_0=0.40
- Earthquake Magnitude : 7.00 < M < 7.50

STEEL (from TS648)

S235 : f_y = 355MPa f_u = 510MPa ($t \leq 40$mm)

EARTHQUAKE PARAMETER

- Regional Earthquake Level : DD-2
- Effective Ground Acceleration Coefficient : A_0=0.40
- Building Important Coefficient : 1
- Seismic Modification Coefficent : 6

8. LOAD ANALYSIS

8.1. DEAD LOAD

Concrete Unit Weight	2.50 t/m^3
Marble	2.60 t/m^3
Mortar	2.10 t/m^3
Plaster	2.00 t/m^3
Course Aggregate	1.80 t/m^3
Water Resistance Mebrane	0.5 t/m^3
Termal Isolation Material	0.10 t/m^3

Normal Floor Slab

15 mm Marble : 26 x 0.015 = 0.39 kN/m^2

30 mm Mortar : 21 x 0.03 = 0.39 kN/m^2

180 mm RC Slab : 25 x 0.18 = 4.5 kN/m^2

20 mm Plaster : 20 x 0.02 = 0.4 kN/m^2

Total Dead Load (G) = 5.92 kN/m^2

Design Load = 1.4 * 5.92 + 1.6 * 5 = 16.288 kN/m^2

Roof Floor Slab

50 mm Course Aggregate : 18 x 0.05 = 0.9 kN/m^2

5 mm Water Resistance Mebrane : 5 x 0.005 = 0.025 kN/m^2

20 mm Termal Isolation Material : 1 x 0.02 = 0.02 kN/m^2

30 mm Mortar : 21 x 0.03 = 0.63 kN/m^2

18 mm RC Slab : 25 x 0.18 = 4.5 kN/m^2

20 mm Plaster : 20 x 0.02 = 0.4 kN/m^2

Total Dead Load (G) = 6.475 kN/m^2

Design Load = 1.4 * 6.475 + 1.6 * 5 = 17.065 kN/m^2

8.2. LIVE LOAD

Static load that affect the structural elements and relocated from time to time

Machine Load	0,50 kN/m^2
Snow Load	0,75 kN/m^2
Wind Load	0,50 kN/m^2

The wind load to be taken in analysis and design of building elements according to TS 498 is summarized in Table.

Heigth from Foundaiton (m)	Wind Speed v (m/s)	Absorbtion q (kN/m^2)
0-8	28	0.5
9-20	36	0.8
21-100	42	1.1
>100	46	1.3

8.3. EARTQUAKE LOAD

DEFINITION OF ELASTIC EARTHQUAKE LOADS: SPECTRAL ACCELERATION COEFFICIENT

The Spectral Acceleration Coefficient, A (T), which shall be considered as the basis for the determination of seismic loads is given by Equation (2.1). Elastic Spectral Acceleration, Sae (T) which is the ordinate of Elastic Acceleration Spectrum defined for 5 % damped rate is derived by multiplying Spectral Acceleration Coefficient with gravity, g.

$$A(T) = A_0 * I * S(T) \qquad \textbf{(2.1)}$$

$$S_{ae}(T) = A(T) * g$$

- **Effective Ground Acceleration Coefficient**

The Effective Ground Acceleration Coefficient, Ao, appearing in Equation (2.1) is specified in Table 2.

TABLE 2.2 - EFFECTIVE GROUND ACCELERATION COEFFICIENT (Ao)

Eartquake Zone	A0
1	0.4
2	0.3
3	0.2
4	0.1

- **Building Importance Coefficient**

The Building Importance Factor, I, appearing in Equation (2.1) is specified in Table 2.3.

TABLE 2.3 - BUILDING IMPORTANT COEFFICIENT (I)

Purpose of Occupancy or Type of Building	Building Importance Factor
1. Buildings required to be utilized after the earthquake and buildings containing hazardous materials a) Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) b) Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
2. Intensively and long-term occupied buildings and buildings preserving valuable goods a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. b) Museums	1.4
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc	1.2
4. Other buildings Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

- **Spectrum Coefficient**

The Spectrum Coefficient, S(T), appearing in Equation (2.1) shall be determined by Equation (2.2), depending on the local site conditions and the building natural period, T (Figure 2.5):

$$S(T) = 1 + 1.5 * T / T_A \quad (0 \leq T \leq T_A)$$

$$S(T) = 2.5 \quad (T_A < T \leq T_B) \qquad \textbf{(2.2)}$$

$$S(T) = 2.5 * (T_B / T)^{0.8} \quad (T_B < T)$$

Spectrum Characteristic Periods, T_A and T_B , appearing in Equation (2.2) are specified in Table 2.4, depending on Local Site Classes defined in Table 6.2 of Chapter 6.

TABLE 2.4 - SPECTRUM CHARACTERISTIC PERIODS (T_A, T_B)

Local Soil Class	T_A(s)	T_B(s)
Z1	0.1	0.3
Z2	0.15	0.4
Z3	0.15	0.6
Z4	0.2	0.9

In case where the requirements specified in 6.2.1.2 and 6.2.1.3 of Chapter 6 are not met, spectrum characteristic periods defined in Table 2.4 for local site class Z4 shall be used.

- **Special Design Acceleration Spectra**

In required cases, elastic acceleration spectrum may be determined through special investigations by considering local seismic and site conditions. However spectral acceleration coefficients corresponding to so obtained acceleration spectrum ordinates shall in no case be less than those determined by **Equation (2.1)** based on relevant characteristic periods specified in **Table 2.4.**

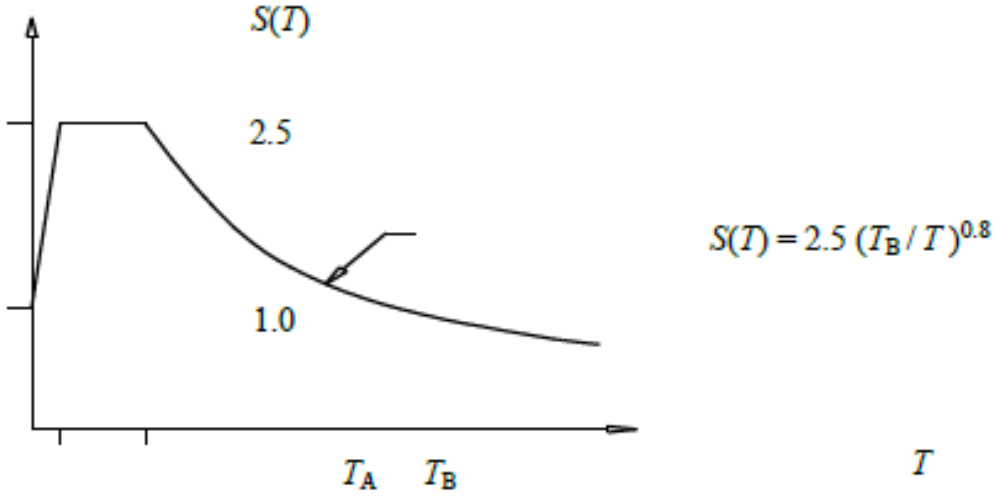

S(T)
2.5
1.0
S(T) = 2.5 (T_B / T)^0.8
T_A
T_B
T

TABLE 2.5 - STRUCTURAL SYSTEM BEHAVIOUR FACTORS (R)

BUILDING STRUCTURAL SYSTEM	*Systems of Nominal Ductility Level*	*Systems of High Ductility Level*
(1) <u>CAST-IN-SITE REINFORCED CONCRETE BUILDINGS</u>		
(1.1) Buildings in which seismic loads are fully resisted by frames..	4	8
(1.2) Buildings in which seismic loads are fully resisted by coupled structural walls..	4	7
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls..	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and / or coupled structural walls..	4	7
<u>(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS</u>		
(2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer	3	7
(2.2) Single-storey buildings in which seismic loads are fully resisted by columns with hinged upper connections	-	3
(2.3) Prefabricated buildings with hinged frame connections in which seismic loads are fully resisted by prefabricated or cast – in – situ solid structural walls and / or coupled structural walls.	-	5
(2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and / or coupled structural walls	3	6

3) STRUCTURAL STEEL BUILDINGS **(3.1)** Buildings in which seismic loads are fully resisted by frames.. **(3.2)** Single – storey buildings in which seismic loads are fully resisted by columns with connections hinged at the top.. **(3.3)** Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls **(a)** Centrically braced frames.. **(b)** Eccentrically braced frames.. **(c)** Reinforced concrete structural walls.. **(3.4)** Buildings in which seismic loads are jointly resisted by structural steel braced frames or cast-in-situ reinforced concrete structural walls **(a)** Centrically braced frames.. **(b)**Eccentrically braced frames.. **(c)** Reinforced concrete structural walls..	5 - 4 - 4 5 - 4	8 4 5 7 6 6 8 7

TABLE 2.7 - LIVE LOAD PARTICIPATION FACTOR (n)

Purpose of Occupancy of Building	***n***
Depot, warehouse, etc.	0.8
School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0.6
Residence, office, hotel, hospital, etc.	0.3

Storey weights wi shall be calculated by Equation (2.6).

$$wi = gi + n\ qi \qquad \textbf{(2.6)}$$

Live Load Participation Factor, n , appearing in Equation (2.6) is given in Table 2.7. In industrial buildings, n = 1 shall be taken for fixed equipment weights while crane payloads shall not be taken into account in the calculation of storey weights 30 % of snow loads shall be considered in calculation of weight of roof top to be used in determination of seismic loads.

8.4. SEISMIC SOIL PRESSURES AND SOIL RETAINING STRUCTURES

6.4.1 - Total Active and Passive Pressure Coefficients

6.4.1.1 – *Total Active Pressure Coefficient*, K_{at}, and *Total Passive Pressure Coefficient*, K_{pt}, which shall be used to calculate the sum of static soil pressure and additional dynamic soil pressure induced by earthquake are given by **Equation (6.1)**, by neglecting the soil cohesion in order to remain on the conservative side.

$$K_{at} = \frac{(1 \pm C_v)\cos^2(\varphi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \left[1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - \lambda - i)}{\cos(\delta + \alpha + \lambda)\cos(i - \alpha)}}\right]^{-2} \quad \textbf{(6.1a)}$$

$$K_{pt} = \frac{(1 \pm C_v)\cos^2(\varphi - \lambda + \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta - \alpha + \lambda)} \left[1 - \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - \lambda + i)}{\cos(\delta - \alpha + \lambda)\cos(i - \alpha)}}\right]^{-2} \quad \textbf{(6.1b)}$$

6.4.1.2 – The angle l in **Equation (6.1)** is defined by **Equation (6.2)**.

(a) For dry soils,

$$\lambda = \arctan\left[\frac{C_h}{(1 \pm C_v)}\right] \quad \textbf{(6.2a)}$$

(b) For submerged soils,

$$\lambda = \arctan\left[\frac{\gamma_s}{\gamma_b}\,\frac{C_h}{(1 \pm C_v)}\right] \quad \textbf{(6.2b)}$$

6.4.1.3 –In the case of submerged or saturated soils, d in **Equation (6.1)** shall be replaced by d/2.

6.4.1.4 – Equivalent lateral seismic coefficient, *C*h, appearing in **Equation (6.2)** is defined by **Equation (6.3)**.

(a) In soil retaining structures behaving as vertical free cantilevers,

$$C_h = 0.2\,(I + 1)\,A_o \quad \textbf{(6.3a)}$$

(b) In soil retaining structures and elements horizontally supported by building floors or soil anchors,

$$C_h = 0.3\,(I + 1)\,A_o \quad \textbf{(6.3b)}$$

6.4.1.5 – Equivalent vertical seismic coefficient, Cv, appearing in **Equation (6.1)** and **Equation (6.2)** is defined by **Equation (6.4)**. However, it shall be Cv = 0 in basement walls which are horizontally supported by building floors.

$$\mathrm{Cv} = \frac{\mathbf{2*Ch}}{\mathbf{3}} \qquad \textbf{(6.4)}$$

The cases + Cv or - Cv shall be considered as consistent with **Equation (6.2)** to yield more unfavorable lateral soil pressure by **Equation (6.1)**.

6.4.2. Dynamic Active and Passive Soil Pressures

6.4.2.1 – Dynamic active pressure coefficient, Kad, and dynamic passive pressure coefficient, K_{pd}, induced by earthquake shall be determined by **Equation (6.5)**.

$$K_{ad} = K_{at} - K_{as} \quad \textbf{(6.5a)}$$

$$K_{pd} = K_{pt} - K_{ps} \quad \textbf{(6.5b)}$$

Static active pressure coefficient, Kas, and static passive pressure coefficient, K_{ps}, appearing in **Equation (6.5)** may be obtained by substituting l = 0 and Cv = 0 in **Equation (6.1)**.

6.4.2.2 – Variation of dynamic active and passive soil pressures along the depth of soil which is induced by soil mass in addition to static soil pressure in case of earthquake which are induced in addition to static soil pressure by the soil mass during earthquake, is defined by **Equation (6.6)**.

$$p_{ad}(z) = 3\, K_{ad}\,(1 - z / H)\, p\mathrm{v}\,(z) \quad \textbf{(6.6a)}$$

$$p_{pd}(z) = 3\, K_{pd}\,(1 - z / H)\, p\mathrm{v}\,(z) \quad \textbf{(6.6b)}$$

In the special case of uniform and dry soil, the positive value of the resultant P_{ad} of dynamic active soil pressure and the negative value of the resultant P_{pd} of dynamic passive soil pressure which are induced in addition to static soil because of earthquake and z_{cd} which indicates the depths of such resultants measured from the top soil level, are obtained as given

by **Equation (6.7)** and **Equation (6.8)**, respectively, by integrating **Equation (6.6)** along the soil depth by taking $p_v(z) = \gamma z$;

$$P_{ad} = 0.5\ \gamma K_{ad} H2 \quad \textbf{(6.7a)}$$

$$P_{pd} = 0.5\ \gamma K_{pd} H2 \quad \textbf{(6.7b)}$$

$$z_{cd} = H / 2 \quad \textbf{(6.8)}$$

In the case of submerged soil, gb shall be considered in lieu of g in determining *p*v (*z*) and hydrodynamic pressure of water shall not be calculated additionally. In the case of saturated soil, gs shall be used in lieu of γ.

6.4.2.3 – Variation of dynamic active and passive pressures along the depth of soil which are induced in addition to static soil pressure by uniformly distributed external loads in case of earthquake, are defined by **Equation (6.9)**.

$$q_{ad}(z) = 2\ q_o K_{ad} (1 - \gamma / H) \cos\alpha / \cos(\alpha - i) \quad \textbf{(6.9a)}$$

$$q_{pd}(z) = 2\ q_o K_{pd} (1 - \gamma / H) \cos\alpha / \cos(\alpha - i) \quad \textbf{(6.9b)}$$

In the case where soil characteristics are uniform, the resultants *Q*ad and *Q*pd of active (positive) and passive (negative) soil pressures which are induced in addition to static soil pressure by contribution of earthquake and zcd which indicates the depths of such resultants measured from the top soil level, are obtained as given by **Equation (6.10)** and **Equation (6.11)**, respectively, by integrating **Equation (6.9)** along the soil depth.

$$Q_{ad} = q_o K_{ad} H \cos\alpha / \cos(\alpha - i) \quad \textbf{(6.10a)}$$

$$Q_{pd} = q_o K_{pd} H \cos\alpha / \cos(\alpha - i) \quad \textbf{(6.10b)}$$

$$z_{cd} = H / 3 \quad \textbf{(6.11)}$$

9. Seismic Hazard Map Detail Report

T.C. BAŞBAKANLIK
Afet ve Acil Durum Yönetimi Başkanlığı

Türkiye Deprem Tehlike Haritaları İnteraktif Web Uygulaması

Kullanıcı Girdileri

Rapor Başlığı	Bitirme	
Deprem Yer Hareketi Düzeyi	DD-2	50 yılda aşılma olasılığı %10 (tekrarlanma periyodu 475 yıl) olan deprem yer hareketi düzeyi
Yerel Zemin Sınıfı	ZB	Az ayrışmış, orta sağlam kayalar
Enlem	40.825°	
Boylam	29.345°	

Çıktılar

$S_S = 1.247$ $S_1 = 0.343$ $PGA=0.507$ $PGV=34.612$

S_S : Kısa periyod bölgesi için harita spektral ivme katsayısı(boyutsuz)

S_1 : 1.0 saniye periyod için harita spektral ivme katsayısı(boyutsuz)

PGA : En büyük yer ivmesi (g)

PGV : En büyük yer hızı (cm/sn)

Yerel Zemin Sınıfları

Yerel zemin sınıfı	Zemin cinsi	Üst 30 metrede ortalama		
		$(V_S)_{30}$ [m/s]	$(N_{60})_{30}$ [darbe/30 cm]	$(C_u)_{30}$ [kPa]
ZA	Sağlam, sert kayalar	> 1500	-	-
ZB	Az ayrışmış, orta sağlam kayalar	760 - 1500	-	-
ZC	Çok sıkı kum, çakıl ve sert kil tabakaları veya ayrışmış, çok çatlaklı zayıf kayalar	360 - 760	> 50	> 250
ZD	Orta sıkı – sıkı kum, çakıl veya çok katı kil tabakaları	180 - 360	15 - 50	70 - 250
ZE	Gevşek kum, çakıl veya yumuşak – katı kil tabakaları veya PI > 20 ve w > % 40 koşullarını sağlayan toplamda 3 metreden daha kalın yumuşak kil tabakası (c_u < 25 kPa) içeren profiller	< 180	< 15	< 70
ZF	Sahaya özel araştırma ve değerlendirme gerektiren zeminler: 1) Deprem etkisi altında çökme ve potansiyel göçme riskine sahip zeminler (sıvılaşabilir zeminler, yüksek derecede hassas killer, göçebilir zayıf çimentolu zeminler vb), 2) Toplam kalınlığı 3 metreden fazla turba ve/veya organik içeriği yüksek killer, 3) Toplam kalınlığı 8 metreden fazla olan yüksek plastisiteli (PI > 50) killer, 4) Çok kalın (> 35 m) yumuşak veya orta katı killer.			

Yerel Zemin Etki Katsayıları

Yerel Zemin Sınıfı	Kısa periyod bölgesi için Yerel Zemin Etki Katsayısı F_S					
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 1.50$
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	Sahaya özel zemin davranış analizi yapılacaktır					

Yerel Zemin Sınıfı ZB ve S_S =1.247 için F_S=0.900

Yerel Zemin Sınıfı	1.0 saniye periyod için Yerel Zemin Etki Katsayısı F_1					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	Sahaya özel zemin davranış analizi yapılacaktır					

Yerel Zemin Sınıfı ZB ve S_1 =0.343 için F_1=0.800

Faya Yakınlık Katsayısı

y_F= 1.2	$L_F \leq$ 15km
y_F= 1.2 - 0.02(L_F - 15)	15km < $L_F \leq$ 25km
DD-3 ve DD-4 için y_F 1.0 alınacaktır	
L_F : Fay düzlemine olan mesafe (km) y_F : Faya yakınlık katsayısı	
L_F = 10.5 Km için y_F = 1.200	

Tasarım Spektral İvme Katsayıları

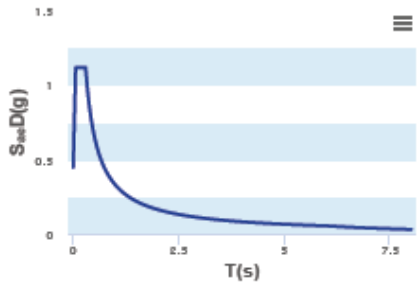
$$S_{DS} = S_S F_S = 1.247 \text{ x } 0.900 = 1.122$$

$$S_{D1} = S_1 y_F F_1 = 0.343 \text{ x } 1.200 \text{ x } 0.800 = 0.329$$

S_{DS} : Kısa periyod bölgesi için tasarım spektral ivme katsayısı (boyutsuz)

S_{D1} : 1.0 saniye periyod için tasarım spektral ivme katsayısı (boyutsuz)

Yatay Elastik Tasarım Spektrumu



$S_{ae}(T)=(0.4+0.6(T/T_A))S_{DS}$
$S_{ae}(T)=S_{DS}$
$S_{ae}(T)=S_{D1}/T$
$S_{ae}(T)=S_{D1}T_L/T^2$

$T_A = 0.2S_{D1}/S_{DS}$ $T_B = S_{D1}/S_{DS}$

T(S)	
0	0.44892
0.05	1.022698
0.05868	1.122113
0.1	1.1223
0.15	1.1223
0.2	1.1223
0.25	1.1223
0.293398	1.12228
0.3	1.0976
0.35	0.9408
0.4	0.8232
0.45	0.731733
0.5	0.65856
0.55	0.598691
0.6	0.5488
0.65	0.506585
0.7	0.4704
0.75	0.43904

0.8	0.4116
0.85	0.387388
0.9	0.365867
0.95	0.346611
1	0.32928
1.05	0.3136
1.1	0.299345
1.15	0.28633
1.2	0.2744
1.25	0.263424
1.3	0.253292
1.35	0.243911
1.4	0.2352
1.45	0.22709
1.5	0.21952
1.55	0.212439
1.6	0.2058
1.65	0.199564
1.7	0.193694
1.75	0.18816
1.8	0.182933
1.85	0.177989
1.9	0.173305
1.95	0.168862
2	0.16464
2.05	0.160624
2.1	0.1568
2.15	0.153153
2.2	0.149673

2.25	0.146347
2.3	0.143165
2.35	0.140119
2.4	0.1372
2.45	0.1344
2.5	0.131712
2.55	0.129129
2.6	0.126646
2.65	0.124257
2.7	0.121956
2.75	0.119738
2.8	0.1176
2.85	0.115537
2.9	0.113545
2.95	0.11162
3	0.10976
3.05	0.107961
3.1	0.106219
3.15	0.104533
3.2	0.1029
3.25	0.101317
3.3	0.099782
3.35	0.098293
3.4	0.096847
3.45	0.095443
3.5	0.09408
3.55	0.092755
3.6	0.091467
3.65	0.090214

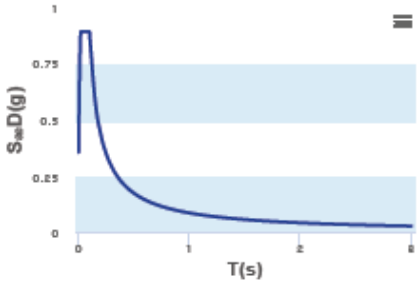
3.7	0.088995
3.75	0.087808
3.8	0.086653
3.85	0.085527
3.9	0.084431
3.95	0.083362
4	0.08232
4.05	0.081304
4.1	0.080312
4.15	0.079345
4.2	0.0784
4.25	0.077478
4.3	0.076577
4.35	0.075697
4.4	0.074836
4.45	0.073996
4.5	0.073173
4.55	0.072369
4.6	0.071583
4.65	0.070813
4.7	0.07006
4.75	0.069322
4.8	0.0686
4.85	0.067893
4.9	0.0672
4.95	0.066521
5	0.065856
5.05	0.065204
5.1	0.064565

5.15	0.063938
5.2	0.063323
5.25	0.06272
5.3	0.062128
5.35	0.061548
5.4	0.060978
5.45	0.060418
5.5	0.059869
5.55	0.05933
5.6	0.0588
5.65	0.05828
5.7	0.057768
5.75	0.057266
5.8	0.056772
5.85	0.056287
5.9	0.05581
5.95	0.055341
6	0.05488
6.05	0.053977
6.1	0.053095
6.15	0.052236
6.2	0.051396
6.25	0.050577
6.3	0.049778
6.35	0.048997
6.4	0.048234
6.45	0.047489
6.5	0.046762
6.55	0.04605

6.6	0.045355
6.65	0.044676
6.7	0.044012
6.75	0.043362
6.8	0.042727
6.85	0.042105
6.9	0.041497
6.95	0.040902
7	0.04032
7.05	0.03975
7.1	0.039192
7.15	0.038646
7.2	0.038111
7.25	0.037587
7.3	0.037074
7.35	0.036571
7.4	0.036079
7.45	0.035596
7.5	0.035123
7.55	0.03466
7.6	0.034205
7.65	0.033759
7.7	0.033322
7.75	0.032894
7.8	0.032473
7.85	0.032061
7.9	0.031656
7.95	0.03126
8	0.03087

T_A = 0.059 (s) T_B = 0.293 (s) T_L = 6.000 (s)

Düşey Elastik Tasarım Spektrumu



$S_{aeD}(T)=(0.32+0.48(T/T_{AD}))S_{DS}$

$S_{aeD}(T)=0.8S_{DS}$

$S_{aeD}(T)=0.8S_{DS}T_{BD}/T$

$T_{AD} = T_A/3$ $T_{BD} = T_B/3$

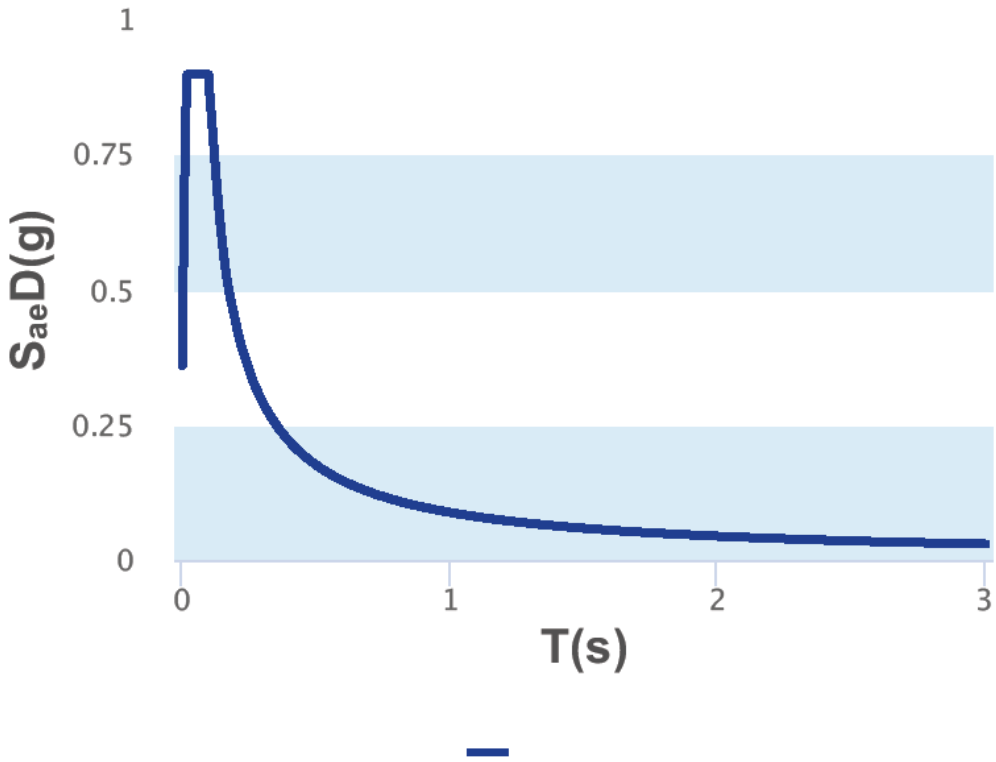
T(s)	
0	0.359136
0.01956	0.897835
0.05	0.89784
0.097799	0.897825
0.1	0.87808
0.15	0.585387
0.2	0.43904
0.25	0.351232
0.3	0.292693
0.35	0.25088
0.4	0.21952
0.45	0.195129
0.5	0.175616
0.55	0.159651
0.6	0.146347
0.65	0.135089

0.7	0.12544
0.75	0.117077
0.8	0.10976
0.85	0.103304
0.9	0.097564
0.95	0.092429
1	0.087808
1.05	0.083627
1.1	0.079825
1.15	0.076355
1.2	0.073173
1.25	0.070246
1.3	0.067545
1.35	0.065043
1.4	0.06272
1.45	0.060557
1.5	0.058539
1.55	0.05665
1.6	0.05488
1.65	0.053217
1.7	0.051652
1.75	0.050176
1.8	0.048782
1.85	0.047464
1.9	0.046215
1.95	0.04503
2	0.043904
2.05	0.042833

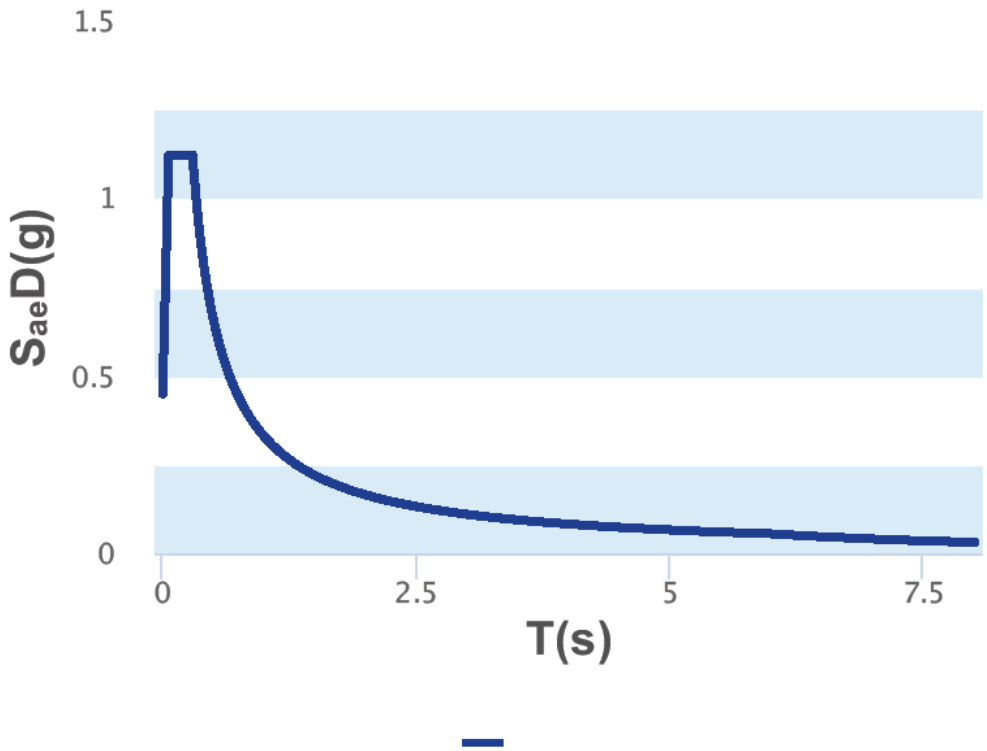
2.1	0.041813
2.15	0.040841
2.2	0.039913
2.25	0.039026
2.3	0.038177
2.35	0.037365
2.4	0.036587
2.45	0.03584
2.5	0.035123
2.55	0.034435
2.6	0.033772
2.65	0.033135
2.7	0.032521
2.75	0.03193
2.8	0.03136
2.85	0.03081
2.9	0.030279
2.95	0.029765
3	0.029269

T_{AD} = 0.020 (s) T_{BD} = 0.098 (s) T_{LD} = 3.000 (s)

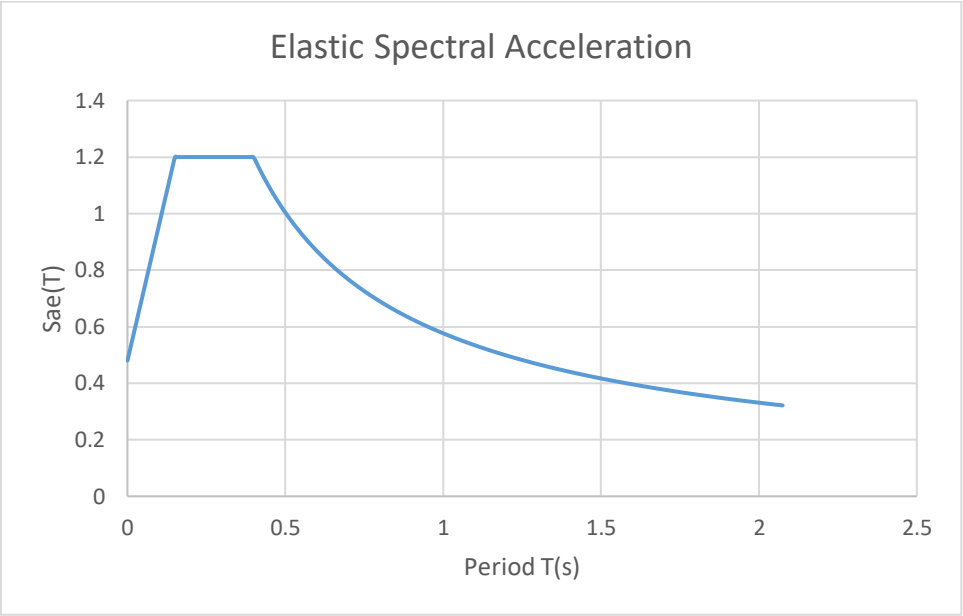
9.1.Vertical Spectrum



9.2.Horizontal Spectrum



9.3.Elastic Spectral Acceleration Diagram



10. PRELIMINARY DESIGN

10.1. Column Design

10.1.1. Column Dimension Calculation

	Slab														
Column Name	g(kN)	q(kN)	Parapet(kN)	Roof Weight (kN	Beam Weight(kN)	Wall Weight(kN)	Column Weight(kN)	The Load From Upper Stories(kN)	Total Weight(kN)	Ac(mm^2	a	a*Ac(mm^2)	Ac cm^2	Choosen Column	
301	0	0		44,75	0	0	96	0	197,05	13136,7	1,8	23646,00	236,46	70x70	4900 cm^2
302	0	0		44,75	0	0	96	0	197,05	13136,7	1,8	23646,00	236,46	70x70	
303	58,28	45	12	44,75	19,45	0	96	0	394,67	26311,5	1,8	47360,64	473,61	70x70	
304	116,55	90			28,35	0	96	0	481,26	32084	1,8	57751,20	577,51	70x70	
305	116,55	90	24		28,35	0	96	0	514,86	34324	1,8	61783,20	617,83	70x70	
306	58,28	45	12		18,9	0	96	0	331,25	22083,5	1,8	39750,24	397,50	70x70	
307	233,1	180			37,8	0	96	0,00	801,66	53444	1,8	96199,20	961,99	70x70	
308	61,65	45,91	12		28,35	0	96	0,00	350,66	23377,1	1,8	42078,72	420,79	70x70	
201	53,28	45			18,9	108	64	197,05	610,90	40726,8	1,8	73308,24	733,08	70x70	
202	106,56	90			28,35	138	64	197,05	812,72	54181,6	1,8	97526,88	975,27	70x70	
203	106,56	90			28,35	138	64	394,67	1010,35	67356,4	1,8	121241,52	1212,42	70x70	
204	213,12	180			37,8	120	64	481,26	1378,15	91876,5	1,8	165377,76	1653,78	70x70	
205	106,56	90			28,35	138	64	514,86	1130,53	75368,9	1,8	135664,08	1356,64	70x70	
206	53,28	45			18,9	108	64	331,25	745,10	49673,6	1,8	894,12,48	894,12	70x70	
207	213,12	180			37,8	120	64	801,66	1698,55	113237	1,8	203825,76	2038,26	70x70	
208	106,56	90			28,35	138	64	350,66	966,33	64422	1,8	115959,60	1159,60	70x70	
209	106,56	90			28,35	138	64	0,00	615,67	41044,9	1,8	73880,88	738,81	70x70	
210	213,12	180			37,8	120	64	0,00	896,89	59792,5	1,8	107626,56	1076,27	70x70	
211	213,12	180			37,8	120	64	0,00	896,89	59792,5	1,8	107626,56	1076,27	70x70	
101	53,28	45			18,9	108	64	610,90	1024,75	68316,9	1,8	122970,48	1229,70	70x70	
102	106,56	90			28,35	138	64	812,72	1428,40	95226,5	1,8	171407,76	1714,08	70x70	
103	106,56	90			28,35	138	64	1010,35	1626,02	108401	1,8	195122,40	1951,22	70x70	
104	213,12	180			37,8	120	64	1378,15	2275,04	151669	1,8	273004,32	2730,04	70x70	
105	106,56	90			28,35	138	64	1130,53	1746,21	116414	1,8	209544,96	2095,45	70x70	
106	53,28	45			18,9	108	64	745,10	1158,96	77263,7	1,8	139074,72	1390,75	70x70	
107	213,12	180			37,8	120	64	1698,55	2595,44	173029	1,8	311452,32	3114,52	70x70	
108	106,56	90			28,35	138	64	966,33	1582,00	105467	1,8	189840,48	1898,40	70x70	
109	106,56	90			28,35	138	64	615,67	1231,35	82089,9	1,8	147761,76	1477,62	70x70	
110	213,12	180			37,8	120	64	896,89	1793,78	119585	1,8	215253,12	2152,53	70x70	
111	213,12	180			37,8	120	64	896,89	1793,78	119585	1,8	215253,12	2152,53	70x70	

10.1.2. Column Weight

- Third Story Columns

Total SteelRoof Weight

18 . 18 . 1,1 = 356,4 kN ⇒ steel roof weight

$\frac{356,4}{8}$ = 44,55 kN (for each column at the roof)

C301

Roof: 44,55 kN

Column self weight: 96 kN

C302

Roof = 44,55 kN

Column self weight: 96 kN

C303

Slab: Ng: 6,475 . 3 . 3 = 58,275 kN

Nq: 5 . 3 . 3 = 45 kN

Roof: 44,55 kN

Paropet: 3 . 4 = 12 kN

Beam: 0,3 (0,6 - 0,18) . (3+3) . 25 = 18,9 kN

Column self weight: 96 kN

C304

Slab: Ng: 6,475 . 6 . 3 = 116,55 kN

Nq: 5 . 6 . 3 = 90 kN

Roof: 44,55 kN

Beam: 0,3 . (0,6 - 0,18) . (9) . 25 = 28,35 kN

Column self weight: 96 kN

C305

Slab: Ng = 6,475 . 6 . 3 = 116,55 kN

Nq = 5 . 6 . 3 = 90 kN

Paropet: 6 . 4 = 24 kN

Beam: 0,3 (0,6 - 0,18) . 9 . 25 = 28,35 kN

Column self weight: 96 kN

C306

Slab: Ng = 6,475 . 3 . 3 = 58,275 kN

Nq = 5 . 3 . 3 = 45 kN

Paropet: 3 . 4 = 12 kN

Beam: 0,3 . (0,6 - 0,18) . 6 . 25 = 18,9

Column self weight: 96 kN

C307

Slab: Ng = 6,475 . 6 . 6 = 233,1 kN

Nq = 5 . 6 . 6 = 180 kN

Beam: 0,3 . (0,6 - 0,18) . (6+6) . 25 = 37,8 kN

Column self weight = 96 kN

C308

Slab: Ng = 6,475 . 6 . 3 = 116,55 kN

Nq = 5 . 6 . 3 = 90 kN

Paropet: 3 . 4 = 12 kN

Beam: 0,3 . (0,6 - 0,18)(6+3) 25 = 28,35 kN

Column self weight = 96 kN

- **Second Story Columns**

C201	
Slab	N_q = 5 x 3 x 3 = 45 kN N_g = 5,92 x 3 x 3 = 53,28 kN
Wall	6 x 4,5 x 4 = 108 kN
Beam	0,3 x (0,6 – 0,18) x (3+3) x 25 = 18,9 kN
Column Self Weight	64 kN

C202	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4 x 4,5 + 3 x 4 x 2 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C203	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4 x 4,5 + 3 x 4 x 2 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C204	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4 x 2,5 = 120 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

C205	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4 x 4,5 + 3 x 4 x 2 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C206	
Slab	N_q = 5 x 3 x 3 = 45 kN N_g = 5,92 x 3 x 3 = 53,28 kN
Wall	6 x 4,5 x 4 = 108 kN
Beam	0,3 x (0,6 – 0,18) x (3+3) x 25 = 18,9 kN
Column Self Weight	64 kN

C207	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4 x 2,5 = 120 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

C208	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4 x 4,5 + 3 x 4 x 2 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C209	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4 x 4,5 + 3 x 4 x 2 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C210	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4 x 2,5 = 120 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

C211	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4 x 2,5 = 120 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

- **First Story Columns**

C101	
Slab	N_q = 5 x 3 x 3 = 45 kN N_g = 5,92 x 3 x 3 = 53,28 kN
Wall	6 x 4,5 x 4,6 = 124,2 kN
Beam	0,3 x (0,6 – 0,18) x (3+3) x 25 = 18,9 kN
Column Self Weight	64 kN

C102	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4,6 x 4,5 + 3 x 4,6 x 2,5 = 158,7 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C103	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4,6 x 4,5 + 3 x 4,6 x 2,5 = 158,7 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C104	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4,6 x 2,5 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

C105	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4,6 x 4,5 + 3 x 4,6 x 2,5 = 158,7 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C106	
Slab	N_q = 5 x 3 x 3 = 45 kN N_g = 5,92 x 3 x 3 = 53,28 kN
Wall	6 x 4,5 x 4,6 = 124,2 kN
Beam	0,3 x (0,6 – 0,18) x (3+3) x 25 = 18,9 kN
Column Self Weight	64 kN

C107	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4,6 x 2,5 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

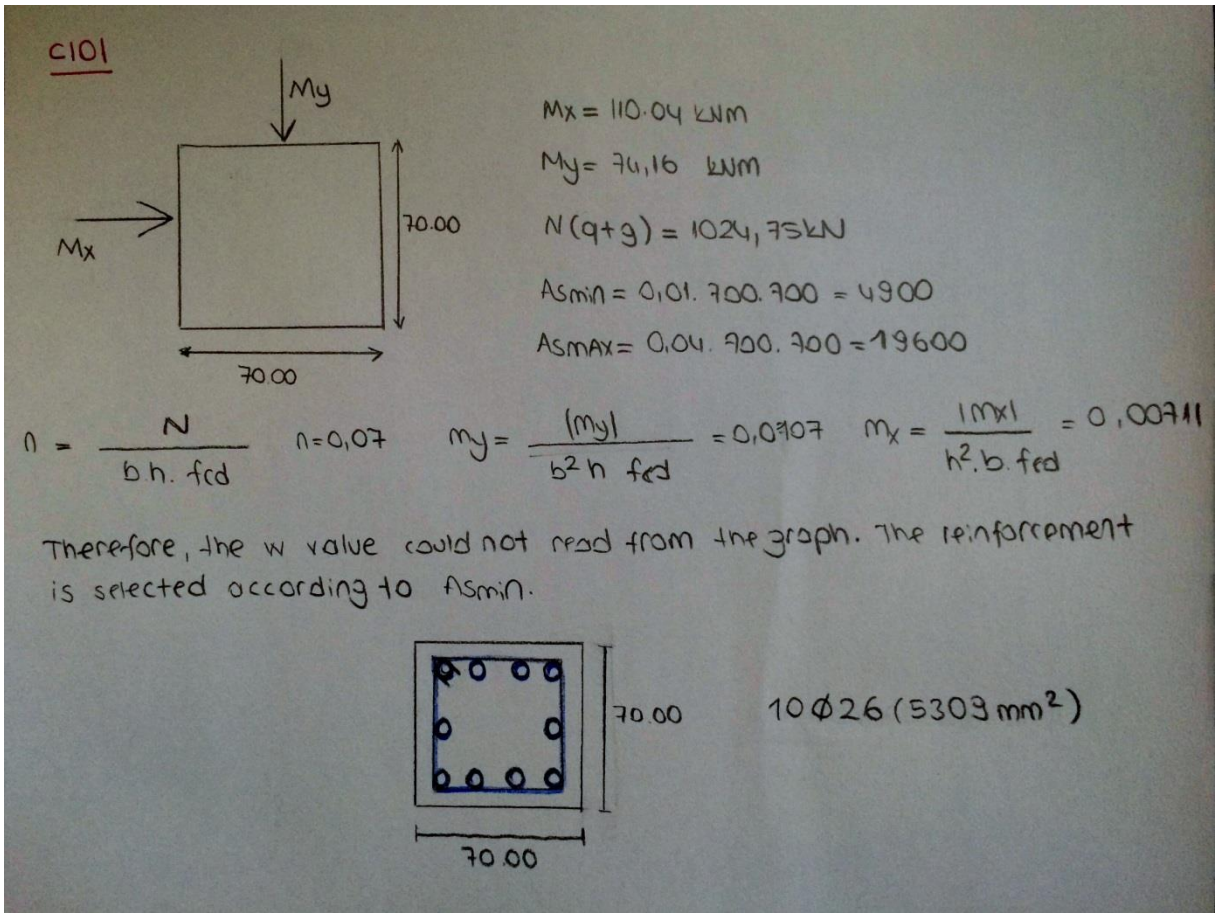
C108	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4,6 x 4,5 + 3 x 4,6 x 2,5 = 158,7 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C109	
Slab	N_q = 5 x 6 x 3 = 90 kN N_g = 5,92 x 6 x 3 = 106,56 kN
Wall	6 x 4,6 x 4,5 + 3 x 4,6 x 2,5 = 158,7 kN
Beam	0,3 x (0,6 – 0,18) x (6+3) x 25 = 28,35 kN
Column Self Weight	64 kN

C110	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4,6 x 2,5 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

C111	
Slab	N_q = 5 x 6 x 6 = 180 kN N_g = 5,92 x 6 x 6 = 213,12 kN
Wall	12 x 4,6 x 2,5 = 138 kN
Beam	0,3 x (0,6 – 0,18) x (6+6) x 25 = 37,8 kN
Column Self Weight	64 kN

10.1.3. Reinforcement for Columns



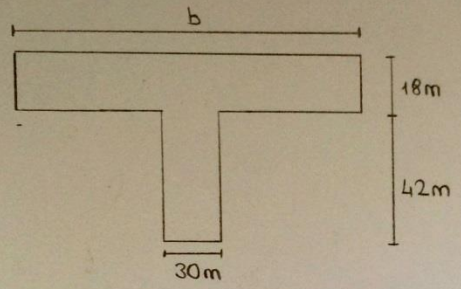
When n, m_y and m_x values are calculated, the w value could not read from the graph. The reinforcement is selected according to As_{min} for C102, C102', C103, C104, C105, C106, C107, C108, C109, C110, C111.

10 Φ 26 (5309 mm^2) ➔ is selected

10.2. Beam Design

10.2.1. Effective Flange of Beams

Effective Flange of Beams



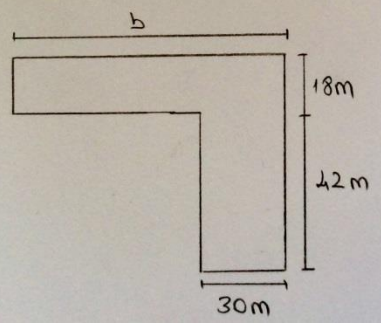
$b = b_w + \frac{l_p}{5}$

$l_p = a \cdot l$ $\quad a = 0{,}8 \rightarrow$ if exterior span of continuous beam

$a = 0{,}6 \rightarrow$ if interior span of continuous beam

$l_p = 0{,}8 \cdot 6 = 4{,}8\ m$

$b = 0{,}3 + \frac{4{,}8}{5} = 1{,}26\ m$



$b = b_w + \frac{l_p}{10}$ for asymetric

$l_p = 0{,}6 \cdot 6 = 3{,}6\ m$

$b = 0{,}3 + \frac{3{,}6}{10} = 0{,}66\ m$

Beams moment of Inertia

- Interior Beam

$\frac{b_w}{b} = \frac{0{,}3}{1{,}26} = 0{,}2381$

$\frac{h_f}{h} = \frac{0{,}18}{0{,}6} = 0{,}3$

$\mu = 27{,}8$

$I = \frac{bh^3}{\mu} = \frac{(1{,}26)(0{,}6)^3}{27{,}8} = 0{,}0098$

- Exterior Beam

$\frac{b_w}{b} = \frac{0{,}3}{0{,}66} = 0{,}4545$

$\frac{h_f}{h} = \frac{0{,}18}{0{,}6} = 0{,}3$

$\mu = 18{,}6$

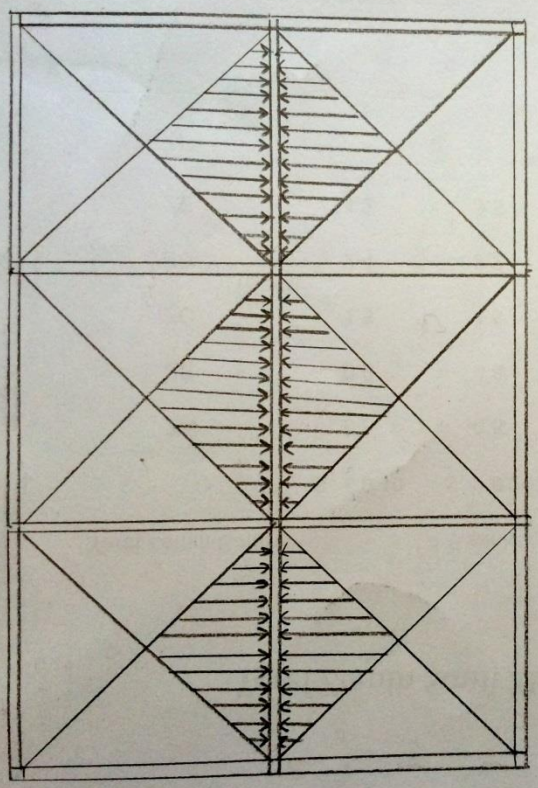
$I = \frac{bh^3}{\mu} = \frac{(0{,}66)(0{,}6)^3}{18{,}6} = 0{,}0077$

Colums moment of Inertia

Third story column $\Rightarrow$ $I_c = \frac{(0{,}7)(0{,}7)^3}{12} = 0{,}02$

- column = 2,61 I
- Exterior Beam = I
- Interior Beam = 1,28 I

10.2.2. Load Calculation of Beams



• Third Story Beams

Exterior Beam

Slab: $g = \frac{(6,475)(6)}{3} = 12,95$

$q = \frac{5.6}{3} = 10$

Parapet: 4 kN

Beam: 0,3 . 0,42 . 25 = 3,15 kN

$\Sigma g = 20,10$ kN $\quad \Sigma q = 10$ kN

Interior Beam

Slab: $g = \frac{(6,475)(6)}{3} = 25,9$

$q = \frac{5.6}{3} = 10$

Beam: 0,3 . 0,42 . 25 = 3,15 kN

$\Sigma g = 29,05$ kN $\quad \Sigma q = 10$ kN

• Second Story Beams

Exterior Beam

Slab: $g = \frac{(5,92)(6)}{3} = 11,84$ kN

$q = \frac{5.6}{3} = 10$ kN

Wall: (4)(5) . (4 − 0,6) = 15,3 kN

Beam: (0,3) (0,42) (25) = 3,15 kN

$\Sigma g = 30,29$ kN $\quad \Sigma q = 10$ kN

Interior Beam (Steel Roof Area)

Slab: $g = \frac{(5,92)(6)}{3} = 11,84$ kN

$q = \frac{5.6}{3} = 10$ kN

Beam: (0,3) (0,42) (25) = 3,15 kN

$\Sigma g = 26,83$ kN $\quad \Sigma q = 10$ kN

Interior Beam (RC Frame Area)

Slab: $g = \frac{(5,92)(6)}{3} . 2 = 23,68$ kN

$q = \frac{5.6}{3} = 10$ kN

Wall: (2) . (5) . (4 − 0,6) = 8,5 kN

beam: 0,3 . 0,42 . 25 = 3,15 kN

$\Sigma g = 35,29$ kN $\quad \Sigma q = 10$ kN

10.2.3. Reinforcement for Beams

- Reinforcement for Third Story Beams

Beam Name: B305 – Span **b**: 0,3 m **h**: 0,6 m **M**: 88,333 kNm

K: $\frac{b\times(d^2)}{M}$ = 106,5061 **k_s** = 2,86 (is read from reinforced concrete tables)

A_s= ($k_s \times M$) / d = 451,13 mm^2

Reinforcement: 3Φ16 (603 mm^2)

Beam Name	b	h	M	K	ks	As	Reinforcement	Hangers	Additional
B305-span	0,3	0,6	88,333	106,5061	2,86	451,13	3Φ16 (603 mm^2)		
B305-support	0,3	0,6	135,455	69,4548	2,89	699,04		2Φ12(113mm^2	3Φ16(603mm^2)
B306-span	0,3	0,6	70,691	133,0862	2,84	358,50	2Φ16 (402 mm^2)		
B306-support	0,3	0,6	127,942	73,53332	2,89	660,27		2Φ12(113mm^2	3Φ16(603mm^2)
B307-span	0,3	0,6	119,645	78,63262	2,8	598,23	3Φ16 (603 mm^2)		
B307-support	0,3	0,6	192,231	48,94112	2,95	1012,65		2Φ12(113mm^2	5Φ16(1005mm^2)
B308-span	0,3	0,6	80,245	117,2409	2,85	408,39	3Φ16(603 mm^2)		
B308-support	0,3	0,6	150,045	62,70119	2,91	779,70		2Φ12(113mm^2	4Φ16(806mm^2)
B309-span	0,3	0,6	88,333	106,5061	2,86	451,13	3Φ16 (603 mm^2)		
B309-support	0,3	0,6	135,455	69,4548	2,89	699,04		2Φ12(113mm^2	3Φ16(603mm^2)
B310-span	0,3	0,6	70,691	133,0862	2,84	358,50	3Φ16(603mm^2)		
B310-support	0,3	0,6	127,942	73,53332	2,89	660,27		2Φ12(113mm^2	3Φ16(603mm^2)
B317-span	0,3	0,6	88,49	106,3171	2,86	451,93	3Φ16(603mm^2)		
B317-support	0,3	0,6	139,309	67,53333	2,88	716,45		2Φ12(113mm^2	3Φ16(603mm^2)
B318-span	0,3	0,6	69,04	136,2688	2,84	350,13	2Φ16(402mm^2)		
B318-support	0,3	0,6	139,309	67,53333	2,9	721,42		2Φ12(113mm^2	3Φ16(603mm^2)
B319-span	0,3	0,6	158,873	59,21711	2,91	825,57	5Φ16 (1005 mm^2)		
B319-support	0,3	0,6	155,52	60,49383	2,91	808,15		2Φ12(113mm^2	4Φ16(804mm^2)
B320-span	0,3	0,6	86,804	108,3821	2,86	443,32	3Φ16 (603 mm^2)		
B320-support	0,3	0,6	184,684	50,94107	2,94	969,59		2Φ12(113mm^2	5Φ16(1005mm^2)

- **Reinforcement for Second Story Beams**

Beam Name	b	h	M	K	ks	As	Reinforcement	Hangers	Additional
B201-span	0,3	0,6	104,278	90,22037	2,87	534,4248	3Φ16 (603 mm^2)		
B201-support	0,3	0,6	178,44	52,7236	2,93	933,6236		2Φ12(113mm^2	5Φ16(1005mm^2)
B202-span	0,3	0,6	91,036	103,3437	2,86	464,9339	3Φ16 (603 mm^2)		
B202-support	0,3	0,6	171,79	54,76454	2,93	898,8298		2Φ12(113mm^2	5Φ16(1005mm^2)
B203-span	0,3	0,6	102,702	91,60484	2,87	526,3478	3Φ16 (603 mm^2)		
B203-support	0,3	0,6	156,67	60,04979	2,91	814,1245		2Φ12(113mm^2	4Φ16(806mm^2)
B204-span	0,3	0,6	69,57	135,2307	2,84	352,8193	2Φ16(402mm^2)		
B204-support	0,3	0,6	150,045	62,70119	2,91	779,6981		2Φ12(113mm^2	4Φ16(806mm^2)
B205-span	0,3	0,6	102,39	91,88397	2,87	524,7488	3Φ16 (603 mm^2)		
B205-support	0,3	0,6	182,558	51,53431	2,93	955,1695		2Φ12(113mm^2	5Φ16(1005mm^2)
B206-span	0,3	0,6	125,61	74,8985	2,89	648,2373	4Φ16(804mm^2)		
B206-support	0,3	0,6	172,79	54,4476	2,93	904,062		2Φ12(113mm^2	5Φ16(1005mm^2)
B207-span	0,3	0,6	207,39	45,36381	2,96	1096,204	6Φ16(1206mm^2)		
B207-support	0,3	0,6	182,588	51,52584	2,93	955,3265		2Φ12(113mm^2	5Φ16(1005mm^2)
B208-span	0,3	0,6	125,61	74,8985	2,89	648,2373	4Φ16(804mm^2)		
B208-support	0,3	0,6	172,79	54,4476	2,93	904,062		2Φ12(113mm^2	5Φ16(1005mm^2)
B209-span	0,3	0,6	104,208	90,28098	2,87	534,066	3Φ16 (603 mm^2)		
B209-support	0,3	0,6	202,558	46,44596	2,93	1059,812		2Φ12(113mm^2	4Φ16(806mm^2)
B210-span	0,3	0,6	90,827	103,5815	2,86	463,8665	3Φ16 (603 mm^2)		
B210-support	0,3	0,6	174,26	53,98829	2,93	911,7532		2Φ12(113mm^2	4Φ16(806mm^2)
B211-span	0,3	0,6	226,515	41,53367	2,98	1205,383	6Φ16(1206mm^2)		
B211-support	0,3	0,6	187,82	50,09051	2,94	986,055		2Φ12(113mm^2	5Φ16(1005mm^2)
B212-span	0,3	0,6	85,015	110,6628	2,86	434,1838	3Φ16 (603 mm^2)		
B212-support	0,3	0,6	180,23	52,19997	2,93	942,9891		2Φ12(113mm^2	5Φ16(1005mm^2)
B213-span	0,3	0,6	60	156,8	2,83	303,2143	2Φ16(402mm^2)		
B213-support	0,3	0,6	131,04	71,79487	2,89	676,26		2Φ12(113mm^2	5Φ16(1005mm^2)
B217-span	0,3	0,6	89,522	105,0915	2,86	457,2016	3Φ16 (603 mm^2)		
B217-support	0,3	0,6	179,29	52,47365	2,93	938,0709		2Φ12(113mm^2	5Φ16(1005mm^2)
B218-span	0,3	0,6	85,39	110,1768	2,86	436,0989	3Φ16 (603 mm^2)		
B218-support	0,3	0,6	185,64	50,67873	2,93	971,295		2Φ12(113mm^2	5Φ16(1005mm^2)
B214-span	0,3	0,6	93,35	100,782	2,86	476,7518	3Φ16 (603 mm^2)		
B214-support	0,3	0,6	157,84	59,60466	2,91	820,2043		2Φ12(113mm^2	5Φ16(1005mm^2)
B215-span	0,3	0,6	69,76	134,8624	2,84	353,7829	2Φ16(402mm^2)		
B215-support	0,3	0,6	147,26	63,887	2,9	762,5964		2Φ12(113mm^2	4Φ16(806mm^2)
B216-span	0,3	0,6	72,19	130,3228	2,84	366,1064	2Φ16(402mm^2)		
B216-support	0,3	0,6	143,34	65,63416	2,9	742,2964		2Φ12(113mm^2	4Φ16(806mm^2)
B219-span	0,3	0,6	101,93	92,29864	2,87	522,3913	3Φ16 (603 mm^2)		
B219-support	0,3	0,6	192,07	48,98214	2,95	1011,797		2Φ12(113mm^2	5Φ16(1005mm^2)
B220-span	0,3	0,6	95,294	98,72605	2,86	486,6801	3Φ16(603mm^2)		
B220-support	0,3	0,6	210,64	44,66388	2,95	1109,621		2Φ12(113mm^2	5Φ16(1005mm^2)

- **Reinforcement for First Story Beams**

Beam Name	b	h	M	K	ks	As	Reinforcement	Hangers	Additional
B101-span	0,3	0,6	100,666	93,45757257	2,87	515,9133	3Φ16 (603 mm^2)		
B101-support	0,3	0,6	184,84	50,89807401	2,93	967,1093		2Φ12(113mm^2	5Φ16(1005mm^2)
B102-span	0,3	0,6	130,742	71,95851371	2,88	672,3874	4Φ16(804mm^2)		
B102-support	0,3	0,6	171,91	54,72631028	2,93	899,4577		2Φ12(113mm^2	5Φ16(1005mm^2)
B103-span	0,3	0,6	116,62	80,67226891	2,88	599,76	3Φ16 (603 mm^2)		
B103-support	0,3	0,6	211,39	44,50541653	2,96	1117,347		2Φ12(113mm^2	5Φ16(1005mm^2)
B104-span	0,3	0,6	94,13	99,94688197	2,86	480,7354	3Φ16 (603 mm^2)		
B104-support	0,3	0,6	200,35	46,95782381	2,95	1055,415		2Φ12(113mm^2	5Φ16(1005mm^2)
B105-span	0,3	0,6	116,62	80,67226891	2,88	599,76	3Φ16 (603 mm^2)		
B105-support	0,3	0,6	211,39	44,50541653	2,96	1117,347		2Φ12(113mm^2	5Φ16(1005mm^2)
B106-span	0,3	0,6	94,13	99,94688197	2,86	480,7354	3Φ16 (603 mm^2)		
B106-support	0,3	0,6	200,35	46,95782381	2,95	1055,415		2Φ12(113mm^2	5Φ16(1005mm^2)
B107-span	0,3	0,6	116,62	80,67226891	2,88	599,76	3Φ16 (603 mm^2)		
B107-support	0,3	0,6	211,39	44,50541653	2,96	1117,347		2Φ12(113mm^2	5Φ16(1005mm^2)
B108-span	0,3	0,6	94,13	99,94688197	2,86	480,7354	3Φ16 (603 mm^2)		
B108-support	0,3	0,6	200,35	46,95782381	2,95	1055,415		2Φ12(113mm^2	5Φ16(1005mm^2)
B109-span	0,3	0,6	100,66	93,46314325	2,87	515,8825	3Φ16 (603 mm^2)		
B109-support	0,3	0,6	184,83	50,90082779	2,94	970,3575		2Φ12(113mm^2	5Φ16(1005mm^2)
B110-span	0,3	0,6	130,74	71,9596145	2,89	674,7118	4Φ16(804mm^2)		
B110-support	0,3	0,6	178,33	52,75612628	2,93	933,048		2Φ12(113mm^2	5Φ16(1005mm^2)
B111-span	0,3	0,6	100,68	93,44457688	2,87	515,985	3Φ16 (603 mm^2)		
B111-support	0,3	0,6	185	50,85405405	2,94	971,25		2Φ12(113mm^2	5Φ16(1005mm^2)
B112-span	0,3	0,6	85,67	109,8167386	2,86	437,5289	3Φ16 (603 mm^2)		
B112-support	0,3	0,6	178,85	52,60273973	2,93	935,7688		2Φ12(113mm^2	5Φ16(1005mm^2)
B113-span	0,3	0,6	51,73	181,8673884	2,82	260,4975	2Φ16(402mm^2)		
B113-support	0,3	0,6	144,41	65,14784295	2,9	747,8375		2Φ12(113mm^2	4Φ16(806mm^2)
B117-span	0,3	0,6	90,87	103,532519	2,86	464,0861	3Φ16 (603 mm^2)		
B117-support	0,3	0,6	173,88	54,10628019	2,93	909,765		2Φ12(113mm^2	4Φ16(806mm^2)
B118-span	0,3	0,6	85,44	110,1123596	2,86	436,3543	3Φ16 (603 mm^2)		
B118-support	0,3	0,6	185	50,85405405	2,94	971,25		2Φ12(113mm^2	5Φ16(1005mm^2)
B114-span	0,3	0,6	116,45	80,79003864	2,88	598,8857	3Φ16 (603 mm^2)		
B114-support	0,3	0,6	209,23	44,96487119	2,96	1105,93		2Φ12(113mm^2	5Φ16(1005mm^2)
B115-span	0,3	0,6	97,26	96,73041333	2,87	498,4575	3Φ16 (603 mm^2)		
B115-support	0,3	0,6	198,07	47,49835917	2,95	1043,404		2Φ12(113mm^2	5Φ16(1005mm^2)
B116-span	0,3	0,6	93,93	100,1596934	2,86	479,7139	3Φ16 (603 mm^2)		
B116-support	0,3	0,6	199,47	47,16498722	2,95	1050,779		2Φ12(113mm^2	5Φ16(1005mm^2)
B119-span	0,3	0,6	95,77	98,23535554	2,86	489,1111	3Φ16 (603 mm^2)		
B119-support	0,3	0,6	198,92	47,29539513	2,95	1047,882		2Φ12(113mm^2	5Φ16(1005mm^2)
B120-span	0,3	0,6	95,67	98,338037	2,86	488,6004	3Φ16 (603 mm^2)		
B120-support	0,3	0,6	209,29	44,95198051	2,96	1106,247		2Φ12(113mm^2	5Φ16(1005mm^2)

10.3. Slab Design

10.3.1. Slab Moments

			X Direction				Y Direction			
			support		Span		support		Span	
Slab Name	m	Pd*Lsn^2	a	M	a	M	a	M	a	M
S301	1	554,44	0,042	23,29	0,031	17,19	0,042	23,29	0,031	17,19
S302	1	554,44	0,049	27,17	0,037	20,51	0,049	27,17	0,037	20,51
S303	1	554,44	0,033	18,30	0,025	13,86	0,033	18,30	0,025	13,86
S304	1	554,44	0,042	23,29	0,031	17,19	0,042	23,29	0,031	17,19
S201	1	529,20	0,042	22,23	0,031	16,41	0,042	22,23	0,031	16,41
S202	1	529,20	0,049	25,93	0,037	19,58	0,049	25,93	0,037	19,58
S203	1	529,20	0,033	17,46	0,025	13,23	0,033	17,46	0,025	13,23
S204	1	529,20	0,042	22,23	0,031	16,41	0,042	22,23	0,031	16,41
S205	1	529,20	0,042	22,23	0,031	16,41	0,042	22,23	0,031	16,41
S206	1	529,20	0,033	17,46	0,025	13,23	0,033	17,46	0,025	13,23
S101	1	529,20	0,042	22,23	0,031	16,41	0,042	22,23	0,031	16,41
S102	1	529,20	0,049	25,93	0,037	19,58	0,049	25,93	0,037	19,58
S103	1	529,20	0,033	17,46	0,025	13,23	0,033	17,46	0,025	13,23
S104	1	529,20	0,042	22,23	0,031	16,41	0,042	22,23	0,031	16,41
S105	1	529,20	0,042	22,23	0,031	16,41	0,042	22,23	0,031	16,41
S106	1	529,20	0,033	17,46	0,025	13,23	0,033	17,46	0,025	13,23

10.3.2. Reinforcement for Slab

- **for Span:** **d':** 20 mm **h:**180 mm **d:** 160 mm and **d:**150 mm C30/S420

Slab Name: S301

Direction: x

Moment: 17,19 kNm

K: $148,92 \times 10^{-5}$

k_s: 2,83

A_s= ($k_s \times M$) / d = 304,05 mm^2

Reinforcement: 10 φ 200 (10 φ 400 – Longditunal / 10 φ 400 – Bent up)

Slab Name	Direction	Moment kNm/m	d (m)	K (10^-5)	ks	As (mm^2/mm)	Reinforcement	Longitudinal	Bent-up
S301	X	17,19	0,16	148,9238	2,83	304,05	ϕ10/200	ϕ10/400	ϕ10/400
S301	Y	17,19	0,15	130,8901	2,84	325,46	ϕ10/200	ϕ10/400	ϕ10/400
S302	X	20,51	0,16	124,8172	2,85	365,33	ϕ10/200	ϕ10/400	ϕ10/400
S302	Y	20,51	0,15	109,7026	2,86	391,06	ϕ10/200	ϕ10/400	ϕ10/400
S303	X	13,86	0,16	184,7042	2,82	244,28	ϕ10/200	ϕ10/400	ϕ10/400

S303	Y	13,86	0,15	162,3377	2,83	261,49	ϕ10/200	ϕ10/400	ϕ10/400
S304	X	17,19	0,16	148,9238	2,83	304,05	ϕ10/200	ϕ10/400	ϕ10/400
S304	Y	17,19	0,15	130,8901	2,84	325,46	ϕ10/200	ϕ10/400	ϕ10/400
S201	X	16,41	0,16	156,0024	2,83	290,25	ϕ10/200	ϕ10/400	ϕ10/400
S201	Y	16,41	0,15	137,1115	2,84	310,70	ϕ10/200	ϕ10/400	ϕ10/400
S202	X	19,58	0,16	130,7457	2,84	347,55	ϕ10/200	ϕ10/400	ϕ10/400
S202	Y	19,58	0,15	114,9132	2,85	372,02	ϕ10/200	ϕ10/400	ϕ10/400
S203	X	13,23	0,16	193,4996	2,82	233,18	ϕ10/200	ϕ10/400	ϕ10/400
S203	Y	13,23	0,15	170,068	2,83	249,61	ϕ10/200	ϕ10/400	ϕ10/400
S204	X	16,41	0,16	156,0024	2,83	290,25	ϕ10/200	ϕ10/400	ϕ10/400
S204	Y	16,41	0,15	137,1115	2,84	310,70	ϕ10/200	ϕ10/400	ϕ10/400
S205	X	16,41	0,16	156,0024	2,83	290,25	ϕ10/200	ϕ10/400	ϕ10/400
S205	Y	16,41	0,15	137,1115	2,84	310,70	ϕ10/200	ϕ10/400	ϕ10/400
S206	X	13,23	0,16	193,4996	2,82	233,18	ϕ10/200	ϕ10/400	ϕ10/400
S206	Y	13,23	0,15	170,068	2,83	249,61	ϕ10/200	ϕ10/400	ϕ10/400
S101	X	16,41	0,16	156,0024	2,84	291,28	ϕ10/200	ϕ10/400	ϕ10/400
S101	Y	16,41	0,15	137,1115	2,84	310,70	ϕ10/200	ϕ10/400	ϕ10/400
S102	X	19,58	0,16	130,7457	2,84	347,55	ϕ10/200	ϕ10/400	ϕ10/400
S102	Y	19,58	0,15	114,9132	2,85	372,02	ϕ10/200	ϕ10/400	ϕ10/400
S103	X	13,23	0,16	193,4996	2,82	233,18	ϕ10/200	ϕ10/400	ϕ10/400
S103	Y	13,23	0,15	170,068	2,83	249,61	ϕ10/200	ϕ10/400	ϕ10/400
S104	X	16,41	0,16	156,0024	2,83	290,25	ϕ10/200	ϕ10/400	ϕ10/400
S104	Y	16,41	0,15	137,1115	2,84	310,70	ϕ10/200	ϕ10/400	ϕ10/400
S105	X	16,41	0,16	156,0024	2,83	290,25	ϕ10/200	ϕ10/400	ϕ10/400
S105	Y	16,41	0,15	137,1115	2,84	310,70	ϕ10/200	ϕ10/400	ϕ10/400
S106	X	13,23	0,16	193,4996	2,82	233,18	ϕ10/200	ϕ10/400	ϕ10/400
S106	Y	13,23	0,15	170,068	2,83	249,61	ϕ10/200	ϕ10/400	ϕ10/400

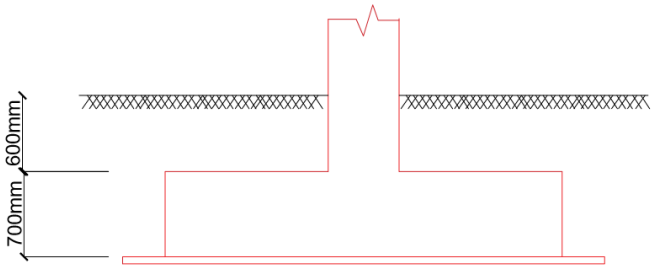
- **For Support: d': 20 mm h:180 mm d: 160 mm C30/S420**

Slab Name	Moment kNm/m	d (m)	K (10^-5)	ks	As (mm^2/mm)	Existing Reinforcement	Additional Reinforcement
S301-S302	27,17	0,16	94,221568	2,87	487,36	ϕ10/400+ϕ10/400	ϕ10/330
S301-S303	21,63	0,16	118,35414	2,85	385,28	ϕ10/400+ϕ10/400	-
S302-S304	27,17	0,16	94,221568	2,87	487,36	ϕ10/400+ϕ10/400	ϕ10/330
S303-S304	21,63	0,16	118,35414	2,85	385,28	ϕ10/400+ϕ10/400	-
S201-S202	25,98	0,16	98,537336	2,87	466,02	ϕ10/400+ϕ10/400	ϕ10/330
S201-S203	20,64	0,16	124,03101	2,85	367,65	ϕ10/400+ϕ10/400	-
S201-S205	22,23	0,16	115,15969	2,85	395,97	ϕ10/400+ϕ10/400	ϕ10/330
S202-S204	25,93	0,16	98,727343	2,87	465,12	ϕ10/400+ϕ10/400	ϕ10/330
S203-S204	20,64	0,16	124,03101	2,85	367,65	ϕ10/400+ϕ10/400	-

S203-S206	17,46	0,16	146,62085	2,84	309,92	ϕ10/400+ϕ10/400	-
S205-S206	20,64	0,16	124,03101	2,85	367,65	ϕ10/400+ϕ10/400	-
S101-S102	25,98	0,16	98,537336	2,87	466,02	ϕ10/400+ϕ10/400	ϕ10/330
S101-S103	20,64	0,16	124,03101	2,85	367,65	ϕ10/400+ϕ10/400	-
S101-S105	22,23	0,16	115,15969	2,85	395,97	ϕ10/400+ϕ10/400	ϕ10/330
S102-S104	25,93	0,16	98,727343	2,87	465,12	ϕ10/400+ϕ10/400	ϕ10/330
S103-S104	20,64	0,16	124,03101	2,85	367,65	ϕ10/400+ϕ10/400	-
S103-S106	17,46	0,16	146,62085	2,84	309,92	ϕ10/400+ϕ10/400	-
S105-S106	20,64	0,16	124,03101	2,85	367,65	ϕ10/400+ϕ10/400	-

10.4.Foundation Dimensioning and RC Design

Before starting to dimension spread foundation depth of foundation is assumed as h=0,70m.



All 1. Story colums are 70*70 therefore spread foundations are selected as sqare shape. Considering an approximate unit weight for the composition of RC and the soil above the footer sections of the footing as:

γ_{soil}= 20 kN/m^3

$\sigma_{z,net}=\sigma_{all}-\gamma*t$=180-20*1,3=154 kN/m^3 t: h+freezen depth= 0,60+0,70= 1,30 m

For the selection of footing dimensions and the bearing stress control, employed non-factored load combinations (Service Loads) and calculated the soil stresses as given below:

$\sigma_{1,2,3,4}=\frac{N_0}{A}\pm\frac{M_x}{Wx}\pm\frac{M_y}{Wy}$ must be less than 154 kN/m^3 so, dimensions of spread foundations are selected.

			Y-Direction									
	C105	C107	C102	C110	C101	C109	C106	C108	C103	C104	C111	C102
T(kN)	23,85	7,47	27,82	28,88	23,86	7,47	23,86	7,47	27,82	28,88	28,88	27,82
M(kN)	36,58	11,47	42,56	44,29	36,58	11,45	36,58	11,45	42,56	44,29	44,29	42,56

N(kN)	1746,21	1698,55	1428,40	1973,78	1024,75	1231,35	1158,96	1582,00	1626,02	2275,04	1973,78	1428,40
Mo(kN)	53,28	16,69	62,04	64,51	53,28	16,67	53,28	16,67	62,04	64,51	64,51	62,04
b(m)	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70
h(m)	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70	0,70

X-Direction

	C105	C107	C102	C110	C101	C109	C106	C108	C103	C104	C111	C102'
T(kN)	5,39	-4,15	5,39	-1,67	23,92	25,74	23,47	17,16	-7,47	-3,64	3,64	6,59
M(kN)	8,27	-6,37	8,27	-2,56	55,02	39,48	36,00	39,47	-11,46	-8,37	8,37	-10,11
N(kN)	1746,21	1698,55	1428,40	1973,78	1024,75	1231,35	1158,96	1582,00	1626,02	2275,04	1973,78	1428,40
Mo(kN)	12,04	-9,28	12,04	-3,73	71,76	57,50	52,43	51,48	-16,69	-10,92	10,92	-5,50

Y Direction G+Q

	C105	C107	C102	C110	C101	C109	C106	C108	C103	C104	C111	C102
T(kN)	15,90	4,98	18,55	19,26	15,91	4,98	15,91	4,98	18,55	19,26	19,26	18,55
M(kN)	24,39	7,64	28,38	29,53	24,39	7,63	24,39	7,63	28,38	29,53	29,53	28,38
N(kN)	1164,14	1132,37	952,27	1315,85	683,17	820,90	772,64	1054,67	1084,01	1516,69	1315,85	952,27
Mo(kN)	35,52	11,13	41,36	43,01	35,52	11,12	35,52	11,12	41,36	43,01	43,01	41,36
Lx(m)	2,90	2,80	2,70	3,10	2,50	2,50	2,50	2,80	2,80	3,25	3,10	2,80

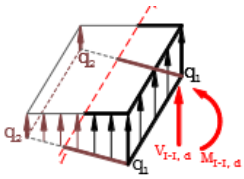
X Direction G+Q

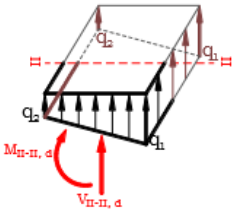
	C105	C107	C102	C110	C101	C109	C106	C108	C103	C104	C111	C102'
T(kN)	3,59	-2,77	3,59	-1,11	15,95	17,16	15,65	11,44	-4,98	-2,43	2,43	4,39
M(kN)	5,51	-4,25	5,51	-1,71	36,68	26,32	24,00	26,31	-7,64	-5,58	5,58	-6,74
N(kN)	1164,14	1132,37	952,27	1315,85	683,17	820,90	772,64	1054,67	1084,01	1516,69	1315,85	952,27
Mo(kN)	8,03	-6,18	8,03	-2,49	47,84	38,33	34,95	34,32	-11,13	-7,28	7,28	-3,66

Soil Streses

	C105	C107	C102	C110	C101	C109	C106	C108	C103	C104	C111	C102
Sigma1	149,14	145,79	145,68	145,09	141,32	150,33	150,68	146,94	146,53	149,84	147,05	131,77
Sigma2	131,66	139,70	120,47	127,76	114,04	141,79	123,40	140,87	123,92	134,80	129,73	109,16
Sigma3	145,19	149,17	140,79	146,09	104,58	120,89	123,84	128,18	152,61	152,38	144,12	133,77
Sigma4	127,71	143,08	115,57	128,76	77,29	112,36	96,56	122,10	130,00	137,35	126,80	111,16

After choosing dimensions, soil stresses q1,q2, q3 and q4 are calculated with employing design loads(1,4G+1,6Q).





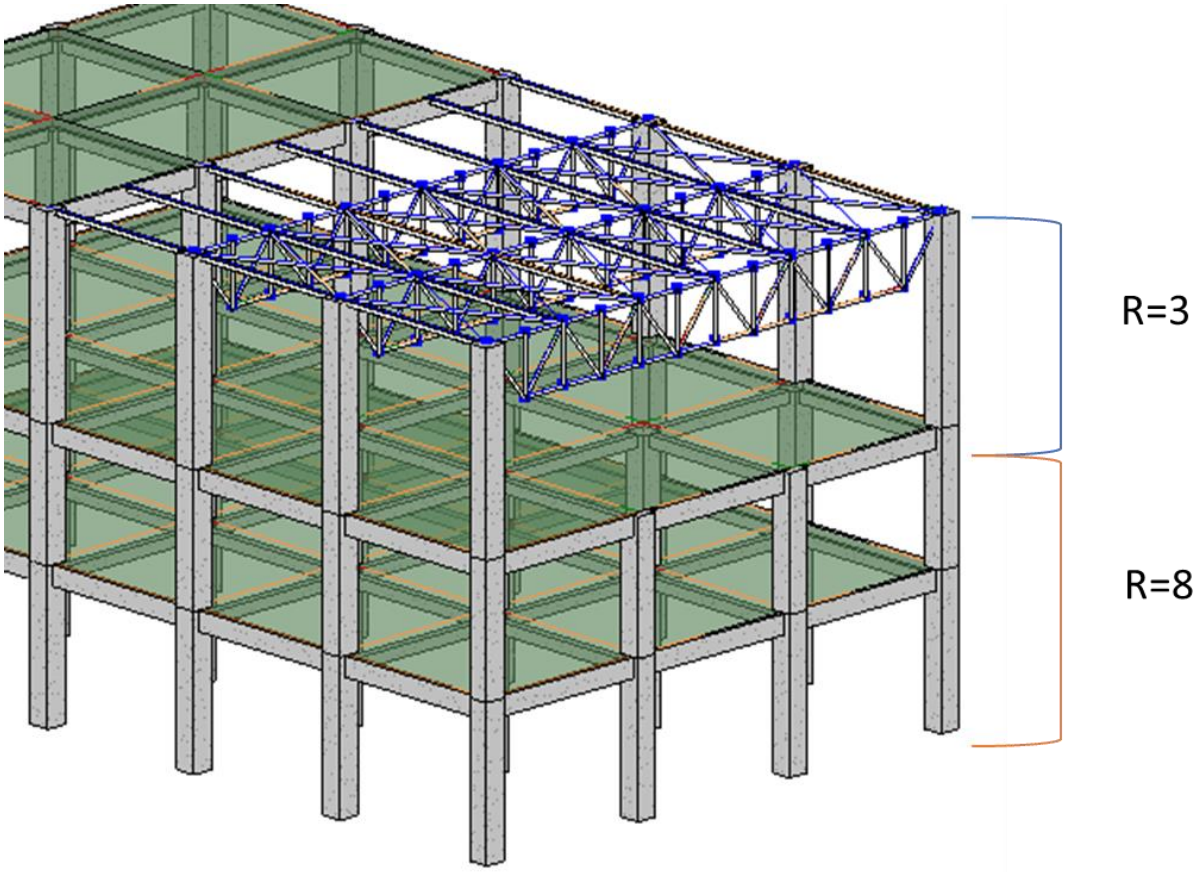
In the critical sections, bending moments and shear forces are calculated. After that according to bending moments, bars are choosen.

10.5. PRELIMINARY DESIGN DIMENSIONS

Member	Dimensions
COLUMN	70x70 cm
BEAM	6030 cm
SLAB	Thickness=18 cm
FOUNDATION	Thickness=70cm, max 3.25x3.25m and min 2.10x2.10
STEEL ROOF	No Proportioning

11. CANTILEVER COLUMN DESİGN

- **Response Modification Coefficient Factor (R)**



The required axial strength of individual cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15 percent of the available axial strength, including slenderness effects.

Cantilever columns; loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity)

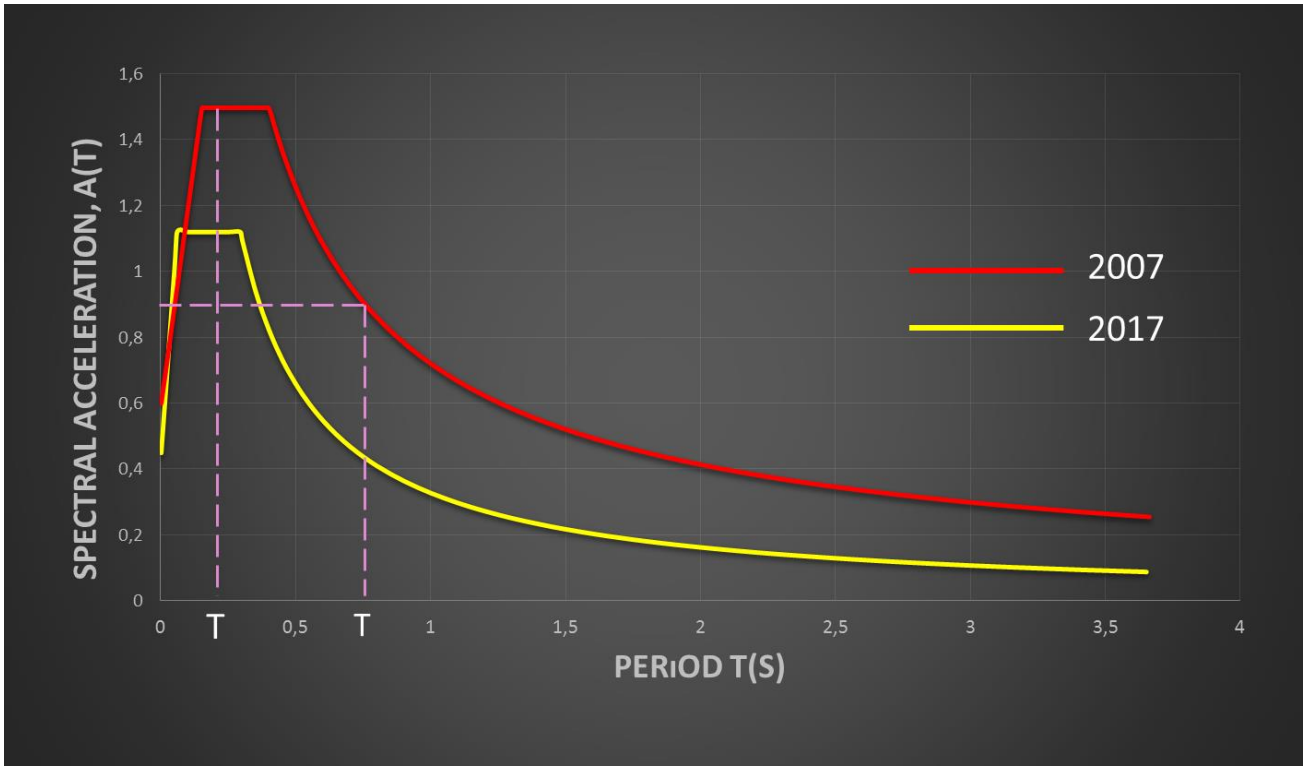
Tablo 4.1. Bina Taşıyıcı Sistemleri için Taşıyıcı Sistem Davranış Katsayısı, Dayanım Fazlalığı Katsayısı ve İzin Verilen Bina Yükseklik Sınıfları

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
A12. Deprem etkilerinin tamamının *süneklik düzeyi yüksek* bağ kirişli (boşluklu) betonarme perdelerle karşılandığı binalar	7	2.5	BYS ≥ 2
A13. Deprem etkilerinin tamamının *süneklik düzeyi yüksek* boşluksuz betonarme perdelerle karşılandığı binalar	6	2.5	BYS ≥ 2
A14. Deprem etkilerinin moment aktaran *süneklik düzeyi yüksek* betonarme çerçeveler ile *süneklik düzeyi yüksek* bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.**4.3.4.5**)	8	2.5	BYS ≥ 2
A15. Deprem etkilerinin moment aktaran *süneklik düzeyi yüksek* betonarme çerçeveler ile *süneklik düzeyi yüksek* boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.**4.3.4.5**)	7	2.5	BYS ≥ 2
A16. Deprem etkilerinin tamamının çatı düzeyindeki bağlantıları mafsallı olan ve yüksekliği 12 m'yi geçmeyen *süneklik düzeyi yüksek* betonarme kolonlar tarafından karşılandığı tek katlı binalar	3	2	–

- Response Modification Coefficient Factor (R) = 3
- Overstrengh Factor $\Omega_0 = 3$

For the frame section with high ductility level R value is 8 and the cantilever section R value is 3. Therefore for all system we choose R value as 6 and take overstrenght factor as 3 for designing cantilever columns.

12. EQ FORCE CALCULATION



$A(T)=A_0 \times I \times S(T)$

$Sae(T)=A(T) \times S(T)$

According to:

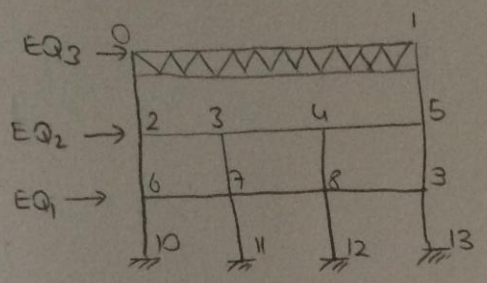
- Z2 soil class (Ta=0,15s Tb=0,40s)
- Importance Factor I1=1,5
- Eq zone A_0=0,40

the Sae(T)-Periot graph is obtained shown as above.

For the EQ force calculation we determined the mass of the building. After that we obtained the total EQ force that affect building on EQ. Then, the total EQ force distributed floors according to mass distribution of floors. For the safe design, we took period as T=0,2 s. Sae(T) is 1,5 considering the obtained period. For this way, we faced with larger EQ forces than the real. Total force is V(t)=6342.2375 kN.

Earthquake Forces

A-A Axis



$EQ_3 = 193,41\,kN$

$EQ_2 = 360,20\,kN$

$EQ_1 = 178,07\,kN$

For EQ3

(0,1) → column : 3 . 0,7 . 0,7 . 25 = 36,75 kN

E. wall : 9 . 3 . 4,5 = 121,5 kN

S. Roof : 9 . 3 . 1 = 27 kN

Snow : 9 . 3 . 0,75 = 20,25 kN

$\Sigma G = 185,25\,kN$ $\Sigma W = 185,25 + 0,3 . 20,25 = 191,325\,kN$

(0,1) → $0,04185\,EQ_3 = 8,09$

at truss → $0,0833\,EQ_3 = 16,11\,kN$

For EQ2

(2,5) → column = (3+2) . 0,7 . 0,7 . 25 = 61,25 kN

E wall = 6 . (3+2) . 4,5 = 135 kN

Slab = 3 . 3 . 5,92 = 53,28 kN

Beam = (3+3) . 0,3 . 0,6 . 25 = 27 kN

$\Sigma G = 276,53\,kN$ $\Sigma W = 276,53 + 0,3 . 45 = 290,03\,kN$

(3,4) → column = 2 . 0,7 . 0,7 . 25 = 24,5 kN Live = 6 . 3 . 5 = 90 kN

Beam = (6+3) . 0,3 . 0,6 . 25 = 40,5 kN

E. wall = 6 . (3+2) . 4,5 = 135 kN

I. wall = 3 . (3+2) . 2,5 = 37,5 kN

Slab = 6 . 3 . 5,92 = 106,56 kN

$\Sigma G = 344,06\,kN$ $\Sigma W = 344,06 + 0,3 . 90 = 371,06\,kN$

(2.5) → 290,03 kN → $0,219\,EQ_2 = 78,88\,kN$

(3.4) → 371,06 kN → $0,281\,EQ_2 = 101,22\,kN$

for EQ1

(6,9) → column = (2+2,3) . 0,7 . 0,7 . 25 = 52,68 kN

Beam = (3+3) . 0,3 . 0,6 . 25 = 27 kN

Slab = 3 . 3 . 5,32 = 53,28 kN

E.Wall = (2+2,3) . (3+3) . 4,5 = 116,1 kN

ΣG = 249,06

Live = 3 . 3 . 5 = 45 kN

SW = 249,06 + 0,3 . 45 = 262,56 kN

(7,8) → column = 52,68 kN

Beam = (6+3) . 0,3 . 0,6 . 25 = 40,5 kN

Slab = 6 . 3 . 5,32 = 79,32 kN

E.Wall = (2+2,3) . 6 = 116,1 kN

I.Wall = (2+2,3) . 3 . 2,5 = 32,25 kN

ΣG = 321,45 kN

SW = 321,45 + 0,3 . 90 = 348,45 kN

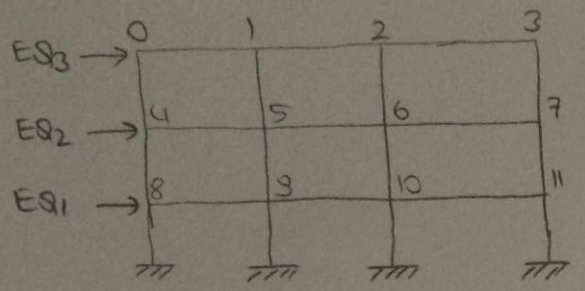
Live Load = 6 . 3 . 5 = 90 kN

(6,9) → 262,56 kN → 0,215 EQ1

(7,8) → 348,45 kN → 0,285 EQ1

* Earthquake forces in B-B Axis are same value of A-A Axis

D-D Axis



$EQ_3 = 462,96 kN$

$EQ_2 = 445,55 kN$

$EQ_1 = 221,05 kN$

for EQ3

(0,3) → column = 3 0,7 0,7 . 25 = 36,75 kN

snow = 0,75 . 6 . 3 =

E.wall = (3-0,6) . 6 . 4,5 =

I. wall = (3-0,6) . 3 . 2,5 =

Live Load = 3 3 5 . 0,3 =

Slab = 3 . 3 . 6,475 =

Roof = 3 . 3 . 1 = 9 kN

$\Sigma W = 213,825 kN$

(1,2) → column = 3 . 0,7 . 0,7 . 25 = 36,75 kN

slab = 6 . 3 . 6,475 =

Roof = 3 6 . 1 = 18 kN

snow = 0,75 . 6 6 = (

Live Load = 6 . 3 . 5 . 0,3 =

I. wall = (3-0,6) 12 . 2,5 =

$\Sigma W = 297,3 kN$

(0,3) → 0,2 EQ_1 = 92,6 kN

(1,2) → 0,3 EQ_1 = 138,83 kN

for EQ2

(4,7) → column = 0,7 . 0,7 25 . (3+2) =

E.wall = (3+2-0,6) . 6 4,5 =

Slab = 3 . 3 . 5,32 =

Beam = (3+3) . 0,3 . 0,6 . 25 =

Live Load = 0,3 . 3 . 3 5 =

I. wall = (3+2-0,6) . 3 2,5 =

$\Sigma W = 306,83 kN$

(5-6) → column = 0,7.0,7.25.(3+2) =
I wall = (3+2-0,6).6.2,5 =
Slab = 6.6.5,92 =
Beam = (6+6).0,3.0,6.25 =
Live Load = 0,3.6.6.5 =
E.wall = (3+2-0,6).6.4,5 =

ΣW = 514,37 kN

(4,7) ⇒ 0,187 EQ2 = 83,28 kN
(5-6) ⇒ 0,313 EQ2 = 139,39 kN

for EQ1

(8,11) → column = 25.0,7.0,7.(2+2,3) =
E.wall = 6.4,5.(2+2,3-0,6) =
Slab = 3.3.5,92 =
Beam = (3+3).0,3.0,6.25 =
Live Load = 0,3.3.3.5 =
I.wall = (2+2,3-0,6).3.2,5 =

ΣW = 374,005 kN

(9-10) → column = (2+2,3).25.0,7.0,7 =
Beam = (6+6).0,3.0,6.25 =
Live Load = 0,3.6.6.5 =
I wall = (2+2,3-0,6).6.2.2,5 =
Slab = 6.6.5,92

ΣW = 487,735 kN

(8,11) → 0,217 EQ1 = 47,37 kN
(9,10) → 0,283 EQ1 = 62,50 kN

E-E Axis

for EQ3

(0,3) → column = 3.0,7.0,7.25 =
Slab = 3.6.6.4,75 =
Snow = 0,75.6.3 =
Live Load = 6.3.5.0,3 =
Beam = (6+3).0,3.0,6.25 =
E.wall = (3-0,6).6.4,5 =
I wall = (3-0,6).3.2,5 =

ΣW = 317,1 kN

(1,2) → Column = 3 0,7.0,7.25 =
slab = 6.6.6,475 =
snow = 6.6.0,75 =
Live Load = 6.6.5.0,3 =
Beam = (6+6). 0,3.0,6.25 =
1 wall = (3-0,6). 12.25 =

$\Sigma W = 476,85$ kN

(0,3) → 0.2 EQ3 = 125,72 kN
(1,2) → 0,3 EQ3 = 188,574 kN

for EQ2
(4,7) → 77,87 kN
(5,6) → 130,34 kN

for EQ1
(8,11) → 44,83 kN
(9,10) → 58,47 kN

F.F Axis

for EQ3 (0,3) → Column = 3.0,7.0,7.25 =
slab = 3.3.6,475 =
snow = 0,75.3.3 =
Live Load = 3.3.5.0,3 =
Beam = (3+3). 0,6 0,3.25 =
E.wall = (3-0,6). 6,475 =

$\Sigma W = 207,075$ kN

(1,2) → Column = 3 0,7.0,7.25 =
Slab = 3.6. 6,475 =
snow = 0,75.6.3 =
Live Load = 6 3 5 0,3 =
Beam = (6+3). 0,6.0,3.25 =
E.wall = (3-0,6). 6 4,5 =
1 wall = (3-0,6). 3.2,5 =

$\Sigma W = 317,1$ kN

(0,3) → 0,197 EQ3 = 85,94 kN
(1,2) → 0,302 EQ3 = 131,75 kN

for EQ2
(4,7) → 0,213 EQ2 = 67,63 kN
(5,6) → 0,281 EQ2 = 86,85 kN

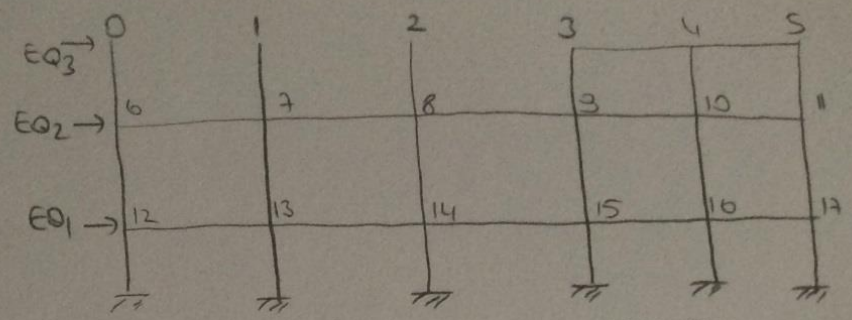
For EQ1
(8,11) → 0,215 EQ1 = 32,85 kN
(9,10) → 0,285 EQ1 = 43,55 kN

* 1. ve 2. story's values is taken from value of A-A Axis.

Y Direction EQ Loads

1-1 Axis

for EQ3



EQ3 = 537,28kN EQ2 = 505,08kN

EQ1 = 251,66kN

(0) → column = 3 0,7.0,7.25

E.Wall = (3-0,6). 3.4,5

TNSS = 3.3.1 =

Snow = 3.3.0,75

ΣW = 81,9 kN

(3) → column = 3.0,7.0,7.25 =

E.Wall = (3-0,6). 3.4,5 =

TNSS = 3.3.1 =

slab = 3.3.5,32

Snow = 0,75.3.6

Live Load = 0,3. 3.6.5

ΣW = 171,93kN

(1,2) → column = 3.0,7.0,7.25

E.Wall = (3-0,6). 3.4,5

Snow = 0,75.6.3

TNSS = 6.3.1

ΣW = 100,65kN

(4) → column = 3.0,7.0,7.25

E.Wall = (3-0,6). 3 4,5

Snow = 0,75.3.6

Live Load = 0,3. 3.6.5

ΣW = 109,65kN

(5) → column = 3.0,7.0,7.25

E.Wall = (3-0,6). 3.4,5

Snow = 0,75.3.3

Live Load = 0,3.3.3.5

ΣW = 83,5 kN

ΣW = 657,18kN

(0) → 0,13EQ3 = 69,85kN

(1,2) → 0,153EQ3 = 82,20kN

(3) → 0,261 EQ3 = 140,23kN

(4) → 0,167 EQ3 = 83,72kN

(5) → 0,136EQ3 = 73,07kN

for EQ2

(6-11) → column = (3+2). 0,7.0,7.25
E.wall = (3+2-0,6). 6. 4,5
slab = 3. 3. 5,92
Beam = (3+3). 0,3. 0,6. 25
Live Load = 0,3. 3. 3.5

$\Sigma W = 273,83 kN$

(7,8,9,10) → column = (3+2). 0,7.0,7.25
E.wall = (3+2-0,6). 6. 4,5
slab = 3.6. 5,92
Beam = (6+3). 0,3. 0,6. 25
Live Load = 0,3. 6. 3.5

$\Sigma W = 354,11 kN$

(6,11) → 0,139 EQ2 = 70,21 kN
(7,8,9,10) → 0,18 EQ2 = 90,91 kN

for EQ1

(12,17) → column = (2+2,3). 0,7.0,7.25
E.wall = (2+2,3-0,6). 6. 4,5
slab = 3. 3. 5,92
Beam = (3+3). 0,3. 0,6. 25
Live Load = 0,3. 3. 3.5

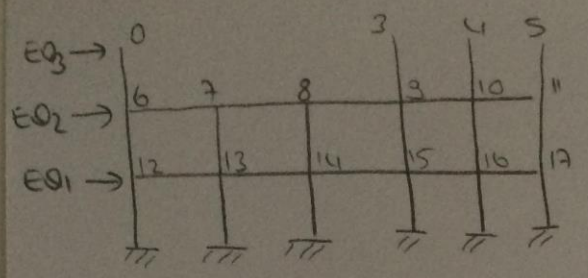
$\Sigma W = 246,355 kN$

(13,14,15,16) → column = (2+2,3). 0,7.0,7.25
E.wall = (2+2,3-0,6). 6. 4,5
slab = 3. 6. 5,92
Beam = (6+3). 0,3. 0,6. 25
Live Load = 0,3. 6. 3.5

$\Sigma W = 326,639 kN$

(12,17) → 0,137 EQ1 = 34,48 kN
(13,14,15,16) → 0,181 EQ1 = 45,55 kN

2-2 Axis



for EQ3

(3,5) → column = 36,75kN
Slab = 6 3.5,32
E.wall = 3.6.4,5
I wall = 3. 3. 2,5
Beam = (6+3). 0,3.0,6.25

Live Load = 6. 3. 5 . 0,3
Snow = 6.3. 0,75 . 0,3

ΣW = 321,06

(4) → column = 36,75 kN
Slab = 6.6.5,32
Beam = (6+6). 0,3.0,6,25
I. wall = (6+6). 3.2,5

Live = 6.6. 5
Snow = 6. 6 .0,75

ΣW = 455,97kN

(3,5) → 0,232 EQ3 = 178,02kN
(4) → 0,415 EQ3 = 253,16kN

for EQ2

(6) → column = (2+3). 0,7.0,7.25 = 61,25kN
Beam = (6+3). 0,3. 0,6 . 25 = 40,5 kN
Slab = 6. 3. 5,32 = 106,56kN
E.wall = 6. (2+3). 4,5 = 135kN
I wall = 3. (2+3). 2,5 = 37,5kN

Live = 6.3.5 = 90kN

ΣW = 407.81 kN

(7,8) → column = 2. 0,7. 0,7. 25 = 24,5kN
Beam = (6+6). 0,3. 0,6. 25 = 54kN
Slab = 6. 6. 5,32 = 213,12 kN
I. wall = (6+6). 2. 2,5 = 180kN

Live = 6.6 5 = 180kN

ΣW = 524,62kN

(9) → column = (3+2). 0,7. 0,7 25 = 61,25kN
Beam = 54kN
Slab = 213,12kN
I. wall = 112,5kN

Live Load = 6 6 5 = 180kN

ΣW = 494,87kN

(10) → column = 61,25 kN

Beam = (6+6). 0,3 0,6. 25 = 54 kN

Slab = 213,12 kN

I. wall = (6+6) (3+2). 25 = 150 kN

Live = 6. 6. 5 = 180 kN

ΣW = 532,37 kN

(11) → column = 61,25 kN

Beam = 54 kN

Slab = 213,12 kN

I. wall = 3. (3+2). 2,5 = 37,5 kN

E. wall = 6. (3+2). 4,5 = 135 kN

Live = 6. 3. 5 = 90 kN

ΣW = 527,87 kN

(6) → 0,137 EQ2 = 102,66 kN

(7,8) → 0,174 EQ2 = 132,32 kN

(9) → 0,164 EQ2 = 124,72 kN

(10) → 0,177 EQ2 = 134,60 kN

(11) → 0,175 EQ2 = 133,08 kN

for EQ1

(12,17) → column = (2+2,3). 0,7 0,7. 25 = 52,68 kN

Beam = (6+3). 0,3. 0,6. 25 = 40,5 kN

Slab = 6. 3. 5,32 = 106,56 kN

E. wall = 6,4,5. (2+2,3) = 116,1 kN

I. wall = (2+2,3). 3. 2,5 = 32,25 kN

Live Load = 6. 3. 5 = 90 kN

ΣW = 375,09 kN

(13,14,15,16) → column = 52,68 kN

Beam = 6. 6. 0,3. 25 = 81,0 kN

Slab = 6. 6. 5,32 = 213,12 kN

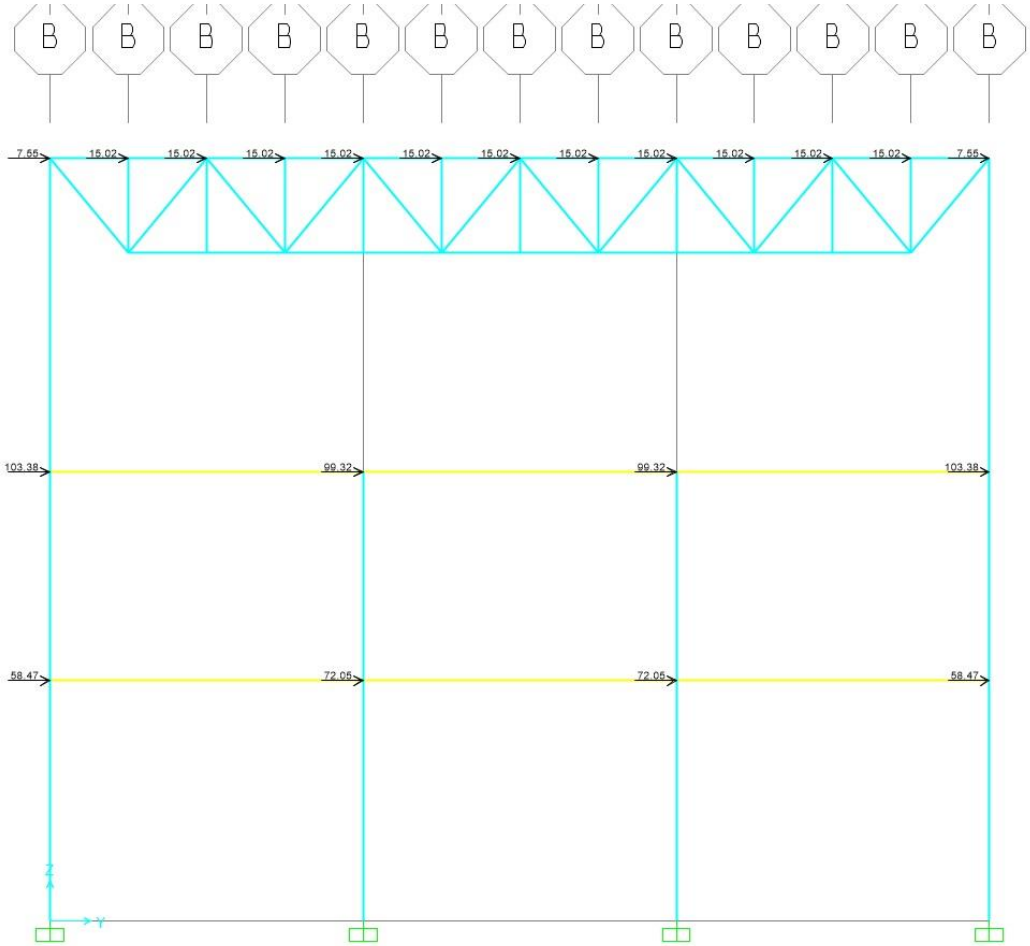
I. wall = (2+2,3). (6+6). 2,5 = 129 kN

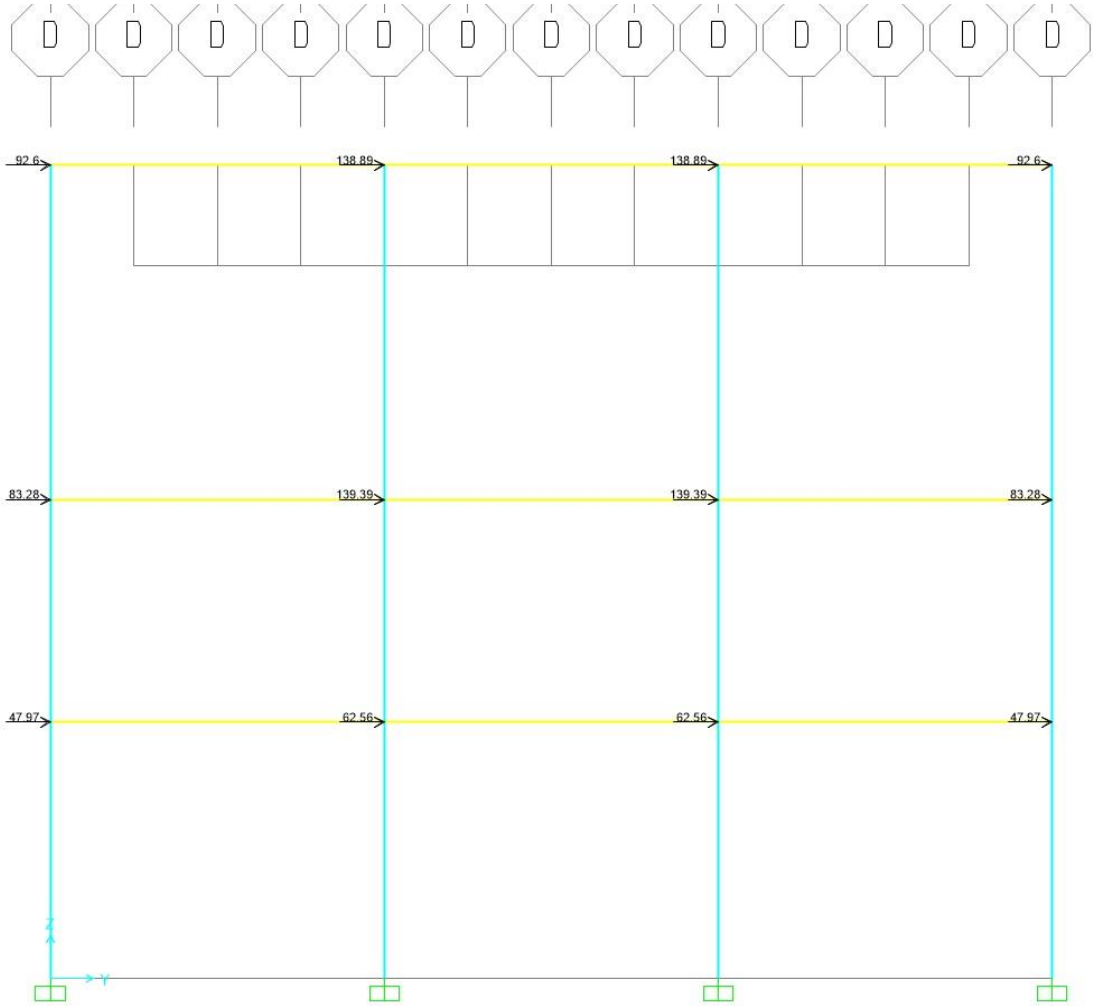
Live = 6. 6. 5 = 180 kN

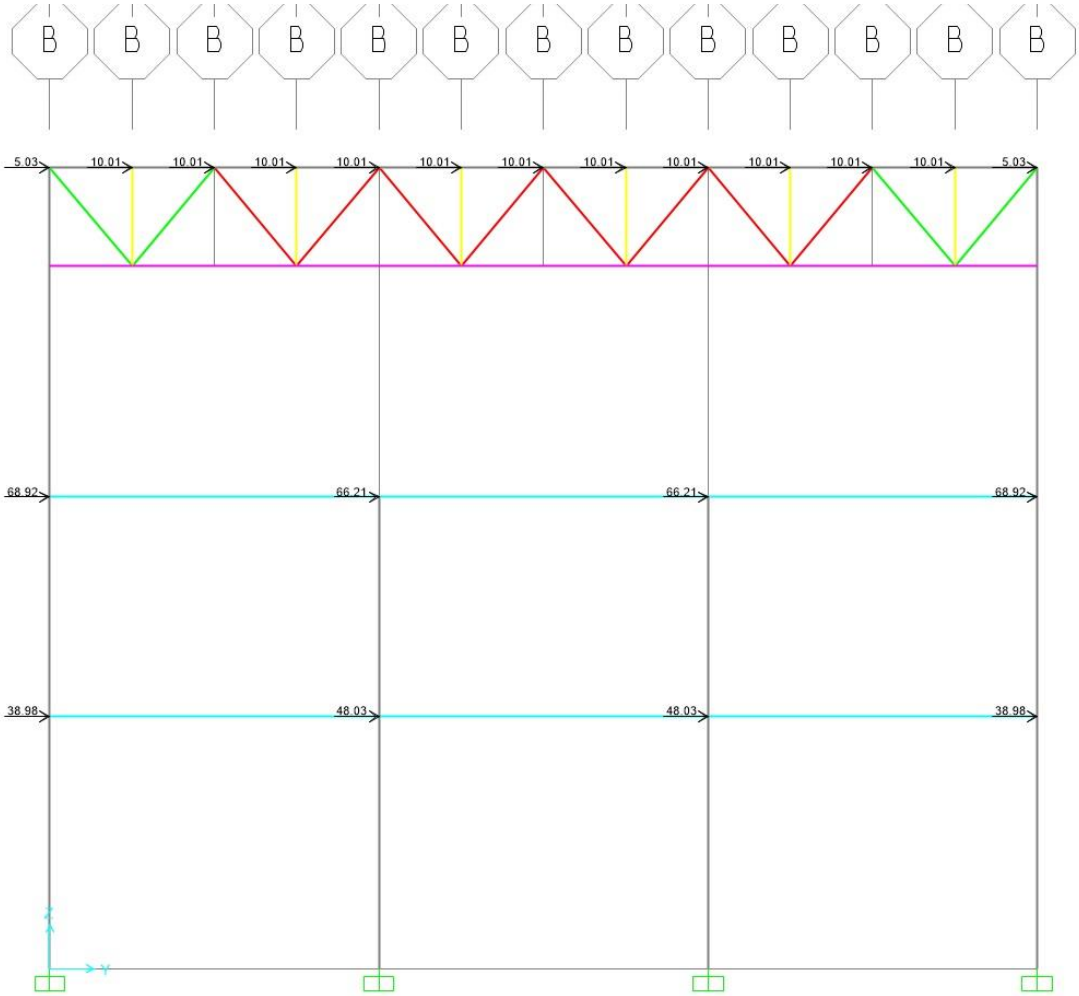
ΣW = 529,8 kN

(12,17) → 0,13 EQ1 = 43,58 kN

(13,14,15,16) → 0,185 EQ1 = 70,56 kN

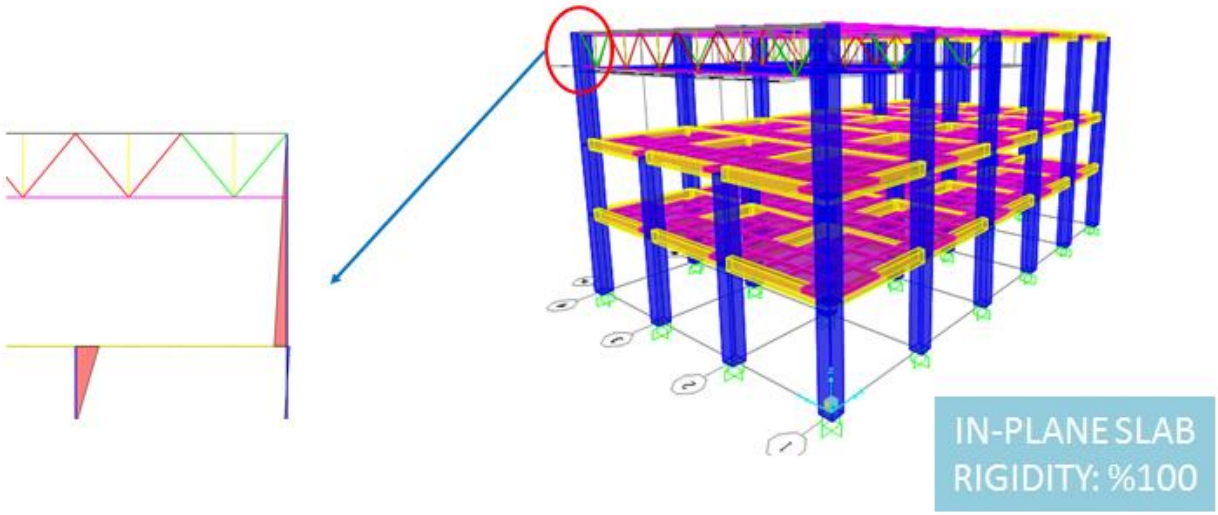

B B B B B B B B B B B B B
7.55 15.02 15.02 15.02 15.02 15.02 15.02 15.02 15.02 15.02 15.02 15.02 7.55
103.38 99.32 99.32 103.38
58.47 72.05 72.05 58.47
Z
Y


D D D D D D D D D D D D D
92.6 138.89 138.89 92.6
83.28 139.39 139.39 83.28
47.97 62.56 62.56 47.97
Z
Y

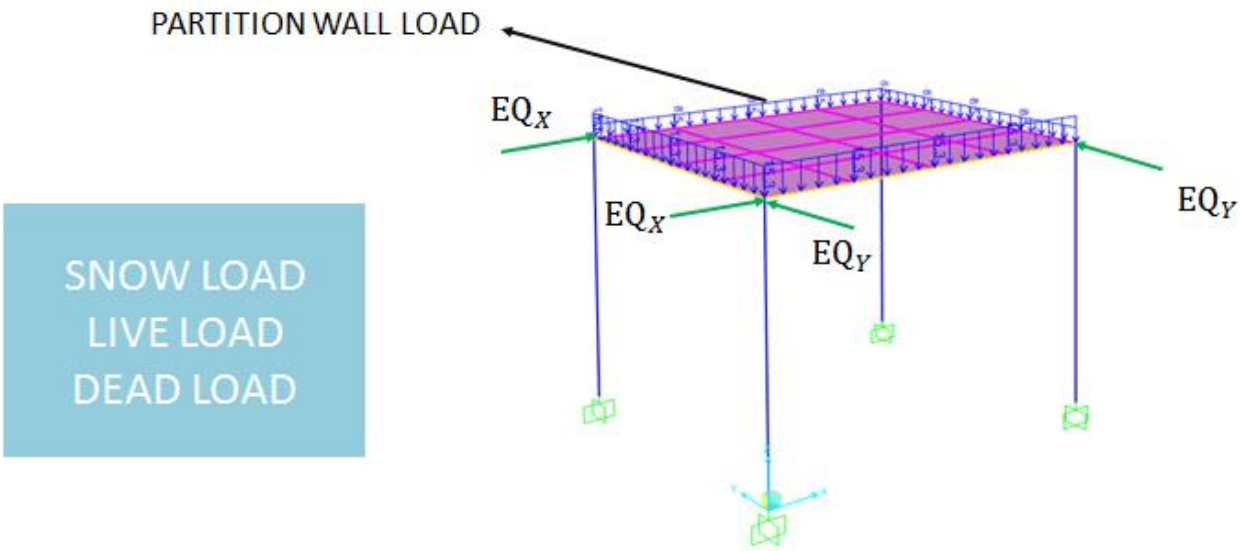

B
5.03
10.01
68.92
66.21
38.98
48.03

13. SAP2000 MODELS

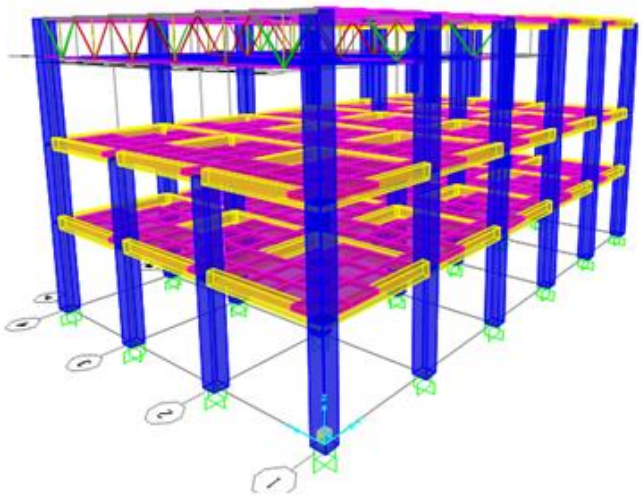
3D RIGID SLAB SAP2000 MODEL



3D RIGID SLAB SAP2000 MODEL LOADS

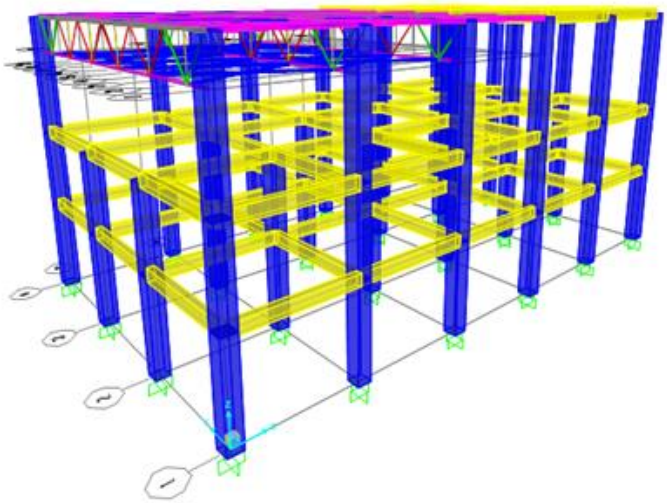


3D FLEXIBLE SAP2000 MODEL



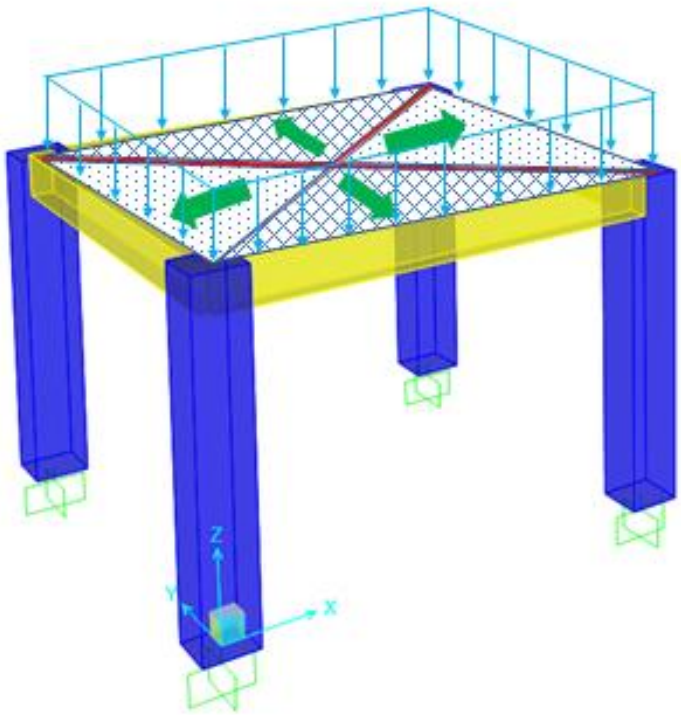
IN-PLANE & OUT-PLANE SLAB RIGIDITY: %1

3D WITHOUT SLAB SAP2000 MODEL

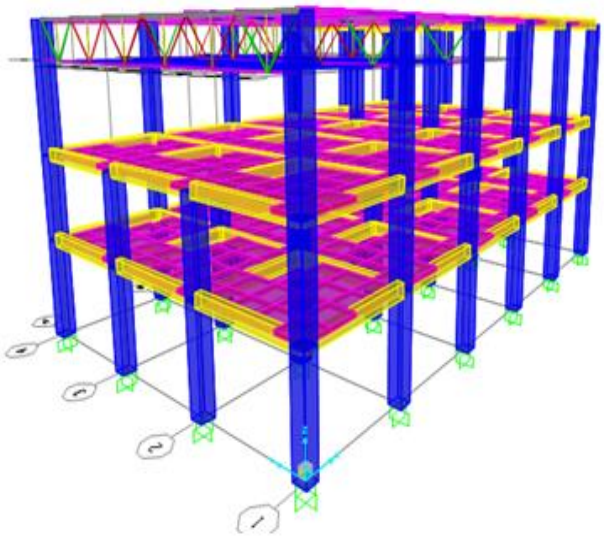


WITHOUT SLAB

WITHOUT SLAB SAP2000 MODEL LOADS



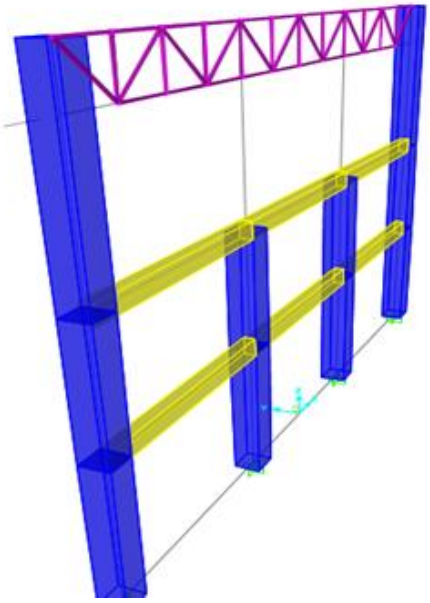
3D EFFECTIVE SECTION SAP2000 MODEL

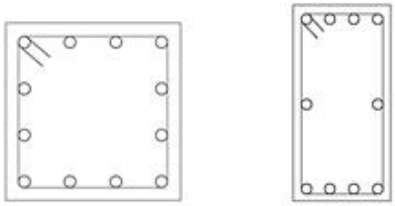


TSC-2017 4.5.8.1

IN-PLANE SLAB RIGIDITY: %25
COLUMN: %70
BEAM: %35

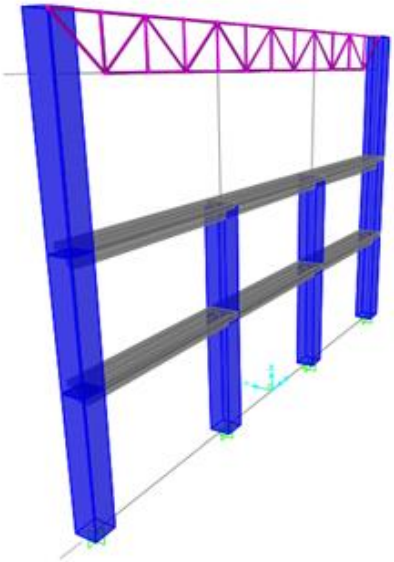
2D RECTANGULAR BEAM SAP2000 MODEL

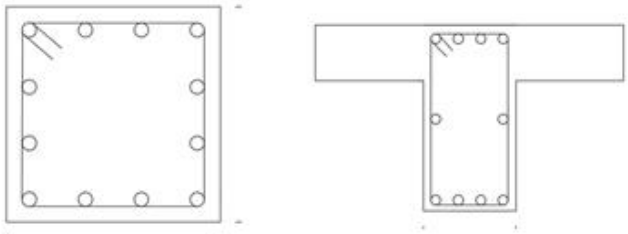


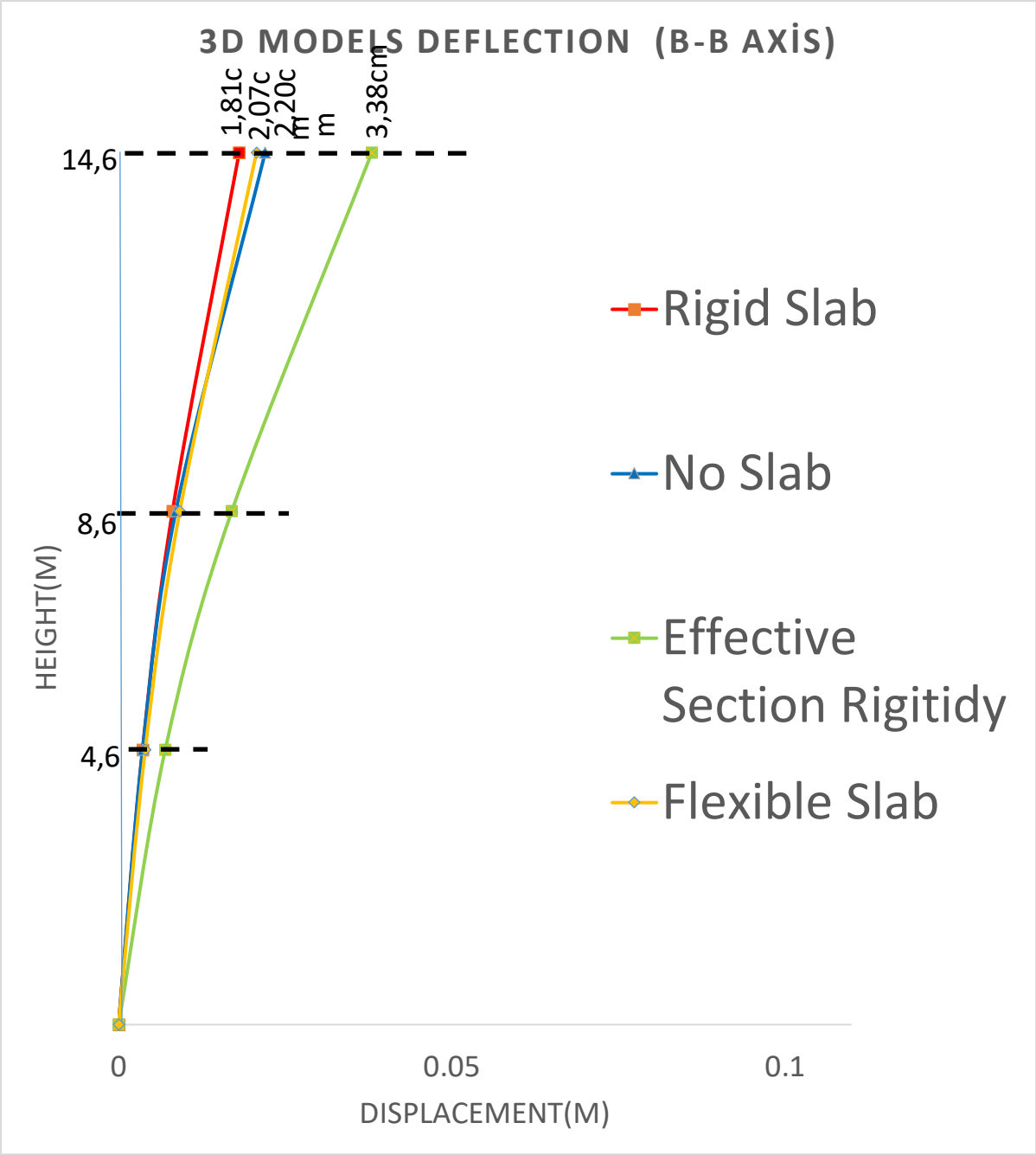


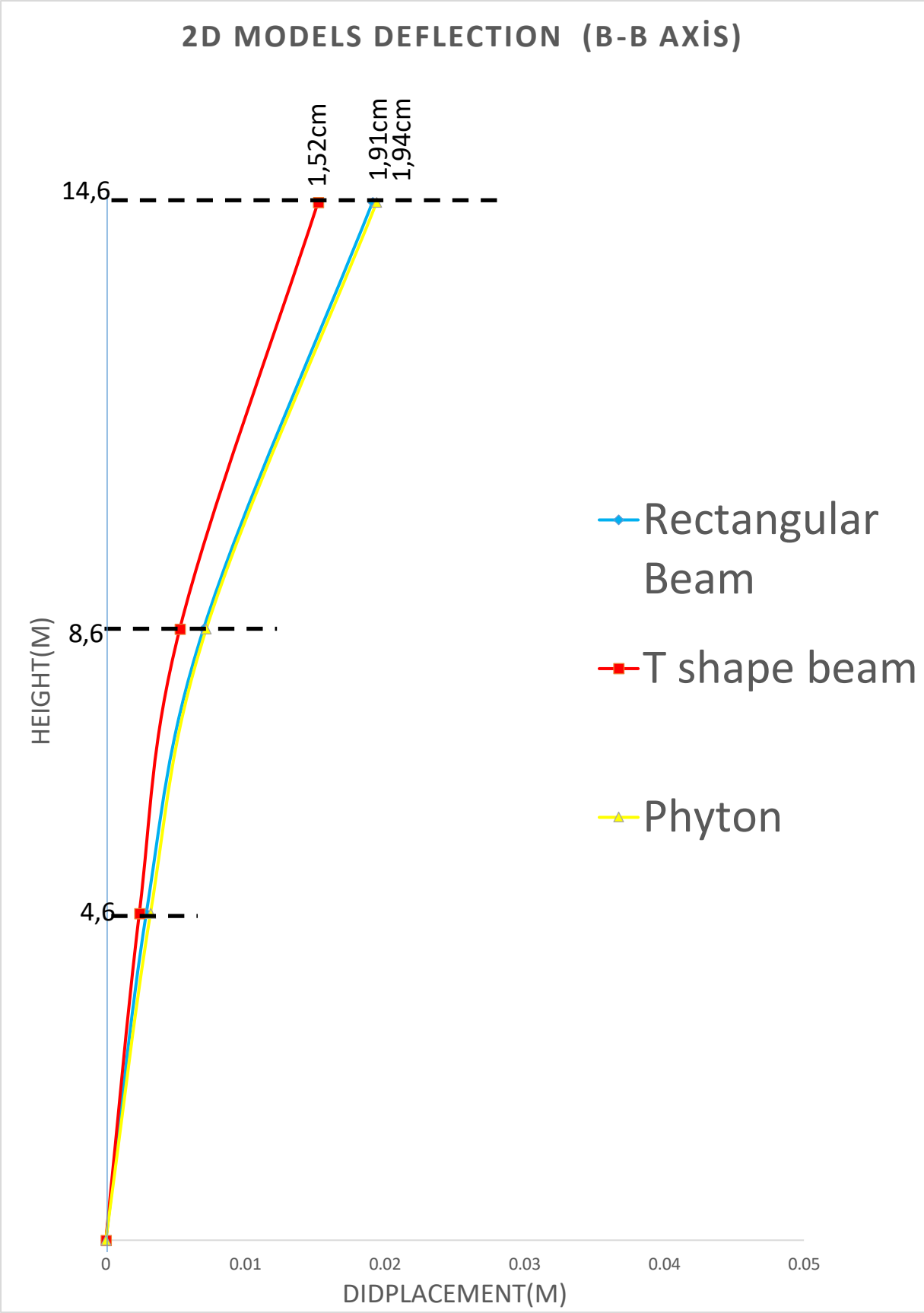
COLUMN & BEAM
RIGIDITY: %100

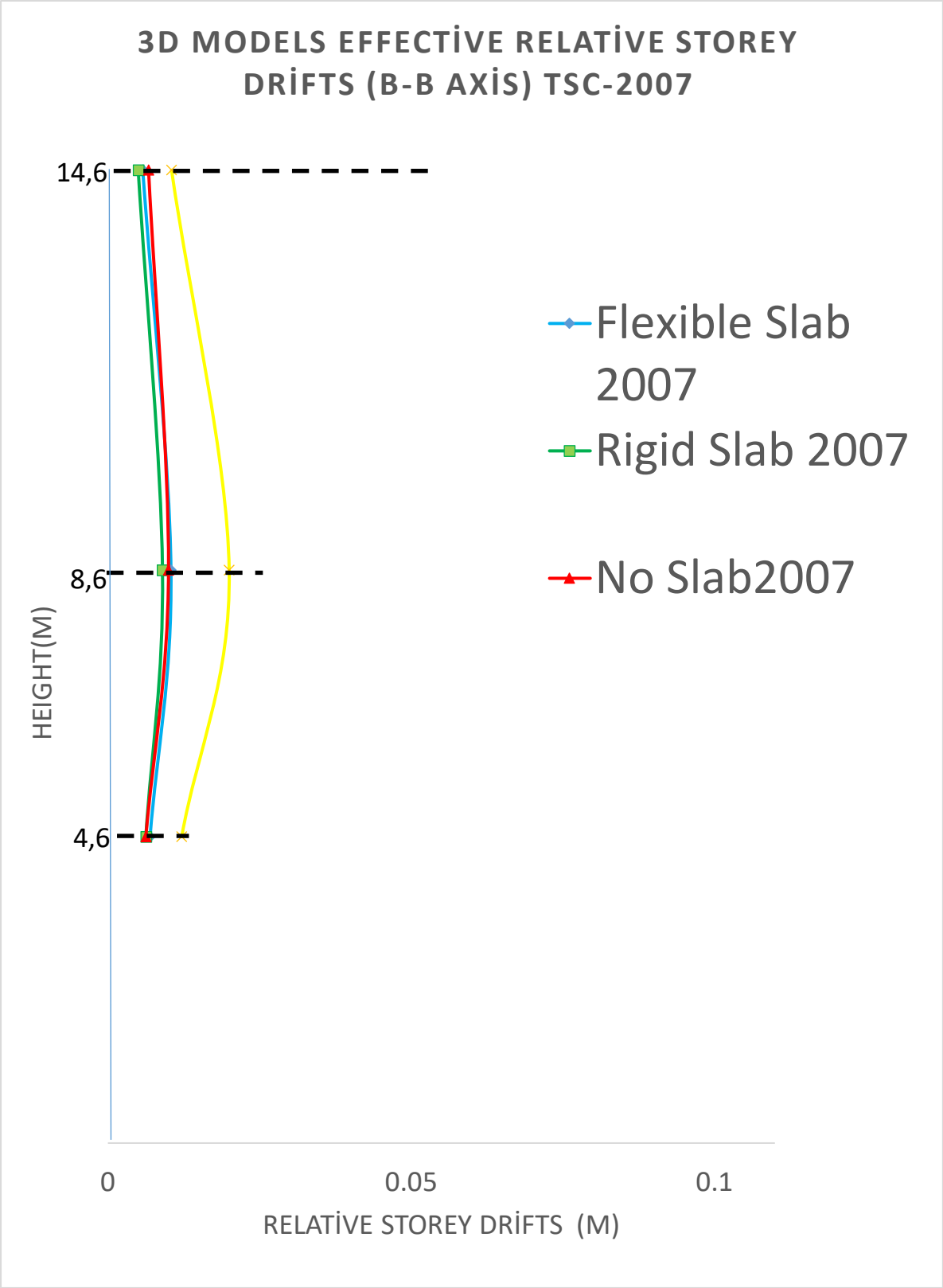
2D T-SHAPE BEAM SAP2000 MODEL

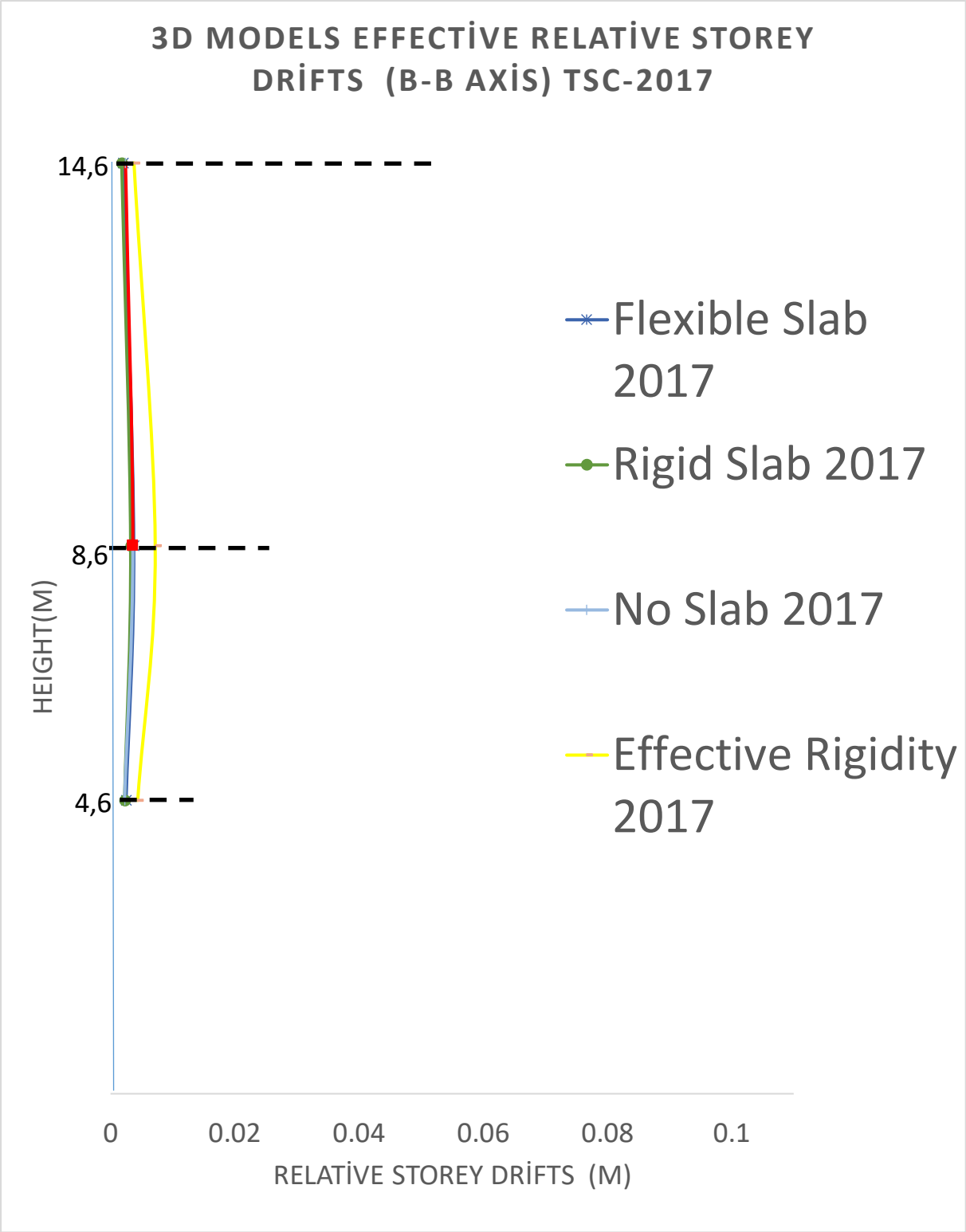


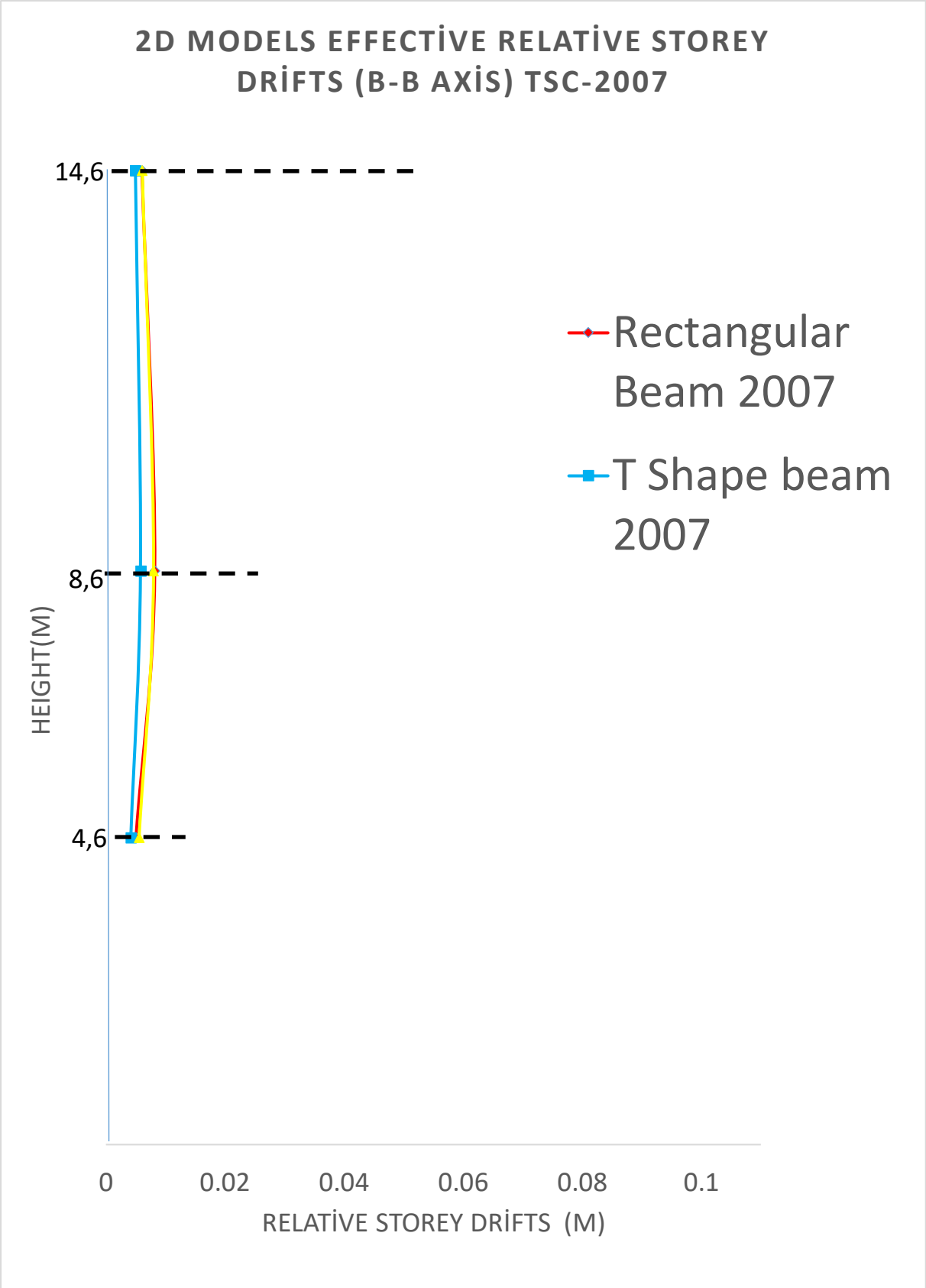


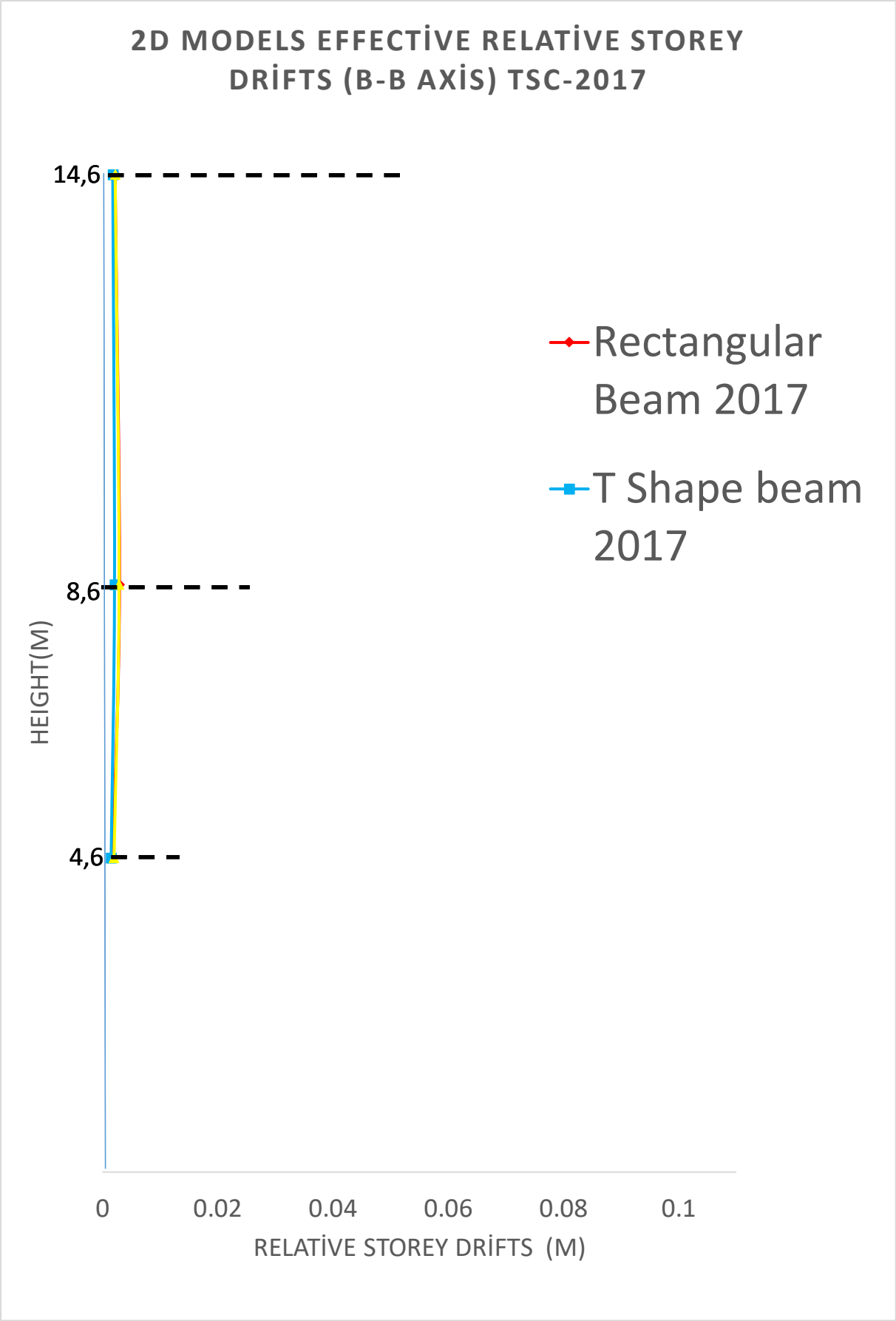

3D MODELS DEFLECTION (B-B AXİS)
1,81cm
2,07cm
2,20cm
3,38cm
14,6
8,6
4,6
HEIGHT(M)
0
0.05
0.1
DISPLACEMENT(M)
Rigid Slab
No Slab
Effective Section Rigitidy
Flexible Slab


2D MODELS DEFLECTION (B-B AXİS)
1,52cm
1,91cm
1,94cm
14,6
8,6
4,6
HEIGHT(M)
0
0.01
0.02
0.03
0.04
0.05
DIDPLACEMENT(M)
Rectangular Beam
T shape beam
Phyton


3D MODELS EFFECTİVE RELATİVE STOREY DRİFTS (B-B AXİS) TSC-2007
14,6
8,6
4,6
HEIGHT(M)
0
0.05
0.1
RELATİVE STOREY DRİFTS (M)
Flexible Slab 2007
Rigid Slab 2007
No Slab2007


3D MODELS EFFECTİVE RELATİVE STOREY DRİFTS (B-B AXİS) TSC-2017
14,6
8,6
4,6
HEIGHT(M)
0
0.02
0.04
0.06
0.08
0.1
RELATİVE STOREY DRİFTS (M)
Flexible Slab 2017
Rigid Slab 2017
No Slab 2017
Effective Rigidity 2017


2D MODELS EFFECTİVE RELATİVE STOREY DRİFTS (B-B AXİS) TSC-2007
14,6
8,6
4,6
HEIGHT(M)
0
0.02
0.04
0.06
0.08
0.1
RELATİVE STOREY DRİFTS (M)
Rectangular Beam 2007
T Shape beam 2007


2D MODELS EFFECTİVE RELATİVE STOREY DRİFTS (B-B AXİS) TSC-2017
14,6
8,6
4,6
HEIGHT(M)
0
0.02
0.04
0.06
0.08
0.1
RELATİVE STOREY DRİFTS (M)
Rectangular Beam 2017
T Shape beam 2017

14. Stiffness Method

For B-B axis, structural analysis was evaluated by using stiffness method. This method was applied via python coding. 2D B-B axis model was created also on SAP 2000. Python and SAP 2000 models were compared and results of these models was same in the ratio of %99.7.

Structure of the Program

For B-B axis, elements and nodes were enumerated. Firstly node and element dictionaries were created on python. Node dictionary include coordinate, external force and restrained situation index of each nodes. Element dictionary include connection nodes, material and section index of each element. Configuration parameters were defined to obtain element matrix dimension. Length, sin and cos function were defined to obtained stiffness matrix function of elements.

$$[KL^e] = \begin{bmatrix} EA/L & 0 & 0 & -EA/L & 0 & 0 \\ 0 & 12EI/L^3 & 6EI/L^2 & 0 & -12EI/L^3 & -6EI/L^2 \\ 0 & 6EI/L^2 & 4EI/L & 0 & -6EI/L^2 & 2EI/L \\ -EA/L & 0 & 0 & EA/L & 0 & 0 \\ 0 & -12EI/L^3 & -6EI/L^2 & 0 & 12EI/L^3 & -6EI/L^2 \\ 0 & 6EI/L^2 & 2EI/L & 0 & -6EI/L^2 & 4EI/L \end{bmatrix}$$

Stiffness Matrix

Stiffness matrix of each element was defined as a function. Transformation matrix function defined to transform local coordinate system to global coordinate system.

$$[T^e] = \begin{bmatrix} \cos x & \sin x & 0 & 0 & 0 & 0 \\ -\sin x & \cos x & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos x & \sin x & 0 \\ 0 & 0 & 0 & -\sin x & \cos x & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

Transformation Matrix

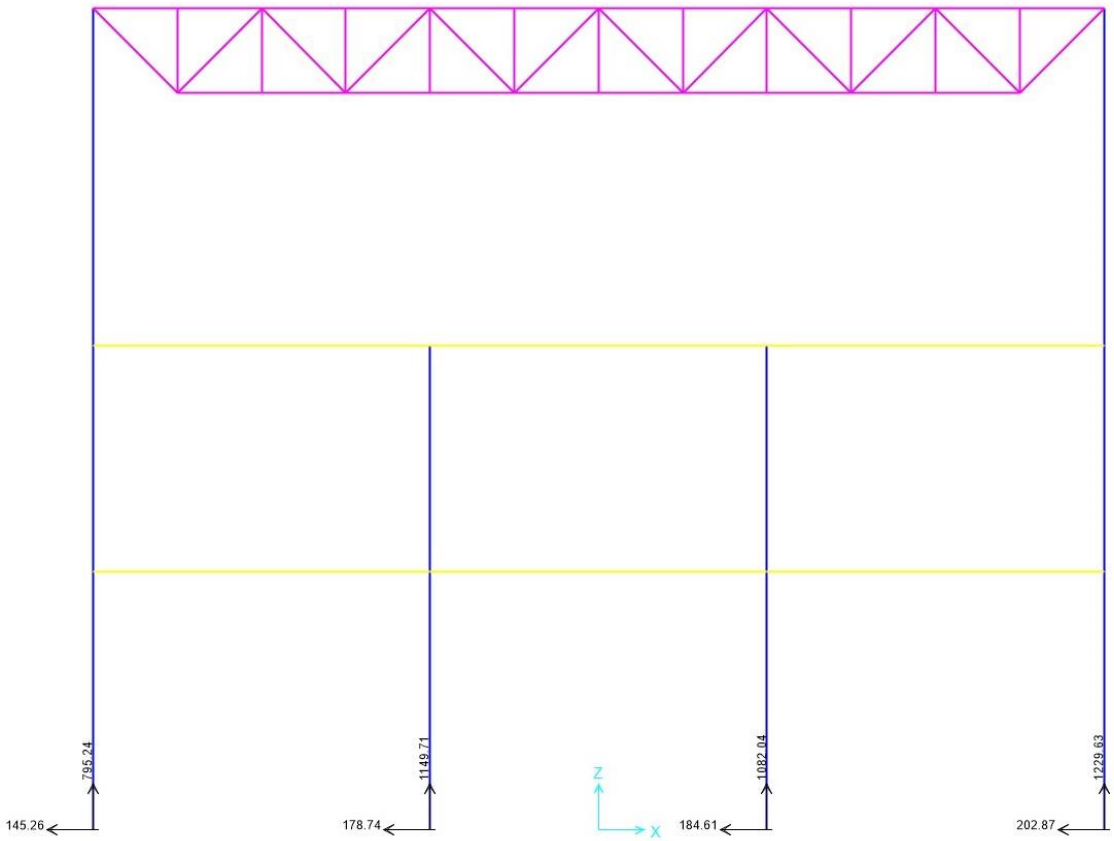
After obtaining of transformation matrix, global stiffness matrix of each element were created with equation …

$$[KLG^e]=[T^e]^T*[KL^e]*[T^e]. \qquad \text{Eq…}$$

All element stiffness matrix were assembled together to get global equilibrium as a A Matrix. All external force matrix of elements also were assembled together as RHS matrix. After that according to stiffness equation, the equation was solved and all displacement and forces were obtained.

These processes were rewrite for dead load, live load and earthquake load after that G+Q+E, G+Q-E, 1.4G+1.6Q, 09G+E and 0.9G-E combinations results were obtained.

<u>Comparison of SAP2000 and the stiffness program</u>



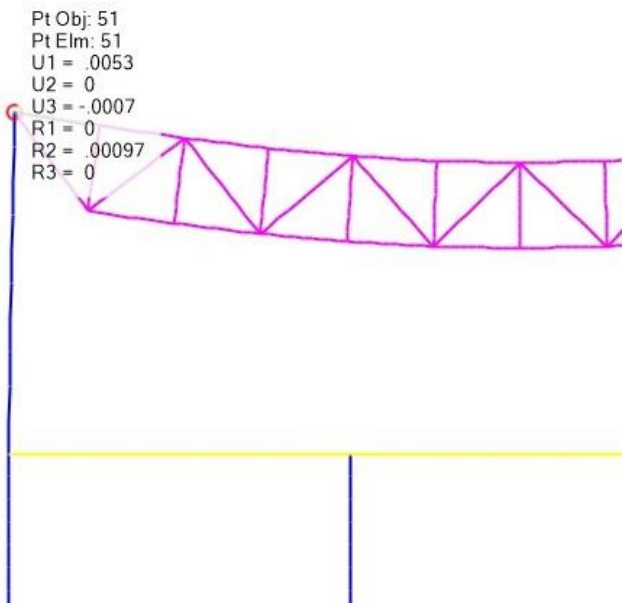
G+Q+E Load Combination Sap2000 Support Reactions

G+Q+E_stiffness program support reactions: [-146.57119421 794.64543349 597.23169592 -177.16541617 1147.98935681 656.11791181 -183.67648127 1082.88623256 669.21980324 -204.06690835 1230.83897714 696.28606896]

First Column Support Axial Force in Sap2000 is 795.24 kN

First Column Support Axial Force in stiffness program is 794.65 kN

Error= (795.24-794.65)/795.24*100= %0.07



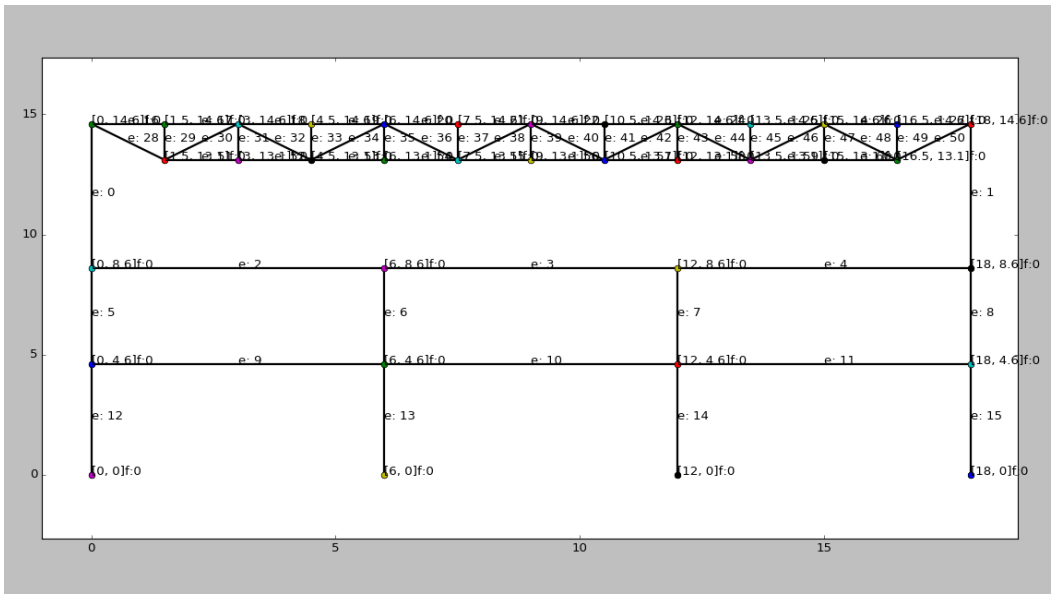
1.4G+1.6Q Load Combination Cantilever Column Deflections

In the stiffness program, 1.4G+1.6Q Load Combination Cantilever Column top point displacement in the z direction is -6.87409664e-04m

In the Sap2000, 1.4G+1.6Q Load Combination Cantilever Column top point displacement in the z direction is -0,0007m

Error = [(-6.87409664e-04) -(-0,0007)]/(-0,0007m)* 100=%1.79

Script of the Program




```
import numpy as np
import matplotlib.pyplot as plt

#Dictionary Keywords
code,rest, nodes, dof, Force, ForceQ, x = 'code','rest', 'nodes', 'dof', 'Force','ForceQ', 'x'
mat, kes = 'mat', 'kes'

#Node and Element Dictionaries
NDG = {0: {x: [0, 14.6], Force :[0, -4.3, 0]},
    1: {x: [18, 14.6], Force:[0, -4.3, 0]},
    2: {x: [0, 8.6], Force:[0, -382.56, -120.06]},
    3: {x: [6, 8.6], Force:[0, -267.04, 0]},
    4: {x: [12, 8.6], Force:[0, -267.04, 0]},
    5: {x: [18, 8.6], Force :[0, -382.56, 120.06]},
    6: {x: [0, 4.6], Force: [0, -369.71, -145.53]},
    7: {x: [6, 4.6], Force: [0, -474.47, 0]},
    8: {x: [12, 4.6], Force: [0, -474.47, 0]},
    9: {x: [18, 4.6], Force: [0, -369.71, 145.53]},
   10: {x: [0, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
```

```
    11: {x: [6, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
    12: {x: [12, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
    13: {x: [18, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
    14: {x: [1.50, 14.6], Force: [0, -8.35, 0]},
    15: {x: [1.50, 13.1], Force: [0, -0.60, 0]},
    16: {x: [3, 14.6], Force: [0, -8.65, 0]},
    17: {x: [3, 13.1], Force: [0, -0.45, 0]},
    18: {x: [4.5, 14.6], Force: [0, -8.35, 0]},
    19: {x: [4.5, 13.1], Force: [0, -0.60, 0]},
    20: {x: [6, 14.6], Force: [0, -8.65, 0]},
    21: {x: [6, 13.1], Force: [0, -0.45, 0]},
    22: {x: [7.5, 14.6], Force: [0, -8.35, 0]},
    23: {x: [7.5, 13.1], Force: [0, -0.60, 0]},
    24: {x: [9, 14.6], Force: [0, -8.65, 0]},
    25: {x: [9, 13.1], Force: [0, -0.45, 0]},
    26: {x: [10.5, 14.6], Force: [0, -8.35, 0]},
    27: {x: [10.5, 13.1], Force: [0, -0.60, 0]},
    28: {x: [12, 14.6], Force: [0, -8.65, 0]},
    29: {x: [12, 13.1], Force: [0, -0.45, 0]},
    30: {x: [13.50, 14.6], Force: [0, -8.35, 0]},
    31: {x: [13.50, 13.1], Force: [0, -0.60, 0]},
    32: {x: [15, 14.6], Force: [0, -8.65, 0]},
    33: {x: [15, 13.1], Force: [0, -0.45, 0]},
    34: {x: [16.50, 14.6], Force: [0, -8.35, 0]},
    35: {x: [16.5, 13.1], Force: [0, -0.60, 0]}
     }

mater1 = {'E' : 32e6, 'v': 0.2}
mater2 = {'E':210e6, 'v':0.3}

kesit1 = {'A':0.7*0.7, 'I':0.7**3*0.7/12}
kesit2 = {'A' : 0.6*0.3,'I':0.6**3*0.3/12}
kesit3 = {'A' : 0.002199, 'I': 0.00000137}
```

EDG = {0: {nodes: [2, 0], mat: mater1, kes: kesit1},
1: {nodes: [1, 5], mat: mater1, kes: kesit1},
2: {nodes: [2, 3], mat: mater1, kes: kesit2},
3: {nodes: [3, 4], mat: mater1, kes: kesit2},
4: {nodes: [4, 5], mat: mater1, kes: kesit2},
5: {nodes: [6, 2], mat: mater1, kes: kesit1},
6: {nodes: [7, 3], mat: mater1, kes: kesit1},
7: {nodes: [8, 4], mat: mater1, kes: kesit1},
8: {nodes: [5, 9], mat: mater1, kes: kesit1},
9: {nodes: [6, 7], mat: mater1, kes: kesit2},
10: {nodes: [7, 8], mat: mater1, kes: kesit2},
11: {nodes: [8, 9], mat: mater1, kes: kesit2},
12: {nodes: [10, 6], mat: mater1, kes: kesit1},
13: {nodes: [11, 7], mat: mater1, kes: kesit1},
14: {nodes: [12, 8], mat: mater1, kes: kesit1},
15: {nodes: [9, 13], mat: mater1, kes: kesit1},
16: {nodes: [0, 14], mat: mater2, kes: kesit3},
17: {nodes: [14, 16], mat: mater2, kes: kesit3},
18: {nodes: [16, 18], mat: mater2, kes: kesit3},
19: {nodes: [18, 20], mat: mater2, kes: kesit3},
20: {nodes: [20, 22], mat: mater2, kes: kesit3},
21: {nodes: [22, 24], mat: mater2, kes: kesit3},
22: {nodes: [24, 26], mat: mater2, kes: kesit3},
23: {nodes: [26, 28], mat: mater2, kes: kesit3},
24: {nodes: [28, 30], mat: mater2, kes: kesit3},
25: {nodes: [30, 32], mat: mater2, kes: kesit3},
26: {nodes: [32, 34], mat: mater2, kes: kesit3},
27: {nodes: [34, 1], mat: mater2, kes: kesit3},
28: {nodes: [0, 15], mat: mater2, kes: kesit3},
29: {nodes: [14, 15], mat: mater2, kes: kesit3},
30: {nodes: [16, 15], mat: mater2, kes: kesit3},
31: {nodes: [16, 17], mat: mater2, kes: kesit3},
32: {nodes: [16, 19], mat: mater2, kes: kesit3},
33: {nodes: [18, 19], mat: mater2, kes: kesit3},

34: {nodes: [20, 19], mat: mater2, kes: kesit3},

35: {nodes: [20, 21], mat: mater2, kes: kesit3},

36: {nodes: [20, 23], mat: mater2, kes: kesit3},

37: {nodes: [22, 23], mat: mater2, kes: kesit3},

38: {nodes: [24, 23], mat: mater2, kes: kesit3},

39: {nodes: [24, 25], mat: mater2, kes: kesit3},

40: {nodes: [24, 27], mat: mater2, kes: kesit3},

41: {nodes: [26, 27], mat: mater2, kes: kesit3},

42: {nodes: [28, 27], mat: mater2, kes: kesit3},

43: {nodes: [28, 29], mat: mater2, kes: kesit3},

44: {nodes: [28, 31], mat: mater2, kes: kesit3},

45: {nodes: [30, 31], mat: mater2, kes: kesit3},

46: {nodes: [32, 31], mat: mater2, kes: kesit3},

47: {nodes: [32, 33], mat: mater2, kes: kesit3},

48: {nodes: [32, 35], mat: mater2, kes: kesit3},

49: {nodes: [34, 35], mat: mater2, kes: kesit3},

50: {nodes: [1, 35], mat: mater2, kes: kesit3},

51: {nodes: [15, 17], mat: mater2, kes: kesit3},

52: {nodes: [17, 19], mat: mater2, kes: kesit3},

53: {nodes: [19, 21], mat: mater2, kes: kesit3},

54: {nodes: [21, 23], mat: mater2, kes: kesit3},

55: {nodes: [23, 25], mat: mater2, kes: kesit3},

56: {nodes: [25, 27], mat: mater2, kes: kesit3},

57: {nodes: [27, 29], mat: mater2, kes: kesit3},

58: {nodes: [29, 31], mat: mater2, kes: kesit3},

59: {nodes: [31, 33], mat: mater2, kes: kesit3},

60: {nodes: [33, 35], mat: mater2, kes: kesit3}

}

```
#Configuration Parameters
NODOF = 3  #Number of global degree of freedom per node
NON   = 2  #Number of nodes
EDIM  = NODOF * NON #Element matrix dimension


def LL(e): return (DX(e)**2+DY(e)**2)**0.5
def DX(e):return NDG[e[nodes][1]][x][0]-NDG[e[nodes][0]][x][0]
def DY(e):return NDG[e[nodes][1]][x][1]-NDG[e[nodes][0]][x][1]
def nX(e):return DX(e)/LL(e)
def nY(e):return DY(e)/LL(e)

def KLG(e):
    E = e[mat]['E']
    A = e[kes]['A']
    I = e[kes]['I']
    L = LL(e)
    L2 = L*L
    L3 = L2* L
    EA_L = E *A / L
    EI_L3 = E*I/L3
    EI_L2 = E * I / L2
    EI_L = E * I / L

    return np.asarray([EA_L, 0,        0,       -EA_L,    0,       0,
                       0,   12*EI_L3,  6*EI_L2,  0,      -12*EI_L3, 6*EI_L2,
                       0,   6*EI_L2,   4*EI_L,   0,      -6*EI_L2,  2*EI_L,
                      -EA_L, 0,        0,        EA_L,    0,       0,
                       0,   -12*EI_L3, -6*EI_L2, 0,       12*EI_L3, -6*EI_L2,
                       0,   6*EI_L2,   2*EI_L,   0,      -6*EI_L2,  4*EI_L]).reshape(6,6)
```

```
def T(e):
    nx, ny = nX(e), nY(e)
    return np.asarray([nx,  ny, 0,  0,   0,  0,
                       -ny, nx, 0,  0,   0,  0,
                       0,   0,  1,  0,   0,  0,
                       0,   0,  0,  nx,  ny, 0,
                       0,   0,  0,  -ny, nx, 0,
                       0,   0,  0,  0,   0,  1]).reshape(6,6)


def KGG(e): return T(e).T@KLG(e)@T(e)


for n in NDG.values():
    n[dof] = [0]*NODOF
    n[Force] = n.get(Force, [0]*NODOF)
    n[rest] = n.get(rest, [0]*NODOF)

M = 0
N = 0
for kere in [0, 1]:
    for n in NDG.values():
        for i, r in enumerate(n[rest]):
            if r==kere: n[dof][i]=M; M+=1
    if kere==0: N = M

for e in EDG.values():
    e[code] = []
    for n in e[nodes]: e[code] += NDG[n][dof]

print('Number of displacement unknowns: ', N)
print('Number of total unknowns: ', M)

print('Assembling System Equations....')
```

```
RHSG = np.zeros(M)
LHSG = np.zeros(M)
AG = np.zeros(M**2).reshape(M,M)


for e in EDG.values():
    columns = np.asarray(e[code]*EDIM)
    rows = columns.reshape(EDIM, EDIM).T.reshape(EDIM**2)
    AG[rows, columns] += KGG(e).reshape(EDIM**2)


for n in NDG.values():
    for i, d in enumerate(n[dof]): RHSG[d] = n[Force][i]



print('Solving System Equations....')

LHSG[0:N] = np.linalg.solve(AG[0:N,0:N], RHSG[0:N])
RHSG[N:M] = AG[N:M,:]@LHSG

print('G_displacement: ',LHSG[0:N])
print('G_support reactions: ',RHSG[N:M])
print('G_node reactions: ', RHSG[0:M])
print("Dead Load DONE...")


##########################################LIVE
LOAD#################################


#Node and Element Dictionaries
NDQ = {0: {x: [0, 14.6], ForceQ :[0,-3.375,0]},
```

1: {x: [18, 14.6], ForceQ:[0,-3.375, 0]},

2: {x: [0, 8.6], ForceQ:[0,-90, -90]},

3: {x: [6, 8.6], ForceQ:[0,-180, 0]},

4: {x: [12, 8.6], ForceQ:[0, -180, 0]},

5: {x: [18, 8.6], ForceQ :[0, -90, 90]},

6: {x: [0, 4.6], ForceQ: [0, -90, -90]},

7: {x: [6, 4.6], ForceQ: [0, -180, 0]},

8: {x: [12, 4.6], ForceQ: [0,-180, 0]},

9: {x: [18, 4.6], ForceQ: [0, -90, 90]},

10: {x: [0, 0], ForceQ: [0, 0, 0], rest: [1, 1, 1]},

11: {x: [6, 0], ForceQ: [0, 0, 0], rest: [1, 1, 1]},

12: {x: [12, 0], ForceQ: [0, 0, 0], rest: [1, 1, 1]},

13: {x: [18, 0], ForceQ: [0, 0, 0], rest: [1, 1, 1]},

14: {x: [1.50, 14.6], ForceQ: [0,-6.75, 0]},

15: {x: [1.50, 13.1], ForceQ: [0, 0, 0]},

16: {x: [3, 14.6], ForceQ: [0,-6.75, 0]},

17: {x: [3, 13.1], ForceQ: [0, 0, 0]},

18: {x: [4.5, 14.6], ForceQ: [0,-6.75, 0]},

19: {x: [4.5, 13.1], ForceQ: [0, 0, 0]},

20: {x: [6, 14.6], ForceQ: [0,-6.75, 0]},

21: {x: [6, 13.1], ForceQ: [0, 0, 0]},

22: {x: [7.5, 14.6], ForceQ: [0,-6.75, 0]},

23: {x: [7.5, 13.1], ForceQ: [0, 0, 0]},

24: {x: [9, 14.6], ForceQ: [0, -6.75, 0]},

25: {x: [9, 13.1], ForceQ: [0, 0, 0]},

26: {x: [10.5, 14.6], ForceQ: [0,-6.75, 0]},

27: {x: [10.5, 13.1], ForceQ: [0, 0, 0]},

28: {x: [12, 14.6], ForceQ: [0,-6.75, 0]},

29: {x: [12, 13.1], ForceQ: [0, 0, 0]},

30: {x: [13.50, 14.6], ForceQ: [0,-6.75, 0]},

31: {x: [13.50, 13.1], ForceQ: [0, 0, 0]},

32: {x: [15, 14.6], ForceQ: [0,-6.75, 0]},

33: {x: [15, 13.1], ForceQ: [0, 0, 0]},

34: {x: [16.50, 14.6], ForceQ: [0,-6.75, 0]},

35: {x: [16.50, 13.1], ForceQ: [0, 0, 0]}
}

EDQ = {0: {nodes: [2, 0], mat: mater1, kes: kesit1},
1: {nodes: [1, 5], mat: mater1, kes: kesit1},
2: {nodes: [2, 3], mat: mater1, kes: kesit2},
3: {nodes: [3, 4], mat: mater1, kes: kesit2},
4: {nodes: [4, 5], mat: mater1, kes: kesit2},
5: {nodes: [6, 2], mat: mater1, kes: kesit1},
6: {nodes: [7, 3], mat: mater1, kes: kesit1},
7: {nodes: [8, 4], mat: mater1, kes: kesit1},
8: {nodes: [5, 9], mat: mater1, kes: kesit1},
9: {nodes: [6, 7], mat: mater1, kes: kesit2},
10: {nodes: [7, 8], mat: mater1, kes: kesit2},
11: {nodes: [8, 9], mat: mater1, kes: kesit2},
12: {nodes: [10, 6], mat: mater1, kes: kesit1},
13: {nodes: [11, 7], mat: mater1, kes: kesit1},
14: {nodes: [12, 8], mat: mater1, kes: kesit1},
15: {nodes: [9, 13], mat: mater1, kes: kesit1},
16: {nodes: [0, 14], mat: mater2, kes: kesit3},
17: {nodes: [14, 16], mat: mater2, kes: kesit3},
18: {nodes: [16, 18], mat: mater2, kes: kesit3},
19: {nodes: [18, 20], mat: mater2, kes: kesit3},
20: {nodes: [20, 22], mat: mater2, kes: kesit3},
21: {nodes: [22, 24], mat: mater2, kes: kesit3},
22: {nodes: [24, 26], mat: mater2, kes: kesit3},
23: {nodes: [26, 28], mat: mater2, kes: kesit3},
24: {nodes: [28, 30], mat: mater2, kes: kesit3},
25: {nodes: [30, 32], mat: mater2, kes: kesit3},
26: {nodes: [32, 34], mat: mater2, kes: kesit3},
27: {nodes: [34, 1], mat: mater2, kes: kesit3},
28: {nodes: [0, 15], mat: mater2, kes: kesit3},
29: {nodes: [14, 15], mat: mater2, kes: kesit3},

30: {nodes: [16, 15], mat: mater2, kes: kesit3},

31: {nodes: [16, 17], mat: mater2, kes: kesit3},

32: {nodes: [16, 19], mat: mater2, kes: kesit3},

33: {nodes: [18, 19], mat: mater2, kes: kesit3},

34: {nodes: [20, 19], mat: mater2, kes: kesit3},

35: {nodes: [20, 21], mat: mater2, kes: kesit3},

36: {nodes: [20, 23], mat: mater2, kes: kesit3},

37: {nodes: [22, 23], mat: mater2, kes: kesit3},

38: {nodes: [24, 23], mat: mater2, kes: kesit3},

39: {nodes: [24, 25], mat: mater2, kes: kesit3},

40: {nodes: [24, 27], mat: mater2, kes: kesit3},

41: {nodes: [26, 27], mat: mater2, kes: kesit3},

42: {nodes: [28, 27], mat: mater2, kes: kesit3},

43: {nodes: [28, 29], mat: mater2, kes: kesit3},

44: {nodes: [28, 31], mat: mater2, kes: kesit3},

45: {nodes: [30, 31], mat: mater2, kes: kesit3},

46: {nodes: [32, 31], mat: mater2, kes: kesit3},

47: {nodes: [32, 33], mat: mater2, kes: kesit3},

48: {nodes: [32, 35], mat: mater2, kes: kesit3},

49: {nodes: [34, 35], mat: mater2, kes: kesit3},

50: {nodes: [1, 35], mat: mater2, kes: kesit3},

51: {nodes: [15, 17], mat: mater2, kes: kesit3},

52: {nodes: [17, 19], mat: mater2, kes: kesit3},

53: {nodes: [19, 21], mat: mater2, kes: kesit3},

54: {nodes: [21, 23], mat: mater2, kes: kesit3},

55: {nodes: [23, 25], mat: mater2, kes: kesit3},

56: {nodes: [25, 27], mat: mater2, kes: kesit3},

57: {nodes: [27, 29], mat: mater2, kes: kesit3},

58: {nodes: [29, 31], mat: mater2, kes: kesit3},

59: {nodes: [31, 33], mat: mater2, kes: kesit3},

60: {nodes: [33, 35], mat: mater2, kes: kesit3}

}

```
def KLQ(eQ):
    E = eQ[mat]['E']
    A = eQ[kes]['A']
    I = eQ[kes]['I']
    L = LL(eQ)
    L2 = L*L
    L3 = L2* L
    EA_L = E *A / L
    EI_L3 = E*I/L3
    EI_L2 = E * I / L2
    EI_L = E * I / L

    return np.asarray([EA_L, 0,        0,       -EA_L,    0,       0,
                       0,   12*EI_L3,  6*EI_L2,  0,      -12*EI_L3, 6*EI_L2,
                       0,   6*EI_L2,   4*EI_L,   0,      -6*EI_L2,  2*EI_L,
                      -EA_L, 0,        0,        EA_L,    0,       0,
                       0,   -12*EI_L3, -6*EI_L2, 0,       12*EI_L3, -6*EI_L2,
                       0,   6*EI_L2,   2*EI_L,   0,      -6*EI_L2,  4*EI_L]).reshape(6,6)



def KGQ(eQ): return T(eQ).T@KLQ(eQ)



for nQ in NDQ.values():
    nQ[dof] = [0]*NODOF
    nQ[ForceQ] = nQ.get(ForceQ, [0]*NODOF)
    nQ[rest] = nQ.get(rest, [0]*NODOF)

M = 0
N = 0
for kere in [0, 1]:
```

```
    for n in NDQ.values():
        for i, r in enumerate(n[rest]):
            if r==kere: n[dof][i]=M; M+=1
    if kere==0: N = M

for eQ in EDQ.values():
    eQ[code] = []
    for nQ in eQ[nodes]: eQ[code] += NDQ[nQ][dof]


print('Assembling System Equations....')

RHSQ = np.zeros(M)
LHSQ = np.zeros(M)
AQ = np.zeros(M**2).reshape(M,M)


for eQ in EDQ.values():
    columnsQ = np.asarray(eQ[code]*EDIM)
    rowsQ = columnsQ.reshape(EDIM, EDIM).T.reshape(EDIM**2)
    AQ[rowsQ, columnsQ] += KGQ(eQ).reshape(EDIM**2)


for nQ in NDQ.values():
    for iQ, dQ in enumerate(nQ[dof]): RHSQ[dQ] = nQ[ForceQ][iQ]


print('Solving System Equations....')

LHSQ[0:N] = np.linalg.solve(AQ[0:N,0:N], RHSQ[0:N])
RHSQ[N:M] = AQ[N:M,:]@LHSQ

print('Q_displacement: ',LHSQ[0:N])
```

```
print('Q_support reactions: ',RHSQ[N:M])
print('Q_node reactions: ', RHSQ[0:M])
print("Live Load DONE...")

#########################################Pozitif Earthquake
Load#################################


#Node and Element Dictionaries
NDE = {0: {x: [0, 14.6], Force :[7.55,0, 0]},
    1: {x: [18, 14.6], Force:[7.55, 0, 0]},
    2: {x: [0, 8.6], Force:[82.7, 0, 0]},
    3: {x: [6, 8.6], Force:[76.46, 0, 0]},
    4: {x: [12, 8.6], Force:[76.46, 0, 0]},
    5: {x: [18, 8.6], Force :[82.7, 0, 0]},
    6: {x: [0, 4.6], Force: [48.78, 0, 0]},
    7: {x: [6, 4.6], Force: [57.64, 0, 0]},
    8: {x: [12, 4.6], Force: [57.64, 0, 0]},
    9: {x: [18, 4.6], Force: [48.78, 0, 0]},
   10: {x: [0, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
   11: {x: [6, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
   12: {x: [12, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
   13: {x: [18, 0], Force: [0, 0, 0], rest: [1, 1, 1]},
   14: {x: [1.50, 14.6], Force: [15.02, 0, 0]},
   15: {x: [1.50, 13.1], Force: [0, 0, 0]},
   16: {x: [3, 14.6], Force: [15.02, 0, 0]},
   17: {x: [3, 13.1], Force: [0, 0, 0]},
   18: {x: [4.5, 14.6], Force: [15.02, 0, 0]},
   19: {x: [4.5, 13.1], Force: [0, 0, 0]},
   20: {x: [6, 14.6], Force: [15.02, 0, 0]},
   21: {x: [6, 13.1], Force: [0, 0, 0]},
   22: {x: [7.5, 14.6], Force: [15.02, 0, 0]},
   23: {x: [7.5, 13.1], Force: [0, 0, 0]},
   24: {x: [9, 14.6], Force: [15.02, 0, 0]},
```

25: {x: [9, 13.1], Force: [0, 0, 0]},

26: {x: [10.5, 14.6], Force: [15.02, 0, 0]},

27: {x: [10.5, 13.1], Force: [0, 0, 0]},

28: {x: [12, 14.6], Force: [15.02, 0, 0]},

29: {x: [12, 13.1], Force: [0, 0, 0]},

30: {x: [13.50, 14.6], Force: [15.02, 0, 0]},

31: {x: [13.50, 13.1], Force: [0, 0, 0]},

32: {x: [15, 14.6], Force: [15.02, 0, 0]},

33: {x: [15, 13.1], Force: [0, 0, 0]},

34: {x: [16.50, 14.6], Force: [15.02, 0, 0]},

35: {x: [16.50, 13.1], Force: [0, 0, 0]}

}

EDE = {0: {nodes: [2, 0], mat: mater1, kes: kesit1},

1: {nodes: [1, 5], mat: mater1, kes: kesit1},

2: {nodes: [2, 3], mat: mater1, kes: kesit2},

3: {nodes: [3, 4], mat: mater1, kes: kesit2},

4: {nodes: [4, 5], mat: mater1, kes: kesit2},

5: {nodes: [6, 2], mat: mater1, kes: kesit1},

6: {nodes: [7, 3], mat: mater1, kes: kesit1},

7: {nodes: [8, 4], mat: mater1, kes: kesit1},

8: {nodes: [5, 9], mat: mater1, kes: kesit1},

9: {nodes: [6, 7], mat: mater1, kes: kesit2},

10: {nodes: [7, 8], mat: mater1, kes: kesit2},

11: {nodes: [8, 9], mat: mater1, kes: kesit2},

12: {nodes: [10, 6], mat: mater1, kes: kesit1},

13: {nodes: [11, 7], mat: mater1, kes: kesit1},

14: {nodes: [12, 8], mat: mater1, kes: kesit1},

15: {nodes: [9, 13], mat: mater1, kes: kesit1},

16: {nodes: [0, 14], mat: mater2, kes: kesit3},

17: {nodes: [14, 16], mat: mater2, kes: kesit3},

18: {nodes: [16, 18], mat: mater2, kes: kesit3},

19: {nodes: [18, 20], mat: mater2, kes: kesit3},

20: {nodes: [20, 22], mat: mater2, kes: kesit3},

21: {nodes: [22, 24], mat: mater2, kes: kesit3},
22: {nodes: [24, 26], mat: mater2, kes: kesit3},
23: {nodes: [26, 28], mat: mater2, kes: kesit3},
24: {nodes: [28, 30], mat: mater2, kes: kesit3},
25: {nodes: [30, 32], mat: mater2, kes: kesit3},
26: {nodes: [32, 34], mat: mater2, kes: kesit3},
27: {nodes: [34, 1], mat: mater2, kes: kesit3},
28: {nodes: [0, 15], mat: mater2, kes: kesit3},
29: {nodes: [14, 15], mat: mater2, kes: kesit3},
30: {nodes: [16, 15], mat: mater2, kes: kesit3},
31: {nodes: [16, 17], mat: mater2, kes: kesit3},
32: {nodes: [16, 19], mat: mater2, kes: kesit3},
33: {nodes: [18, 19], mat: mater2, kes: kesit3},
34: {nodes: [20, 19], mat: mater2, kes: kesit3},
35: {nodes: [20, 21], mat: mater2, kes: kesit3},
36: {nodes: [20, 23], mat: mater2, kes: kesit3},
37: {nodes: [22, 23], mat: mater2, kes: kesit3},
38: {nodes: [24, 23], mat: mater2, kes: kesit3},
39: {nodes: [24, 25], mat: mater2, kes: kesit3},
40: {nodes: [24, 27], mat: mater2, kes: kesit3},
41: {nodes: [26, 27], mat: mater2, kes: kesit3},
42: {nodes: [28, 27], mat: mater2, kes: kesit3},
43: {nodes: [28, 29], mat: mater2, kes: kesit3},
44: {nodes: [28, 31], mat: mater2, kes: kesit3},
45: {nodes: [30, 31], mat: mater2, kes: kesit3},
46: {nodes: [32, 31], mat: mater2, kes: kesit3},
47: {nodes: [32, 33], mat: mater2, kes: kesit3},
48: {nodes: [32, 35], mat: mater2, kes: kesit3},
49: {nodes: [34, 35], mat: mater2, kes: kesit3},
50: {nodes: [1, 35], mat: mater2, kes: kesit3},
51: {nodes: [15, 17], mat: mater2, kes: kesit3},
52: {nodes: [17, 19], mat: mater2, kes: kesit3},
53: {nodes: [19, 21], mat: mater2, kes: kesit3},
54: {nodes: [21, 23], mat: mater2, kes: kesit3},

```
    55: {nodes: [23, 25], mat: mater2, kes: kesit3},
    56: {nodes: [25, 27], mat: mater2, kes: kesit3},
    57: {nodes: [27, 29], mat: mater2, kes: kesit3},
    58: {nodes: [29, 31], mat: mater2, kes: kesit3},
    59: {nodes: [31, 33], mat: mater2, kes: kesit3},
    60: {nodes: [33, 35], mat: mater2, kes: kesit3}

     }


def KLE(e):
    E = e[mat]['E']
    A = e[kes]['A']
    I = e[kes]['I']
    L = LL(e)
    L2 = L*L
    L3 = L2* L
    EA_L = E *A / L
    EI_L3 = E*I/L3
    EI_L2 = E * I / L2
    EI_L = E * I / L

    return np.asarray([EA_L, 0,        0,       -EA_L,    0,       0,
                    0,   12*EI_L3,  6*EI_L2,  0,      -12*EI_L3, 6*EI_L2,
                    0,   6*EI_L2,   4*EI_L,   0,      -6*EI_L2,  2*EI_L,
                   -EA_L, 0,        0,       EA_L,    0,       0,
                    0,   -12*EI_L3, -6*EI_L2, 0,      12*EI_L3, -6*EI_L2,
                    0,   6*EI_L2,   2*EI_L,   0,      -6*EI_L2,  4*EI_L]).reshape(6,6)



def KGE(e): return T(e).T@KLE(e)@T(e)
```

```
for n in NDE.values():
    n[dof] = [0]*NODOF
    n[Force] = n.get(Force, [0]*NODOF)
    n[rest] = n.get(rest, [0]*NODOF)

M = 0
N = 0
for kere in [0, 1]:
    for n in NDE.values():
        for i, r in enumerate(n[rest]):
            if r==kere: n[dof][i]=M; M+=1
    if kere==0: N = M

for e in EDE.values():
    e[code] = []
    for n in e[nodes]: e[code] += NDE[n][dof]


print('Assembling System Equations....')

RHSE = np.zeros(M)
LHSE = np.zeros(M)
AE = np.zeros(M**2).reshape(M,M)


for e in EDE.values():
    columns = np.asarray(e[code]*EDIM)
    rows = columns.reshape(EDIM, EDIM).T.reshape(EDIM**2)
    AE[rows, columns] += KGE(e).reshape(EDIM**2)


for n in NDE.values():
    for i, d in enumerate(n[dof]): RHSE[d] = n[Force][i]
```

```
print('Solving System Equations....')

LHSE[0:N] = np.linalg.solve(AE[0:N,0:N], RHSE[0:N])
RHSE[N:M] = AE[N:M,:]@LHSE

print('E_displacement: ',LHSE[0:N])
print('E_support reactions: ',RHSE[N:M])
print('E_node reactions: ', RHSE[0:M])
print("Eartquake Load DONE...")


##############################G+Q+E
Combination##########################################

RHS1 = np.zeros(M)
LHS1 = np.zeros(M)

LHS1[0:N] = LHSG[0:N]+LHSQ[0:N]+LHSE[0:N]
RHS1[N:M] = RHSG[N:M]+RHSQ[N:M]+RHSE[N:M]
RHS1[0:M] = RHSG[0:M]+RHSQ[0:M]+RHSE[0:M]

print('G+Q+E_displacement: ',LHS1[0:N])
print('G+Q+E_support reactions: ',RHS1[N:M])
print('G+Q+E_node reactions: ', RHS1[0:M])
print("G+Q+E Combination DONE...")

#######################Negatif Earthquake##############################3
#Node and Element Dictionaries
NDE2 = {0: {x: [0, 14.6], Force :[-7.55,0, 0]},
    1: {x: [18, 14.6], Force:[-7.55, 0, 0]},
    2: {x: [0, 8.6], Force:[-82.7, 0, 0]},
```

3: {x: [6, 8.6], Force:[-76.46, 0, 0]},

4: {x: [12, 8.6], Force:[-76.46, 0, 0]},

5: {x: [18, 8.6], Force :[-82.7, 0, 0]},

6: {x: [0, 4.6], Force: [-48.78, 0, 0]},

7: {x: [6, 4.6], Force: [-57.64, 0, 0]},

8: {x: [12, 4.6], Force: [-57.64, 0, 0]},

9: {x: [18, 4.6], Force: [-48.78, 0, 0]},

10: {x: [0, 0], Force: [0, 0, 0], rest: [1, 1, 1]},

11: {x: [6, 0], Force: [0, 0, 0], rest: [1, 1, 1]},

12: {x: [12, 0], Force: [0, 0, 0], rest: [1, 1, 1]},

13: {x: [18, 0], Force: [0, 0, 0], rest: [1, 1, 1]},

14: {x: [1.50, 14.6], Force: [-15.02, 0, 0]},

15: {x: [1.50, 13.1], Force: [0, 0, 0]},

16: {x: [3, 14.6], Force: [-15.02, 0, 0]},

17: {x: [3, 13.1], Force: [0, 0, 0]},

18: {x: [4.5, 14.6], Force: [-15.02, 0, 0]},

19: {x: [4.5, 13.1], Force: [0, 0, 0]},

20: {x: [6, 14.6], Force: [-15.02, 0, 0]},

21: {x: [6, 13.1], Force: [0, 0, 0]},

22: {x: [7.5, 14.6], Force: [-15.02, 0, 0]},

23: {x: [7.5, 13.1], Force: [0, 0, 0]},

24: {x: [9, 14.6], Force: [-15.02, 0, 0]},

25: {x: [9, 13.1], Force: [0, 0, 0]},

26: {x: [10.5, 14.6], Force: [-15.02, 0, 0]},

27: {x: [10.5, 13.1], Force: [0, 0, 0]},

28: {x: [12, 14.6], Force: [-15.02, 0, 0]},

29: {x: [12, 13.1], Force: [0, 0, 0]},

30: {x: [13.50, 14.6], Force: [-15.02, 0, 0]},

31: {x: [13.50, 13.1], Force: [0, 0, 0]},

32: {x: [15, 14.6], Force: [-15.02, 0, 0]},

33: {x: [15.25, 13.1], Force: [0, 0, 0]},

34: {x: [16.50, 14.6], Force: [-15.02, 0, 0]},

35: {x: [16.5, 13.1], Force: [0, 0, 0]}

}

EDE2 = {0: {nodes: [2, 0], mat: mater1, kes: kesit1},

1: {nodes: [1, 5], mat: mater1, kes: kesit1},

2: {nodes: [2, 3], mat: mater1, kes: kesit2},

3: {nodes: [3, 4], mat: mater1, kes: kesit2},

4: {nodes: [4, 5], mat: mater1, kes: kesit2},

5: {nodes: [6, 2], mat: mater1, kes: kesit1},

6: {nodes: [7, 3], mat: mater1, kes: kesit1},

7: {nodes: [8, 4], mat: mater1, kes: kesit1},

8: {nodes: [5, 9], mat: mater1, kes: kesit1},

9: {nodes: [6, 7], mat: mater1, kes: kesit2},

10: {nodes: [7, 8], mat: mater1, kes: kesit2},

11: {nodes: [8, 9], mat: mater1, kes: kesit2},

12: {nodes: [10, 6], mat: mater1, kes: kesit1},

13: {nodes: [11, 7], mat: mater1, kes: kesit1},

14: {nodes: [12, 8], mat: mater1, kes: kesit1},

15: {nodes: [9, 13], mat: mater1, kes: kesit1},

16: {nodes: [0, 14], mat: mater2, kes: kesit3},

17: {nodes: [14, 16], mat: mater2, kes: kesit3},

18: {nodes: [16, 18], mat: mater2, kes: kesit3},

19: {nodes: [18, 20], mat: mater2, kes: kesit3},

20: {nodes: [20, 22], mat: mater2, kes: kesit3},

21: {nodes: [22, 24], mat: mater2, kes: kesit3},

22: {nodes: [24, 26], mat: mater2, kes: kesit3},

23: {nodes: [26, 28], mat: mater2, kes: kesit3},

24: {nodes: [28, 30], mat: mater2, kes: kesit3},

25: {nodes: [30, 32], mat: mater2, kes: kesit3},

26: {nodes: [32, 34], mat: mater2, kes: kesit3},

27: {nodes: [34, 1], mat: mater2, kes: kesit3},

28: {nodes: [0, 15], mat: mater2, kes: kesit3},

29: {nodes: [14, 15], mat: mater2, kes: kesit3},

30: {nodes: [16, 15], mat: mater2, kes: kesit3},

31: {nodes: [16, 17], mat: mater2, kes: kesit3},

32: {nodes: [16, 19], mat: mater2, kes: kesit3},

```
    33: {nodes: [18, 19], mat: mater2, kes: kesit3},
    34: {nodes: [20, 19], mat: mater2, kes: kesit3},
    35: {nodes: [20, 21], mat: mater2, kes: kesit3},
    36: {nodes: [20, 23], mat: mater2, kes: kesit3},
    37: {nodes: [22, 23], mat: mater2, kes: kesit3},
    38: {nodes: [24, 23], mat: mater2, kes: kesit3},
    39: {nodes: [24, 25], mat: mater2, kes: kesit3},
    40: {nodes: [24, 27], mat: mater2, kes: kesit3},
    41: {nodes: [26, 27], mat: mater2, kes: kesit3},
    42: {nodes: [28, 27], mat: mater2, kes: kesit3},
    43: {nodes: [28, 29], mat: mater2, kes: kesit3},
    44: {nodes: [28, 31], mat: mater2, kes: kesit3},
    45: {nodes: [30, 31], mat: mater2, kes: kesit3},
    46: {nodes: [32, 31], mat: mater2, kes: kesit3},
    47: {nodes: [32, 33], mat: mater2, kes: kesit3},
    48: {nodes: [32, 35], mat: mater2, kes: kesit3},
    49: {nodes: [34, 35], mat: mater2, kes: kesit3},
    50: {nodes: [1, 35], mat: mater2, kes: kesit3},
    51: {nodes: [15, 17], mat: mater2, kes: kesit3},
    52: {nodes: [17, 19], mat: mater2, kes: kesit3},
    53: {nodes: [19, 21], mat: mater2, kes: kesit3},
    54: {nodes: [21, 23], mat: mater2, kes: kesit3},
    55: {nodes: [23, 25], mat: mater2, kes: kesit3},
    56: {nodes: [25, 27], mat: mater2, kes: kesit3},
    57: {nodes: [27, 29], mat: mater2, kes: kesit3},
    58: {nodes: [29, 31], mat: mater2, kes: kesit3},
    59: {nodes: [31, 33], mat: mater2, kes: kesit3},
    60: {nodes: [33, 35], mat: mater2, kes: kesit3}

    }


def KLE2(e):
    E = e[mat]['E']
```

```
    A = e[kes]['A']
    I = e[kes]['I']
    L = LL(e)
    L2 = L*L
    L3 = L2* L
    EA_L = E *A / L
    EI_L3 = E*I/L3
    EI_L2 = E * I / L2
    EI_L = E * I / L

    return np.asarray([EA_L, 0,        0,       -EA_L,    0,       0,
                       0,   12*EI_L3,  6*EI_L2,  0,      -12*EI_L3, 6*EI_L2,
                       0,   6*EI_L2,   4*EI_L,   0,      -6*EI_L2,  2*EI_L,
                      -EA_L, 0,        0,       EA_L,    0,       0,
                       0,   -12*EI_L3, -6*EI_L2, 0,      12*EI_L3, -6*EI_L2,
                       0,   6*EI_L2,   2*EI_L,   0,      -6*EI_L2,  4*EI_L]).reshape(6,6)




def KGE2(e): return T(e).T@KLE2(e)@T(e)



for n in NDE2.values():
    n[dof] = [0]*NODOF
    n[Force] = n.get(Force, [0]*NODOF)
    n[rest] = n.get(rest, [0]*NODOF)

M = 0
N = 0
for kere in [0, 1]:
    for n in NDE2.values():
        for i, r in enumerate(n[rest]):
            if r==kere: n[dof][i]=M; M+=1
```

```
    if kere==0: N = M

for e in EDE2.values():
    e[code] = []
    for n in e[nodes]: e[code] += NDE2[n][dof]


print('Assembling System Equations....')

RHSE2 = np.zeros(M)
LHSE2 = np.zeros(M)
AE2 = np.zeros(M**2).reshape(M,M)


for e in EDE2.values():
    columns = np.asarray(e[code]*EDIM)
    rows = columns.reshape(EDIM, EDIM).T.reshape(EDIM**2)
    AE2[rows, columns] += KGE2(e).reshape(EDIM**2)


for n in NDE2.values():
    for i, d in enumerate(n[dof]): RHSE2[d] = n[Force][i]



print('Solving System Equations....')

LHSE2[0:N] = np.linalg.solve(AE2[0:N,0:N], RHSE2[0:N])
RHSE2[N:M] = AE2[N:M,:]@LHSE2

print('-E_displacement: ',LHSE2[0:N])
print('-E_support reactions: ',RHSE2[N:M])
print('-E_node reactions: ', RHSE2[0:M])
print("-Eartquake Load DONE...")
```

```
##############################G+Q+E
Combination##########################################

RHS1 = np.zeros(M)
LHS1 = np.zeros(M)

LHS1[0:N] = LHSG[0:N]+LHSQ[0:N]+LHSE[0:N]
RHS1[N:M] = RHSG[N:M]+RHSQ[N:M]+RHSE[N:M]
RHS1[0:M] = RHSG[0:M]+RHSQ[0:M]+RHSE[0:M]

print('G+Q+E_displacement: ',LHS1[0:N])
print('G+Q+E_support reactions: ',RHS1[N:M])
print('G+Q+E_node reactions: ', RHS1[0:M])
print("G+Q+E Combination DONE...")

#########################G+Q-E
Combination##################################################


RHS2 = np.zeros(M)
LHS2 = np.zeros(M)

LHS2[0:N] = LHSG[0:N]+LHSQ[0:N]+LHSE2[0:N]
RHS2[N:M] = RHSG[N:M]+RHSQ[N:M]+RHSE2[N:M]
RHS2[0:M] = RHSG[0:M]+RHSQ[0:M]+RHSE2[0:M]


print('G+Q-E_displacement: ',LHS2[0:N])
print('G+Q-E_support reactions: ',RHS2[N:M])
print('G+Q-E_node reactions: ', RHS2[0:M])
print("G+Q-E Combination DONE...")
```

```
#####################1.4G+1.6Q
Combination#####################################

RHS3 = np.zeros(M)
LHS3 = np.zeros(M)

LHS3[0:N] = 1.4*LHSG[0:N]+1.6*LHSQ[0:N]
RHS3[N:M] = 1.4*RHSG[N:M]+1.6*RHSQ[N:M]
RHS3[0:M] = 1.4*RHSG[0:M]+1.6*RHSQ[0:M]

print('1.4G+1.6Q_displacement: ',LHS3[0:N])
print('1.4G+1.6Q_support reactions: ',RHS3[N:M])
print('1.4G+1.6Q_node reactions: ', RHS3[0:M])
print("1.4G+1.6Q Combination DONE...")

###########################09G+E
Combination##################################

RHS4 = np.zeros(M)
LHS4 = np.zeros(M)

LHS4[0:N] = 0.9*LHSG[0:N]+LHSE[0:N]
RHS4[N:M] = 0.9*RHSG[N:M]+RHSE[N:M]
RHS4[0:M] = 0.9*RHSG[0:M]+RHSE[0:M]

print('0.9G+E_displacement: ',LHS4[0:N])
print('0.9G+E_support reactions: ',RHS4[N:M])
print('0.9G+E_node reactions: ', RHS4[0:M])
print("0.9G+E Combination DONE...")

###########################0.9G-E
Combination################################

RHS5 = np.zeros(M)
```

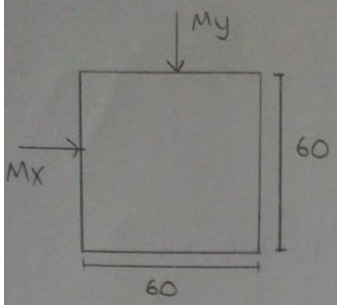
```
LHS5 = np.zeros(M)

LHS5[0:N] = 0.9*LHSG[0:N]+LHSE2[0:N]
RHS5[N:M] = 0.9*RHSG[N:M]+RHSE2[N:M]
RHS5[0:M] = 0.9*RHSG[0:M]+RHSE2[0:M]

print('0.9G+E_displacement: ',LHS5[0:N])
print('0.9G+E_support reactions: ',RHS5[N:M])
print('0.9G+E_node reactions: ', RHS5[0:M])
print("0.9G+E Combination DONE...")
```

15. Detail Design

Column (60x60) Reinforcement calculation (T=0,2s)



125. Column (Third Story)

At the beginning, Turkish seismic code state that two seismic modification factor, R=8 and R=2. However; In the project R=6 is assumed. In the cantilever column design, the code state that overstrength factor should be taken D=3.

Therefore; w value could not read from the graph. The reinforcement is selected according to Asmin.

$A_{smin} = 0{,}01 \cdot 600 \cdot 600 = 3600\ mm^2$

$12\,\Phi 20\ (3769{,}9\ mm^2)$

$A_{smax} = 0{,}04 \cdot 600 \cdot 600 = 144{,}00\ mm^2$

Controls to meet High Ductility Level conditions

1. Cross Section condition

- 600mm ≥ 300mm ; in a cross section, shorter edge should not be less than 300mm. (7.3.1.1.)
- $A_c \geq N_{dm} / (0{,}4 \cdot f_{ck})$ $\quad N_{dm} = 143{,}143\ kN$

$A_c = 600 \cdot 600 = 360000 \geq 143{,}143 / (0{,}4 \cdot 30 \cdot 10^{-3}) = 11928{,}58\ mm^2$

(7.3.1.2) (✓)

2. Transverse Reinforcement (conditional)

$k_1 = 1{,}8 \cdot 10^{-3}\ m^4$

784,17 kNm = Mp

$k_2 = 2{,}7 \cdot 10^{-3}\ m^4$

$M_p = 1{,}43 \cdot (784{,}70) = 1122{,}12\ kNm$

$M_{down} = 1122{,}12 \cdot \dfrac{2{,}7 \cdot 10^{-3}}{(2{,}7 + 1{,}8) \cdot 10^{-3}} = 673{,}27\ kNm$

$M_{up} = 0\ kNm$

$V_e = \dfrac{M_{up} + M_{down}}{h} = \dfrac{673{,}27}{6} = 112{,}21\ kN$

$V = 112{,}21\ kN\ (G + Q - 3E_x - 0{,}3E_y)$

so; we select $\boxed{V_e = 112{,}21\ kN}$

i. $V_e \leq V_r$ (7.3.7.5)

ii. $V_e \leq 0.85 . A_w . \sqrt{f_{ck}}$ (7.7.)

$V_e = 112,21 \, kN$

$V_c = 0,8 . 0,65 . f_{ctd} . b_w . d = 0,8 . 0,65 . 1,25 . 600 . 580 = 226,2 \, kN$

$V_r = V_c + V_w$

$V_e - V_c \leq V_w \rightarrow 112,21 - 226,2 \leq V_w$

so that stirrup is designed according to min condition.

In a confined zone; $s \leq 150mm$; $\frac{600}{3} = 200mm$; $6 . 20(\phi) = 120mm$

$s \geq 50mm$

$s = 120mm$ (confined zone) (7.3.4.1.a)

In a unconfined zone; $s \leq 200mm$, $12 . 20(\phi) = 240mm$ (TS500) (7.4.1)

$s = 200mm$ (unconfined zone)

2 leg $\phi 10$ (stirrup)

- $\frac{A_{sw}}{s} \geq 0,3 . \frac{f_{ctd}}{f_{ywd}} . b_w = \frac{2.79}{200} \geq 0,3 . \frac{1,25}{365} . 600$

$0,79 \geq 0,62$ (✓) (TS500)

- $V_d \leq 0,22 . f_{cd} . b_w . d = 0,22 . 20 . 600 . 580 = 1531,2 \, kN$

$V_d = 112,21 < 1531,2 \, kN$

overlop should not be less than $1{,}5 . \ell_b$ (TS500)

$$\ell_b = 0{,}12 \frac{f_{yd}}{f_{ctk}} . \phi \geq 20\phi$$

$$\ell_b = 0{,}12 . \frac{365}{1{,}25} . 20 \geq 20 . 20$$

$$\ell_b = 700{,}8 mm \geq 400 mm$$

$$\ell_b = 70 cm \cong 71 cm$$ (TS500 - 3.1.2.a)

$$\ell_b = 1{,}5 . 71 = 106{,}5 cm$$

overlop length = 110cm (7.3.3.1)

overlop length should not be more than $\frac{600}{3} = 200mm, 150mm$

Distance between stirrup = 120mm (✓) (7.3.3.1)

- 135° (stirrup and crosstie hooks) (7.3.4) (✓)

- confined zone length should be less than two times of longer edge of column.

2 . 60 = 120 cm confined zone length = 120 mm (✓) (7.3.4.1.)

cross tie

$S_c < \min(b_{min}/3 = 600/3 = 200mm,\ 100mm)$

$S_c = 100mm$ $\quad \emptyset 10$

$A_{cl} = 560.560 - 12.\frac{20^2}{4}.\pi - 4.560.2.10$

$A_{cl} = 265030{,}08 mm^2$

$b_l = b - 2d' = 600 - 2.20 = 560mm$

$N_d \leq 0.2.A_c.f_{ck} = 2160kN$ (Earthquake Revolution 2017) (7.3.4.1.d)

$N_d = \quad \leq 2160kN$ (✓)

$A_{sh} \geq 0{,}3\ s_c.b_l\left[\frac{A_c}{A_{cl}} - 1\right].\frac{f_{ck}}{f_{ywd}} = 0{,}3.100.560.\left[\frac{600.600}{265030{,}08} - 1\right].\frac{30}{420}$

$A_{sh} \geq 43000mm^2$ (✓)

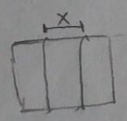
$A_{sh} = 43000\ mm^2 \rightarrow \emptyset 10$ (2 leg stirrup) + $2\emptyset 10$ (cross tie)

$A_{sh}' = \frac{2}{3}.43000 = 286{,}66mm^2 < 2.79 + 157 = 315mm^2$

(7.3.4.1.d)

$A_{sh} \geq 0{,}075.s_c.b_l\ (f_{ck}/f_{ywd}) = 0{,}075.100.560.30/420$

$A_{sh} \geq 300mm^2$

Two cross tie 

$x = \frac{560}{3} = 186{,}67cm \leq 25.10(\emptyset) = 250cm$

diameter of stirrup

(7.3.4.2)

Column middle zone → Dontuse stirrup less than ϕ8

Distance between stirrup and cross tie should not be more than

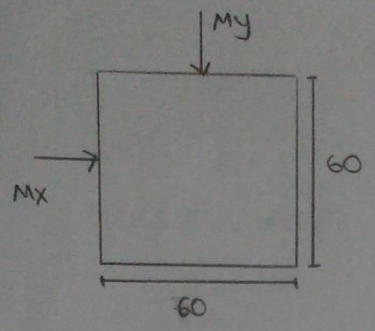
$\frac{600}{2} = 300mm$ and 200mm (7.3.4.2) (✓)

P-M-M Curve

from P-M-M curve of the section that has 12Φ20 bars, the point of PM was out of P-M capacity curve therefore section bars are selected as 16Φ26.

Columns

37. Column (First Story)



$n = \frac{N}{b.h.f_{cd}}$ $\quad m_x = \frac{|M_x|}{b\,h^2 f_{cd}}$ $\quad m_y = \frac{|M_y|}{b^2.h.f_{cd}}$

$A_{smin} = 0{,}01.\ 600.\ 600 = 3600 mm^2$

$A_{smax} = 0{,}04.\ 600.\ 600 = 14400 mm^2$

Therefore; w value couldnot read from the graph. The reinforcement is selected according to Asmin.

$12 \phi 20$ ($3769{,}9\ mm^2$)

Controls to meet High Ductility Level Conditions

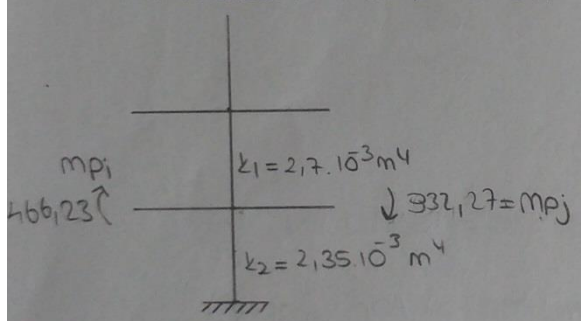
1. Cross Section condition

- $600mm \geq 300mm$; In a cross section, shorter edge should not be less than 300mm (7.3.1.1.)
- $A_c \geq N_{dm} / (0{,}4.f_{ck})$ $\quad N_{dm} = 2237{,}59\ kN$

  $A_c = 600.\ 600 = 360000 \geq 2237.59 / (0{,}4.30.10^{-3}) = 186465{,}8$

  (7.3.1.2.)

2. Transerve Reinforcement



$\Sigma M_p = (1{,}43)(932{,}27 + 466{,}23) = 1399{,}85\ kNm$

$M_{up} = 1399{,}85 \cdot \frac{2{,}35.10^{-3}}{(2{,}35 + 2{,}7).10^{-3}} = 930{,}6\ kNm$

$M_{down} = (1{,}43)(436{,}23) = 623{,}81\ kNm$

$V_e = \frac{M_{up} + M_{down}}{l_n} = \frac{930{,}6 + 623{,}81}{4{,}6 - 0{,}6} = 388{,}60\ kN$

$V = 842{,}337\ kN$ $(G + Q - 0{,}3E_x + 3E_y)$

So; we select $\boxed{V_e = 388{,}60 kN}$

i. $V_e \leq V_r$

ii. $V_e \leq 0{,}85 . A_w . \sqrt{f_{ck}}$ (7.3.75)
(Equation 7.7)

$V_e = 388{,}6$ kN

$V_c = 0{,}8 (0{,}65 . f_{ctd} . b_w . d) = 0{,}8 . 0{,}65 . 1{,}25 . 600 . 580 = 226{,}2$ kN

$V_r = V_c + V_w$

$V_e - V_c \leq V_w \rightarrow 388{,}6 - 226{,}2 \leq V_w$

$162{,}40 \leq V_w$

ii. $0{,}85 . 560 . 560 . \sqrt{30} = 1460000$ kN

$388{,}6 \leq 1460000$ (✓)

$V_w \geq 71{,}93$ kN $= \frac{2.79}{S} . 365 . 580$ } 2 kollu $\phi 10$
$S = 46{,}5$ cm
$S = 20$ cm (sorgusuz)
$S = 10$ cm (sorgulu) } stirrup

- $\frac{A_{sw}}{S} \geq 0{,}3 . \frac{f_{ctd}}{f_{ywd}} . b_w \Rightarrow \frac{2.79}{200} \geq 0{,}3 . \frac{1{,}25}{365} . 600$

$0{,}79 \geq 0{,}62$ (✓) (TS500)

- $V_d \leq 0{,}22 . f_{cd} . b_w . d = 0{,}22 . 20 . 600 . 580 = 1531{,}2$ kN

$V_d = 388{,}60 < 1531{,}2$ kN (✓) (Allowable shear force)

- Overlap should not be less than $1{,}5 . \ell_b$ (TS500)

$$\ell_b = 0{,}12 . \frac{f_{yd}}{f_{cdt}} . \phi \geq 20\phi$$

$$\ell_b = 0{,}12 . \frac{365}{1{,}25} . 20 \geq 20 . 20$$

$$\ell_b = 700{,}8 \text{ mm} \geq 400 \text{mm}$$

$\ell_b = 70 \text{cm} \cong 71 \text{cm}$ (TS500 - 9.1.2 (a))

$\ell_b = (1{,}5) . 71 = 106{,}5 \text{cm}$

overlap length = 110cm (7.3.3.1)

$\frac{600}{3} = 200\text{mm}$ and 150mm ; overlap length should not be more than those values.

Distance between stirrup = 100mm (✓) (7.3.3.1)

- 135° (stirrup and cross tie) (7.3.4) (✓)

- Confined zone length should not be less than $\frac{4600}{6} = 766{,}67\text{mm}$, $600 . 1{,}5 = 900\text{mm}$, 500mm, overlap = 110mm

confined zone length is selected "110 mm". (7.3.4.1) (✓)

- Number of cross tie in a confined zones is "3".
in a other zones is "2".

Cross'tie

$S_c < \min(b_{min}/3 = 600/3 = 200mm, 100mm)$

$S_c = 100mm \quad \phi 10$

$A_{ck} = 560.560 - 12.\frac{20^2}{4}.\pi - 4.560.2.10$

$A_{ck} = 265030,08\ mm^2$

$b_k = b - 2d' = 600 - 2.20 = 560mm$

$N_d \leq 0,2.A_c.f_{ck} = 2160kN$

(Earthquake Revolution 2017)

$N_d = 1611,67 \leq 2160kN$ (✓) (7.3.4.1.d)

$A_{sh} \geq 0,3.S_c.b_k\left[\frac{A_c}{A_{ck}} - 1\right]\frac{f_{ck}}{f_{ywd}} = 0,3.100.560.\left[\frac{600.600}{265030,08} - 1\right].\frac{30}{4}$

$A_{sh} \geq 430,00mm^2$ (✓)

$A_{sh} = 430\,00mm^2 \longrightarrow \phi 10$ (iki kollu - etriye) + $2\phi 10$ (ciroz)

$A_{sh}' = \frac{2}{3}.430\,00 = 286,66mm^2 < 2.79 + 157 = 315mm^2$

(7.3.4.1.d)

$A_{sh} \geq 0,075.S_c.b_k(f_{ck}/f_{ywk}) = 0,075.100.560.30/420$

$A_{sh} \geq 300mm^2$

- Column middle zone → Dont use stirrup less than $\phi 8$ (✓)
- Distance between stirrup and cross-tie should not be more than $\frac{600}{2} = 300mm$ and 200mm. (✓) (7.3.4.2)
- Vertical distance of cross-tie should not be more 25 times of stirrup diameter.

$$\left.\begin{array}{l} 10 \cdot 25 = 250mm \\ \frac{560}{3} = 168,8\,mm \end{array}\right\} (7.3.4.2) \quad (✓)$$

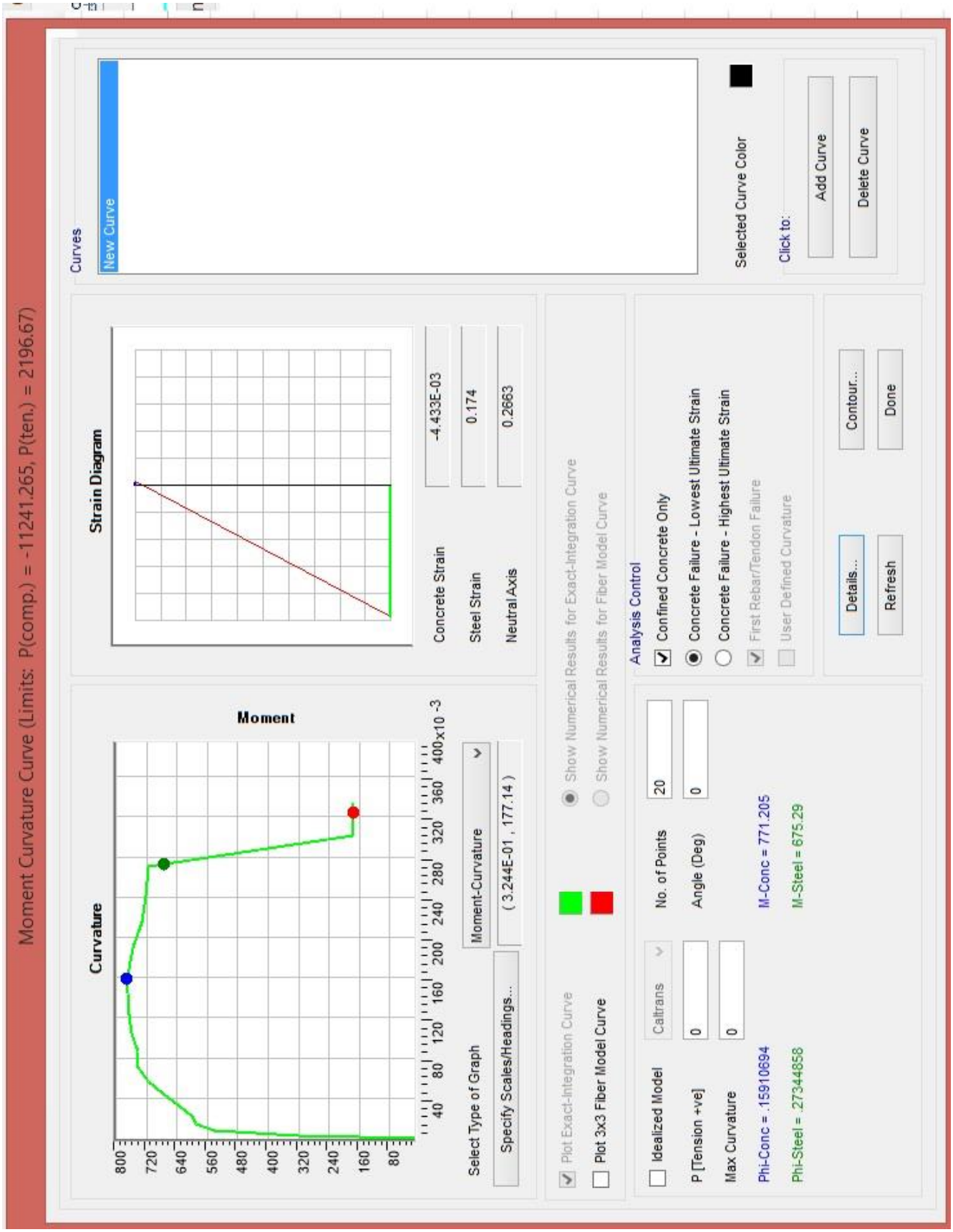
- Columns capasity is stronger than beam capasity (Bearing capasity)

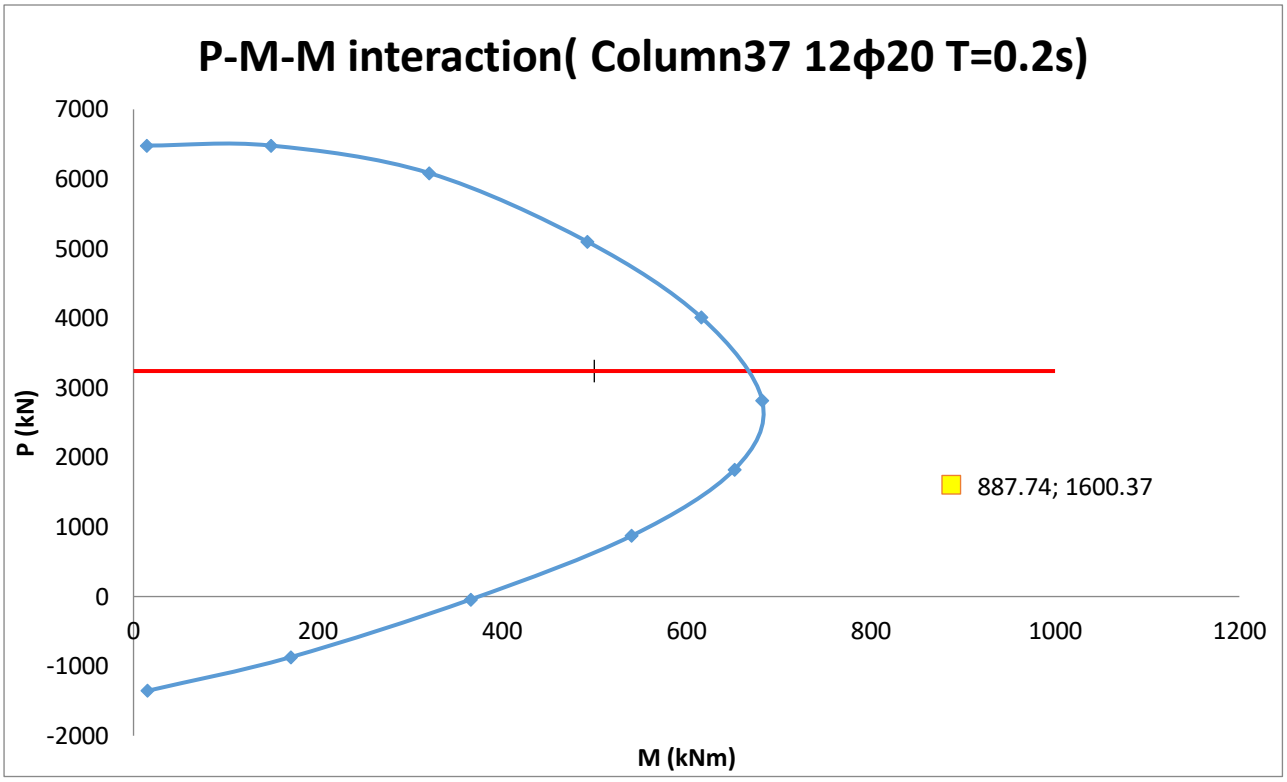
$$M_{ra} + M_{rü} \geq 1.2\,(M_{rj} + M_{ri})$$

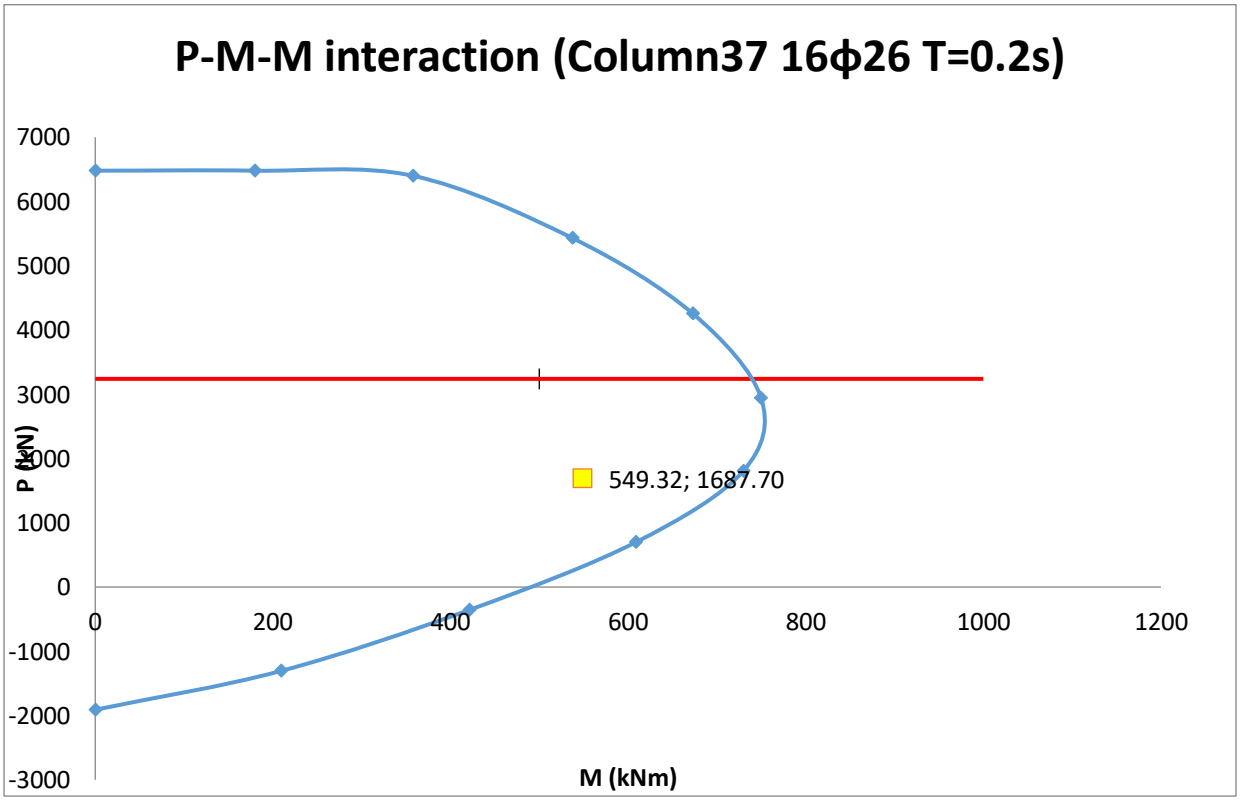
$$1286,76 \geq 1025,52 \qquad (7.3.5.1) \quad (✓)$$

ϕ P-M-M Curve

From P-M-M curve of 37. colum, in 12Φ20 bars section, the P-M point was out of the P-M capacity curve therefore, section bars are selected as 16Φ28.

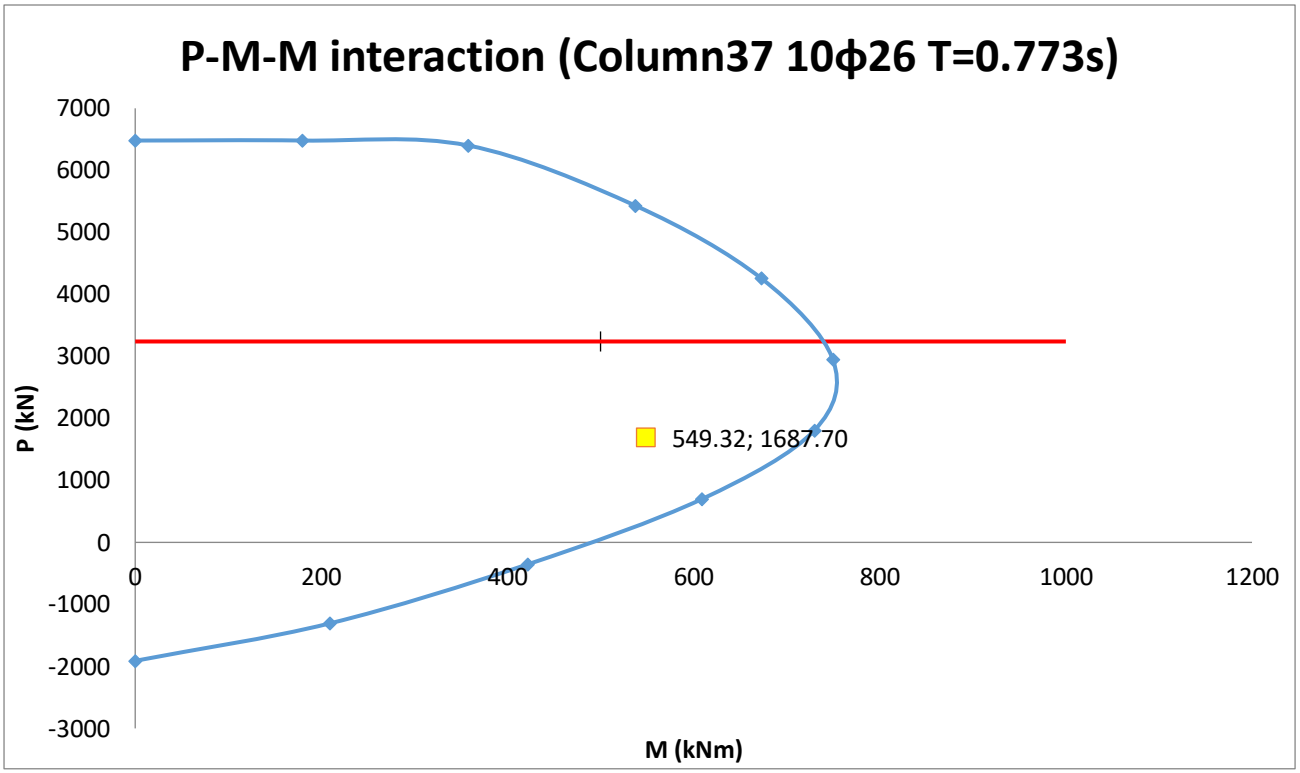

Moment Curvature Curve (Limits: P(comp.) = -11241.265, P(ten.) = 2196.67)
Curvature
Moment
800
720
640
560
480
400
320
240
160
80
40
80
120
160
200
240
280
320
360
400x10 -3
Select Type of Graph
Moment-Curvature
Specify Scales/Headings...
(3.244E-01 , 177.14)
Plot Exact-Integration Curve
Plot 3x3 Fiber Model Curve
Show Numerical Results for Exact-Integration Curve
Show Numerical Results for Fiber Model Curve
Idealized Model
Caltrans
No. of Points
20
P [Tension +ve]
0
Angle (Deg)
0
Max Curvature
0
Phi-Conc = .15910694
M-Conc = 771.205
Phi-Steel = .27344858
M-Steel = 675.29
Strain Diagram
Concrete Strain
-4.433E-03
Steel Strain
0.174
Neutral Axis
0.2663
Analysis Control
Confined Concrete Only
Concrete Failure - Lowest Ultimate Strain
Concrete Failure - Highest Ultimate Strain
First Rebar/Tendon Failure
User Defined Curvature
Details...
Contour...
Refresh
Done
Curves
New Curve
Selected Curve Color
Click to:
Add Curve
Delete Curve


P-M-M interaction(Column37 12ϕ20 T=0.2s)
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
P (kN)
0
200
400
600
800
1000
1200
M (kNm)
887.74; 1600.37


P-M-M interaction (Column37 16ϕ26 T=0.2s)
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
P (kN)
0
200
400
600
800
1000
1200
M (kNm)
549.32; 1687.70

According to TSC2017 on high ductility level design, we want to ductile failure (bending moment failure) instead of brittle failure (shear failure). Therefore, when we desing reinforcement, we use yeilding moment capasity and ultimate shear capasity. To obtain ultimate shear, we multiply the yeilding moment capasity with a overstrenght factor which came from our moment curvature diagram(figure 18).

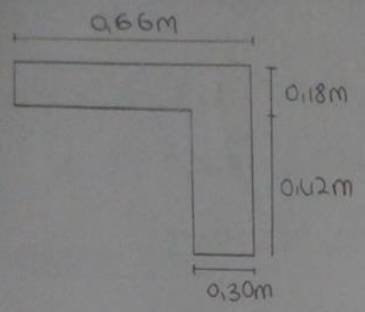
While determining the reinforcement, we looked at the most critical columns which are faced with most bending moment values and less axial force. While evaulating the P-M-M curves, there is two probable problems that we would face with. One of them is exceeding the %30 of the column axial force capasity which will be brittle failure. 60x60 column dimentions are sufficient for this. The capasity is 3240 kN and the most critical axial force is 2394.39 kN. Second one is that while having low axial fore and high moment, P-M-M curve of this section could be exceeded. Firstly, we put 12φ20 for 60x60 column. But, when we checked the P-M-M curve, we saw that on column 37 our column's P-M-M curve was exceeded which will be cause failure of the column. Therefore, we incerased the reinforcement to 16φ26. The update P-M-M curve and previous one shown in figure19 and figure20. This figures contains the yellow points (N-M values) for the EQ forces which is calculated T=0,2s (safe design). The reinforcement ratio is %2.36 which is not applicable for the real life application. Moreover, we want to check tihis section with our real period of building T=0.773 s. The EQ force recalculated and updated to our modal. Then we checked section with 10φ26. The updated N-M values stayed in the border on P-M-M curve. Our updated reinforcement ratio is %1.47 which is suitable for real life applications.


P-M-M interaction (Column37 10ϕ26 T=0.773s)
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
P (kN)
0
200
400
600
800
1000
1200
M (kNm)
549.32; 1687.70

Beam Detail Design

T Shape Beam(60x30) Reinforcement calculation (T=0,2s)

Beam



y-y direction (2. Story / D-D Axis) – span

$M = 168,10$ kNm

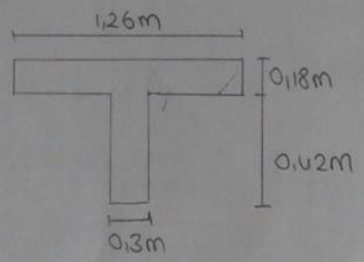
$K = \frac{(0,66).(0,58)^2}{168,10} = 132,1.10^{-5}$

$k_s = 2,84$

$k_x = 0,099$

$A_s = \frac{k_s . M}{d} = \frac{(2,84)(168,10)}{0,58} = 823,11 mm^2$

we choose 3 Ø22 (1140 mm^2)



y-y direction (2. story / E-E Axis) – support

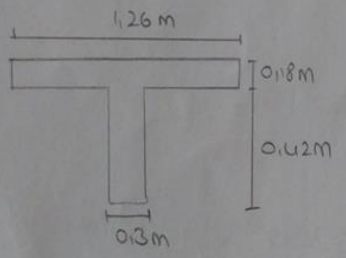
$M = -637,85$ kNm

$K = \frac{(1,26)(0,58)^2}{637,85} = 66,45.10^{5}$

$k_s = 2,96$

$A_s = \frac{k_s . M}{d} = \frac{(2,96)(637,85)}{0,58} = 3255 mm^2$

we choose 9 Ø 22 (3421 mm^2)



y-y direction (1. story / E-E Axis) – support

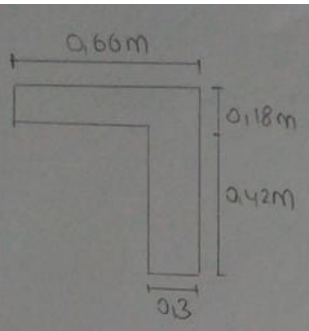
$M = 350,62$ kNm

$K = \frac{(1,26)(0,58)^2}{350,62} = 120,83.10^{-5}$

$k_s = 2.85$

$A_s = \frac{k_s . M}{d} = \frac{(2,85)(350,62)}{0,58} = 1722,87 mm^2$

we choose 5Ø22 (1901 mm^2)



x-x direction (3 Aksi / 2.story) - span

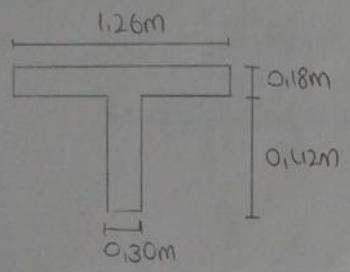
$M = 167,71\ kNm$

$K = \frac{0,66 \cdot 0,58^2}{167,91} = 132,4.10^{-5}$

$ks = 2,84$

$As = \frac{ks.M}{d} = \frac{2,84 \cdot 167,71}{0,58} = 821,20\ mm^2$

we choose $3\phi 22$ ($1140 mm^2$)



x-x direction (2 story) - support

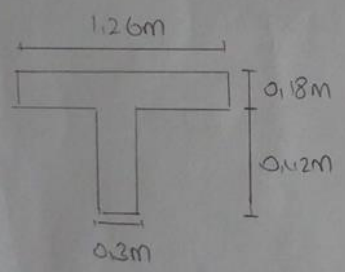
$M = -575,73\ kNm$

$K = \frac{1,26 \cdot 0,58^2}{575,73} = 73,62.10^{-5}$

$ks = 2,88$

$As = \frac{ks.M}{d} = \frac{2,88 \cdot 575,73}{0,58} = 2858,8\ mm^2$

we choose $8\phi 22$ ($3041\ mm^2$)



x-x direction (3.story) - support

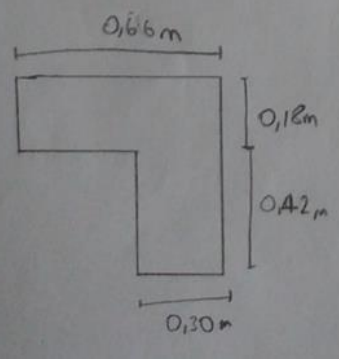
$M = 325,68\ kNm$

$K = \frac{(1,26) \cdot (0,58)^2}{325,68} = 130,15$

$ks = 2,84$

$As = \frac{ks.M}{d} = \frac{(2,84)(325,68)}{0,58} = 1594,71\ mm^2$

we choose $5\phi 22$ ($1901\ mm^2$)



Beam 9 (support)

$M = -658,079\ kNm$

$K = \frac{0,66 \cdot 0,58^2}{658,079} = 33,73\ 10^{-5}$

$ks = 3,05$

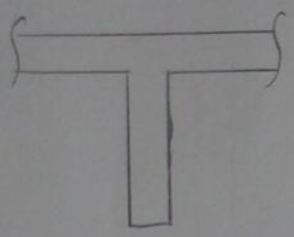
$As = \frac{ks.M}{d} = \frac{(3,05)(658,079)}{0,58} = 3460,58\ mm^2 \rightarrow (3801\ mm^2)$ $10\ \phi 22$

Bearing capasity of Beams

$M_r = A_s \cdot f_{yd} \cdot (d - d')$

$d - d' = 0,58 - 0,02 = 0,56 m$

Internal Beam



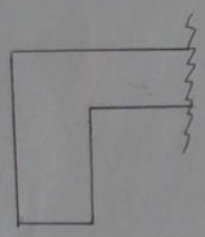
Top reinforcement : $3041 mm^2$ ($8\phi22$)

Bottom Reinforcement. $1521 mm^2$ ($4\phi22$)

$\rightarrow M_r^- = 3041 \cdot 365 \cdot 560 \cdot 10^{-6} = 621,58 kNm$

$\rightarrow M_r^+ = 1521 \cdot 365 \cdot 560 \cdot 10^{-6} = 310,89 kNm$

External Beam



Top Reinforcement : $3801 mm^2$ ($10\phi22$).

Bottom Reinforcement $1140 mm^2$ ($3\phi22$)

$\rightarrow M_r^- = 3801 \cdot 365 \cdot 560 \cdot 10^{-6} = 757,76 kNm$

$\rightarrow M_r^+ = 1140 \cdot 365 \cdot 560 \cdot 10^{-6} = 233,01 kNm$

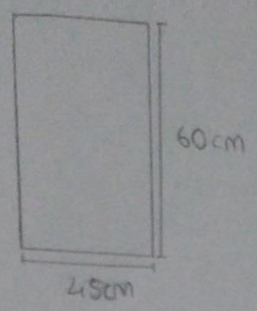
Calculation of beam reinforcement made according to most critical load combination. Efective flange lenght is considered while the calculation of moment of inertia of the beam section(as T section). The calculation is made for T=0,2 s which is for safe design.

Calculation shows that reinforcemnt ratio is %3.92. It is not suitable for the application to real life. Because on the connection point of column and beam, there are lots of reinforcement which are clashed. According to Turkish Seismic code 7.5.2.2, in beam column connection high ductulity level does not be satisfied. Therefore, the beam size is increased to 60x45 from 60x30. Moreover, on SAP2000 we picked up the first period of the building as T=0,773 s. According to new period, the EQ forces is recalculated.

Section of beam revised as 450x600 because 300x600 section does not meet TSC2017 part 7.4.2.2.

Beam

47 Beam



y-y direction (2. story /D-D Axis) - span

$M = 183,41$ kNm

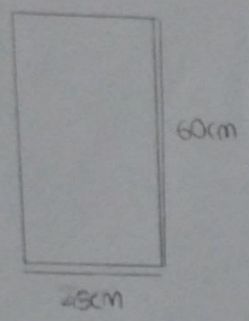
$K = \frac{(0,4)(0,58)^2}{183,41} = 74,6.10^{-5}$

$k_s = 2,88$

$A_s = \frac{k_s M}{d} = \frac{(2,88)(183,41)}{0,58} = 895,63\ mm^2$

we choose $5\ \phi 22\ (1901\ mm^2)$

support



$M = -522,29$ (left) $- 536,09$ (right)

$K = \frac{0,4\ 0,58^2}{536,09} = 25,34.10^{-5}$

$k_s = 3,2$

$A_s = \frac{k_s M}{d} = \frac{(3,20)(536,09)}{0,58} = 2930,13\ mm^2$

we choose $9\ \phi 22\ (3421\ mm^2)$

Bearing capasity

Top reinforcement : $3421\ mm^2$ ($9\ \phi 22$)

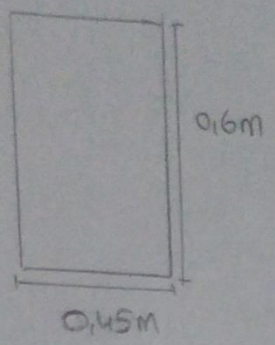
Bottom Reinforcement = $5\ \phi 22$ (1301)

$\rightarrow m_r^- = 3421 \cdot 365 \cdot (560) \cdot 10^{-6} = 699,25$ kNm

$\rightarrow m_r^+ = 1301 \cdot 365 \cdot 560 \cdot 10^{-6} = 388,56$ kNm

($m_r = A_s \cdot f_{yd} (d - d')$
$d - d' = 0,58 - 0,02 = 0,56m$)

204. Beam



X-X direction 3-3 Axis - E/F (2 Story) - span

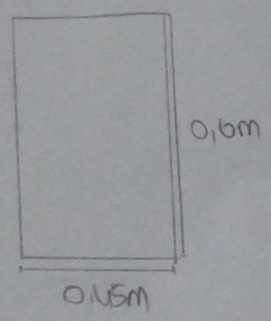
$M = 183,38$ kNm

$K = \frac{(0,4)\ 0,58^2}{183,38} = 74,83$

$ks = 2,88$

$As = \frac{ks\ M}{d} = \frac{(2,88).(183,38)}{0,58} = 892,9\ mm^2$

we choose $4\Phi 22$ ($1521 mm^2$)



X-X direction - support

$M = 486,16$ (left) $- 465,66$ (right)

$K = \frac{(0,4)(0,58)^2}{486,16} = 27,74$

$ks = 3,14$

$As = \frac{ks.\ M}{d} = \frac{(3,14).(486,16)}{0,58} = 2659,1\ mm^2$

we choose $8\Phi 22$ ($3041 mm^2$);

Bearing capasity

Top Reinforcement = $8\Phi 22$ ($3041 mm^2$)

Bottom Reinforcement $4\Phi 22$ ($1521 mm^2$)

$\rightarrow Mr^- = 3041 . 365 . 560.10^{-6} = 621,58$ kNm

$\rightarrow Mr^+ = 1521 . 365 . 560.10^{-6} = 310,83$ kNm

Beam

Controls to meet High Ductility Level condition

- Cross section condition

$b = 450\,mm \geq 250\,mm$ (7.4.1.1. a) (✓)

$b = 450\,mm \leq 600 + 600 = 1200\,mm$

$h = 600\,mm \geq 3.180 = 540\,mm$ (7.4.1.1. b) (✓)

$h = 600\,mm \geq 300\,mm$

$h = 600\,mm \leq \frac{6000}{4} = 1500\,mm$ (7.4.1.1. c) (✓)

$N_d = 21{,}085 \leq 0{,}1 . A_c . f_{ck} = 0{,}1 . 600 . 300 . 20 . 10^{-3} = 360\,kN$

(7.4.1.2) (✓)

- Longidutunal Reinforcement condition

- $\rho \geq 0{,}6 . f_{ctk} / f_{yd} = 0{,}6 . 1{,}25/365 = 2{,}05 . 10^{-3}$

  $\rho = \frac{A_s}{b_w . d} = \frac{4561}{600 . 450} = 1{,}9 . 10^{-2} > 2{,}05 . 10^{-3}$ (7.4.2.1) (✓)

- $\phi = 22 \geq \phi(12)$ (7.4.2.2) (✓)

- (DTS 1a) → seismic Design class

  Top Reinforcement = 12Ø22
  Bottom " = 6 Ø22 } $6\phi22 / 12\phi22 = 0{,}50 \geq \%50$

  (7.4.2.3) (✓)

- $\rho = \frac{A_s}{b_w . d} = 1{,}90\ 10^{-2} = \%1{,}9 < \%2$ (✓)

  (7.4.2.4)

  (TS500) (7.3) (✓)

Transverse Reinforcement condition

Confined zone Length = 2.600 = 1200mm

$V_e = V_{dy} \mp (M_{pi} + M_{pj})/l_n$ (7.4.5.1)

$V_{dy} = 132,575$ kN

$V_e = 132,575 \mp [1.52(699,25 + 388,56)] / (600 - 2.30).10^{-2}$

$\boxed{V_{e_1} = 4,38,77 \text{ kN}}$

$V_{e_2} = -173,62$ kN

* In the Turkish Earthquake code 7.4.5 $M_{pi} \cong 1,4 M_{ri}$, from SAP2000, beam moment curvature graph $M_{ultimate}/M_{yield} \cong 1,51$ and in this case this coefficient is taken as 1,52.

$V = 594,89$ kN $(G + Q + 0,3E_x + 3E_y)$ (7.4.5.2.)

$V_{e_1} = 4,38,77 > 594,89$ kN so $V_e = 4,38,77$ kN ✓

- $V_e \leq V_t$ (Equ. 7.10)
- $V_e \leq 0,85\, b_w . d . \sqrt{f_{ck}}$

V_t = Total seismic Load = 6342,24 kN

$V_e = 4,38,77 \leq V_t = 6342,24$ (✓) (7.4.5.2)

$V_e \leq 0,85 . 600 . 300 . \sqrt{20} = 684,24$ kN (✓)

$V_c = 0,8 . 0,65 . f_{ctd} . b_w . d = 0,8 . 0,65 . 1,25 . 300 . 580 = 113,1$ kN

$V_r = V_c + V_w$

$V_e - V_c \leq V_w$ $\quad 4,38,77 - 113,1 \leq V_w$

$325,67 \leq V_w$

$V_u \geq 281{,}35 = \frac{2.113}{s} . 365.580$ } 2 leg $\phi 12$ / 14cm (unconfined)

$V_d = 335{,}05 \geq 3.113{,}1$; $s \leq 380/4 = 145\,mm$ so:

$s = 140\,mm$ (TS500) (8.1.6) (✓)

$s = 100\,mm$ (confined zone) (TS500) (8.1.6) (✓)

$V_d \leq 0{,}22 . f_{cd} . b_w . d = 0{,}22 . 20 . 300 . 580 = 765{,}6\,kN$

$V_d = 335{,}05 < 765{,}6$ (✓) (Allowable shear force)

Controls to meet High Ductility Level in column-Beam connections

- confined joints $\left(450 \geq 600 \cdot \frac{3}{4}\right)$ (7.5.1.b) (✓)

$V_e = 1{,}25 \cdot f_{yk} \cdot (A_{s1} + A_{s2}) - V_{kol}$ $\qquad V_{kol} = 238{,}13\,kN$

$V_e = 1{,}25 \cdot 420 \cdot (4561.6 + 2280.8) - 238{,}13$

$V_e = 3294.13\ kN$ (7.5.2.1)

- $V_e \leq (1{,}7)\, b_j\, h \sqrt{f_{ck}}$

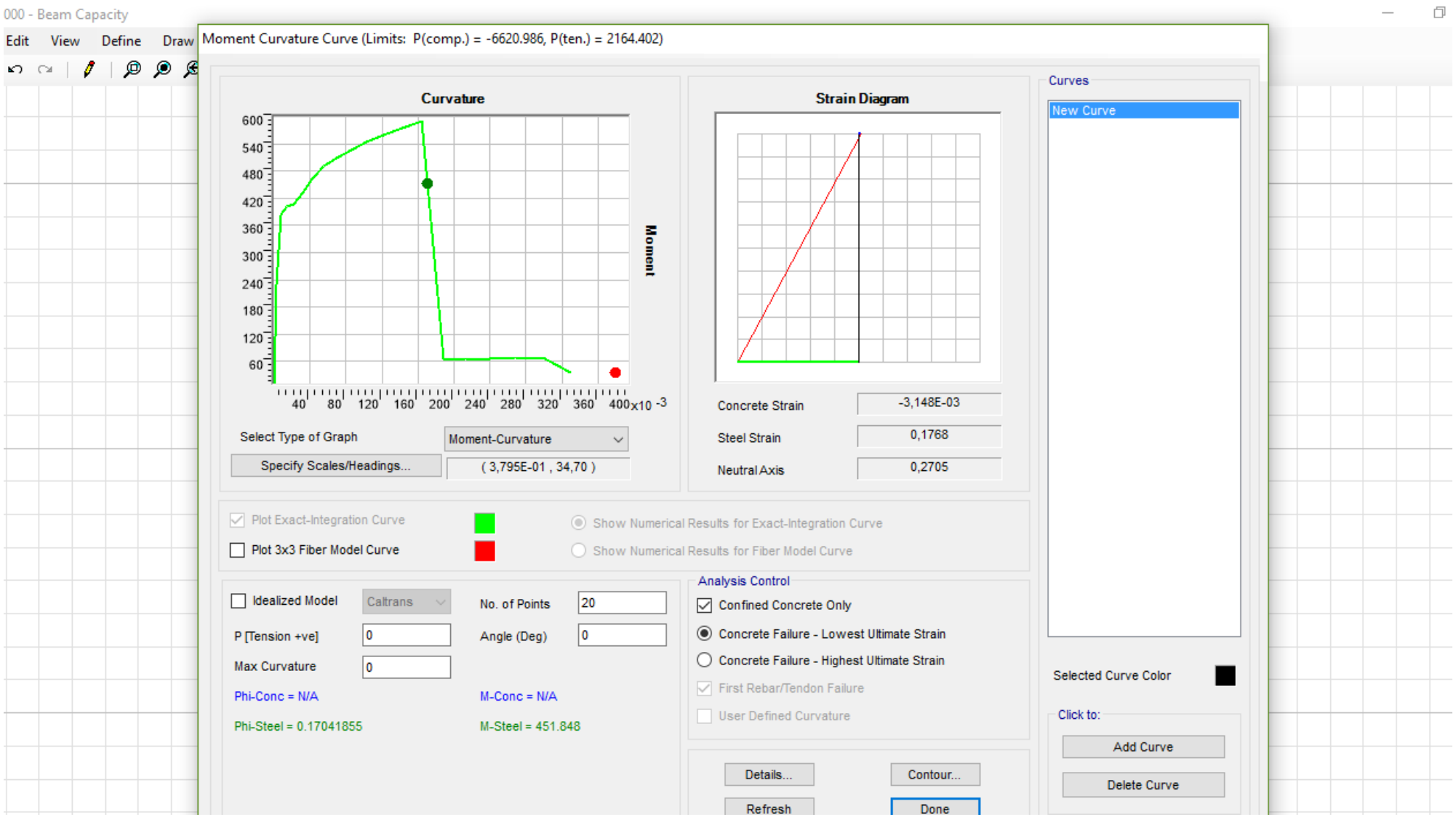
$3294.13 \leq (1{,}7) \cdot (600) \cdot 600 \cdot \sqrt{30}$

$3294.13 \leq 3352{,}06\ kN$ (✓) (7.5.2.2)

- $\phi 10 / 150\,mm$ (beam-column joints stirrup)

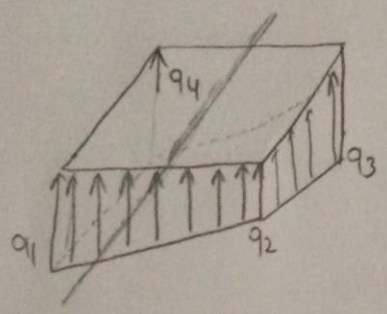
(7.5.2.3-a) ✓

$0{,}33 = \frac{50}{x}$


000 - Beam Capacity
Edit View Define Draw
Moment Curvature Curve (Limits: P(comp.) = -6620.986, P(ten.) = 2164.402)
Curvature
Moment
x10 -3
Strain Diagram
Concrete Strain -3,148E-03
Steel Strain 0,1768
Neutral Axis 0,2705
Select Type of Graph
Moment-Curvature
Specify Scales/Headings...
(3,795E-01 , 34,70)
Plot Exact-Integration Curve
Plot 3x3 Fiber Model Curve
Show Numerical Results for Exact-Integration Curve
Show Numerical Results for Fiber Model Curve
Idealized Model
Caltrans
No. of Points 20
P [Tension +ve] 0
Angle (Deg) 0
Max Curvature 0
Phi-Conc = N/A
M-Conc = N/A
Phi-Steel = 0.17041855
M-Steel = 451.848
Analysis Control
Confined Concrete Only
Concrete Failure - Lowest Ultimate Strain
Concrete Failure - Highest Ultimate Strain
First Rebar/Tendon Failure
User Defined Curvature
Details...
Contour...
Refresh
Done
Curves
New Curve
Selected Curve Color
Click to:
Add Curve
Delete Curve

Foundation Calculations

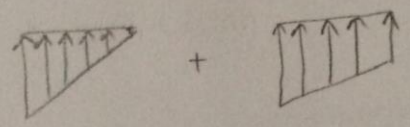
RC calculation of Foundations



correlation between soil stresses ($\sigma_1, \sigma_2, \sigma_3, \sigma_4$)

Then, we selected the Max q (kN/m²)

Max q (kN/m²) = 83,90 kN/m²)



$$= (83{,}90) \cdot \frac{1}{2} \cdot \left(\frac{4{,}1}{2} - 0{,}3\right) \cdot 4{,}1 + (96{,}70 - 83{,}90) \cdot \frac{1}{2} \cdot \left(\frac{4{,}1}{2} - 0{,}3\right) \cdot \frac{2}{3} \cdot 4{,}1$$

$M = 547{,}63$ kNm

$$K = \frac{(4{,}1) \cdot (0{,}7 - 0{,}05)^2}{547{,}63} = 316{,}32$$

$k_s = 2{,}8$

$$A_s = \frac{(2{,}8) \cdot 547{,}63}{0{,}7 - 0{,}05} = 2359{,}00 \text{ mm}^2$$

$A_{s,min} = 0{,}002 \cdot 4{,}1 \cdot (0{,}7 - 0{,}05) = 5330 \text{ mm}^2$

* 18 Ø20 (is choosen) A_s (choosen) = 5654,00 mm²

Control of shear force

$V_{cr} = 0{,}65 \cdot f_{ctd} \cdot b_w \cdot d$

$V(kN) = (96{,}70 + 83{,}9) \cdot 0{,}5 \cdot \left(\frac{4{,}1}{2} - 0{,}3\right) \cdot 4{,}1$

$V_{cr} = 0{,}65 \cdot 1{,}25 \cdot 4{,}1 \cdot 0{,}65 = 2165{,}31$ kN

All shear force of foundations are less than critical shear force of each foundations.

shear forces are controlled according to TS500.

Foundation Design

C14 (60x60)

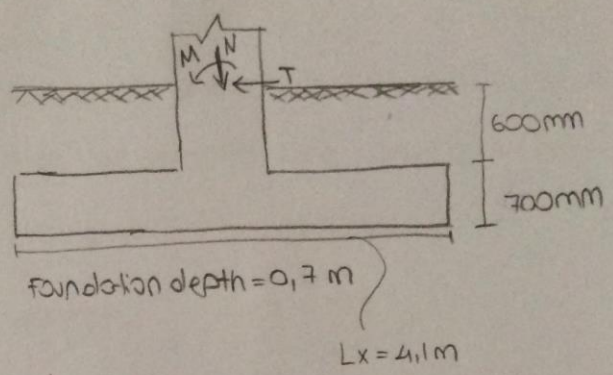
14. Foundation

Y. direction

T = 196,194

M = 564,35

N = 1190,767



$M_O = 564,35 + (0,7).(196,194)$

$M_O = 701,69\ kNm$

$N_d = 1190,767\ kN$

X direction

T = 68,27

M = 117,231

N = 1190,767

$M_O = 117,231 + (0,7).68,27$

$M_O = 225,08\ kNm$

$N_d = 1190,767\ kN$

$\gamma_{soil} = 20\ kN/m^3$

$\sigma_{z,net} = \sigma_{all} - \gamma . t = 180 - 20 . 1,3 = 154\ kN/m^3$

$t = h + \text{freezen depth} = 0,6 + 0,7 = 1,3\ m$

$$\sigma_{1,2,3,4} = \frac{N_d}{A} \mp \frac{M_x}{W_x} \mp \frac{M_y}{W_y}$$ must be less than 154 kN/m^3.

$$\sigma_{1,2,3,4} = \frac{1190,767}{4,1^2} \mp \frac{701,69 . 6}{4,1^3} \mp \frac{225,08 . 6}{4,1^3}$$

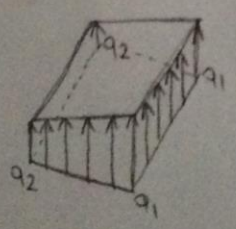
soil stress

$\sigma_1 = 151,52\ kN$

$\sigma_2 = 23,35\ kN$

$\sigma_3 = 112,33\ kN$

$\sigma_4 = -9,84\ kN$



Punching shear control

$d_{mean} = \frac{0,65 + 0,63}{2} = 0,64m$

$b_x = b + d_{mean} = 1,24m$

$b_y = h + d_{mean} = 1,24m$

$u_p = 2 \cdot (b_x + b_y)$ (u_p: punching perimeter)

$u_p = 4,96 m$

$e_x = 0,4 \frac{M_{dx}}{N_{dx}} = 0,059$

$e_{min} = 15mm + 0,03h = 0,033$

$e_y = 0,4 \frac{M_{dy}}{N_{dy}} = 0,189$

$\gamma = 1 / (1 + 1,5((e_x + e_y)/(b_x + b_y))^2) = 0,99$

$V_{pr} = \gamma \cdot f_{ctd} \cdot u_p \cdot d_{mean} = 0,99 \cdot 1,25 \cdot 4,96 \cdot 0,64$

$V_{pr} = 3963,89 kN$

All V_{pr} of columns are bigger than each shear forces

Punching shear is controlled according to TS500.

Y-Direction					
Name	14	17	32	33	31
T(kN)	196.194	195.28	187.87	187.87	174.33
M(kN)	564.35	563.2	538.52	538.51	518.4
N(kN)	1190.767	1124.362	1655.141	1655.143	1403.148
Mo(kN)	701.69	699.90	670.03	670.02	640.43
b	0.60	0.60	0.60	0.60	0.60
h	0.60	0.60	0.60	0.60	0.60
X-Direction					
Name	14	17	32	33	31
T(kN)	68.27	61.975	54.195	54.189	64.031
M(kN)	177.291	154.281	156.045	156.037	183.49
N(kN)	1190.767	1124.362	1655.141	1655.143	1403.148
Mo(kN)	225.08	197.66	193.98	193.97	228.31

Foundation Depth	0.70

Y Direction					
Name	14	17	32	33	31
T(kN)	196.19	195.28	187.87	187.87	174.33
M(kN)	564.35	563.20	538.52	538.51	518.40
N(kN)	1190.77	1124.36	1655.14	1655.14	1403.15
Mo(kN)	701.69	699.90	670.03	670.02	640.43
Lx(m)	4.10	4.10	4.40	4.40	4.20

X Direction					
Name	14	17	32	33	31
T(kN)	68.27	61.98	54.20	54.19	64.03
M(kN)	177.29	154.28	156.05	156.04	183.49
N(kN)	1190.77	1124.36	1655.14	1655.14	1403.15
Mo(kN)	225.08	197.66	193.98	193.97	228.31

Soil Streses					
Name	14	17	32	33	31
Sigma1	151.52	145.02	146.35	146.35	149.90
Sigma2	29.35	23.16	51.96	51.96	46.17
Sigma3	112.33	110.61	119.02	119.02	112.92
Sigma4	-9.84	-11.25	24.64	24.64	9.19

	RC Calculation of Foundations					
	Name	14	17	32	33	31
	q1(kN/m^2)	147.36	141.25	143.68	143.68	146.27
	q2(kN/m^2)	25.19	19.39	49.29	49.29	42.54
	q3(kN/m^2)	116.49	114.39	121.70	121.70	116.55
	q4(kN/m^2)	-5.68	-7.47	27.31	27.31	12.82
	q1-q2(kN/m^2)	96.70	90.72	103.99	103.99	103.05
	q2-q3(kN/m^2)	78.63	75.00	91.25	91.25	85.71
	q3-q4(kN/m^2)	65.83	63.86	82.01	82.01	73.33
	q4-q1(kN/m^2)	54.25	50.01	71.75	71.75	65.45
	q12-q34(kN/m^2)	83.90	84.20	98.64	98.64	95.82
	q23-q41(kN/m^2)	68.52	64.63	83.05	83.05	77.27
	Maxq(kN/m^2)	83.90	84.20	98.64	98.64	95.82
	M(kN)	547.63	524.60	769.56	769.56	647.24
	K	316.32	330.21	241.57	241.57	274.16
Read From RC Tables	ks	2.80	2.80	2.82	2.82	2.81
	As(req)(mm^2)	2359.00	2259.80	3332.81	3338.71	2798.07
	Asmin(mm^2)	5330.00	5330.00	5720.00	5720.00	5460.00
	As(choosen)	18Φ20	18Φ20	19Φ20	19Φ20	18Φ20
		5654.00	5654.00	5969.00	5969.00	5654.00

Control of Shear Force					
Vcr=0,65*fctd*bw*d					
	14	17	32	33	31
V(kN)	629.40	609.60	824.69	824.69	730.85
Vcr	2165.31	2165.31	2323.75	2323.75	2218.13

Punching Shear Control					
Vpr=ɣ*fctd*Up*dmean Up: Perimeter of punching region					
	14	17	32	33	31
bx(m)	1.24	1.24	1.24	1.24	1.24
by(m)	1.24	1.24	1.24	1.24	1.24
Up(m)	4.96	4.96	4.96	4.96	4.96
Vpd(kN)	1379.11	1335.73	1805.66	1805.65	1601.16
ex(m)	0.05956	0.05489	0.03771	0.03771	0.05231
ey(m)	0.18958	0.20036	0.13014	0.13014	0.14778
exmin(mm)	33.00	33.00	33.00	33.00	33.00
eymin(mm)	33.00	33.00	33.00	33.00	33.00
ɣ	0.9990	0.9990	0.9990	0.9990	0.9990
Vpr(kN)	3963.89	3963.89	3963.89	3963.89	3963.89

16. Control of Irregularities According to TSC207

TABLE 2.1 - IRREGULAR BUILDINGS

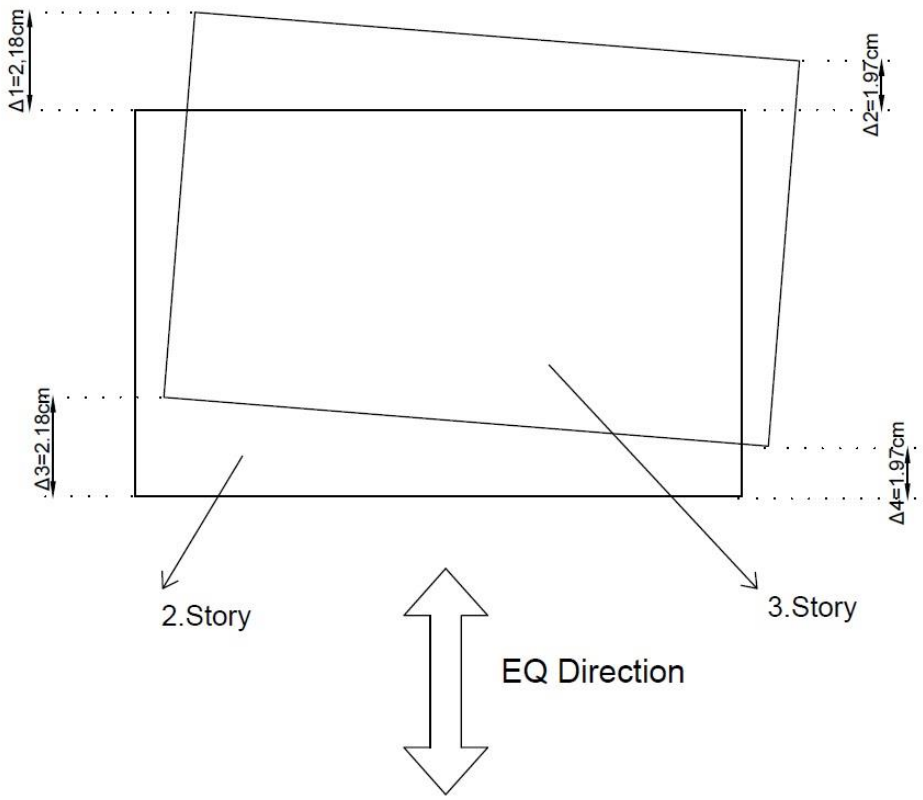
A – *IRREGULARITIES IN PLAN*	***Related Items***
A1 – Torsional Irregularity : The case where *Torsional Irregularity Factor* η_{bi}, which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum relative storey drift at any storey to the average relative storey drift at the same storey in the same direction, is greater than 1.2 (**Figure 2.1**). $[\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{avr} > 1.2]$ *Relative storey drifts shall be calculated in accordance with* **2.7**, *by considering the effects of* ± %5 *additional eccentricities.*	**2.3.2.1**
A2 – Floor Discontinuities : In any floor (**Figure 2.2**); **I -** The case where the total area of the openings including those of stairs and elevator shafts exceeds 1 / 3 of the gross floor area, **II –** The case where local floor openings which make the safe transfer of seismic loads difficult to vertical structural elements, **III –** The cases of abrupt reductions in the in-plane stiffness and strength of floors.	**2.3.2.2**
A3 – Projections in Plan : The cases where dimensions of projections in both of the two perpendicular directions in plan exceed the total plan dimensions of that storey of the building in the respective directions by more than 20% (**Figure 2.3**).	**2.3.2.2**
B – *IRREGULARITIES IN ELEVATION*	***Related Items***
B1 – Interstorey Strength Irregularity (*Weak Storey*) : In reinforced concrete buildings, the case where in each of the orthogonal earthquake directions, *Strength Irregularity Factor* η_{ci} which is defined as the ratio of the *effective shear area* of any storey t the *effective shear area* of the storey immediately above, is less than **0.80**. $[\eta_{ci} = (\sum A_e)_i / (\sum A_e)_{i+1} < 0.80]$ *Definition of effective shear area in any storey :* $\sum A_e = \sum A_w + \sum A_g + 0.15 \sum A_k$ (*See* **3.0** *for notations*)	**2.3.2.3**
B2 – Interstorey Stiffness Irregularity (*Soft Storey*) : The case where in each of the two orthogonal earthquake directions, *Stiffness Irregularity Factor* η_{ki}, which is defined as the ratio of the average relative storey drift at any i'th storey to the average relative storey drift at the storey immediately above or below, is greater than 2.0. $[\eta_{ki} = (\Delta_i / h_i)_{avr} / (\Delta_{i+1} / h_{i+1})_{avr} > 2.0$ or $\eta_{ki} = (\Delta_i / h_i)_{avr} / (\Delta_{i-1} / h_{i-1})_{avr} > 2.0]$ *Relative storey drifts shall be calculated in accordance with* **2.7**, *by considering the effects of* ± %5 *additional eccentricities.*	**2.3.2.1**
B3 - Discontinuity of Vertical Structural Elements : The cases where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath (**Figure 2.4**).	**2.3.2.4**

A)Irregularities in Plan

A1-Torsional Irregularities

On the Y direction, rotation of the building is too much higher than the X direction ones. Therefore we checked only Y direction torsional buckling according to TSC2007. The most critical story drifts are between 2nd and 3rd stories. So torsional irregularities check will be done considering theese stories.

$$\eta bi = \frac{(\Delta i)max}{(\Delta i)ort} > 1.2$$



$$\eta 23 = \frac{2.18}{2.075} = 1.05 < 1.20$$

According to calculation, in our building there is no torsional irregularities.

A2-Floor Discontinuities

In our building there is no floor discontinuities.

A3-Projections in Plan

Our building has not any projections in plan.

B)Irregularities in Elevation

B1-Interstory Strength Irregularity (Weak Store)

In our building, there is no weak story according to shear area ratios between stories.

B2-Interstory Stiffness Irregularity (Soft Storey)

In our building, 1st and 2nd story stiffness are equal and 3rd story stifness is less than the others. Therefore, there is no weak story in our building.

B3- Discontinuties of Vertical Structural Elements

In our building has irregularities on steel roof side on the 3rd story. Some of the 1st and 2nd story columns don't continue on 3rd story. But these irregulaties is not the same with discontinuties of vertical structural element irregularities. Therefore, we could say that there is no discontinuties of vertical structral elements irregularities.

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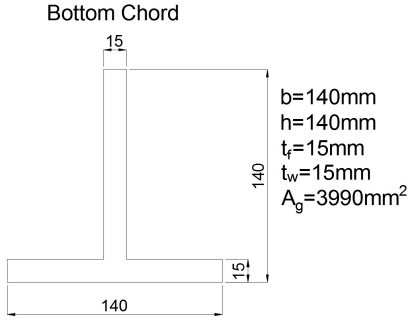
Bottom Chord



Bottom Chord Design

$Tmax := 264\ kN$

$Pmax := 374\ kN$

$L := 3,0\ m$

T 140 is choosen

$b := 140\ mm$ $\quad A := 39,90\ cm^2$ $\quad Ix := 0,66 \cdot 10^{-5}\ m^4$

$h := 140\ mm$ $\quad E := 210000\ MPa$ $\quad Iy := 0,33 \cdot 10^{-5}\ m^4$

$tf := 15\ mm$ $\quad Fy := 355\ MPa$

$tw := 15\ mm$ $\quad Fu := 510\ MPa$

$$rx := \sqrt{\frac{Ix}{A}} = 40,6711\ mm$$

$$ry := \sqrt{\frac{Iy}{A}} = 28,7588\ mm$$

$$yb := \frac{b \cdot tf \cdot \frac{tf}{2} + (h - tf) \cdot tw \cdot \left(\frac{h - tf}{2} + tf\right)}{A} = 40,3665\ mm$$

Desing Under Compression

$Pmax = 374\ kN$ $\quad L = 3\ m$

a) Local Buckling Control

$$\frac{b}{2 \cdot tf} = 4,6667 \qquad \frac{b}{2 \cdot tf} < 0,56 \cdot \sqrt{\frac{E}{Fy}} = 1$$

$$\frac{h}{tw} = 9,3333 \qquad \frac{h}{tw} < 0,75 \cdot \sqrt{\frac{E}{Fy}} = 1$$

The section is not slender, there is no local buckling.

b) Compressive strength of section

$Kx := 1$ $\quad Ky := 1$ $\quad Lx := 3,0\ m$ $\quad Ly := 3,0\ m$

$Pmax = 374\ kN$

$$Kx \cdot \frac{Lx}{rx} = 73,7625 \qquad Ky \cdot \frac{Ly}{ry} = 104,316$$

$$Fe := \frac{\pi^2 \cdot E}{\left(Ky \cdot \frac{Ly}{ry}\right)^2} = 190,466\ MPa$$

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$4,71\cdot\sqrt{\frac{E}{Fy}} = 114,5556 \quad 4,71\cdot\sqrt{\frac{E}{Fy}} > \max\left(\left[\frac{Kx\cdot L}{rx} \quad Ky\cdot\frac{Ly}{ry}\right]\right) = 1$

inelastik mode

$Fcr := 0,658^{\frac{Fy}{Fe}}\cdot Fy = 162,7155\,MPa$

$\Omega c := 1,67$

$Pa := Fcr\cdot\frac{A}{\Omega c} = 388,7633\,kN \qquad Pa > Pmax = 1$

Desing Under Tension

a1) Yeilding Strength

$A = 39,9\,cm^2 \qquad \Omega y := 1,67$

$Rn1 := Fy\cdot\frac{A}{(\Omega y)} = 848,1737\,kN \qquad Rn1 > Tmax = 1$

a2) Rupture Strength

$A = 39,9\,cm^2 \qquad \Omega r := 2$

T profil $\quad b = 0,14\,m \quad 2\cdot\frac{h}{3} = 0,0933\,m \quad b > 2\cdot\frac{h}{3} = 1$

$U := 0,9$

$Ae := A\cdot U = 35,91\,cm^2$

$Rn2 := A\cdot\frac{Fu}{\Omega r} = 1017,45\,kN \qquad Rn2 > Tmax = 1$

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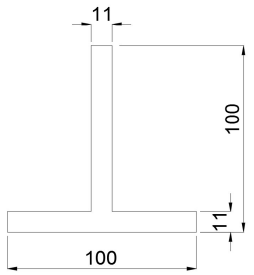
Top Chord



b=100mm
h=100mm
t_f=11mm
t_w=11mm
A_g=2090mm^2

Top Chord Design

$Tmax := 0 \cdot kN$

$Pmax := 50,93\ kN$

$L := 1,50\ m$

T 100 is choosen

$b := 100\ mm$ $\quad A := 20,90\ cm^2$ $\quad Ix := (0,179) \cdot 10^{-5}\ m^4$

$h := 100\ mm$ $\quad E := 210000\ MPa$ $\quad Iy := 0,0885 \cdot 10^{-5}\ m^4$

$tf := 11\ mm$ $\quad Fy := 355\ MPa$

$tw := 11\ mm$ $\quad Fu := 510\ MPa$

$$rx := \sqrt{\frac{Ix}{A}} = 29,2653\ mm$$

$$ry := \sqrt{\frac{Iy}{A}} = 20,5778\ mm$$

$$yb := \frac{b \cdot tf \cdot \frac{tf}{2} + (h - tf) \cdot tw \cdot \left(\frac{h - tf}{2} + tf\right)}{A} = 28,8921\ mm$$

Desing Under Compression

$Pmax = 50,93\ kN$ $\quad L = 1,5\ m$

a) Local Buckling Control

$$\frac{b}{2 \cdot tf} = 4,5455 \qquad \frac{b}{2 \cdot tf} < 0,56 \cdot \sqrt{\frac{E}{Fy}} = 1$$

$$\frac{h}{tw} = 9,0909 \qquad \frac{h}{tw} < 0,75 \cdot \sqrt{\frac{E}{Fy}} = 1$$

The section is not slender, there is no local buckling.

b) Compressive strength of section

$Ky := 1$ $\quad Lx := 1,50\ m$ $\quad Ly := 3,0\ m$ $\quad A := 20,90\ cm^2$

$Pmax = 50,93\ kN$

$$Kx \cdot \frac{Lx}{rx} = 51,2552 \qquad Ky \cdot \frac{Ly}{ry} = 145,7883$$

$$Fe := \frac{\pi^2 \cdot E}{\left(Ky \cdot \frac{Ly}{ry}\right)^2} = 97,5155\ MPa$$

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$4,71\cdot\sqrt{\frac{E}{Fy}} = 114,5556 \quad 4,71\cdot\sqrt{\frac{E}{Fy}} > \max\left(\left[\frac{Kx\cdot L}{rx} \quad Ky\cdot\frac{Ly}{ry}\right]\right) = 0$

elastik mod

$Fcr := 0,877\cdot Fe = 85,5211\ MPa$

$\Omega c := 1,67$

$Pa := Fcr\cdot\frac{A}{\Omega c} = 107,0294\ kN \qquad Pa > Pmax = 1$

Desing Under Tension

a1) Yeilding Strenhth

$A = 20,9\ cm^2 \qquad \Omega y := 1,67$

$Rn1 := Fy\cdot\frac{A}{(\Omega y)} = 444,2814\ kN \qquad Rn1 > Tmax = 1$

a2) Rupture Strength

$A = 20,9\ cm^2 \qquad \Omega r := 2$

T profil $\quad b = 0,1\ m \quad 2\cdot\frac{h}{3} = 0,0667\ m \quad b > 2\cdot\frac{h}{3} = 1$

$U := 0,9$

$Ae := A\cdot U = 18,81\ cm^2$

$Rn2 := A\cdot\frac{Fu}{\Omega r} = 532,95\ kN \qquad Rn2 > Tmax = 1$

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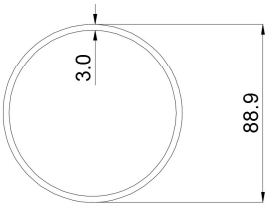
TRUSS MEMBER
(DIAGONAL END)



d=88.9mm
t=3mm
A_g=810mm^2

Truss Members design(end)

$Tmax := 0 \cdot kN$

$Pmax := 189\ kN$

$L := 2,34\ m$

Circular Hallow Section d=88.9mm

$D := 88,9\ mm$ $\quad A := 8,10\ cm^2$ $\quad I := 0,07476 \cdot 10^{-5}\ m^4$

$t := 3,0\ mm$

$E := 210000\ MPa$

$Fy := 355\ MPa$

$Fu := 510\ MPa$

$r := \sqrt{\frac{I}{A}} = 30,3803\ mm$

Desing Under Compression

$Pmax = 189\ kN$ $\quad L = 2,34\ m$

a) Compressive strength of section

$K := 1$ $\quad L = 2,34\ m$ $\quad A := 8,10\ cm^2$

$Pmax = 189\ kN$

$K \cdot \frac{L}{r} = 77,0236$

$$Fe := \frac{\pi^2 \cdot E}{\left(K \cdot \frac{L}{r}\right)^2} = 349,3587\ MPa$$

$$4,71 \cdot \sqrt{\frac{E}{Fy}} = 114,5556 \qquad 4,71 \cdot \sqrt{\frac{E}{Fy}} > \max\left(\left[\frac{Kx \cdot L}{rx} \quad Ky \cdot \frac{Ly}{ry}\right]\right) = 0$$

elastik mode

$Fcr := 0,877 \cdot Fe = 306,3876\ MPa$

$\Omega c := 1,67$

$Pa := Fcr \cdot \frac{A}{\Omega c} = 148,6072\ kN$ $\quad Pa > Pmax = 0$

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Desing Under Tension

a1) Yeilding Strength

$A = 8,1\ cm^2$ $\Omega y := 1,67$

$Rn1 := Fy \cdot \frac{A}{(\Omega y)} = 172,1856\ kN$ $Rn1 > Tmax = 1$

a2) Rupture Strength

$A = 8,1\ cm^2$ $\Omega r := 2$

T profil $b = 0,1\ m$ $2 \cdot \frac{h}{3} = 0,0667\ m$ $b > 2 \cdot \frac{h}{3} = 1$

$U := 0,9$

$Ae := A \cdot U = 7,29\ cm^2$

$Rn2 := A \cdot \frac{Fu}{\Omega r} = 206,55\ kN$ $Rn2 > Tmax = 1$

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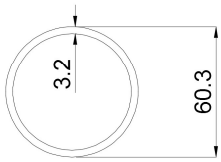
TRUSS MEMBER
(DIAGONAL MIDDLE)



d=60.3mm
t=3.2mm
A_g=869mm^2

Truss Members design(Middle)

$Tmax := 0 \cdot kN$

$Pmax := 87,48\ kN$

Circular Hallow Section d=60.3mm

$D := 60,3\ mm$ $\quad A := 8,69\ cm^2$ $\quad I := 0,05906 \cdot 10^{-5}\ m^4$

$t := 3,2\ mm$

$E := 210000\ MPa$

$Fy := 355\ MPa$

$Fu := 510\ MPa$

$r := \sqrt{\frac{I}{A}} = 26,0697\ mm$

Desing Under Compression

$Pmax = 87,48\ kN$ $\quad L = 2,34\ m$

a) Compressive strength of section

$K := 1$ $\quad L = 2,34\ m$ $\quad A := 9,06\ cm^2$

$Pmax = 87,48\ kN$

$K \cdot \frac{L}{r} = 89,7592$

$$Fe := \frac{\pi^2 \cdot E}{\left(K \cdot \frac{L}{r}\right)^2} = 257,2533\ MPa$$

$$4,71 \cdot \sqrt{\frac{E}{Fy}} = 114,5556 \qquad 4,71 \cdot \sqrt{\frac{E}{Fy}} > \max\left(\left[\frac{Kx \cdot L}{rx} \quad Ky \cdot \frac{Ly}{ry}\right]\right) = 0$$

elastik mode

$Fcr := 0,877 \cdot Fe = 225,6112\ MPa$

$\Omega c := 1,67$

$Pa := Fcr \cdot \frac{A}{\Omega c} = 122,3974\ kN$ $\quad Pa > Pmax = 1$

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Desing Under Tension

a1) Yeilding Strength

$A = 9,06\ cm^2$ $\Omega y := 1,67$

$Rn1 := Fy \cdot \frac{A}{(\Omega y)} = 192,5928\ kN$ $Rn1 > Tmax = 1$

a2) Rupture Strength

$A = 9,06\ cm^2$ $\Omega r := 2$

T profil $b = 0,1\ m$ $2 \cdot \frac{h}{3} = 0,0667\ m$ $b > 2 \cdot \frac{h}{3} = 1$

$U := 0,9$

$Ae := A \cdot U = 8,154\ cm^2$

$Rn2 := A \cdot \frac{Fu}{\Omega r} = 231,03\ kN$ $Rn2 > Tmax = 1$

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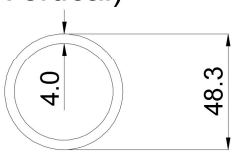
TRUSS MEMBER
(Vertical)



d=48.3mm
t=4.0mm
A_g=557mm^2

Truss Members design(vertical)

$Tmax := 0 \cdot kN$

$Pmax := 25,35\ kN$

$L := 1,8\ m$

d=48.3mm Circular Hallow Section

$D := 48,3\ mm$ $\quad A := 5,57\ cm^2$ $\quad I := 0,01377 \cdot 10^{-5}\ m^4$

$t := 4,0\ mm$

$E := 210000\ MPa$

$Fy := 355\ MPa$

$Fu := 510\ MPa$

$r := \sqrt{\frac{I}{A}} = 15,7231\ mm$

Desing Under Compression

$Pmax = 25,35\ kN$ $\quad L = 1,8\ m$

a) Compressive strength of section

$K := 1$ $\quad L = 1,8\ m$ $\quad A := 5,57\ cm^2$

$Pmax = 25,35\ kN$

$K \cdot \frac{L}{r} = 114,4809$

$$Fe := \frac{\pi^2 \cdot E}{\left(K \cdot \frac{L}{r}\right)^2} = 158,144\ MPa$$

$4,71 \cdot \sqrt{\frac{E}{Fy}} = 114,5556$ $\quad 4,71 \cdot \sqrt{\frac{E}{Fy}} > \max\left(\left(\frac{Kx \cdot L}{rx}\ \ Ky \cdot \frac{Ly}{ry}\right)\right) = 0$

elastik mod

$Fcr := 0,877 \cdot Fe = 138,6923\ MPa$

$\Omega c := 1,67$

$Pa := Fcr \cdot \frac{A}{\Omega c} = 46,2585\ kN$ $\quad Pa > Pmax = 1$

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Desing Under Tension

a1) Yeilding Strength

$A = 5,57\ cm^2$ $\Omega y := 1,67$

$Rn1 := Fy \cdot \frac{A}{(\Omega y)} = 118,4042\ kN$ $Rn1 > Tmax = 1$

a2) Rupture Strength

$A = 5,57\ cm^2$ $\Omega r := 2$

T profil $b = 0,1\ m$ $2 \cdot \frac{h}{3} = 0,0667\ m$ $b > 2 \cdot \frac{h}{3} = 1$

$U := 0,9$

$Ae := A \cdot U = 5,013\ cm^2$

$Rn2 := A \cdot \frac{Fu}{\Omega r} = 142,035\ kN$ $Rn2 > Tmax = 1$

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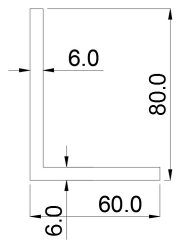
HORIZANTAL STABILITY ELEMENT



b=60mm
h=80mm
t_f=6mm
t_w=6mm
A_g=811mm^2

According to TSC2017 Normal floor and roof palne diagrams should be dimensioned to safely transfer the earhtquake forces to main structural system of the building. This force will be calculated;

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

$F_{px} := 1181\ kN$

$$0.2 S_{DS} I w_{px} \le F_{px} \le 0.4 S_{DS} I w_{px}$$

1878.36kN<1181kN)<3756.735kN

Therefore, when the calculation are doing, for the diagram force we will use 1878.36 kN. But, When we will make dimensioning the most critical forces come from the 1.4G+1.6Q load combination. The calculations are made for 18 kN.

Horizantal Stability Member Desing (Diagonal)

$Tmax := 0\ kN$
$Pmax := 18\ kN$
$L := 3,35\ m$

L section 80x60x6

$b := 80\ mm$ $A := 8,11\ cm^2$ $Ix := 0,0514 \cdot 10^{-5}\ m^4$
$h := 60\ mm$ $E := 210000\ MPa$ $Iy := 0,0248 \cdot 10^{-5}\ m^4$
$tf := 6\ mm$ $Fy := 355\ MPa$
$tw := 6\ mm$ $Fu := 510\ MPa$

$$rx := \sqrt{\frac{Ix}{A}} = 25,1751\ mm$$

$$ry := \sqrt{\frac{Iy}{A}} = 17,487\ mm$$

$$yb := \frac{b \cdot tf \cdot \frac{tf}{2} + (h - tf) \cdot tw \cdot \left(\frac{h - tf}{2} + tf\right)}{A} = 14,9593\ mm$$

Desing Under Compression

$Pmax = 18\ kN$ $L = 3,35\ m$

a) Local Buckling Control

$$\frac{b}{2 \cdot tf} = 6,6667 \qquad \frac{b}{2 \cdot tf} < 0,56 \cdot \sqrt{\frac{E}{Fy}} = 1$$

$$\frac{h}{tw} = 10 \qquad \frac{h}{tw} < 0,75 \cdot \sqrt{\frac{E}{Fy}} = 1$$

The section is not slender, there is no local buckling.

b) Compressive strength of section

$Kx := 1$ $Ky := 1$ $Lx := 3,35\ m$ $Ly := 3,335\ m$ $A := 8,11\ cm^2$

$Pmax = 18\ kN$

$$Kx \cdot \frac{Lx}{rx} = 133,068 \qquad Ky \cdot \frac{Ly}{ry} = 190,7131$$

$$Fe := \frac{\pi^2 \cdot E}{\left(Ky \cdot \frac{Ly}{ry}\right)^2} = 56,9847\ MPa$$

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$4,71\cdot\sqrt{\frac{E}{Fy}} = 114,5556 \quad 4,71\cdot\sqrt{\frac{E}{Fy}} > \max\left(\left[\frac{Kx\cdot L}{rx} \quad Ky\cdot\frac{Ly}{ry}\right]\right) = 0$

elastic mode

$Fcr := 0,877\cdot Fe = 49,9756\,MPa$ $\qquad \Omega c := 1,67$

$Pa := Fcr\cdot\frac{A}{\Omega c} = 24,2696\,kN$ $\qquad Pa > Pmax = 1$

Desing Under Tension

a1) Yeilding Strength

$A = 8,11\,cm^2$ $\qquad \Omega y := 1,67$

$Rn1 := Fy\cdot\frac{A}{(\Omega y)} = 172,3982\,kN$ $\qquad Rn1 > Tmax = 1$

a2) Rupture Strength

$A = 8,11\,cm^2$ $\qquad \Omega r := 2$

T profil $\qquad b = 0,08\,m \qquad 2\cdot\frac{h}{3} = 0,04\,m \qquad b > 2\cdot\frac{h}{3} = 1$

$U := 0,9$

$Ae := A\cdot U = 7,299\,cm^2$

$Rn2 := A\cdot\frac{Fu}{\Omega r} = 206,805\,kN$ $\qquad Rn2 > Tmax = 1$

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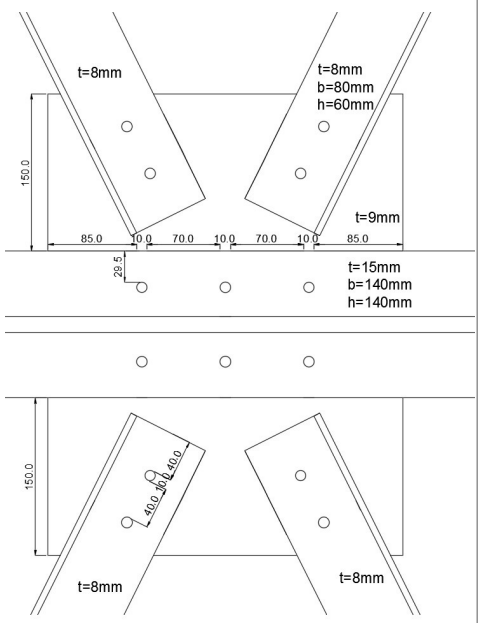

Connection Design

Gusset Plate(bottom) Botton Chord Connection Design

$Ty := 8\ kN$ $Tx := 10,67\ kN$ $T := 13,33\ kN$ $Fy = 355\ MPa$ $Fu = 510\ MPa$

8.8 M10 bolds are choosen.

$Fub := 800\ MPa$

For high strength bolds,treads are out of the shear plane;

$Fnv := 0,563 \cdot Fub = 450,4\ MPa$

$d := 10\ mm$ $\Omega := 2$

Shear Strength of Bolds

$Ab := \frac{\pi \cdot d^2}{4} = 78,5398\ mm^2$

$Rn := Ab \cdot Fnv = 35,3743\ kN$ $Ra1 := \frac{Rn}{\Omega} = 17,6872\ kN$

$\frac{T}{Ra1} = 0,7537$ 3 bolds are used.

Beraring Strength of Gusset Plate

$Lc1 := 10000\ mm$

$Lc2 := 70\ mm$

$Lc := \min([Lc1\ \ Lc2]) = 70\ mm$

$tmin := 9\ mm$

$dh := 11\ mm$

$1,2 \cdot Lc \cdot tmin \cdot Fu = 385,56\ kN$

$2,4 \cdot d \cdot tmin \cdot Fu = 110,16\ kN$ $Rn2 := 110,16\ kN$

$Ra2 := \frac{Rn2}{\Omega} \cdot 3 = 165,24\ kN$ $Ra2 > Tx = 1$

Bearing Strength of Bottom Chord

$Lc1 := 10000\ mm$ $Lc2 := 29,5\ mm$

$Lc := \min([Lc1\ \ Lc2]) = 29,5\ mm$

$tmin := 9\ mm$

$dh := 11\ mm$

$1,2 \cdot Lc \cdot tmin \cdot Fu = 162,486\ kN$

$2,4 \cdot d \cdot tmin \cdot Fu = 110,16\ kN$ $Rn2 := 110,16\ kN$

$Ra2 := \frac{Rn2}{\Omega} \cdot 3 = 165,24\ kN$ $Ra2 > Ty = 1$

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Block Shaer(Bottom Chord)

$theb := 15\ mm \qquad tp := 9\ mm \qquad d_h := 11\ mm \qquad b_{flange} := 140\ mm$

$Anv := \left(\frac{b_{flange}}{4} - d_h\right) \cdot 2 \cdot theb = 720\ mm^2$

$Ant := \left(162\ mm - 2 \cdot d_h\right) \cdot theb = 2100\ mm^2 \qquad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 1224,36\ kN$

$Ra1 := \frac{Rn1}{\Omega} = 612,18\ kN \qquad Ra1 > T = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 1291,32\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 645,66\ kN \qquad Ra2 > Ty = 1$

Block Shear(Gusset Plate)

$theb := 15\ mm \qquad tp := 9\ mm \qquad d_h := 11\ mm \qquad b_{plate} := 310\ mm$

$Anv := 0$

$Ant := \left(b_{plate} - 3 \cdot d_h\right) \cdot tp = 2493\ mm^2 \qquad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 1271,43\ kN$

$Ra1 := \frac{Rn1}{\Omega} = 635,715\ kN \qquad Ra1 > T = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 1271,43\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 635,715\ kN \qquad Ra2 > Tx = 1$

Strenght of Tension Member

$tp = 9\ mm \qquad b_{plate} = 310\ mm \qquad n := 3$ number of bolts

$Ag := tp \cdot b_{plate} = 2790\ mm^2$

$An := \left(b_{plate} - n \cdot d_h\right) \cdot tp = 2493\ mm^2$

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a) Yeilding Strength

$Rn1 := \frac{Fy \cdot Ag}{\Omega y} = 593,0838\ kN$ $\Omega y := 1,67$ $Rn1 > T = 1$

b) Repture Strength

$U := 1$ $\Omega r = 2$

$Ae := U \cdot An = 2493\ mm^2$

$Rn2 := \frac{Ae \cdot Fu}{\Omega r} = 635,715\ kN$ $Rn2 > T = 1$

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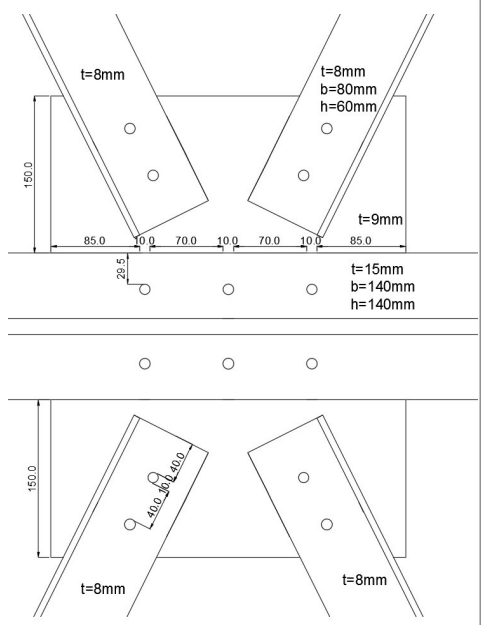

Horizantal Stability Botton Gusset Plate Connection Desing

$T := 9\ kN$ $\quad Fy = 355\ MPa$ $\quad Fu = 510\ MPa$

8.8 M10 Bolds are choosen.

$Fub := 800\ MPa$

For high strength bolds, treads are out of the shear plane;

$Fnv := 0,563 \cdot Fub = 450,4\ MPa$

$d := 10\ mm$ $\quad \Omega := 2$

Shear Strength of Bolds

$Ab := \frac{\pi \cdot d^2}{4} = 78,5398\ mm^2$

$Rn := Ab \cdot Fnv = 35,3743\ kN$ $\quad Ra1 := \frac{Rn}{\Omega} = 17,6872\ kN$

$\frac{T}{Ra1} = 0,5088$ $\quad$ 2 bulon kullanıldı.

Beraring Strength of Gusset Plate

$Lc1 := 40\ mm$ $\quad Lc2 := 40\ mm$

$Lc := \min([Lc1\ \ Lc2]) = 40\ mm$

$tmin := 6\ mm$

$dh := 11\ mm$

$1,2 \cdot Lc \cdot tmin \cdot Fu = 146,88\ kN$

$2,4 \cdot d \cdot tmin \cdot Fu = 73,44\ kN$ $\quad Rn2 := 73,44\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 36,72\ kN$ $\quad Ra2 > T = 1$

Bearing Strength of Bottom Chord

$theb := 6\ mm$ $\quad tp := 9\ mm$ $\quad d_h := 11\ mm$

$Anv := (96,5\ mm - 1,5 \cdot d_h) \cdot theb = 480\ mm^2$

$Ant := (30\ mm - 1 \cdot d_h) \cdot theb = 114\ mm^2$ $\quad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 160,38\ kN$

$Ra1 := \frac{Rn1}{\Omega} = 80,19\ kN$ $\quad Ra1 > T = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 205,02\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 102,51\ kN$ $\quad Ra2 > Ty = 1$

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Block Shaer(Bottom Chord)

$theb := 6\ mm \qquad tp := 9\ mm \qquad d_h := 11\ mm$

$Anv := \left(96,5\ mm - 1,5 \cdot d_h\right) \cdot theb = 480\ mm^2$

$Ant := \left(30\ mm - 1 \cdot d_h\right) \cdot theb = 114\ mm^2 \qquad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 160,38\ kN$

$Ra1 := \frac{Rn1}{\Omega} = 80,19\ kN \qquad Ra1 > T = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 205,02\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 102,51\ kN \qquad Ra2 > Ty = 1$

Strenght of Gusset Plate

$tp = 9\ mm \qquad b_{plate} = 310\ mm \qquad n := 2$ number of bolts

$Ag := tp \cdot b_{plate} = 2790\ mm^2$

$An := \left(b_{plate} - n \cdot d_h\right) \cdot tp = 2592\ mm^2$

a) Yeilding Strenght

$Rn1 := \frac{Fy \cdot Ag}{\Omega y} = 593,0838\ kN \qquad \Omega y := 1,67 \qquad Rn1 > T = 1$

b) Repture Strenght

$U := 1 \qquad \Omega r = 2$

$Ae := U \cdot An = 2592\ mm^2$

$Rn2 := \frac{Ae \cdot Fu}{\Omega r} = 660,96\ kN \qquad Rn2 > T = 1$

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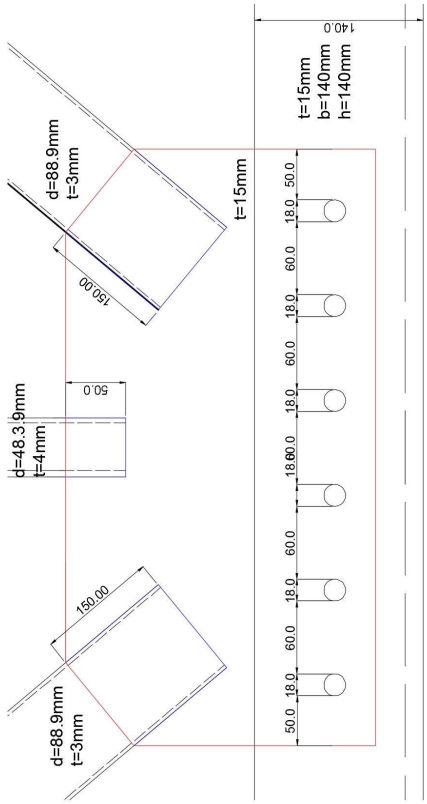

Truss Connection Design

Vertical Gusset Plate-Bottom Chord Connection

$Ty := 58,194\ kN$ $Tx := 7,975\ kN$ $T := 58,74\ kN$ $Fy = 355\ MPa$

10.9 M18 bolds are choosen. $Fu = 510\ MPa$

$Fub := 1000\ MPa$

For high strength bolds, treads are out of the shear plane;

$Fnv := 0,563 \cdot Fub = 563\ MPa$

$d := 18\ mm$ $\Omega := 2$

Shear Strength of Bolds

$$Ab := \frac{\pi \cdot d^2}{4} = 254,469\ mm^2$$

$Rn := Ab \cdot Fnv = 143,266\ kN$ $Ra1 := \frac{Rn}{\Omega} = 71,633\ kN$

$\frac{T}{Ra1} = 0,82$ 6 bolds are used.

Beraring Strength of Gusset Plate

$Lc1 := 50\ mm$

$Lc2 := 60\ mm$

$Lc := \min([Lc1\ \ Lc2]) = 50\ mm$

$tmin := 15\ mm$

$dh := 20\ mm$

$1,2 \cdot Lc \cdot tmin \cdot Fu = 459\ kN$

$2,4 \cdot d \cdot tmin \cdot Fu = 330,48\ kN$ $Rn2 := 330,48\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 165,24\ kN$ $Ra2 > Tx = 1$

Bearing Strength of Bottom Chord

$Lc := 52,5\ mm$

$tmin := 15\ mm$

$dh := 20\ mm$

$1,2 \cdot Lc \cdot tmin \cdot Fu = 481,95\ kN$

$2,4 \cdot d \cdot tmin \cdot Fu = 330,48\ kN$ $Rn2 := 330,48\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 165,24\ kN$ $Ra2 > Ty = 1$

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Block Shaer(Bottom Chord)

$theb := 15\ mm \qquad tp := 15\ mm \qquad d_h := 20\ mm$

$Anv := \left(70\ mm - \frac{d_h}{2}\right) \cdot 2 \cdot tp = 1800\ mm^2$

$Ant := \left(300\ mm - 5 \cdot d_h\right) \cdot tp = 3000\ mm^2 \qquad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 1913,4\ kN$

$Ra1 := \frac{Rn1}{\Omega} = 956,7\ kN \qquad Ra1 > Ty = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 2080,8\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 1040,4\ kN \qquad Ra2 > Ty = 1$

Block Shear(Gusset Plate)

$theb := 15\ mm \qquad tp := 15\ mm \qquad d_h := 20\ mm$

$Anv := \left(70\ mm - \frac{d_h}{2}\right) \cdot 2 \cdot tp = 1800\ mm^2$

$Ant := \left(300\ mm - 5 \cdot d_h\right) \cdot tp = 3000\ mm^2 \qquad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 1913,4\ kN$

$Ra1 := \frac{Rn1}{\Omega} = 956,7\ kN \qquad Ra1 > Ty = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 2080,8\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 1040,4\ kN \qquad Ra2 > Ty = 1$

Strenght of Tension Member

$tp = 15\ mm \qquad b_{plate} = 310\ mm \qquad n := 6$ number of bolts

$Ag := tp \cdot b_{plate} = 4650\ mm^2$

$An := \left(b_{plate} - n \cdot d_h\right) \cdot tp = 2850\ mm^2$

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a) Yeilding Strenght

$Rn1 := \frac{Fy \cdot Ag}{\Omega y} = 988,4731\,kN$ $\Omega y := 1,67$ $Rn1 > Ty = 1$

b) Repture Strenght

$U := 1$ $\Omega r = 2$

$Ae := U \cdot An = 2850\,mm^2$

$Rn2 := \frac{Ae \cdot Fu}{\Omega r} = 726,75\,kN$ $Rn2 > Ty = 1$

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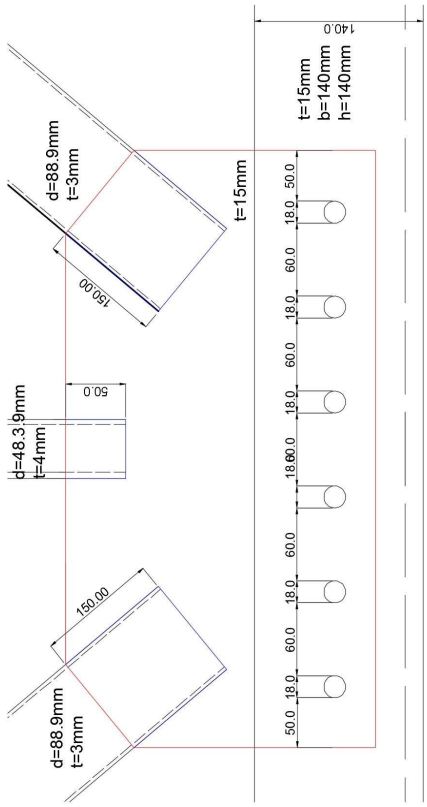

Truss gusset plate-diagonal and vertical member weld

Diagonal Truss Member-Gusset Plate

$Fexx := 480\ MPa$ E70 Electrode

$T_{diagonal} := 197\ kN$

İnce olan elemanın kalınlığı: 3mm

Kalın olan elemanın kalınlığı: 15 mm

$w := 3\ mm$ $t := 0,707 \cdot w = 2,121\ mm$ $\Omega weld := 2$

$F_{nw} := 0,6 \cdot Fexx = 288\ MPa$

$n_l := 4$ number of longitudinal weld $n_t := 2$ number oft transvers weld

$l_l := 150\ mm$ $l_t := 88,9\ mm$

$A_{wel} := n_l \cdot l_l \cdot t = 1272,6\ mm^2$ $A_{wet} := n_t \cdot l_t \cdot t = 377,1138\ mm^2$

$R_{nwl} := F_{nw} \cdot A_{wel} = 366,5088\ kN$ $R_{nt} := F_{nw} \cdot A_{wet} = 108,6088\ kN$

$R_{n1} := R_{nwl} + R_{nt} = 475,1176\ kN$

$R_{n2} := 0,85 \cdot R_{nwl} + 1,5 \cdot R_{nt} = 474,4456\ kN$

$$R_n := \frac{\max\left(\left[R_{n1}\ R_{n2}\right]\right)}{\Omega weld} = 237,5588\ kN$$

a) Yeilding Strenght

$Ag := A_{wel} + A_{wet} = 1649,7138\ mm^2$

$Rn1 := \frac{Fy \cdot Ag}{\Omega y} = 350,6877\ kN$ $\Omega y := 1,67$ $Rn1 > T_{diagonal} = 1$

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b) Repture Strenght

$U := 1 \quad \Omega r = 2$

$An := Ag = 1649,7138\, mm^2$

$Ae := U \cdot An = 1649,7138\, mm^2$

$Rn2 := \frac{Ae \cdot Fu}{\Omega r} = 420,677\, kN \qquad Rn2 > T_{diagonal} = 1$

c) Block Shear (88.9 hallow section)

$t := 3\, mm$

$Anv := n_l \cdot (l_l) \cdot t = 1800\, mm^2$

$Ant := n_t \cdot l_t \cdot t = 533,4\, mm^2 \qquad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 655,434\, kN$

$Ra1 := \frac{Rn1}{\Omega} = 327,717\, kN \qquad Ra1 > T_{diagonal} = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 822,834\, kN$

$Ra2 := \frac{Rn2}{\Omega} = 411,417\, kN \qquad Ra2 > T_{diagonal} = 1$

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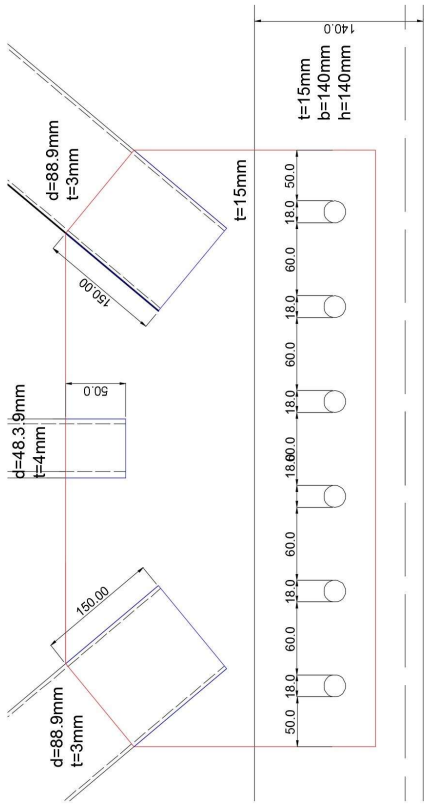

Vertical Member-Gusset Plate

$Fexx := 480\ MPa$ E70 Electrode

$T_{vertical} := 22,95\ kN$

Thiner member tickness: 4mm

Thicker member tickness: 15 mm

$w := 4\ mm$ $t := 0,707 \cdot w = 2,828\ mm$ $\Omega weld := 2$

$F_{nw} := 0,6 \cdot Fexx = 288\ MPa$

$n_l := 4$ number of longitudinal weld $n_t := 2$ number of transvers weld

$l_l := 50\ mm$ $l_t := 48,3\ mm$

$A_{wel} := n_l \cdot l_l \cdot t = 565,6\ mm^2$ $A_{wet} := n_t \cdot l_t \cdot t = 273,1848\ mm^2$

$R_{nwl} := F_{nw} \cdot A_{wel} = 162,8928\ kN$ $R_{nt} := F_{nw} \cdot A_{wet} = 78,6772\ kN$

$R_{n1} := R_{nwl} + R_{nt} = 241,57\ kN$

$R_{n2} := 0,85 \cdot R_{nwl} + 1,5 \cdot R_{nt} = 256,4747\ kN$

$R_n := \frac{\max\left(\left[R_{n1}\ R_{n2}\right]\right)}{\Omega weld} = 128,2374\ kN$ $Rn > T_{vertical} = 1$

a) Yeilding Strenght

$Ag := A_{wel} + A_{wet} = 838,7848\ mm^2$

$Rn1 := \frac{Fy \cdot Ag}{\Omega y} = 178,3046\ kN$ $\Omega y := 1,67$ $Rn1 > T_{vertical} = 1$

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b) Repture Strenght

$U := 1 \quad \Omega r = 2$

$An := Ag = 838,7848\ mm^2$

$Ae := U \cdot An = 838,7848\ mm^2$

$Rn2 := \frac{Ae \cdot Fu}{\Omega r} = 213,8901\ kN \quad Rn2 > T_{vertical} = 1$

c) Blok Kayma (88.9 hallow section)

$t := 4\ mm$

$Anv := n_l \cdot (l_l) \cdot t = 800\ mm^2$

$Ant := n_t \cdot l_t \cdot t = 386,4\ mm^2 \quad Ubs := 1$

$Rn1 := 0,6 \cdot Fy \cdot Anv + Ubs \cdot Fu \cdot Ant = 367,464\ kN$

$Ra1 := \frac{Rn1}{\Omega} = 183,732\ kN \quad Ra1 > T_{vertical} = 1$

$Rn2 := 0,6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant = 441,864\ kN$

$Ra2 := \frac{Rn2}{\Omega} = 220,932\ kN \quad Ra2 > T_{vertical} = 1$

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AŞIK SİSTEMLERİ

HEB 180 seçildi:

$tf := 14\ mm$ $bf := 180\ mm$ $E := 210000\ MPa$

$tw := 8,5\ mm$ $h := 180\ mm$ $hw := 152\ mm$

$Zx := 481,4\ cm^3$ $Zy := 231\ cm^3$ $ry := 4,57\ cm$ $rx := 7,66\ cm$

$Sx := 241\ cm^3$ $Ix := 3831\ cm^4$ $J := 42,16\ cm^4$

$Iy := 1363\ cm^4$ $ho := 166\ mm$ $Fy := 355\ MPa$

$Cb := 1$ $c := 1$ $\Omega := 1,67$

Design Under Compression

$Pmax := 30,21\ kN$ $L := 6\ m$

a) Local Buckling Control

$\frac{bf}{2 \cdot tf} = 6,4286$ $\frac{b}{2 \cdot tf} < 0,56 \cdot \sqrt{\frac{E}{Fy}} = 1$

$\frac{hw}{tw} = 17,8824$ $\frac{hw}{tw} < 0,75 \cdot \sqrt{\frac{E}{Fy}} = 1$

The section is non-slender, there is no local buckling.

b) Compressive Strength of section

$Kx := 1$ $Ky := 1$ $Lx := 6,0\ m$ $Ly := 6,0\ m$

$Pmax = 30,21\ kN$

$Kx \cdot \frac{Lx}{rx} = 78,329$ $Ky \cdot \frac{Ly}{ry} = 131,291$

$$Fe := \frac{\pi^2 \cdot E}{\left(Ky \cdot \frac{Ly}{ry}\right)^2} = 120,24\ MPa$$

$4,71 \cdot \sqrt{\frac{E}{Fy}} = 114,5556$ $4,71 \cdot \sqrt{\frac{E}{Fy}} > \max\left(\left[\frac{Kx \cdot L}{rx}\ \ Ky \cdot \frac{Ly}{ry}\right]\right) = 0$

elastik mode

$Fcr := 0,877 \cdot Fe = 105,4505\ MPa$ $\Omega c := 1,67$

$Pc := Fcr \cdot \frac{A}{\Omega c} = 51,2098\ kN$ $Pc > Pmax = 1$

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$\lambda pw := 3,76 \cdot \sqrt{\frac{E}{Fy}} = 91,4499$

$\lambda pf := 0,38 \cdot \sqrt{\frac{E}{Fy}} = 9,2423$

$\frac{bf}{2 \cdot tf} = 6,4286$ Flange is compact.

$\frac{hw}{tw} = 17,8824$ Web is compact.

Calculation of load coming to purlin

$q_{line} := \frac{1,4 \cdot (0,008 \cdot 27 \cdot 3 \cdot 6) + 1,6 \cdot (0,75 \cdot 3 \cdot 6)}{6} \frac{kN}{m}$

$q_{line} = 4,5072 \frac{kN}{m}$

$L := 6\ m$ Length between truss

$Max := q_{line} \cdot \frac{L^2}{8} = 20,2824\ kN\ m$

Yeilding :

Mp :=Fy*Zx

$Mp := Fy \cdot Zx = 170,897\ kN\ m$

Lateral Torsional Buckling

$Lb := L = 6\ m$

$Lp := 1,76 \cdot ry \cdot \sqrt{\frac{E}{Fy}} = 1,9562\ m$

$rts := \sqrt{Iy \cdot \frac{ho}{2 \cdot Sx}} = 0,0685\ m$

$$Lr := 1,95 \cdot rts \cdot \frac{E}{0,7 \cdot Fy} \cdot \sqrt{\frac{J \cdot c}{Sx \cdot ho} + \sqrt{\left(\frac{J \cdot c}{Sx \cdot ho}\right)^2 + 6,76 \cdot \left(\frac{0,7 \cdot Fy}{E}\right)^2}}$$

$Lr = 16,5613\ m$

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$Lb > Lr = 0 \quad Lr > Lp = 1 \quad (Lr > Lb) > Lp$

$$Mn := Cb \cdot \left(Mp - \left((Mp - 0,7 \cdot Fy \cdot Sx) \cdot \left(\frac{Lb - Lp}{Lr - Lp}\right)\right)\right) = 140,1617\ kN\ m$$

$\Omega = 1,67$

$Mn < Mp \qquad Mcx := \frac{Mn}{\Omega} = 83,9292\ kN\ m \qquad Mcx > Max = 1$

$Mcy := Fy \cdot Zy = 82,005\ kN\ m$

$Mcy\_control := 1,6 \cdot Fy \cdot Zy = 131,208\ kN\ m$

$Mcy > Mcy\_control = 0$

$\frac{Mcy}{\Omega} = 49,1048\ kN\ m$

$Pc := Fcr \cdot \frac{A}{\Omega c} = 51,2098\ kN$

$Pmax = 30,21\ kN$

$\frac{Pmax}{Pc} = 0,5899 > 0.2 \qquad My := 0\ kN\ m$

$$\frac{Pa}{Pc} + \frac{8}{9} \cdot \left(\frac{Max}{Mcx} + \frac{My}{Mcy}\right) = 0,6887 < 1$$

Control of Deflection

$q_{line} = 4,5072 \frac{kN}{m}$

$$fx := 0,0065 \cdot q_{line} \cdot \frac{L^4}{E \cdot Ix} = 0,0047\ m \qquad fy := 0,0065 \cdot q_{line} \cdot \frac{\left(\frac{L}{3}\right)^4}{E \cdot Iy} = 0,0002\ m$$

$\frac{L}{300} = 0,02\ m$

$f := \sqrt{fx^2 + fy^2} = 0,0047\ m$

$\frac{L}{300} > f = 1$ Deflection is in the allowable limits.

$q_{line} = 4,5072 \frac{kN}{m}$

$$Max := 3 \cdot q_{line} \cdot \frac{L^2}{8} = 60,8472\ kN\ m \qquad Mcx > Max = 1$$

$$May := q_{line} \cdot \frac{\left(\frac{L}{2}\right)^2}{8} = 5,0706\ kN\ m \qquad Mcy > May = 1$$

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