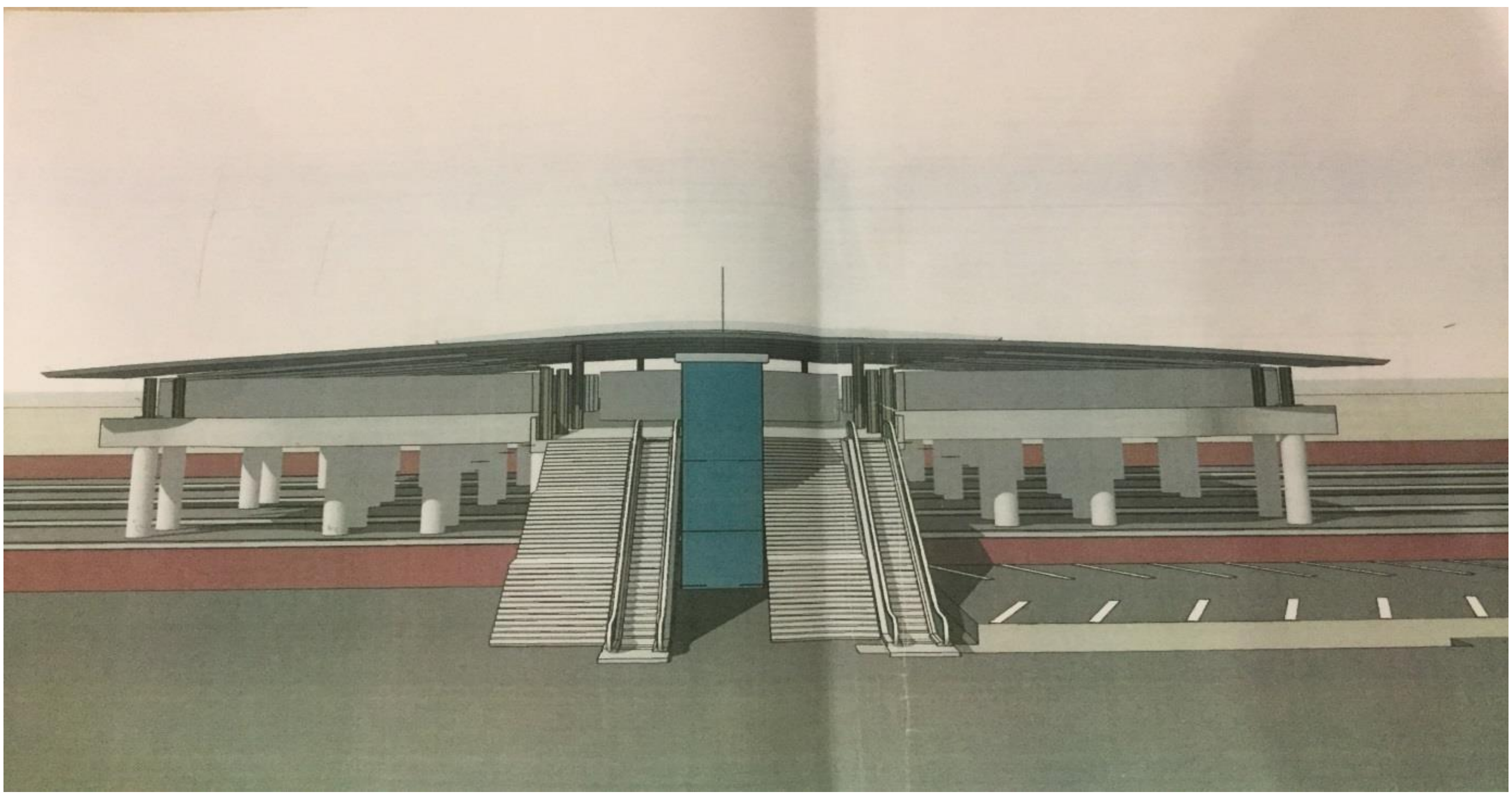
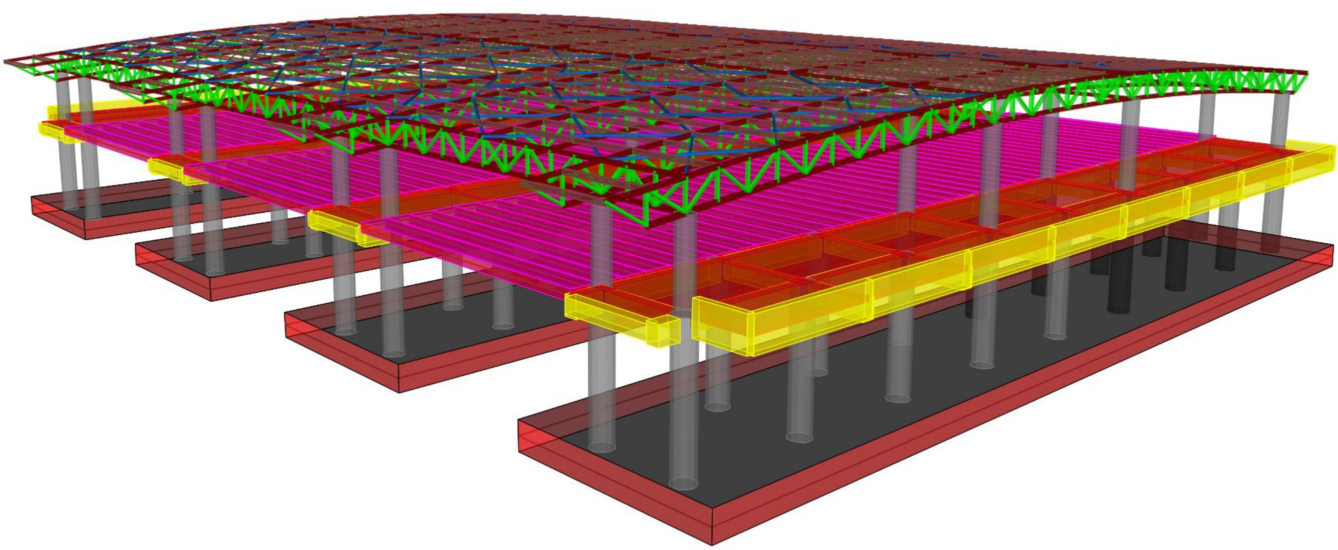
Contents

MARMARAY FENERYOLU STATION



MARMARAY FENERYOLU STATION



INTRODUCTION

This report introduces the final design details of Marmaray Feneryolu Station, one of the 40 stations of Marmaray Project in Feneryolu/Istanbul.

MARMARAY PROJECT

Marmaray is a transportation line which starts at Halkalı/Istanbul (European Side of Istanbul) and ends at Tershane/Istanbul (Anatolian Side of Istanbul) by passing under Marmara Sea. Line of communication has a length of 76.6 km and it includes 37 surface stations and 3 underground stations. This line of communication will serve 75000 passengers per hour, resulting in 1.5 million passengers per day after it is completed.

Line of communication of Marmaray Project and expected location of Marmaray Feneryolu Station are shown in Figure-1 and Figure-2, respectively.

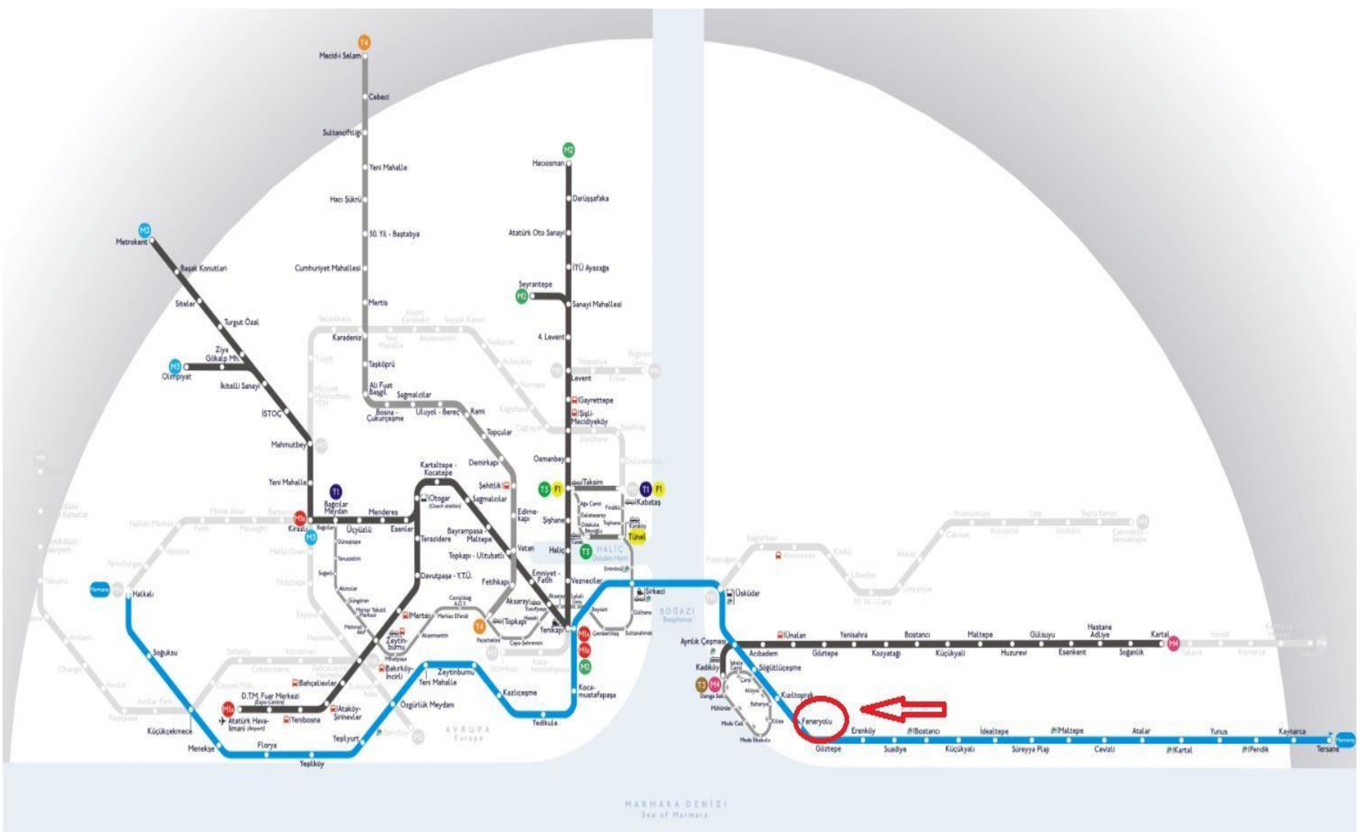


Figure 1: Line of Communication of Marmaray Project

Figure 2: Expected Location of Marmaray Feneryolu Station

ARCHITECTURAL LAYOUT

Architectural views of Marmaray Feneryolu Station are provided in the following 3 figures.

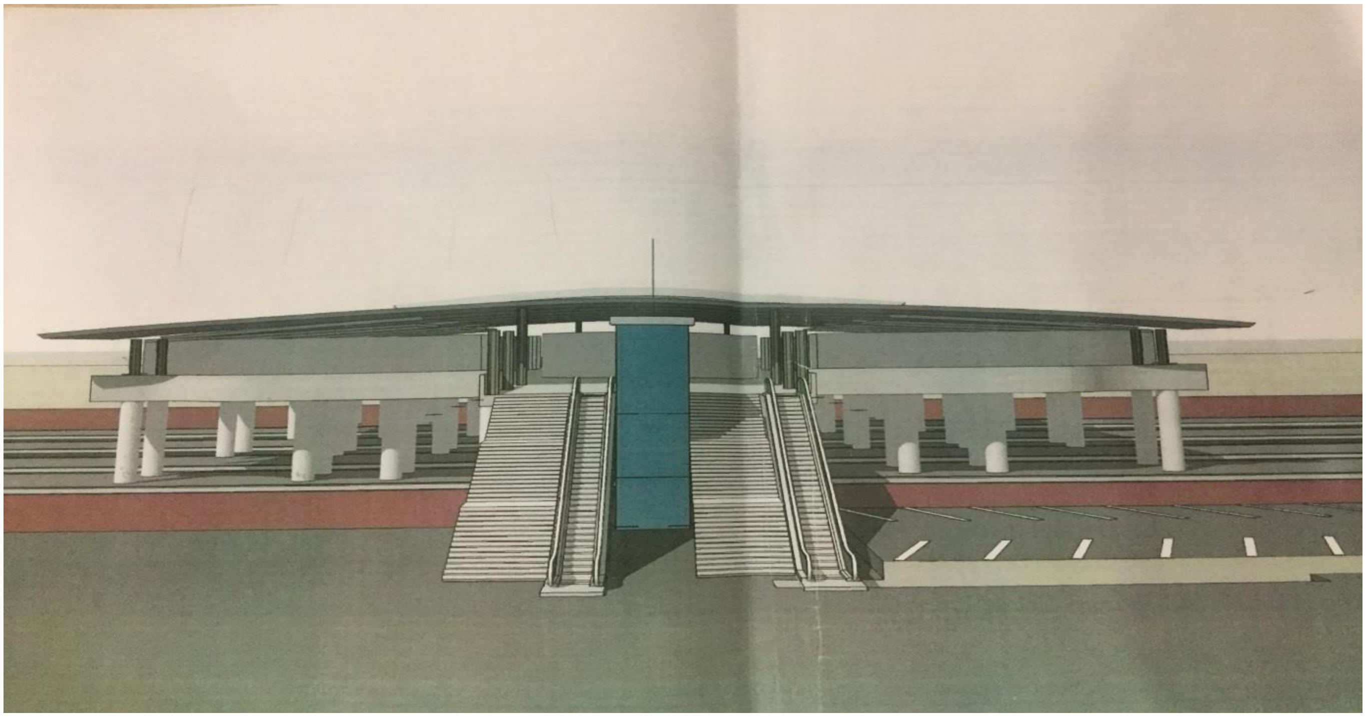


Figure 3: 3D View of Marmaray Feneryolu Station

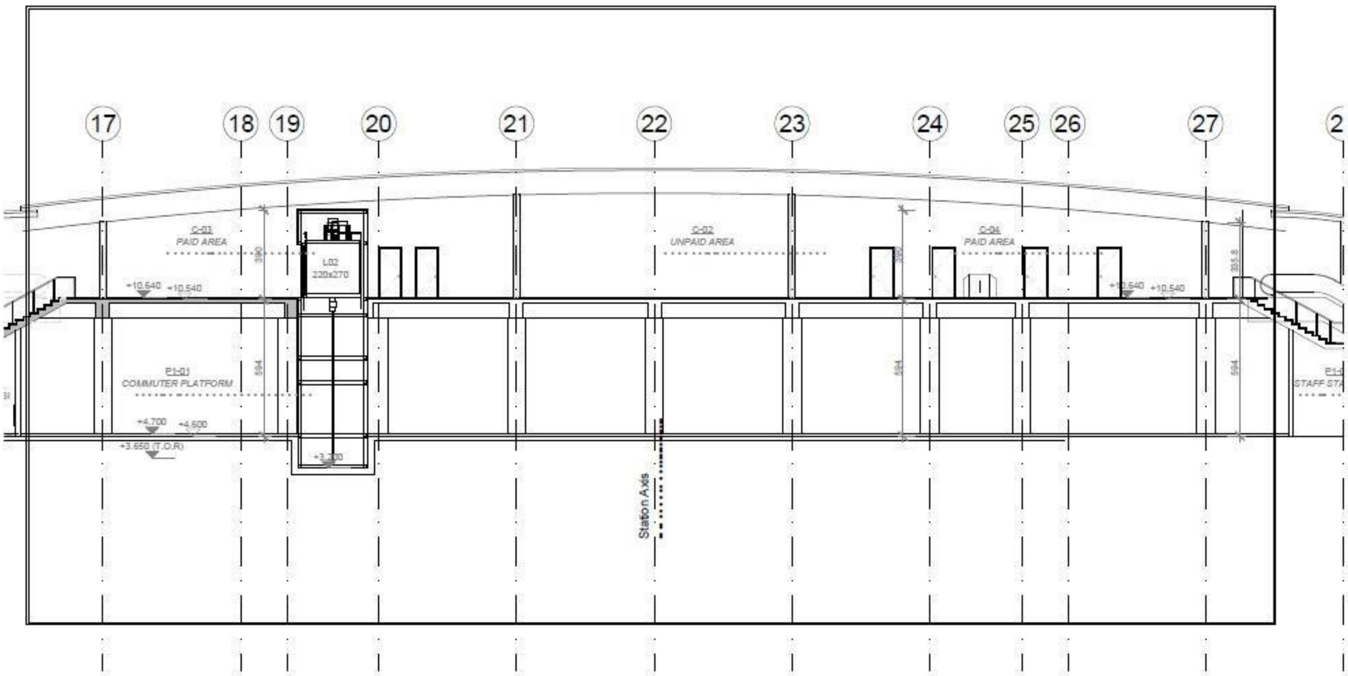


Figure 4: Front View of Marmaray Feneryolu Station According to Figure-3

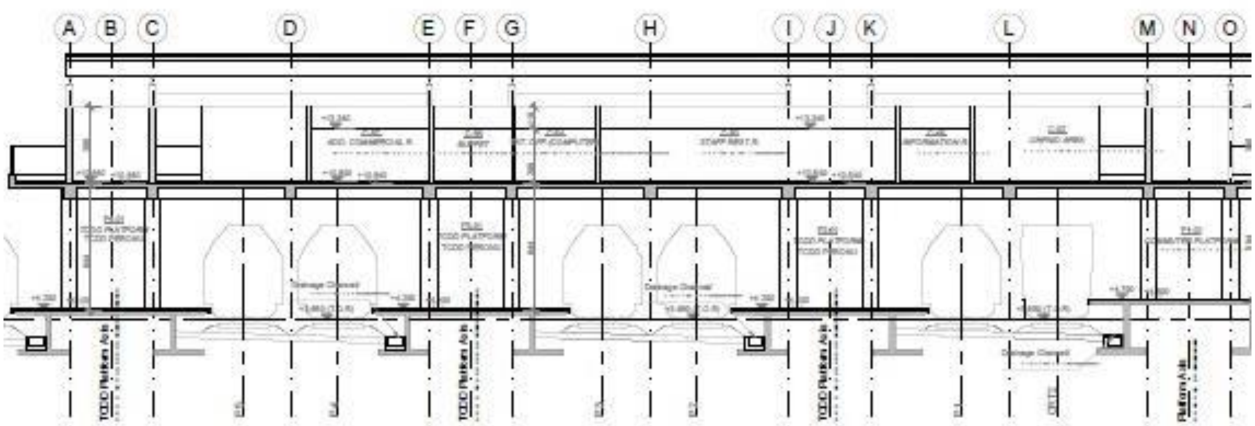


Figure 5: Side View of Marmaray Feneryolu Station According to Figure-3

STRUCTURAL LAYOUT

Structural system of the structure is compatible with the architectural system. The structure includes a steel truss roof system and reinforced concrete columns, beams and slabs. Moreover, there are precast slabs between the platforms.

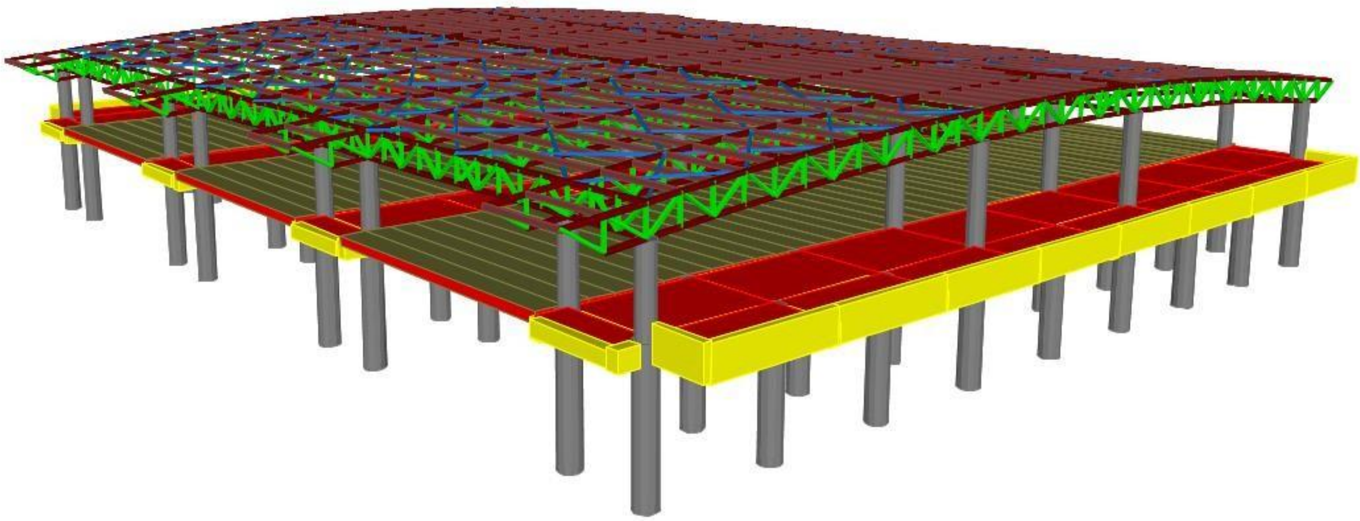


Figure 6: Structural 3D View of the Marmaray Feneryolu Station taken from SAP2000

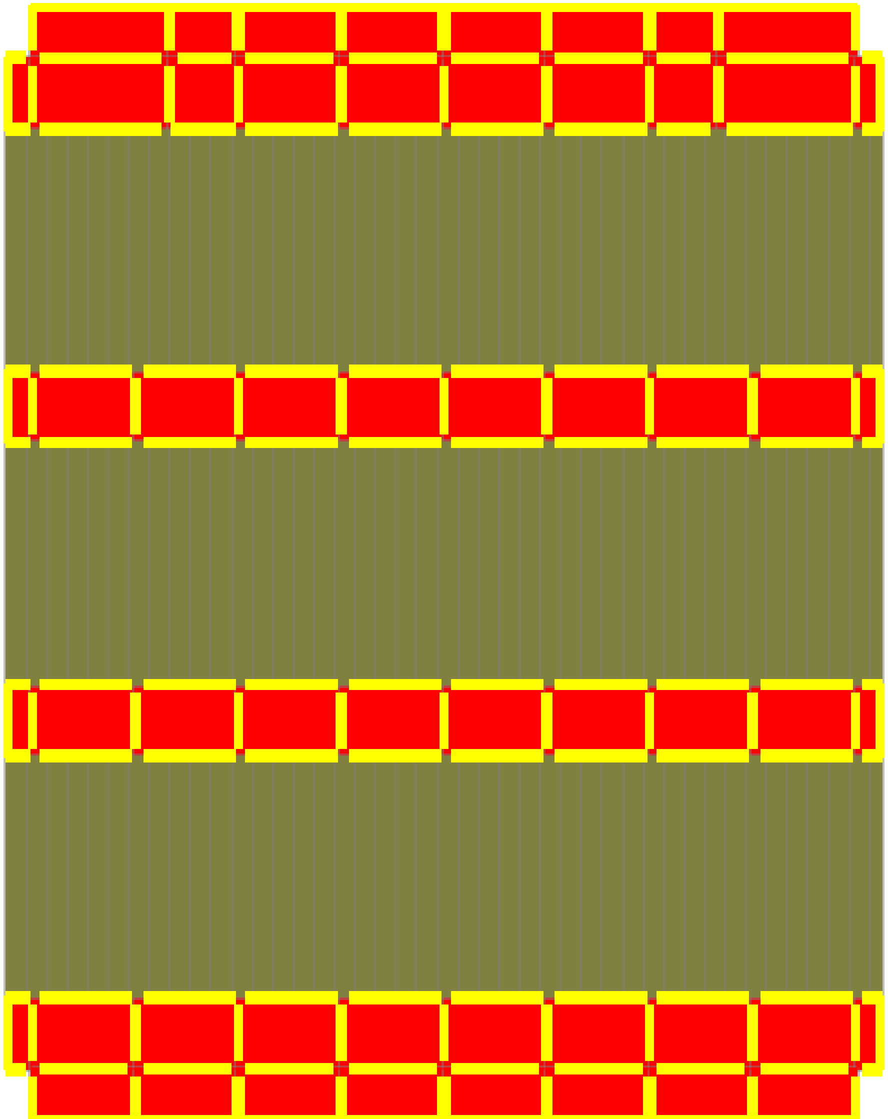


Figure 7: Concourse Floor Layout taken from SAP2000

In Figure-7, red parts represent the normal reinforced concrete slabs which have a thickness of 20 cm. Green parts represent the precast slabs which have 46 cm thickness. Furthermore, the yellow parts represent the beams which have 50x60 cm and 60x80 cm dimensions at different locations. The detailed properties of the slabs and beams will be introduced in the following sections.

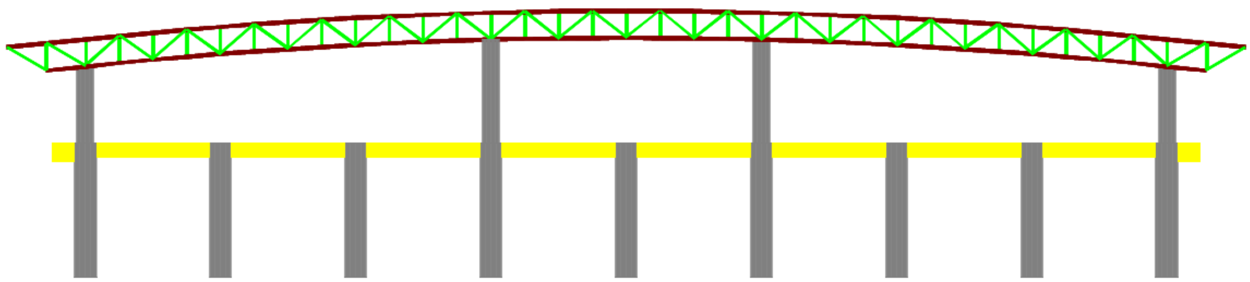


Figure 8: Structural Front View of the Marmaray Feneryolu Station According to Figure-6

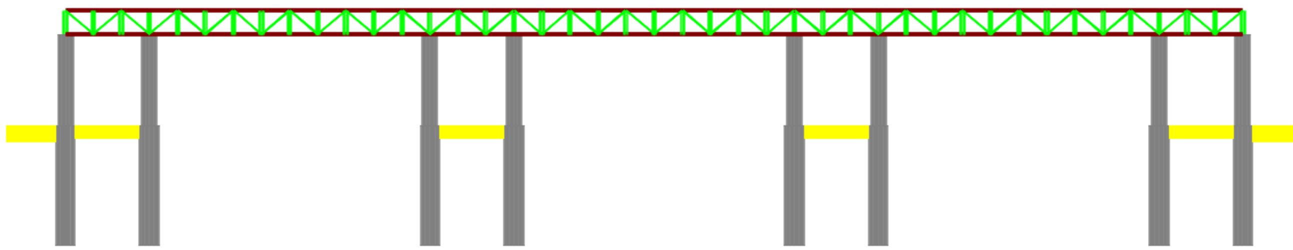


Figure 9: Structural Side View of the Project According to Figure-6

In Figure-8 and 9, grey parts represent the columns which have a diameter of 100 cm and 80 cm at platform level and concourse level, respectively. Moreover, the green and brown parts represent the truss system of Marmaray Feneryolu Station. The detailed design details of the columns and truss system will be provided in the following sections.

DESIGN CRITERIA

This section will provide necessary information about the material properties, design codes, loads and load combinations. In the design process of the project, Ultimate Strength Design (USD) and Load and Resistance Factor Design (LRFD) are considered in concrete design and steel design, respectively.

ANALYSIS AND SHOP DRAWING TOOLS

SAP2000 v19 is utilized as the analysis software and AutoCAD 2016 is used to provide shop drawings.

DESIGN CODES

The standards which are tabulated in Table-1 are used in the design of Marmaray Feneryolu Station.

Table 1: Design Codes Utilized in the Project

DISCIPLINE	STANDARD
GENERAL	• TS-TURKISH STANDARDS
LOADING	• TS498-DESIGN LOADS FOR BUILDING • TSC2007-TURKISH SEISMIC CODE
CONCRETE DESIGN	• TS500-REQUIREMENTS FOR DESIGN AND CONSTRUCTION OF REINFORCED CONCRETE STRUCTURES • TSC2007-TURKISH SEISMIC CODE
STEEL DESIGN	• DESIGN AND CONSTRUCTION SPECIFICATIONS FOR STEEL STRUCTURES by MINISTIRY OF ENVIORENMENT AND URBAN PLANNING (September, 2016)

MATERIAL PROPERTIES

STRUCTURAL STEEL PROPERTIES

S275 hot rolled steel sections are utilized in the design of curved roof part. Table-3 and Table-4 show the strength values for different types of structural steels in accordance with Design and Construction Specifications for Steel Structures by Ministry of Environment and Urban Planning (September, 2016)

Table 2: Mechanical Properties of Hot Rolled Steel Sections

Property	Value	Units
Modulus of Elasticity (E)	200000	MPa
Shear Modulus (G)	76923.08	MPa
Poisson's Ratio	0.30	-
Coefficient of Thermal Expansion (α)	1.0×10^{-5}	/ ºC

Table 3: Nominal Values of Yield Strength f_y and Ultimate Tensile Strength f_u for Hot Rolled Structural Steel

Standard and Steel Grade	Nominal Thickness of the Element t [mm]			
	$t \leq 40mm$		$40mm < t \leq 80mm$	
	Fy(N/mm^2)	Fu(N/mm^2)	Fy(N/mm^2)	Fu(N/mm^2)
EN 10025-2				
S235	235	360	215	360
S275	275	430	255	410
S355	355	510	355	470
S450	450	550	410	550
EN 10025-3				
S275 N/NL	275	390	255	370
S355 N/NL	355	490	355	470
S420 N/NL	420	520	390	520
S460 N/NL	460	540	430	540
EN 10025-4				
S275 M/ML	275	370	255	360
S355 M/ML	355	470	335	450
S420 M/ML	420	520	390	500
S460 M/ML	460	540	430	530

EN 10025-5 S235 W S355 W	235 355	360 510	215 355	340 490
EN 10025-6 S460 Q/QL/QL1	460	570	440	550

Table 4: (Table 3 is continued) Nominal Values of Yield Strength f_y and Ultimate Tensile Strength f_u for Hot Rolled Structural Steel

Standard and Steel Grade	Nominal Thickness of the Element t [mm]			
	t ≤ 40mm		40mm < t ≤ 80mm	
	Fy(N/mm^2)	Fu(N/mm^2)	Fy(N/mm^2)	Fu(N/mm^2)
EN 100210-1 S235 H S275 H S355 H S275 NH/NLH S355 NH/NLH S420 NH/NLH S460 NH/NLH	235 275 355 275 355 420 460	360 430 510 390 490 540 560	215 255 335 255 335 390 430	340 410 490 370 470 520 550
EN 10019-1 S235 H S275 H S355 H S275 NH/NLH S355 NH/NLH S460 NH/NLH S275 MH/MLH S355 MH/MLH S420 MH/MLH S460 MH/MLH	235 275 355 275 355 460 275 355 420 460	360 430 510 370 470 550 360 470 500 530		

STEEL CONNECTION MATERIAL PROPERTIES

High strength structural bolts, nuts and washers which conforms to ASTM A325 and ASTM A490 will be used in the design of connections. Bolts will also conform to Grade A, ASTM A307.

Table 5: Types of Bolts with their Yield and Ultimate Strength

Bolt Type	4.6	4.8	5.6	5.8	6.8	8.8	10.9
Fy (MPa)	240	320	300	400	480	640	900
Fu (MPa)	400	400	500	500	600	800	1000

In weld design, E70XX Electrode which has 500 MPa strength value is utilized, and S355 steel grade is selected as gusset plate in all connections. Thickness of the plates at different connections will be provided in the following sections. M24 6.8 anchor rods are used in the steel truss and reinforced concrete column connection. Moreover, M50 6.8 rods are selected in the design of main pin connection.

CONCRETE PROPERTIES

C30 concrete class is selected for the design of Marmaray Feneryolu Station

Table 6: Mechanical Properties of C30 Concrete Class

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

Table 7: Concrete Classes and their 28 Day Strength taken from TS500-2000

Concrete Class	28-Day Strength (MPa)			
	150x300 mm Cylinder Compressive Strength f_{ck}	Equivalent Compressive Strength (150 mm cube)	Uniaxial Tensile Strength f_{ctk}	Modulus of Elasticity E_{c28}
C16	16	20	1.4	27000
C18	18	22	1.5	27500
C20	20	25	1.6	28000
C25	25	30	1.8	30000
C30	30	37	1.9	32000
C35	35	45	2.1	33000
C40	40	50	2.2	34000
C45	45	55	2.3	36000
C50	50	60	2.5	37000

REINFORCING STEEL PROPERTIES

Reinforcing steel is selected as S420a for the design of Marmaray Feneryolu Station.

Table 8: Mechanical Properties of Reinforcing Steel S420a

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

Table 9: Mechanical Properties of Reinforcing Steel taken from TS500-2000

Mechanical Properties	Reinforcing Bars				Welded Wire Mesh	
	Hot Rolled			Cold Worked		
	S220a	S420a	S500a	S420b	S500bs	S500bk
Minimum Yield Strength, f_{yk} (MPa)	220	420	500	420	500	500
Maximum Strength, f_{su} (MPa)	340	500	550	550	550	550
Minimum Strain Capacity, ε_{su} $\Phi \leq 32$ mm	0.18	0.12	0.12	0.10	0.08	0.05
Minimum Strain Capacity, ε_{su} $32 < \Phi \leq 32$ mm	0.18	0.10	0.10	0.10	0.08	0.05

LOADS AND LOAD COMBINATIONS

The loads listed below with their abbreviations are used in the design of Marmaray Station Building

G: Dead Load

Q: Live Load

Q_r: Roof Live Load

S: Snow Load

T: Temperature Load

W: Wind Load

E: Seismic Load

DEAD LOAD

Dead load of the structure results from self-weight of the materials used and cover load on the roof.

SELF-WEIGHT

Table 10: Parts that contribute to the Self-Weight of the Structure

	Unit Weight	**Thickness (cm)**
Structural Steel (kN/m^3)	78.5	-
Concrete (kN/m^3)	25	-
Precast Slab (kN/m^2)	7.05	46
Partition Wall (kN/m^2)	3	20
Floor Cover (kN/m^3)	27	2
Plaster (kN/m^3)	15	2
Mortar (kN/m^3)	16	2
Marble (kN/m^3)	27	2
Topping (kN/m^3)	1.2	5

COVER LOAD

According to TS ISO 9194, cover load is chosen as 0.15 kN/m^2. Corrugated sheet metal having a 2 mm thickness is selected in order to cover the roof.

LIVE LOAD

Live loads acting on the structure are taken from TS498.

Concourse Floor Live Load

Floor live load is chosen as 5 kN/m^2 from Table-7 in TS498. There is no description for metro station buildings in TS498, but the structure is considered as a public place and 5 kN/m^2 is selected.

Roof Live Load

Roof live load is selected as 0.7 kN/m^2. It is important to note that only one person per meter square is considered on the roof, and therefore; 70 kg/m^2 is taken as roof live load.

SNOW LOAD

The structure is in Feneryolu/Istanbul which is in the second snow region according to TS498 Appendix 1 and the region has an altitude less than 200 meters. Thus, snow load acting on the structure is chosen as P_{k0}=0.75 kN/m^2 from Table 4 in TS498.

$$P_k = m * P_{k0} \rightarrow m = 1 - \frac{\alpha - 30°}{40°}$$

m=1 is selected from Table 3 in TS498 since the roof angle (α) is smaller than 30º. Thus, snow load used in design is calculated as the following;

P_k=0.75 kN/m^2

TEMPERATURE LOAD

The highest temperature and the lowest temperature are 41.5 ^{o}C (in July) and -11.5 ^{o}C (in January) according to Turkish State Meteorological Service and it is assumed that the 20ºC is the base temperature and resulting in temperature change of +21.5 ^{o}C and -31.5 ^{o}C. ±26.5 ^{o}C temperature load is applied to the roof of the structure.

WIND LOAD

Wind load is only applied to the roof of the structure since the structure is open to the atmosphere. Since the structure is open all around, there exists a big pressure under the roof which can result in destruction on the roofs. In order to explain the behavior of this pressure, 0.96q suction is applied on the roof. q=0.8 kPa is taken since the height of the structure is in between 9 and 20 meters. (see Figure 10)

Zeminden Yükseklik m	Rüzgar Hızı v m/s	Emme q (kN/m^2)
0 - 8	28	0,5
9 - 20	36	0,8
21 - 100	42	1,1
> 100	46	1,3

Figure 10: Wind Load Acting on Per Meter Square According to TS498 (Table-5)

SEISMIC LOAD

Total Equivalent Seismic Load (Base Shear)

$$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10 A_0 I W$$

Spectral Acceleration Coefficient

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

Seismic Zone	A_o
1	0.40
2	0.30
3	0.20
4	0.10

Figure 11: Effective Ground Acceleration Coefficients taken from Table 2.2 in TSC2007

Since the structure is located in Feneryolu/Istanbul which is in the 1st seismic zone, $A_0 = 0.4$ is chosen.

Spectrum Coefficient

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad 0 \le T \le T_A$$

$$S(T) = 2.5 \qquad T_A < T \le T_B$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \qquad T_B < T$$

Spectrum equations taken from section 2.4.3 in TSC2007

Local Site Class according to Table 6.2	T_A *(second)*	T_B *(second)*
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Figure 12: Spectrum Characteristic Periods taken from Table 2.4 in TSC2007

In accordance with the soil report prepared by TOKER Drilling and Construction Engineering Consulting CO, structure is located on Z2 local site class. Therefore, TA=0.15 and TB=0.40 are selected.

Building Importance Factor

Purpose of Occupancy or Type of Building	*Importance Factor (I)*
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

Figure 13: Building Importance Factors taken from Table 2.3 in TSC2007

It is important to note that Marmaray Feneryolu Station is a structure in which people occupy intensively but short-term; therefore, $I = 1.2$ is selected.

Seismic Load Reduction Factor

The followings are the seismic load reduction factor equations taken from section 2.5 in TSC2007

$$R_a(T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad 0 \leq T \leq T_A$$

$$R_a(T) = 5 \qquad T_A < T$$

BUILDING STRUCTURAL SYSTEM	*Systems of Nominal Ductility Level*	*Systems of High Ductility Level*
(1) CAST-IN-SITE REINFORCED CONCRETE BUILDINGS		
(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
(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS		
(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..........	5	8
(3.2) Single – storey buildings in which seismic loads are fully resisted by columns with connections hinged at the top..........	—	4
(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*	4	5
(b) *Eccentrically braced frames*	—	7
(c) *Reinforced concrete structural walls*..........	4	6
(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*..........	5	6
(b) *Eccentrically braced frames*..........	—	8
(c) *Reinforced concrete structural walls*..........	4	7

Figure 14: Structural System Behavior Factors

Total Building Weight

$W = \sum_{i=1}^{N} w_i$

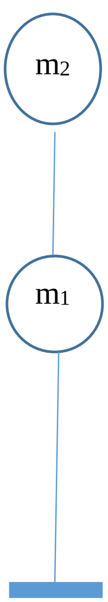
$w_i = g_i + nq_i \rightarrow n \; is \; the \; live \; load \; partication \; factor$

Purpose of Occupancy of Building	*n*
Depot, warehouse, etc.	0.80
School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0.60
Residence, office, hotel, hospital, etc.	0.30

Figure 15: Live Load Participation Factors taken from TSC2007

It is important to note that Marmaray Feneryolu station is a crowded place; therefore, live load participation factor is selected as n=0.60 for the concourse floor. Furthermore, n=0.30 is taken as the snow load participation factor on the roof in accordance with TSC2007 (last sentence on page 16).

The structural system of Marmaray Feneryolu Station is described multi degrees of freedom as shown in below figure. It is important to note that m_1 and m_2 represents concourse floor and roof of the structure, respectively.



Masses Contributing to m_1 and m_2

m_1= Normal Slab Weight + Precast Slab Weight + Partition Wall Weight + Half Weight of the Above Columns + Half Weight of the Below Columns + Weight of Beams without Slab+ Marble Weight + Floor Cover Weight + Plaster + Mortar + Topping+0.6*(Live Load)

m_2= Steel Roof Self-Weight + Roof Cover Weight + Half Weight of the Below Columns +0.3* (Snow Load)

<u>Mass of Concourse Floor</u>

$w_1 = g_1 + nq_1 \; where \; n = 0.60$

$Concourse\ Floor\ Area = 65.2 * 51.60 = 3364.30\ m^2$

$Precast\ Slab\ Area = 50.9 * 13.06 * 3 = 1994.30\ m^2$

$Normal\ Slab\ Area = 50.9 * 4.8 * 4 + 2.95 * 48 * 2 = 1260.5\ m^2$

$Total\ Area\ of\ Concourse\ Floor\ Columns = 72 * \frac{\pi * D^2}{4} = 72 * \frac{\pi * 1^2}{4} = 72 * 0.785 = 56.55\ m^2$

$Total\ Area\ of\ Roof\ Columns = 32 * \frac{\pi * D^2}{4} = 32 * \frac{\pi * 0.8^2}{4} = 32 * 0.283 = 16.08\ m^2$

Normal Slab Weight $= 1260.5 * 0.2 * 25 = 4311\ kN$

$Precast\ Slab\ Weight = 1994.30 * 7 = 13960.1\ kN$

$Half\ Weight\ of\ Concourse\ Floor\ Columns = 25 * 56.55 * 5.95 = 4205.9\ kN$

$Half\ Weight\ of\ Roof\ Columns = 25 * \frac{16.08}{2} * \frac{4.6}{2} + 25 * \frac{16.05}{2} * \frac{3.4}{2} = 803.36\ kN$

Weight of Concourse Floor Beams $= 44 * 0.5 * 0.4 * 4.2 * 25 + 50.9 * 0.6 * 0.6 * 25 * 8 + 2.95 * 0.5 * 0.4 * 25 * 14 + 0.3 * 1.18 * 2 * 25 * 53.9 = 7451.73$ kN

$Marble = 0.81 * 3364.30 = 2725.01\ kN$

$Plaster = 0.3 * 3364.30 = 1009.3\ kN$

$Floor\ Cover = 0.54 * 3364.30 = 1816.73\ kN$

$Mortar = 0.32 * 3364.30 = 1076.58\ kN$

$Topping = 1.25 * 3364.30 = 4205.375\ kN$

$Partition\ Walls = 4.5 * 472.9 * 3 = 6384.15\ kN$

$Live\ Load = 3364.30 * 5 = 16821.5\ kN$

$w_1 = 6302.5 + 13960.1 + 4205.9 + 803.36 + 2725.01 + 1009.3 + 1816.73 + 1076.58 + 4205.375 + 6384.15 + 0.6 * 16821.5 + 5749.30 = \mathbf{60033.63\ kN}$

Mass of Roof

$w_2 = g_2 + nq_2\ where\ n = 0.30$

$Roof\ Area = 55.2 * 65 - 3.5 * 14.06 * 8 = 3194.3\ m^2$

$Self - Weight\ of\ Steel\ Roof = 1882.90\ kN \rightarrow taken\ from\ SAP2000\ model$

$Weight\ of\ Roof\ Cover = 3194.3 * 0.15 = 479.15\ kN = 47.92\ tons$

$Snow\ Load = 3194.3 * 0.75 = 2395.73\ kN = 239.57\ tons$

$w_2 = 1882.90 + 479.15 + 0.3 * 2395.73 = \mathbf{3080.77\ kN}$

$\boldsymbol{W = \sum_{i=1}^{N} w_i = m_1 + m_2 = 60033.63 + 3080.77 = 63114.4\ kN}$

$T_1 = 0.29\ s \rightarrow taken\ from\ SAP2000\ model$

$T_1 > T_A = 0.15\ s \rightarrow R_a(T_1) = 8\ selected\ for\ the\ longer\ direction$

$S(T_1) = 2.5 \quad since \quad T_A < T \leq T_B$

$A(T_1) = A_0 IS(T_1) = 0.4g * 1.2 * 2.5 = 1.2g$

$$\boldsymbol{V_t = \frac{WA(T_1)}{R_a(T_1)} = \frac{\frac{63114.4}{g} * 1.2g}{8} = 9467.16\ kN}$$

$V_t = \Delta F_N + \sum_{i=1}^{N} F_i$

$\Delta F_N = 0.0075 * N * V_t = 0.0075 * 2 * 9767.16 = 142\ kN$

$$F_i = (V_t - \Delta F_N) \frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j}$$

$$F_1 = (9467.16 - 142) \frac{60033.63 * 5.95}{60033.63 * 5.95 + 3080.77 * 10.55} = 9325.16 * 0.9166 = 8547.44\ kN$$

$$F_2 = (9467.16 - 142) \frac{3080.77 * 10.55}{60033.63 * 5.95 + 3080.77 * 10.55} = 9325.16 * 0.0834 = 777.71\ kN$$

$T_2 = 0.26\ s \rightarrow taken\ from\ SAP2000\ model$

$T_2 > T_A = 0.15\ s \rightarrow R_a(T_2) = 6\ selected\ for\ the\ longer\ direction$

$S(T_2) = 2.5 \quad since \quad T_A < T_2 \leq T_B$

$A(T_1) = A_0 IS(T_1) = 0.4g * 1.2 * 2.5 = 1.2g$

$$\boldsymbol{V_t = \frac{WA(T_2)}{R_a(T_2)} = \frac{\frac{63114.4}{g} * 1.2g}{6} = 12622.88\ kN}$$

$V_t = \Delta F_N + \sum_{i=1}^{N} F_i$

$$\Delta F_N = 0.0075 * N * V_t = 0.0075 * 2 * 12622.88 = 189.35\ kN$$

$$F_i = (V_t - \Delta F_N)\frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j}$$

$$F_1 = (12622.88 - 189.35)\frac{60033.63*5.95}{60033.63*5.95+3080.77*10.55} = 12433.5 * 0.916 = 11396.5\ kN$$

$$F_2 = (12622.88 - 189.35)\frac{60033.63*5.95}{60033.63*5.95+3080.77*10.55} = 12433.5 * 0.0834 = 1036.9\ kN$$

LOAD COMBINATIONS

The design combinations are calculated in accordance with TS500, TSC2007 and Design and Construction Specifications for Steel Structures by Ministry of Environment and Urban Planning (September, 2016).

Table 11: Design Combinations

DISCIPLINE	LOAD COMBINATION	SOURCES
CONCRETE DESIGN ULTIMATE STRENGTH DESIGN	1.4G+1.6Q	TS500 TSC2007 Design and Construction Specifications for Steel Structures by Ministry of Environment and Urban Planning (September, 2016)
	1.0G+1.2Q±1.2T	
	1.0G+1.3Q±1.3W	
	0.9G±1.3W	
SEISMIC DESIGN	1.0G+1.0Q± E_x±0.3E_y	
	1.0G+1.0Q± E_x±0.3E_y	
	0.9G± E_x±0.3E_y	
	0.9G± E_y±0.3E_x	
	1.2G+1.0Q+0.2S± E_x±0.3E_y	
	1.2G+1.0Q+0.2S± E_y±0.3E_x	
STEEL DESIGN LOAD AND RESISTANCE FACTOR DESIGN	1.4G	
	1.2G+1.6Q+0.5(Q_r or S)	
	1.2G+1.6(Q_r or S) +(Q or 0.8W)	
	1.2G+1.0Q+0.5(Q_r or S) +1.6W	

ANALYSIS RESULTS

This part of the report introduces the analysis results which are taken from SAP2000.

FIRST THREE FUNDAMENTAL MODES AND MODE SHAPES

The first three mode shapes and the periods of the structure are shown in the following 3 figures and it is clear that the structure has reasonable mode shapes in x and y directions.

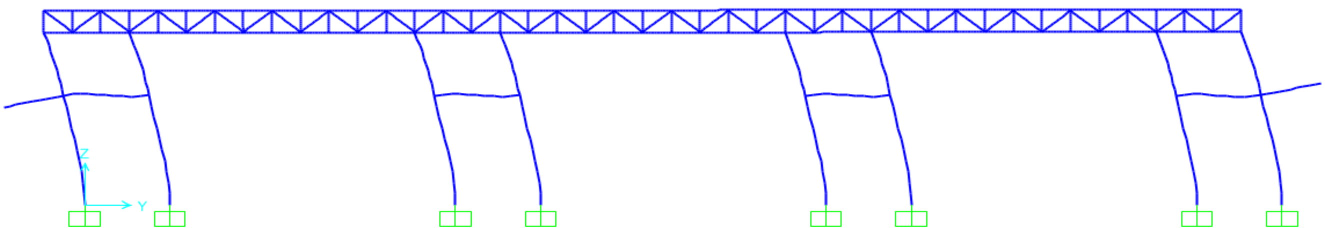


Figure 17: 1st Mode of the Structure in Y-Direction (T_1=0.29 s)

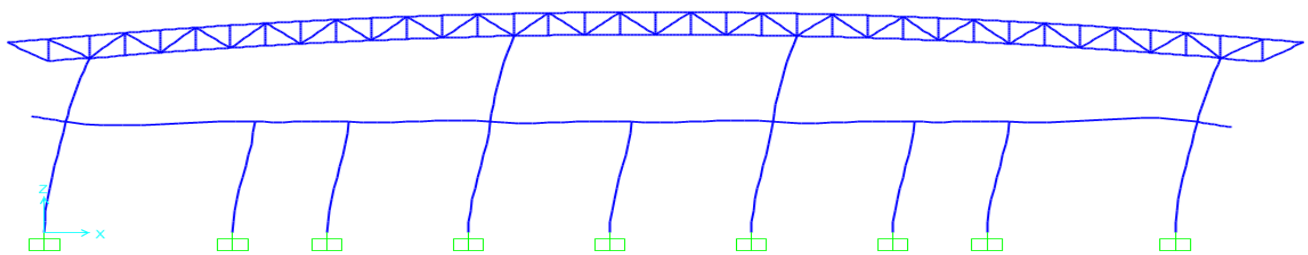


Figure 16: 2nd Mode of the Structure in X-Direction (T_2=0.26 s)

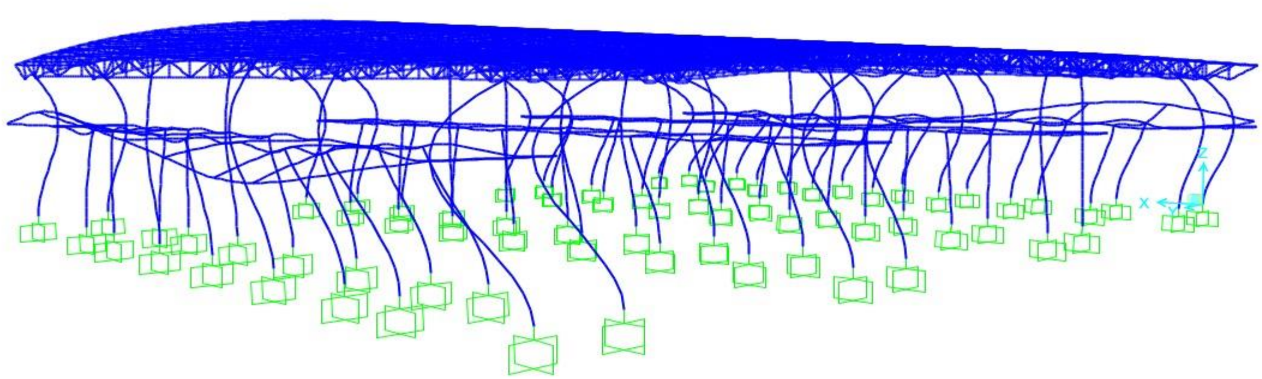
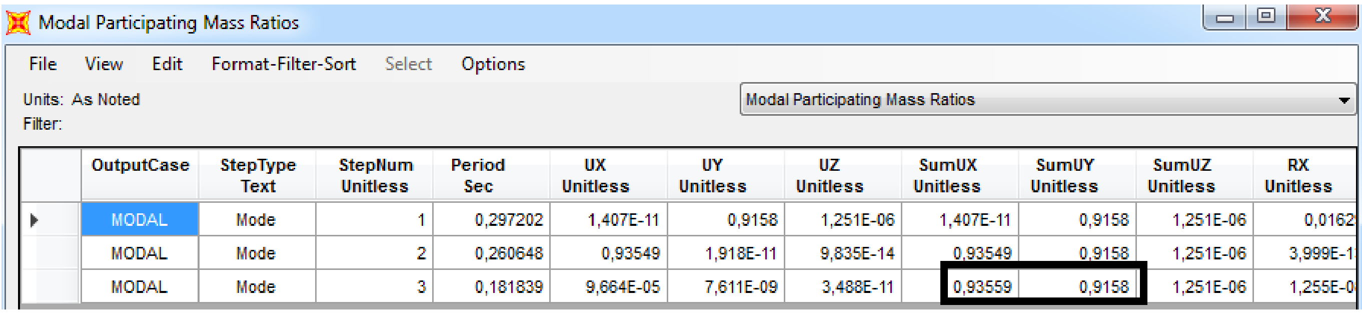


Figure 18: 3rd Mode of the Structure is Torsion (T_3=0.18 s)

MASS PARTICIPATION RATIOS

The mass participation ratios which are taken from SAP2000 are shown in Figure-18 below. Mass participation in x and y directions are above 90 %.


Modal Participating Mass Ratios

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

Modal Participating Mass Ratios

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless	UY Unitless	UZ Unitless	SumUX Unitless	SumUY Unitless	SumUZ Unitless	RX Unitless
MODAL	Mode	1	0,297202	1,407E-11	0,9158	1,251E-06	1,407E-11	0,9158	1,251E-06	0,0162
MODAL	Mode	2	0,260648	0,93549	1,918E-11	9,835E-14	0,93549	0,9158	1,251E-06	3,999E-1
MODAL	Mode	3	0,181839	9,664E-05	7,611E-09	3,488E-11	0,93559	0,9158	1,251E-06	1,255E-0

Figure 19: Mass Participation Ratios in 1st, 2nd and 3rd Mode of the Structure

GOVERNING LOAD COMBINATIONS AND BASE SHEARS IN X AND Y DIRECTIONS

Base shear values are given in Table-12 and the governing load combinations in x and y directions are also provided in this table.

Table 12: Base Shear Values in X and Y Directions

Combinations	Vx (kN)	Vy (kN)
1.0G+1.0Q+1.0EX+0.3	**12622.88**	2839.78
1.0G+1.0Q+1.0EY+0.3	3786.87	**9467.12**

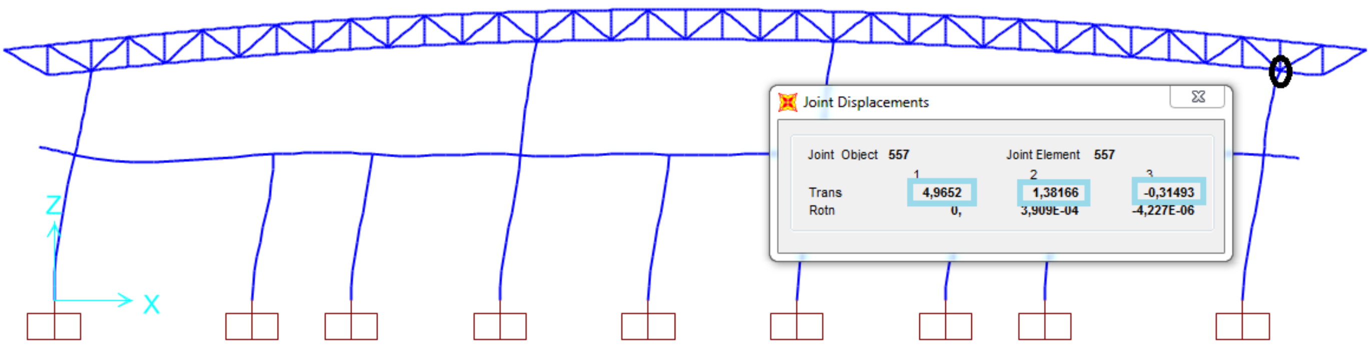


Figure 20: Deflections at the Reinforced Column and Steel Truss Connections owing to 1.0G+1.0Q+1.0EX+0.3EY

According to SAP2000 results, there is a 4.9652 mm deflection in X direction under the effect of 1.0G+1.0Q+1.0EX+0.3EY

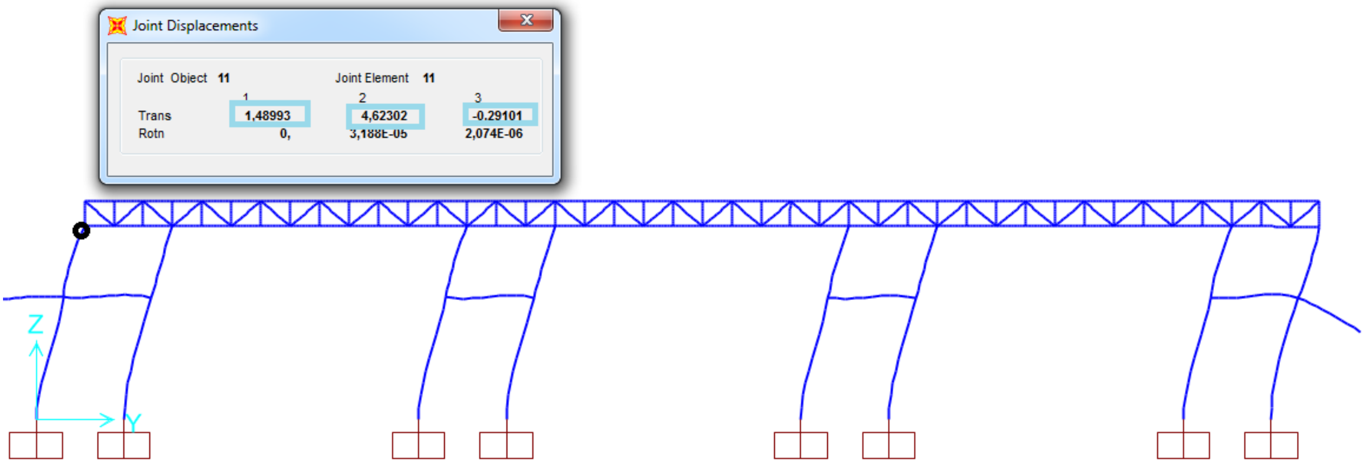


Figure 21: Deflections at the Reinforced Column and Steel Truss Connections Owing to 1.0G+1.0Q+1.0EX+0.3EY

According to SAP2000 results, there is a 4.62302 mm deflection in Y direction under the effect of 1.0G+1.0Q+1.0EY+0.3EX.

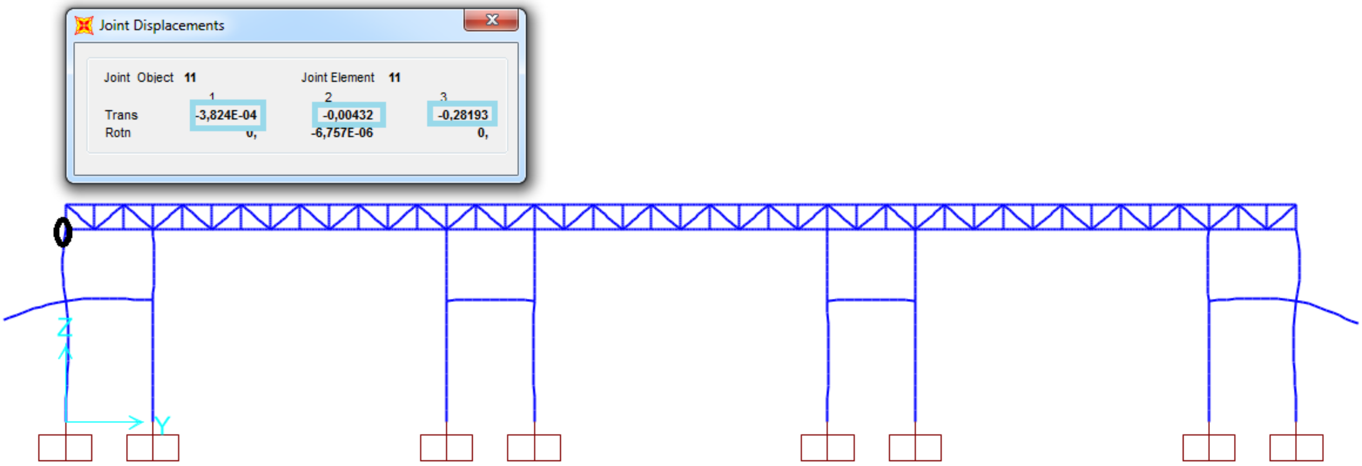


Figure 22: ***Deflections at the Reinforced Column and Steel Truss Connections Owing to 1.2G +1.6Qr +0.8WX +T***

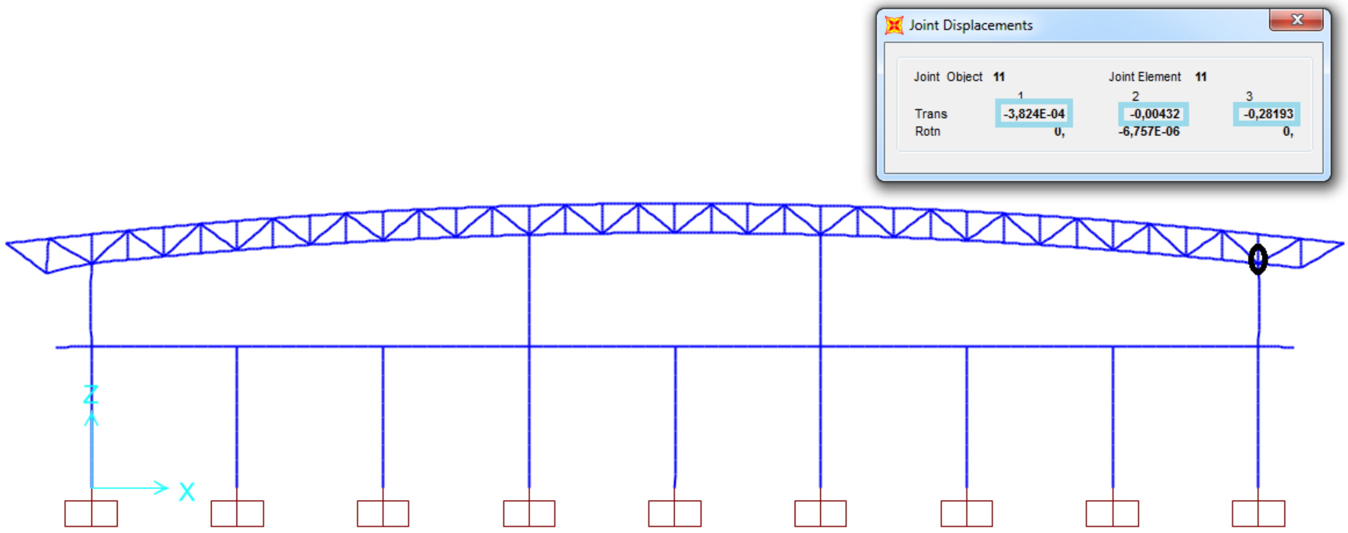


Figure 23: : *Deflections at the Reinforced Column and Steel Truss Connections Owing to 1.2G +1.6Qr +0.8WY +T*

Analysis result shows that deflections due to earthquake are greater than deflections because of temperature effects. Thus, all reinforced column and steel truss system are connected to each other by pin supports. There is no need to use roller support in this design.

It is also important to note that roller supports need more maintenance because it need a frictionless surface not to resist any forces in horizontal direction. Therefore; having a roller support results in a lifelong maintenance cost.

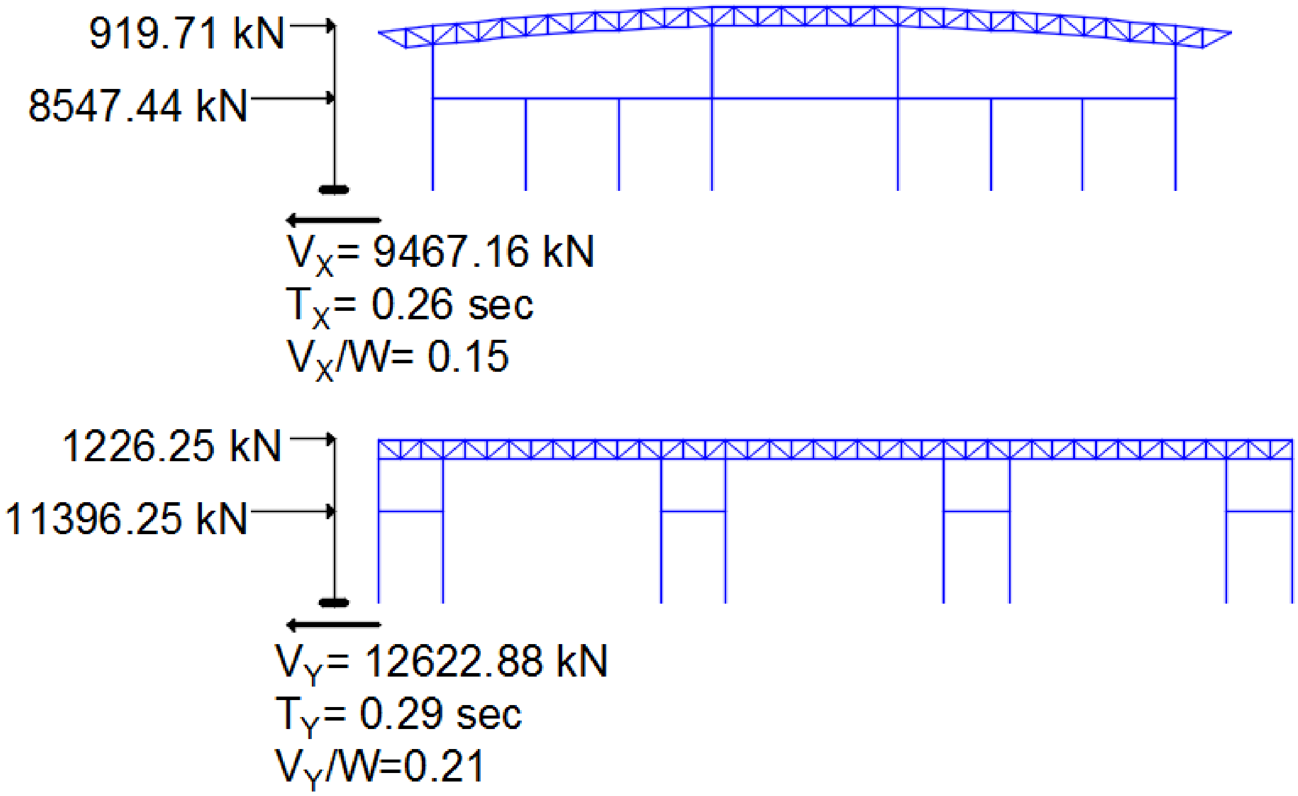


Figure 24: Final Representation of Lateral Load Analysis

FINAL DESIGN

ROOF DESING

Roof of Marmaray Feneryolu Project is made of S275 steel according to architectural desires. The depth of the truss system is 1.2 m at all locations of the roof. There are curved trusses and normal flat trusses in the system.

ROOF COVER

Polycarbonate roof cover sheet is used for the roof

TRUSS DESIGN

In the design of truss system, equations shown below are used and all the calculations are performed by using MS Excel.

Detailed Calculations of Connections

Design of connections are done in accordance with LRFD provisions.

For bolt capacity;

$R_n = F_n * A_b$

F_n: nominal tensile or shear strength of bolt (MPa)

A_b: nominal unthreaded cross section area of bolt (mm^2)

$F_n = 0.563 * F_u \ (Threads\ Excluded)\ (MPa)$

For bearing at bolt holes;

$1.2 * l_c * t * F_u \ \ or\ 2.4 * d_b * t * F_u \ (take\ the\ smaller\ one)$

l_c: clear distance, between edge of the hole and edge of the adjacent hole or the edge of the material (mm)

t: thickness of the connected material (mm)

d_b: nominal bolt diameter (mm)

F_u: minimum tensile strength of the connected material (MPa)

For block shear;

$R_n = 0.6 * A_{gv} * F_y + U_{bs} * F_u * A_{nt}\ or\ R_n = 0.6 * A_{nv} * F_u + U_{bs} * F_u * A_{nt}\ (take\ the\ smaller\ one)$

A_{gv}: gross shear area (mm^2)

A_{nv}: net shear area (mm^2)

A_{nt}: net tension area (mm^2)

F_u: minimum tensile strength of connected material (MPa)

F_y: yield strength of the connected material (MPa)

U_{bs}= 1 if stress is uniform (in most case)

U_{bs}=0.5 if stress is non-uniform

U_{bs}=1 is taken in the calculation

For net area rupture, gross area yielding and shear lag;

$P_n = F_y * A_g$

F_y: specified minimum yield stress (MPa)

A_g: gross area subjected to tension (mm^2)

$P_n = F_u * A_n$

F_u: specified minimum tensile strength (MPa)

A_n: net area subjected to tension (mm^2)

$P_n = F_u * A_e$

F_u: specified minimum tensile strength (MPa)

A_e: effective net area subjected to tension (mm^2)

$A_e = U * A_n$

U: shear lag factor

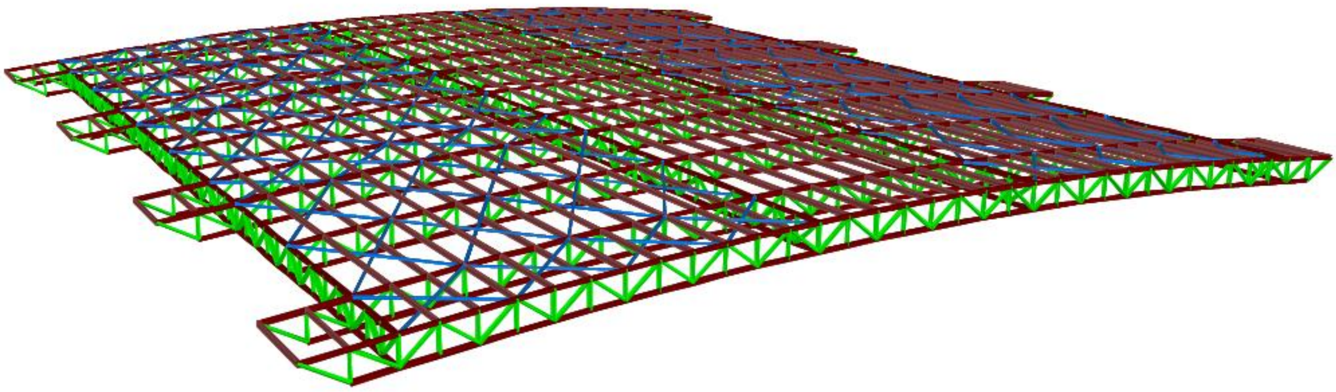


Figure 25: 3D View of Steel Truss Roof System

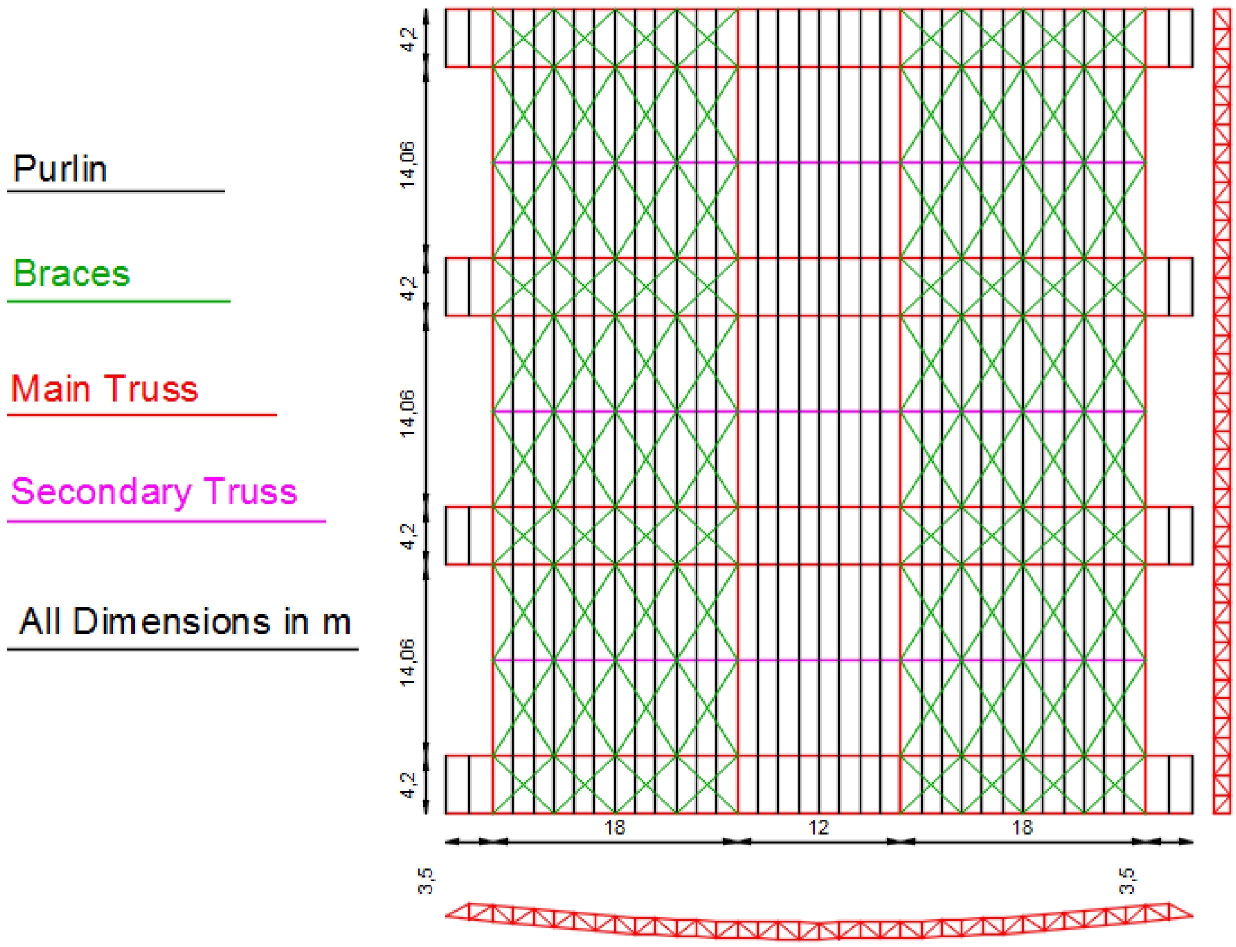


Figure 26: Plan View of the Steel Truss Roof

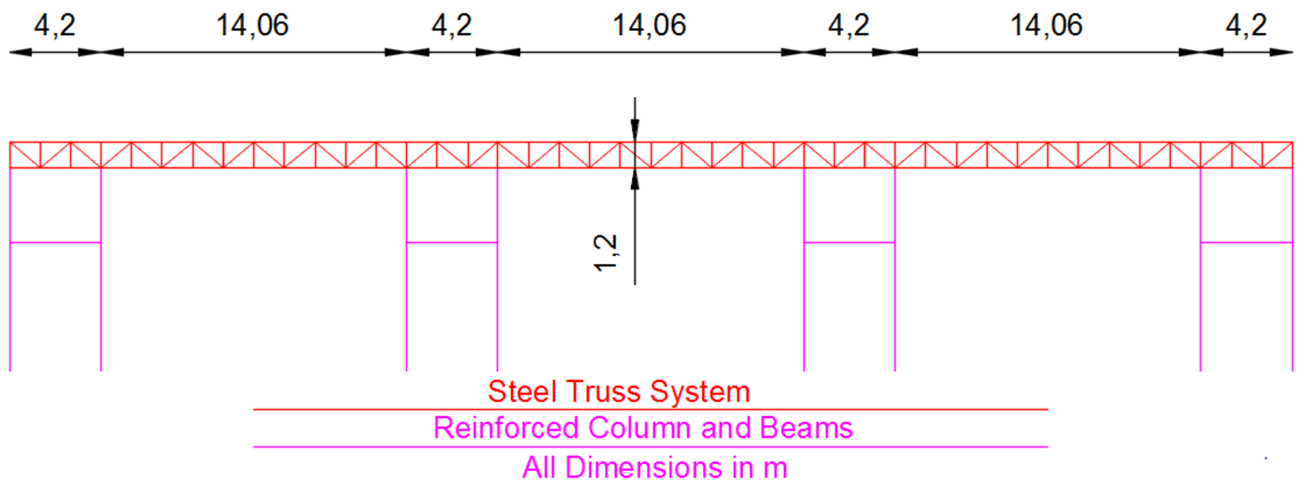


Figure 27: Truss System in Long Direction

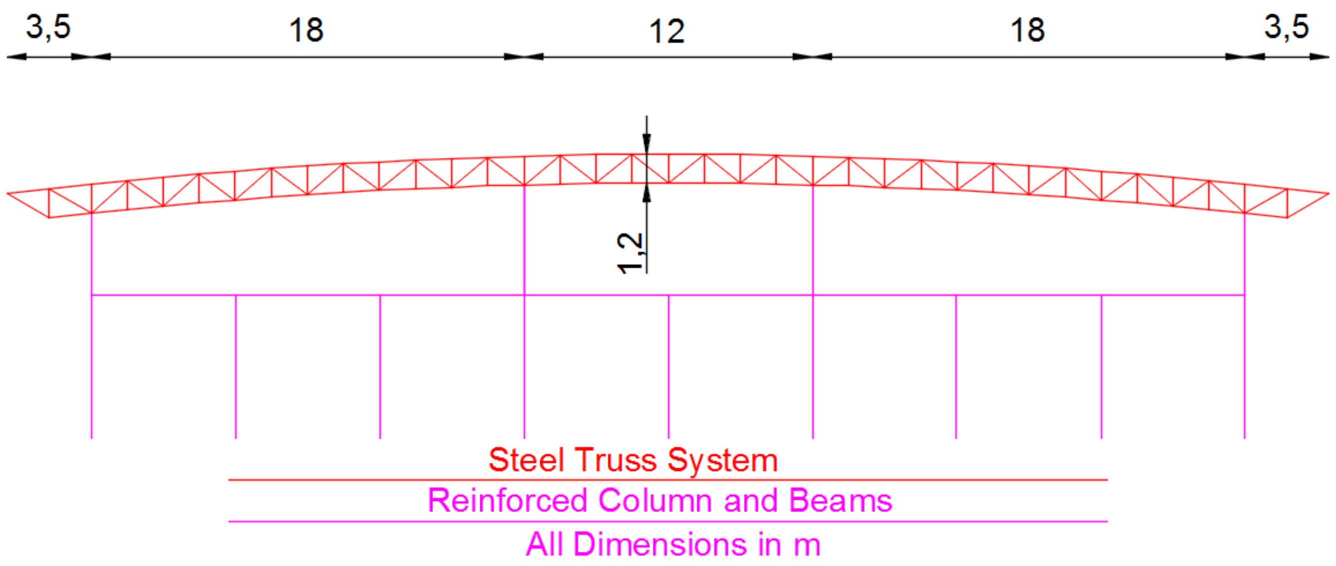


Figure 28: Truss System in Short Direction

TRUSS DESIGN IN SHORT DIRECTION

2UPN140	
Compression Check In Accordance With LRFD	
F_y (Mpa)	275
F_u (Mpa)	430
E (Mpa)	200000
A_g (mm^2)	4080
φ_t	0,75
r_y (mm)	17,50
r_x (mm)	55,00
L (mm)	3000,00
K	1,00
$\frac{KL}{r}$	171,43
$4.71 * \sqrt{\frac{E}{F_y}}$	127,02
Fe (Mpa)	67,17
Fcr (MPa)	49,56
Fcr (MPa)	58,91
Pn (kN)	216,30
SAP2000 Result (kN)	173,00
SECTION IS OK	

Figure 29: Truss Top Chord Design In Accordance With LRFD (Tension Check is in the following sections)

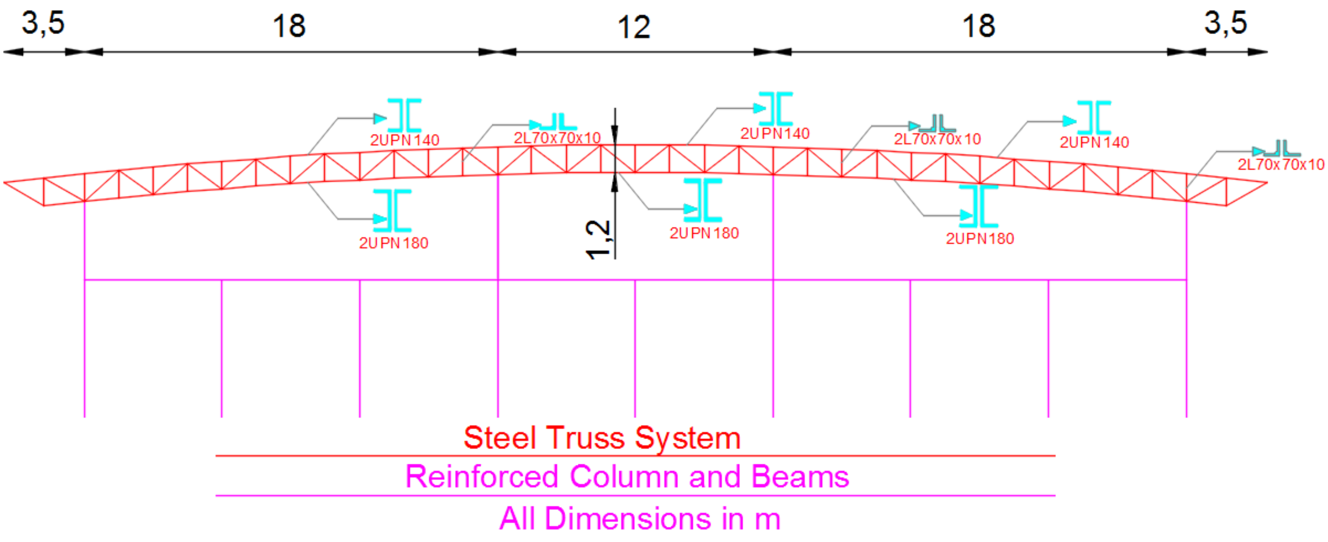


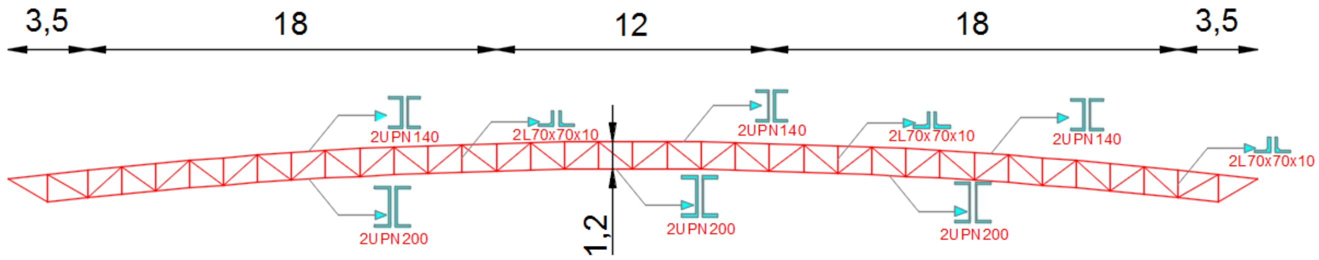
Figure 30: Curved Main Truss Sections

2UPN180	
Compression Check In Accordance With LRFD	
F_y (Mpa)	275
F_u (Mpa)	430
E (Mpa)	200000
A_g (mm^2)	5600
φ_t	0,75
r_y (mm)	20,20
r_x (mm)	69,50
L (mm)	3000,00
K	1,00
$\frac{KL}{r}$	148,51
$4.71 * \sqrt{\frac{E}{F_y}}$	127,02
Fe (Mpa)	89,49
Fcr (MPa)	75,99
Fcr (MPa)	78,49
Pn (kN)	395,57
SAP2000 Result (kN)	318,50
SECTION IS OK	

Figure 31: Truss Bottom Chord Design for Curved Main Truss (Tension Check is in the following sections)

2UPN200	
Compression Check In Accordance With LRFD	
F_y (Mpa)	275
F_u (Mpa)	430
E (Mpa)	200000
A_g (mm^2)	6440
φ_t	0,75
r_y (mm)	21,40
r_x (mm)	77,00
L (mm)	3000,00
K	1,00
$\frac{KL}{r}$	140,19
$4.71 * \sqrt{\frac{E}{F_y}}$	127,02
Fe (Mpa)	100,44
Fcr (MPa)	87,43
Fcr (MPa)	88,09
Pn (kN)	510,56
SAP2000 Result (kN)	453,70
SECTION IS OK	

Figure 32: Truss Bottom Chord Design for Secondary Truss System (Tension Check is in the following sections)



All Dimensions in m

Figure 33: Curved Secondary Truss Sections

TRUSS DESIGN IN LONG DIRECTION

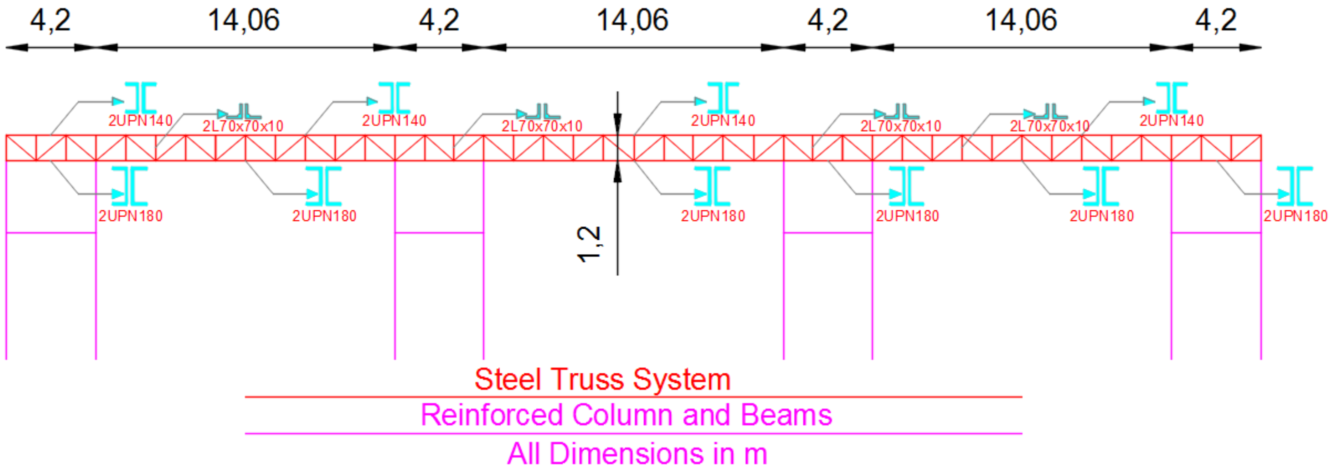


Figure 34: Truss Sections in Long Direction

PURLIN DESIGN

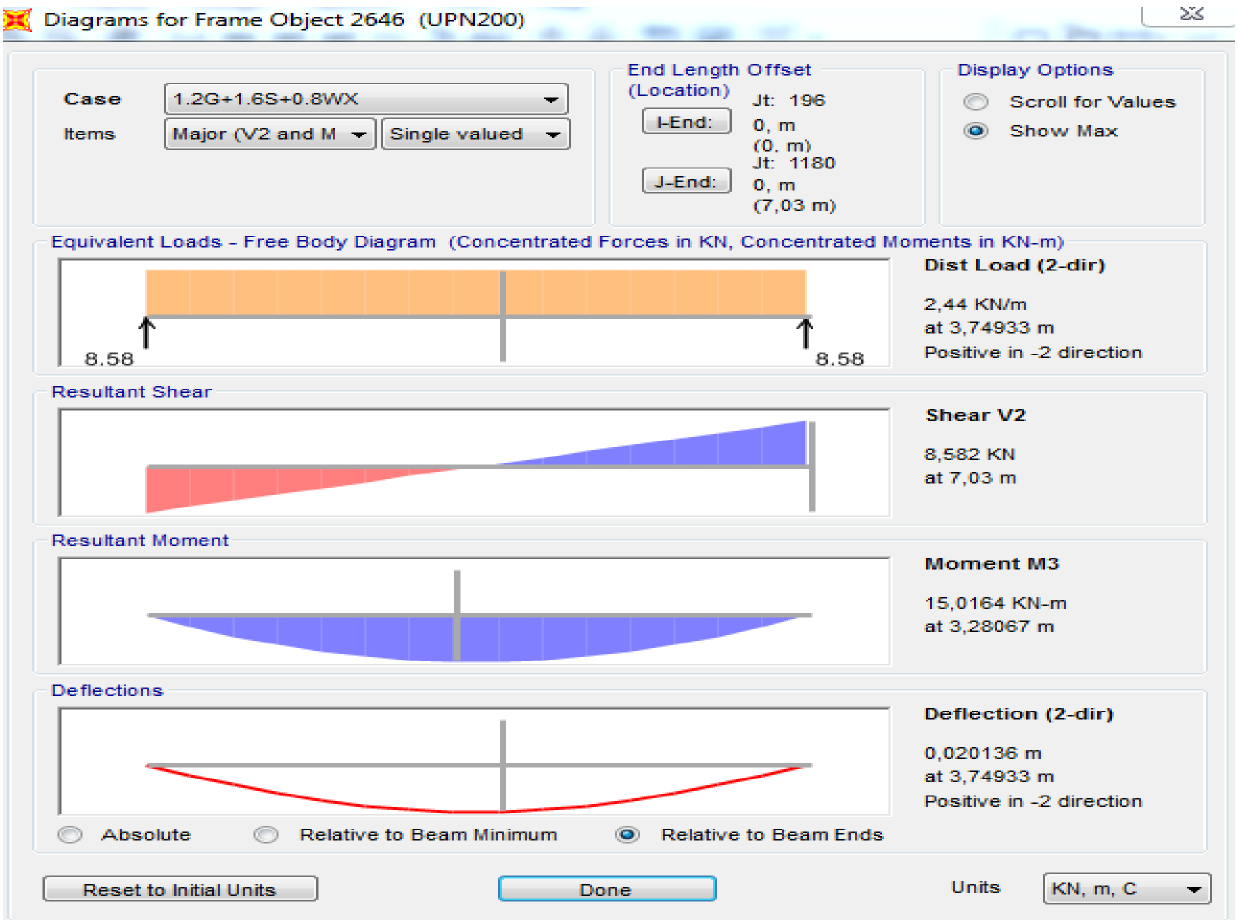


Figure 35: Most Critical Moment Diagram for Purlin Design

Table 13: Purlin Design Calculations

PURLIN DESING IN ACCORDANCE WITH AISC 360-10							
UPN200							
Section Properties	**Values in cm**	**Values in mm**	**Moment and Shear Taken From SAP2000**			**Moment Values for Calculation of Cw**	
d (mm)	20	200	V (kN)		8,58	MA	11,23
b_f (mm)	7,5	75	M (kN*m)		15,02	MB	15,02
t_w (mm)	0,85	8,5	W (kN/m)		2,44	MC	11,23
t_f (mm)	1,15	11,5	**DESIGN**				
A (cm^2)	32,2	3220	M_p (kN)		56,72		
I_x (cm^4)	1910	19100000	**LATERAL TORSIONAL BUCKLING CHECK**				
S_x (cm^3)	191	191000	For Channels c		1,20		
r_x (cm)	7,7	77	L_p (mm)		1015,72		
I_y (cm^4)	148	1480000	r_{ts} (mm)		24,63		
S_y (cm^3)	27	27000	L_r (mm)		4648,88		
r_y (cm)	2,14	21,4	L_b (mm)		7030,00		
x_s (cm)	2,01	20,1	C_w		1,14		
x_m (cm)	3,94	39,4	F_{cr}		141,34		
J (cm^4)	11,9	119000	M_n		24,30		OK
C_w (cm^6)	9070	9,07E+09	**SHEAR DESIGN**				
Z_x (cm^3)	229,155	229155,4	C_v	1,00	20,82	66,33	kv=5 (no Stiffener)
Z_y (cm^4)	No Need		V_n (kN)		280,50		OK

CONNECTION DESIGN

Table 14: Connection Design Detailed Calculation

2L70x70x10		
Tension Check In Accordance With LRFD		
F_y (Mpa)	275	
F_u (Mpa)	430	
E (Mpa)	200000	
A_g (mm^2)	1310	
φ_t	0,90	**→for yielding in the gross section**
φ_t	0,75	**→for rupture in the net section**
Gross Area Yielding		
P_n (kN)	648,45	
Net Section Fracture		
Bolt Type	M14	
Bolt Area (mm^2)	153,94	
Hole Diameter (mm)	24,00	
Number of Holes	1,00	
Connection Lenght (mm)	40,00	
x_s (mm)	20,90	
U	0,48	
t (mm)	10,00	
A_n (mm^2)	1070,00	
A_e (mm^2)	510,93	
P_n (kN)	329,55	
Block Shear		
Outside Length	**50,00**	
Outside Length	**60,00**	
U_{bs}	1,00	
A_{gv} (mm^2)	900,00	
A_{nv} (mm^2)	540,00	
A_{nt} (mm^2)	480,00	
R_n (kN)	345,72	
R_n (kN)	354,90	
P_n (kN)	518,58	
P_n (kN)	329,55	
SAP2000 Result (kN)	293,50	

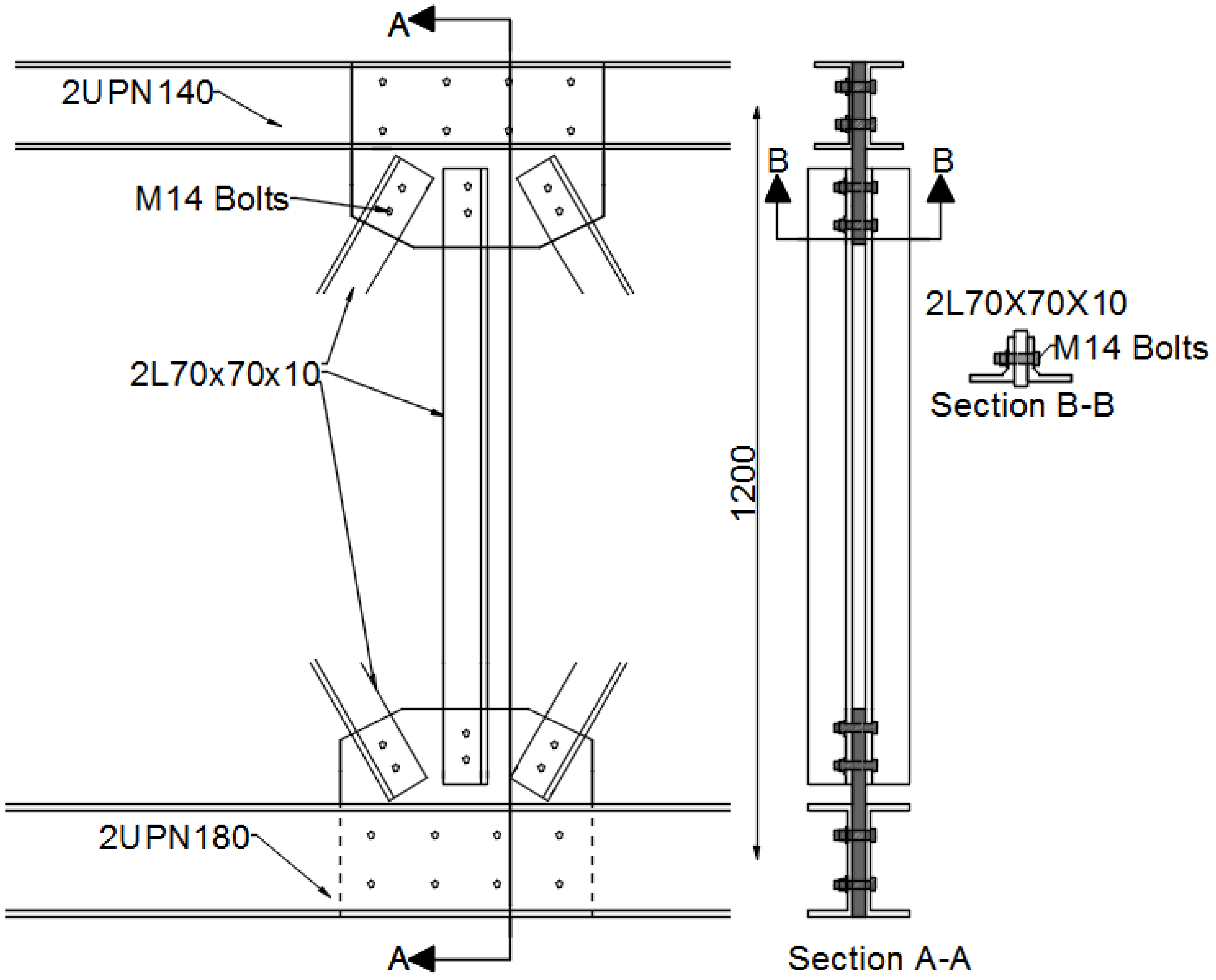


Figure 36: Typical Truss Connection Detail

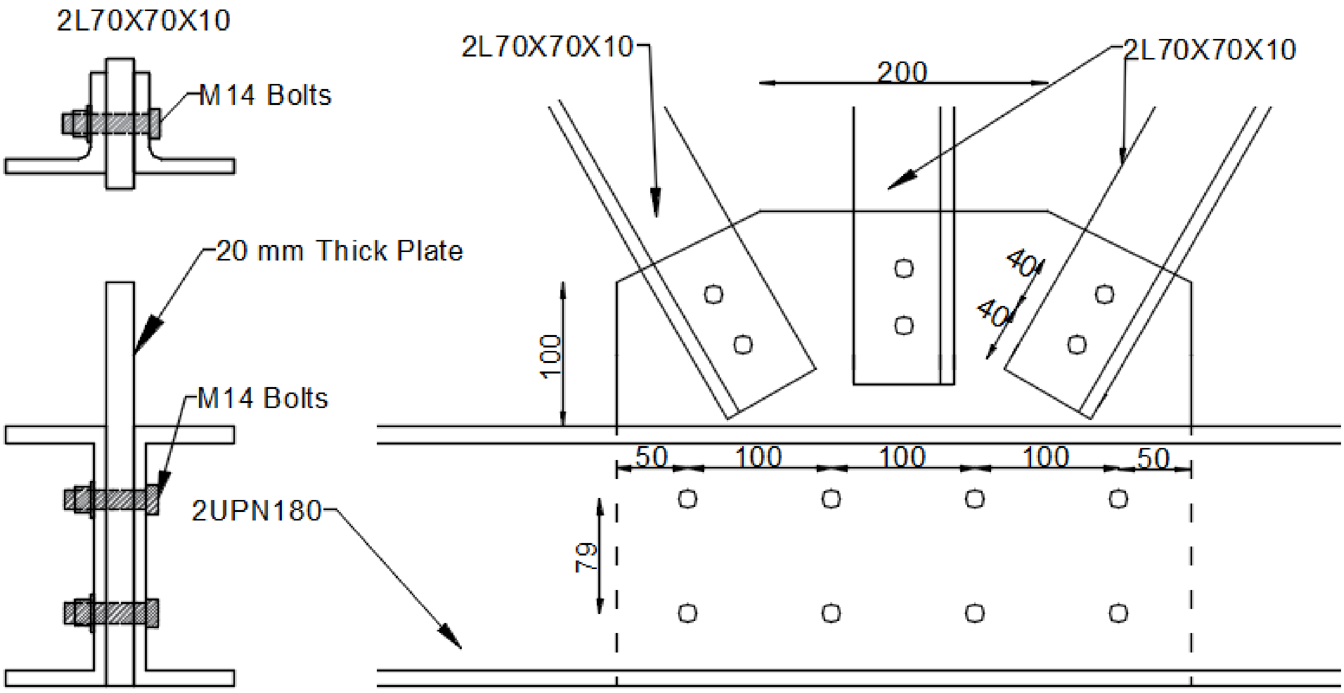


Figure 37: Typical Gusset Plate Diagonal Connection

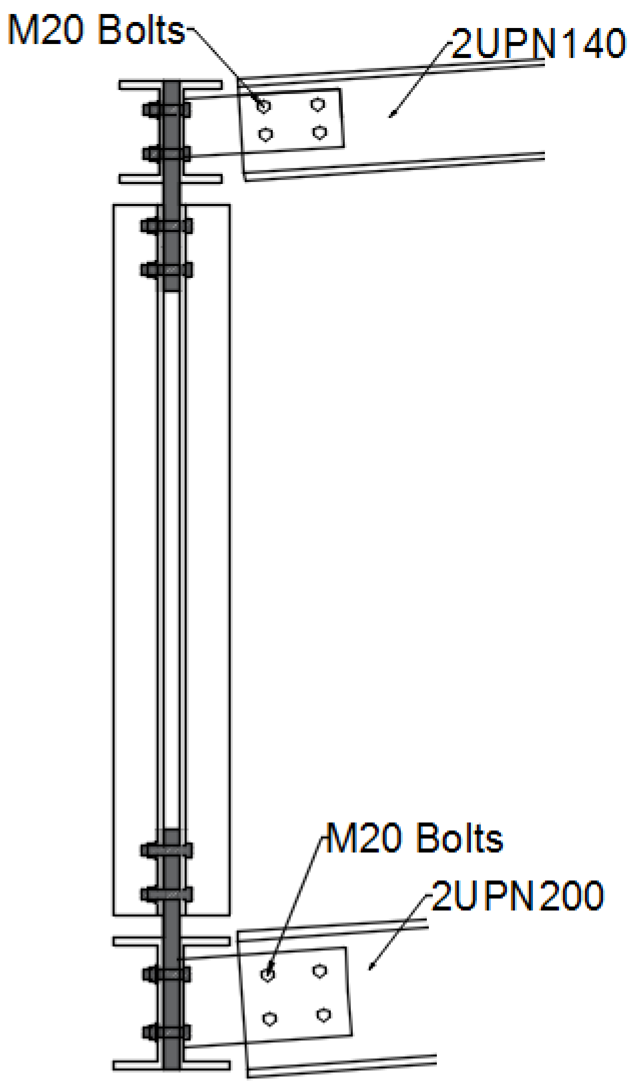


Figure 38: Secondary Truss and Norman Truss Connection

:

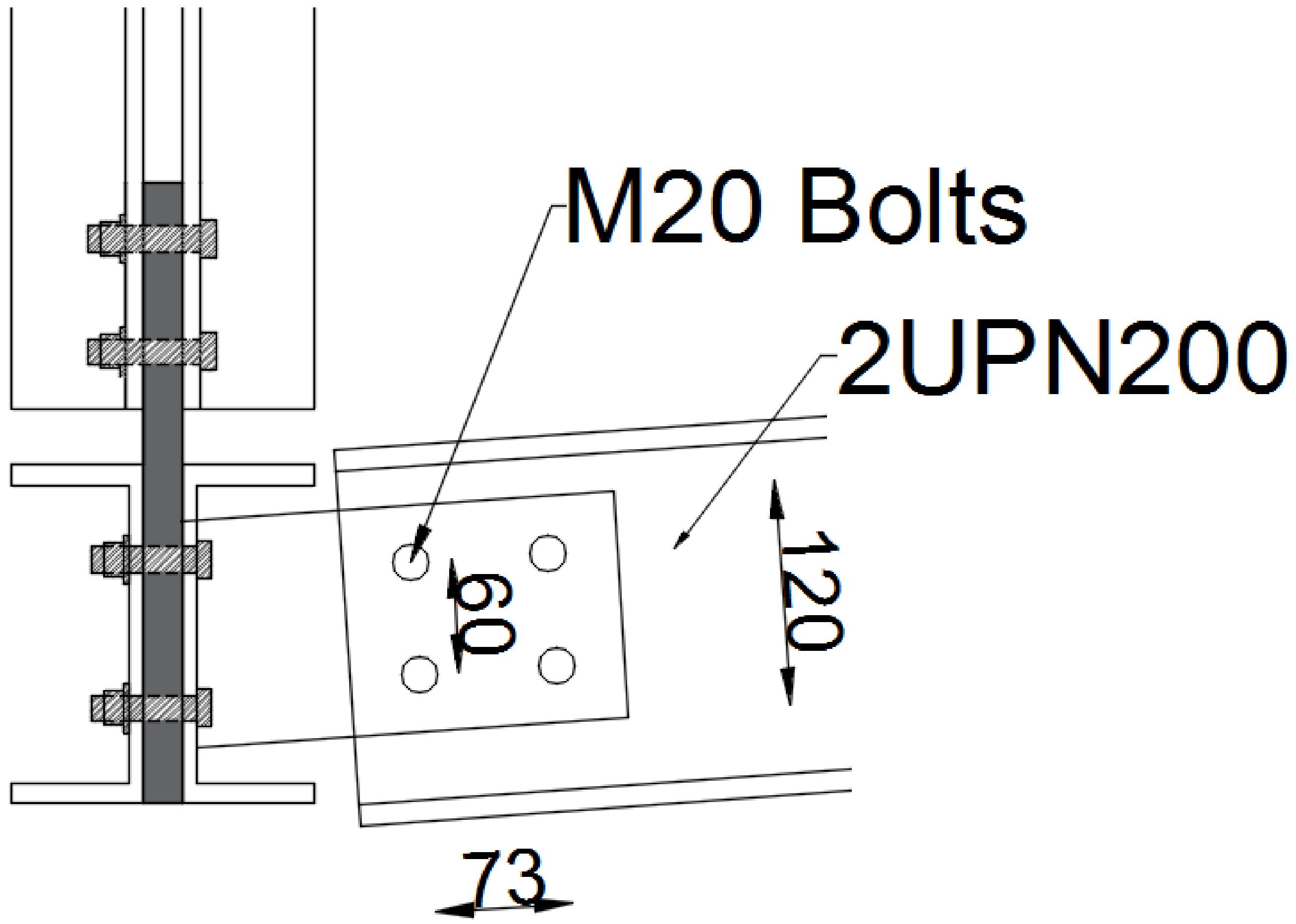


Figure 39: Secondary Truss and Normal Truss Connection Detail (see Figure-38)

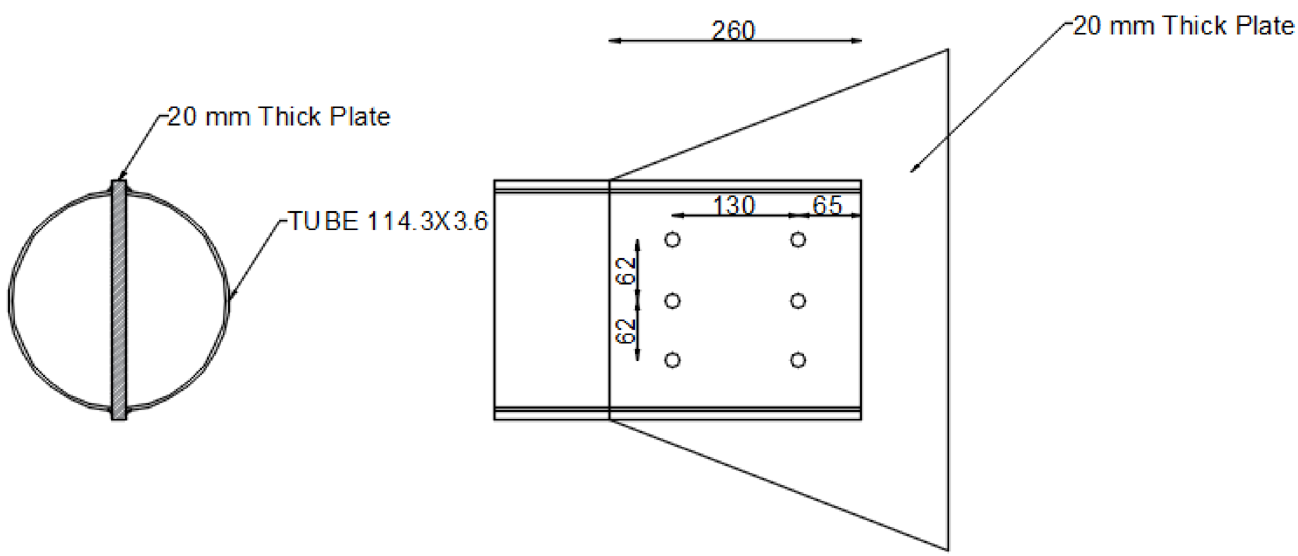


Figure 40: Typical Brace Connection Detail

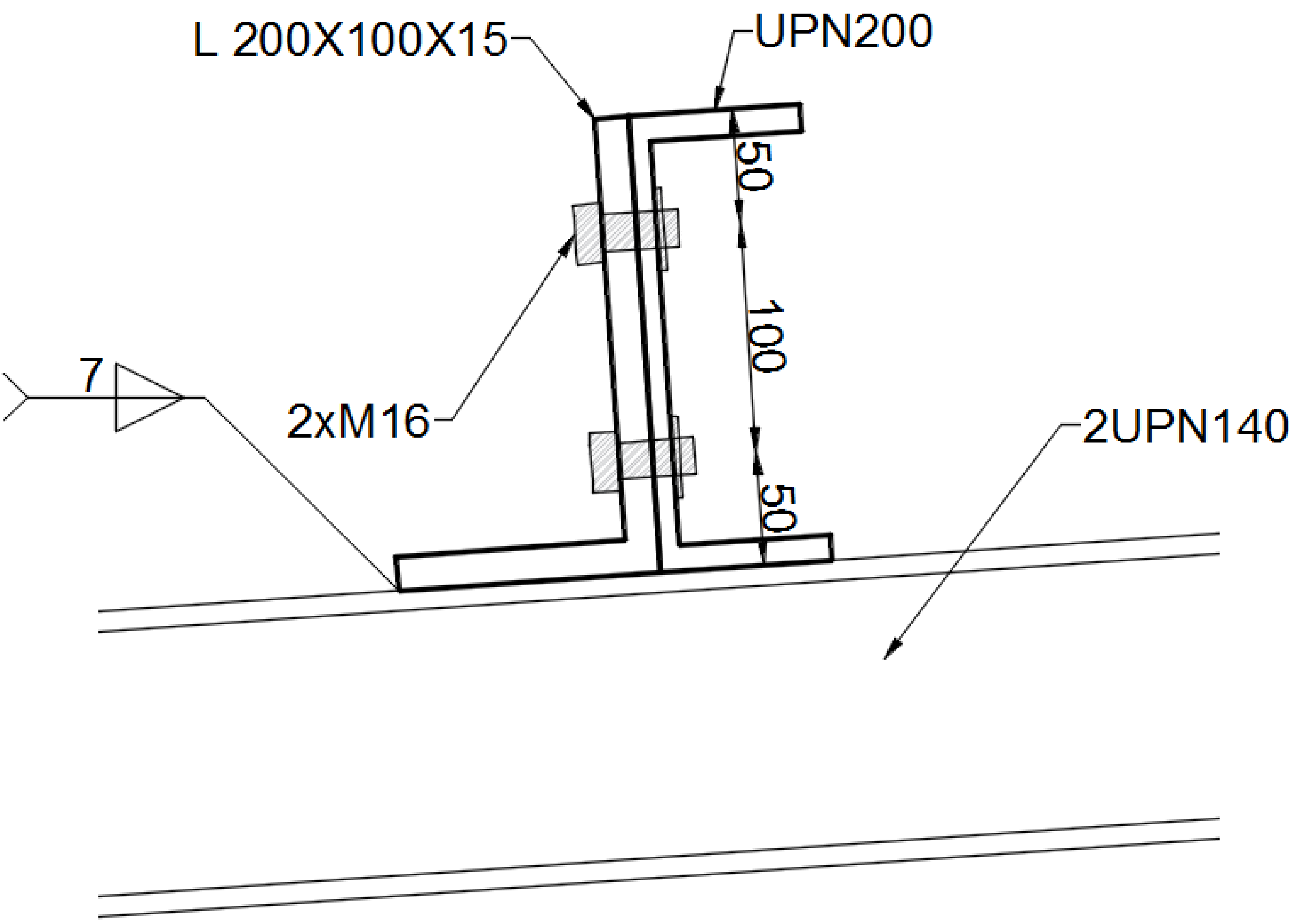


Figure 41: Typical Purlin Connection Detail

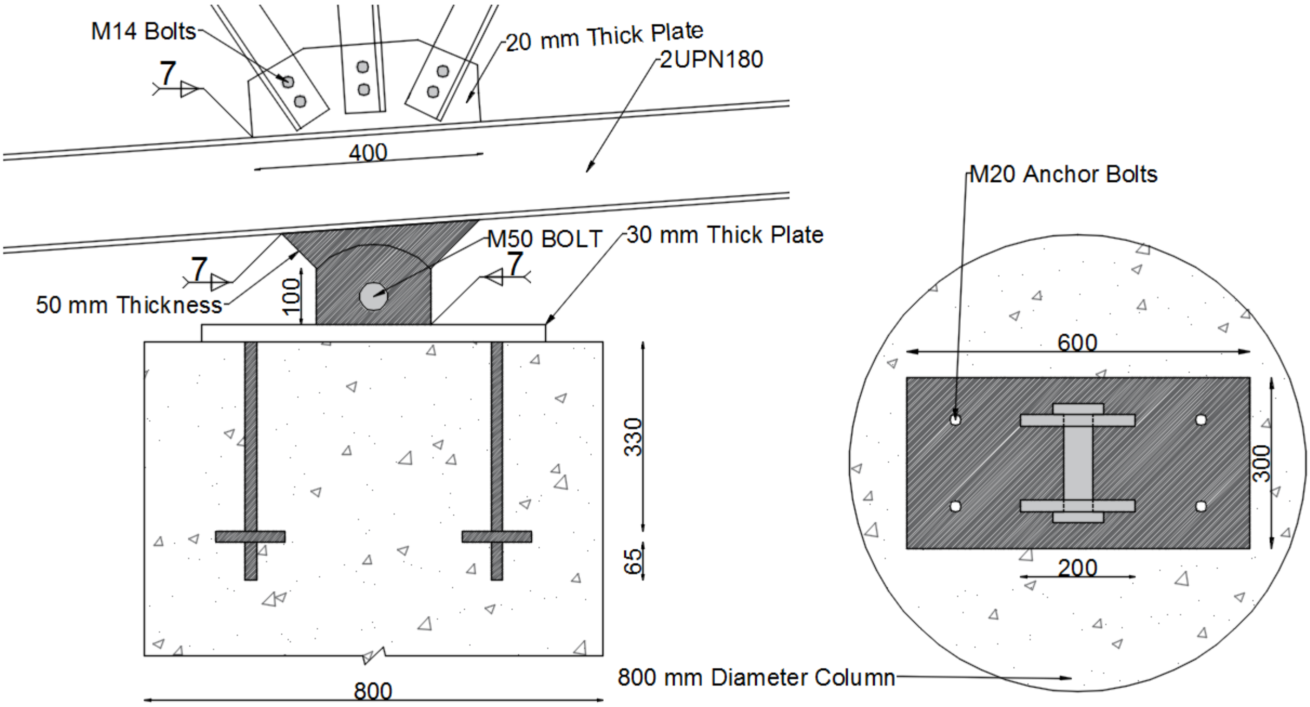


Figure 42: Typical Pin Connection Detail

Calculations are provided below Tables.

Table 15: Pin Connection Bolt Design

PIN CONNECTION DESIGN		
SAP2000 Results	F_x (kN)	408,00
	F_z (kN)	43,00
Design Results	F (kN)	410,26
	Bolt Type	M6.8
	F_u (MPa)	600
	F_y (Mpa)	480
	Bolt Diameter (mm)	50
	Bolt Area (mm^2)	1963,50
	Bolt Shear Capacity	497,45
BOLT IS OK		

Table 16: Anchor Bolt Design

ANCHOR BOLTS	
Bolt Type	M20 ISO 10.9
Bolt Diameter	20
Fu (Mpa)	1000
Fy (Mpa)	900
Minimum Embedded Length (17*d) cm	34
Minimum Embedded Edge Distance (7*d) (cm)	14
Length After the Washer in Concrete	
T=Ag*Ft	
T (kN) (Tension Capacity)	314,16
L_h (Length After the Washer in Concrete)	635,80
Total Anchor Capacity	
Number of Anchors	4
Tension Capacity (kN)	942,48
Shear Capacity (Kn)	530,61

Base Plate Thickness Selection

In this design base plate thickness is selected as 30 mm

According to LRFD;

Allowable bearing stress, $F_P = 0.7*f_C$

Area of plate, $A_1=P/F_P$

Plate dimensions, $N=(A_1)$^0.5 +Δ, Δ=0.5*(0.95*d-0.8*b_f), $B=A_1/N$

Actual bearing pressure, $f_P=P/(N*B)$

m &n, m=(N-d)/2, n=(B-b_f)/2

Plate thickness, larger of m and n,

t_p=(m or n)*($f_P/0.25F_y$)^0.5

Minimum concrete area, because largest bearing stress is selected, A_2=4*N*B

b_f=20 mm and d=400 mm. When calculating according to AISC 360;

Bearing stress capacity, F_P = 0.7*30=21 MPa

Area of plate, A_1=137 kN/21 MPa = 6524 mm^2

Plate dimensions, N=(A_1)^0.5 +Δ, Δ=0.5*(0.95*d-0.8*b_f), B=A_1/N

Δ=182 mm

N=263 mm, B=25 mm

Actual bearing pressure, f_P=137/(263*25)=20.84 MPa

m &n, m=(400-0.95*400)/2=10 mm   n=(20-0.8*20)/2=2 mm

Plate thickness, larger of m and n, t_p=(10)*(20.84/0.25*275)^0.5=5.5 mm

Minimum concrete area, because largest bearing stress is selected, A_2=4*263*25=26300

Column dimensions are 800 mm then dimensions of A_2 are 300 * 90 mm

But we selected plate area as 200 *400 mm and then

A_2<=4*A_1 A_2<=320000 mm^2. Selected area is 300 x 600 mm for concrete foundation and thickness of concrete is arranged according to elevation.

0.35*f_c*(A_2/A_1)^(0.5) =15.75 MPa < 21 MPa (Maximum) then it is OK.

Finally, base plate 300* 600 mm, thickness of plate 30 mm > 5.5 mm

Weld Capacity

Fillet weld was selected. Design is performed in accordance with LRFD. The maximum load is 408 kN (C) from load combinations analysis in SAP2000. So weld design was conducted for this load and applied to other welding connections.

F_u=430 MPa

F_{EXX}=490 MPa

Minimum size of weld (over 6 mm to 13 mm) =5 mm, 7 mm selected

$R_n=0.6*F_u*w*l$ $R_n=0.6*F_{EXX}*a*l$ Where w: size of weld, a: effective throat (mm) $a=0.707*w$, l: total length of weld (mm) $F_u>0.707*F_{EXX}$, no need for base metal check.

$R_n*0.75=408$ then $R_n=544$ kN

$l= R_n/ (0.6*F_{EXX}*0.707*w)$ in mm

l= 544/(0.6*490*0.707*7)=>400 mm. **Weld Length is suggested.**

It is important to note that no need to check the plate because the thickness of plate is greater than total thickness of double sections.

CONCOURSE FLOOR DESIGN

Design of concourse level is performed under the self-weight and the loads shown in Table 17.

Table 17: Loads on the Concourse Level

Loads on the Concourse Level	
Self weight of Precast Slab	8,25 kN/m^2
Mortar	0,32 kN/m^2
Plaster	0,3 kN/m^2
Marble	0,81 kN/m^2
Floor Cover	0,51 kN/m^2
Partition Wall	3 kN/m^2
Live load	5 kN/m^2

PRECAST SLAB DESIGN

In Marmaray Feneryolu Project, precast slabs having a thickness of 46 cm, length of 13.06 m and with of 1.2 m are utilized for the purpose of fast construction stage. These precast slabs are commonly called

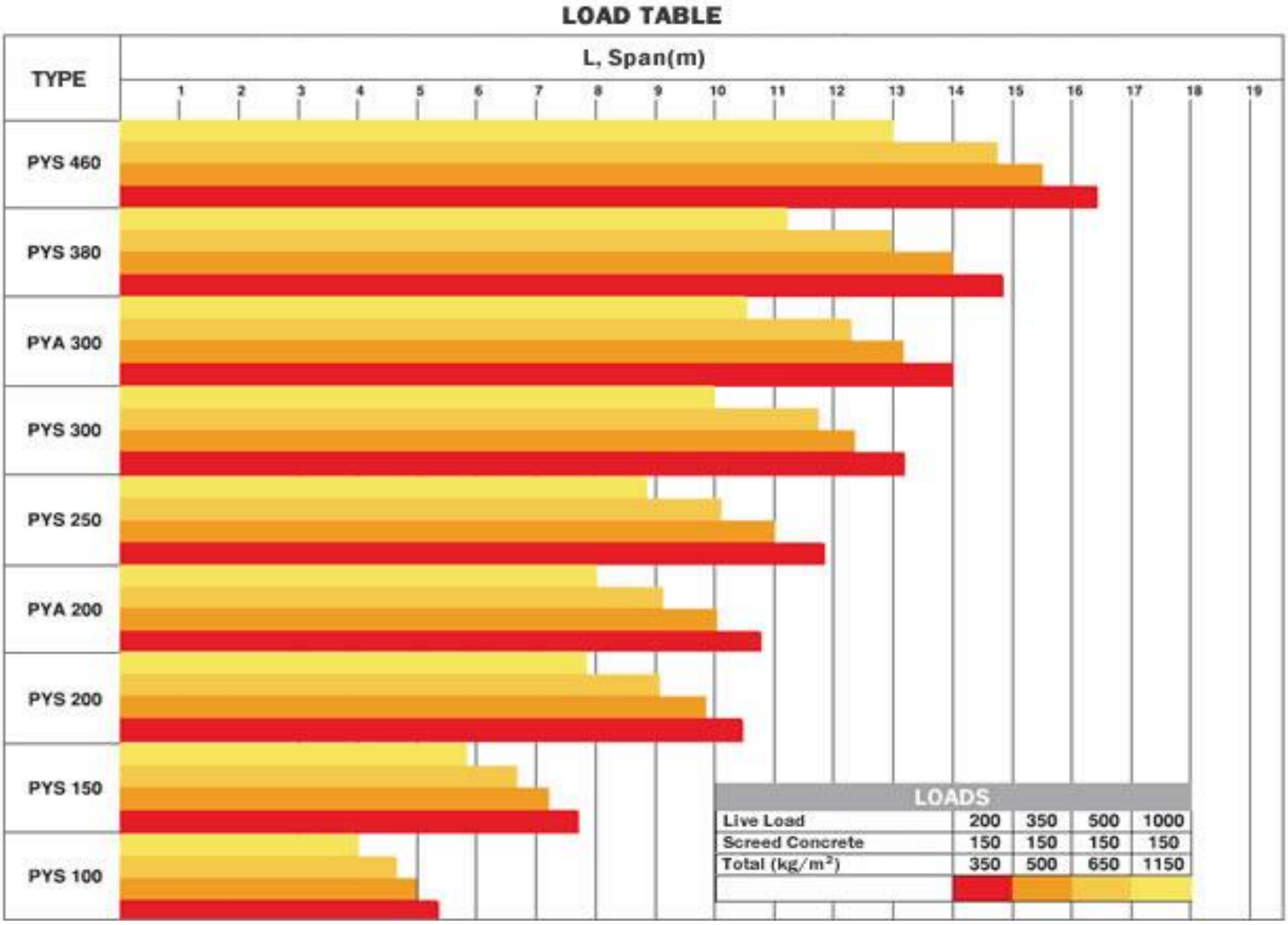


Figure 43: Precast Slabs Load Carrying Capacities

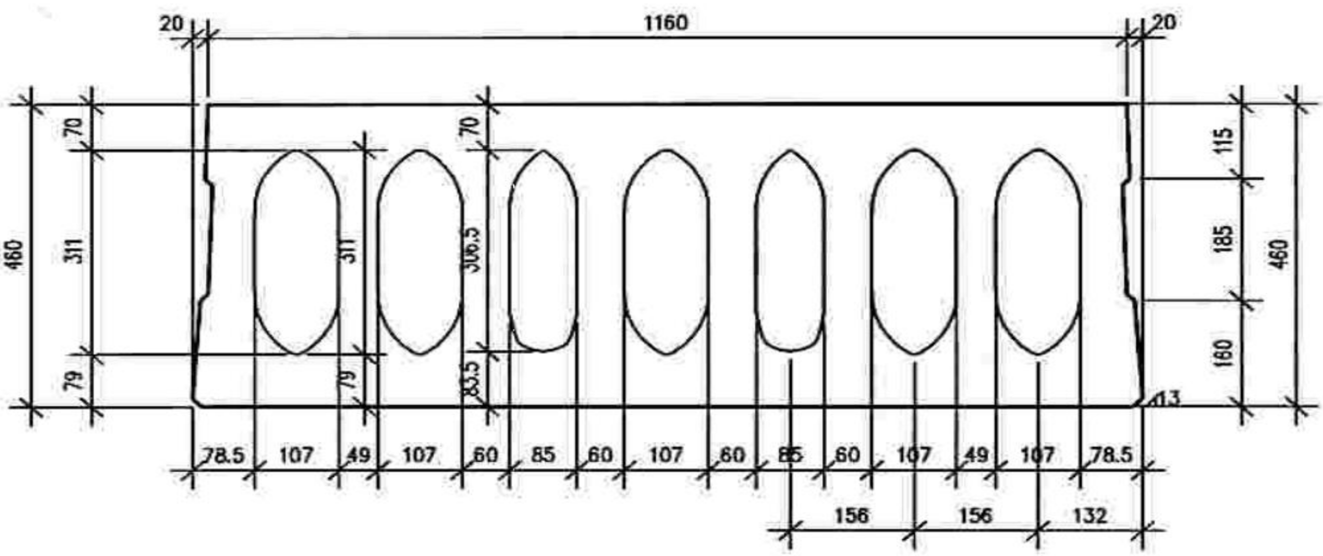


Figure 44: PYS460 Typical Cross Section

REINFORCED CONCRETE SLAB DESIGN

Our concourse floor slab dimensions are 51.60 m length and 65.18 m width. Its area is approximately 3360 m^2. We have 4 different type of slabs. Three of them will be cast in place. And another slab is precast reinforced slab. According to our structural analysis results, the most critical slab is BS113 as can be seen from Figure 41. It has dimensions of 8 m x 4 m.

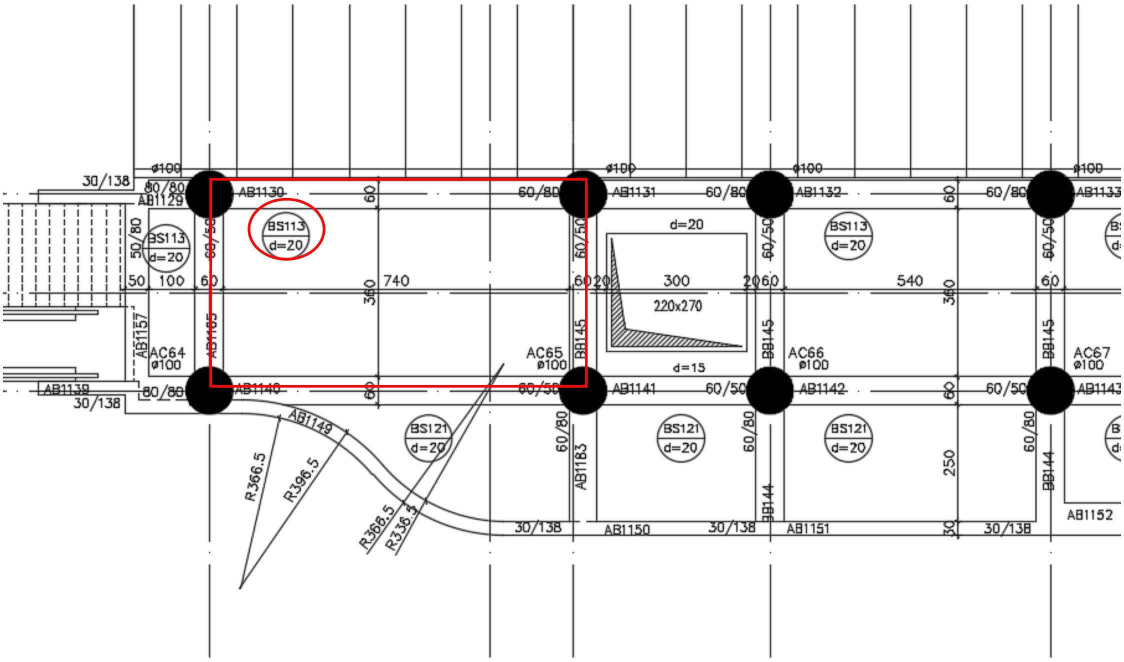


Figure 45: Most Critical Reinforced Concrete Slab

In the Preliminary Design Process, we have made some calculations for different types of slabs.

$l_l = 8$ m, $l_s = 4$ m $\quad$ l_s: Short side of the slab, $\quad$ $m = l_l / l_s = 2$

m is equal to 2.

Hence, Slab-BS113 is two-way slab. The thickness limitation for two-way slab at TS500 is shown below formula.

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

$\alpha_s = 24/24 = 1$

$l_{sn} = 4 - 0{,}6 = 3.4$ m

m: aspect ratio = 2

$$h \geq \frac{3400}{15+\frac{20}{2}} * \left(1-\frac{1}{4}\right) = 102\ mm$$

Minimum thickness for Slab-BS113 is 11 cm.

Since we do not check deflections we had decided slab thickness as 20 cm. After making analysis, we obtain that 20-cm slab thickness is sufficient for Slab-BS113. TS500 design criteria is used for design purpose.

Design Moment Calculation;

$$M_d = \frac{1}{1000} * a * P_d * \ {L_{knet}}^2 \ \ (kNm/m)$$

Slab = 0.2*25 = 5 kN/m^2

Live Load = 5 kN/m^2

P_d =1.4G + 1.6Q

P_d = 18 kN/m^2

$$M_{ym} = 8.2 \ \ (kNm/m)$$

$$M_{ye} = -10.1 \ \ (kNm/m)$$

$$M_{xm} = 12.8 \ (kNm/m)$$

$$M_{xe} = -16.7 \ \ (kNm/m)$$

<u>In x-direction</u>

$$\mathrm{K}_l = \frac{b_w * d^2}{M_d}$$

A_s = 504 mm^2

Φ10/15 cm

<u>In y-direction</u>

A_s = 504 mm^2

Φ10/15 cm

Slab-BS113 reinforcement details can be seen in Figure 43.

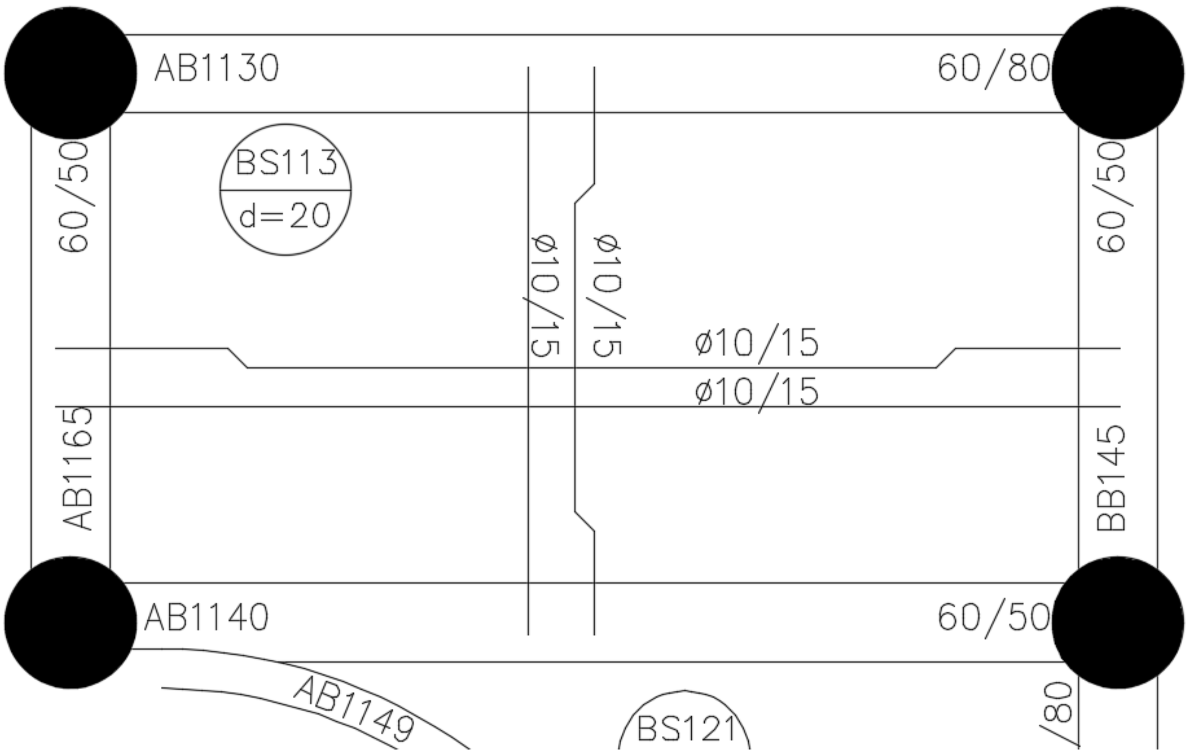


Figure 46: Slab Reinforcement for the Most Critical Slab

BEAM DESIGN

There are 3 type beams which are 50cm/60cm, 60cm/80cm and 30cm/138cm. Beam locations can be seen in Figure 43**.** In beam design stage, longitudinal and shear reinforcements of beams will be calculated.

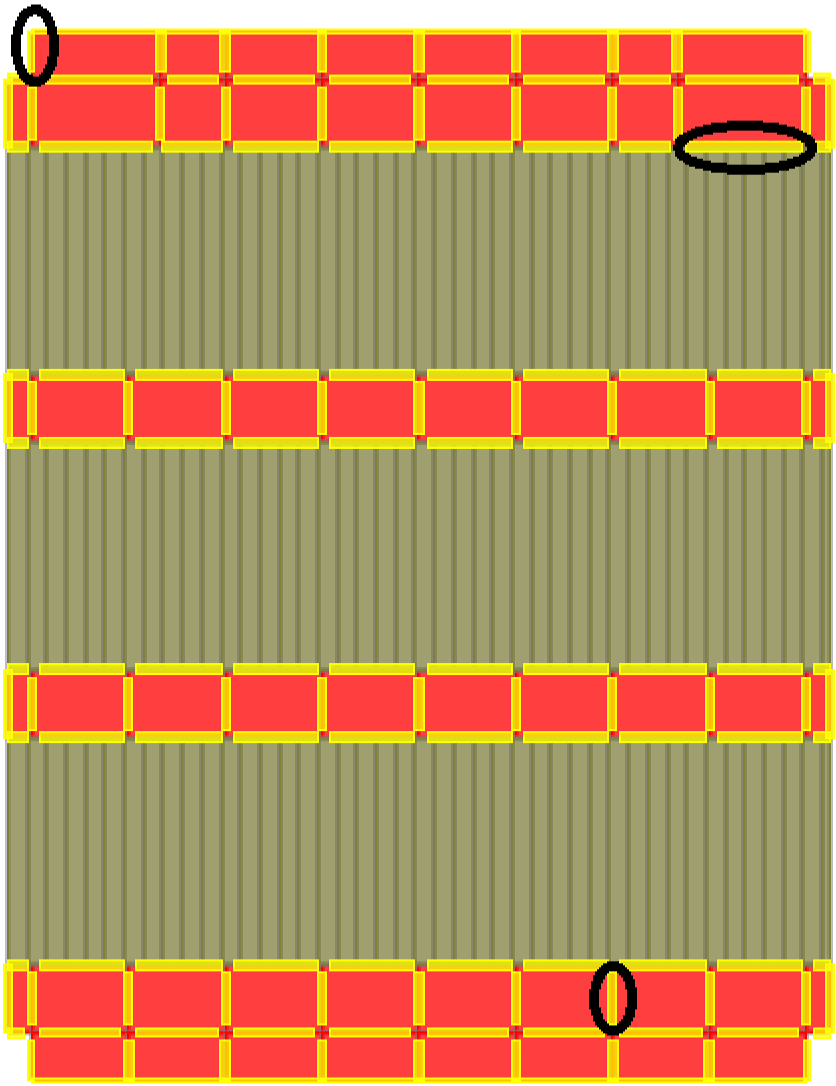


Figure 47: Beam Types and Locations at the Concourse Level

RECTANGULAR BEAM DESIGN

50cm/60cm Beams

All 50cm/60cm Beams are 4,2 meter. According to analysis results, #2040 beam is critical under 1,4G+1,6Q load combination.

Minimum longitudinal reinforcements can be found following formula:

$$(A_s)_{min} = 0{,}8 * \frac{f_{ctd}}{f_{yd}} * b_w * d = 0{,}8 * \frac{1{,}278}{365} * 500 * 560 = 784{,}3 \text{ mm}^2$$

Span Design

Span moment and shear force is **121,94 kNm** and **258,94 kN** according to SAP2000 results for #2040 beam.

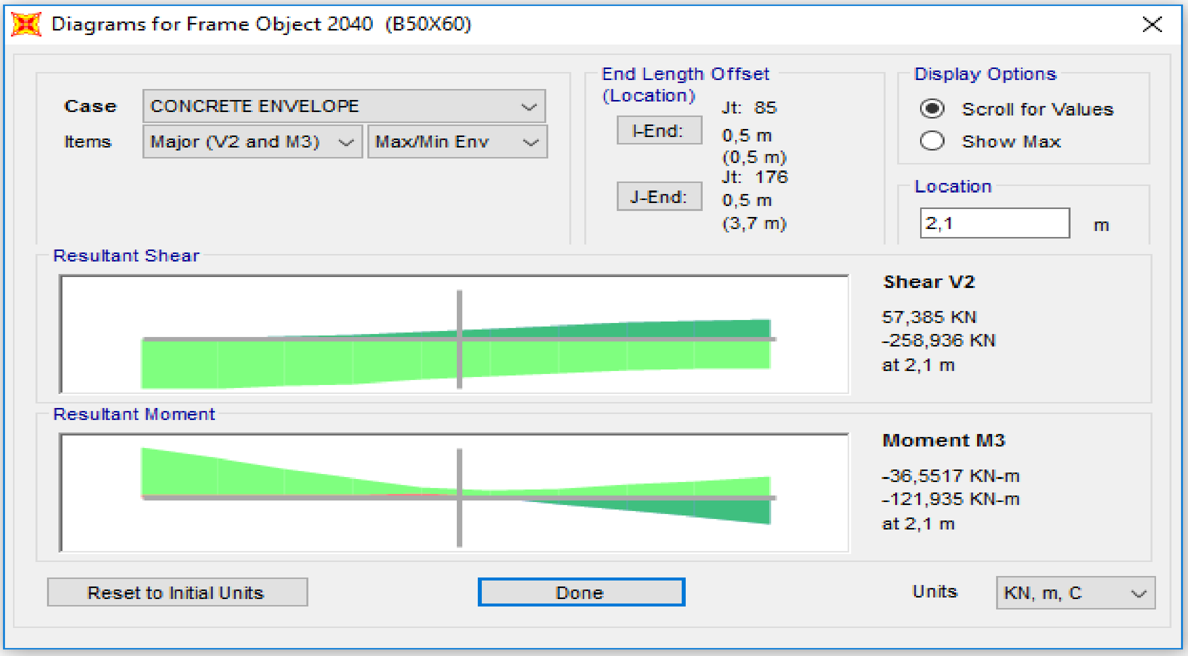


Figure 48: Moment Diagram for Critical Beam taken from SAP2000

$$K = \frac{b_w * d^2}{M_d} = \frac{500 * 560^2}{121{,}94 * 10^3} = 1285{,}8\frac{mm^2}{kN} \geq K_l = 247\frac{mm^2}{kN}$$

There is no need to use compression reinforcements.

$$A_s = \frac{M_d}{f_{yd} * j_l * d} = \frac{121{,}94 * 10^6}{365 * 0{,}86 * 560} = 693{,}7\ mm^2 \leq (A_s)_{min} = 784{,}3mm^2$$

Take minimum reinforcement A_s = 784,3 mm^2

Use **4 φ16** = 804 mm^2 > 784,3 mm^2 for tension zone.

Also, use **4 φ16** for compression zone. (due to shear reinforcement)

According to TS-500, there are necessary longitudinal reinforcements at 'gövde bölgesi' that can be found by using A_{si} = 0,001*b_w*d formula

A_{si} = 0,001*500*560 = 280 mm^2

Use **2φ14** = 308 mm^2 > 280 mm^2

Shear reinforcement

$V_r = V_c + V_w$ where $V_c = 0{,}8*V_{cr}$

$V_{cr} = 0{,}65*f_{ctd}*b_w*d$ = 0,65 * 1,278 * 500 * 560 *10^{-3} = 232,6 kN

V_c = 0,8*232,6 = 186,08 kN and V_r = 258,94 kN

V_w = 72,86 kN

$V_w = \frac{Asw}{s} * fywd * d$

$\frac{Asw}{s} = 0{,}356$

Use **φ14 / s=200 mm**

Support Design

Span moment and shear force of the beam is 726,09 kNm and 335,19 kN. The critical moment and shear for the beam is from 1.0G+1.0Q-1.0E_y+0,3E_x load combination.

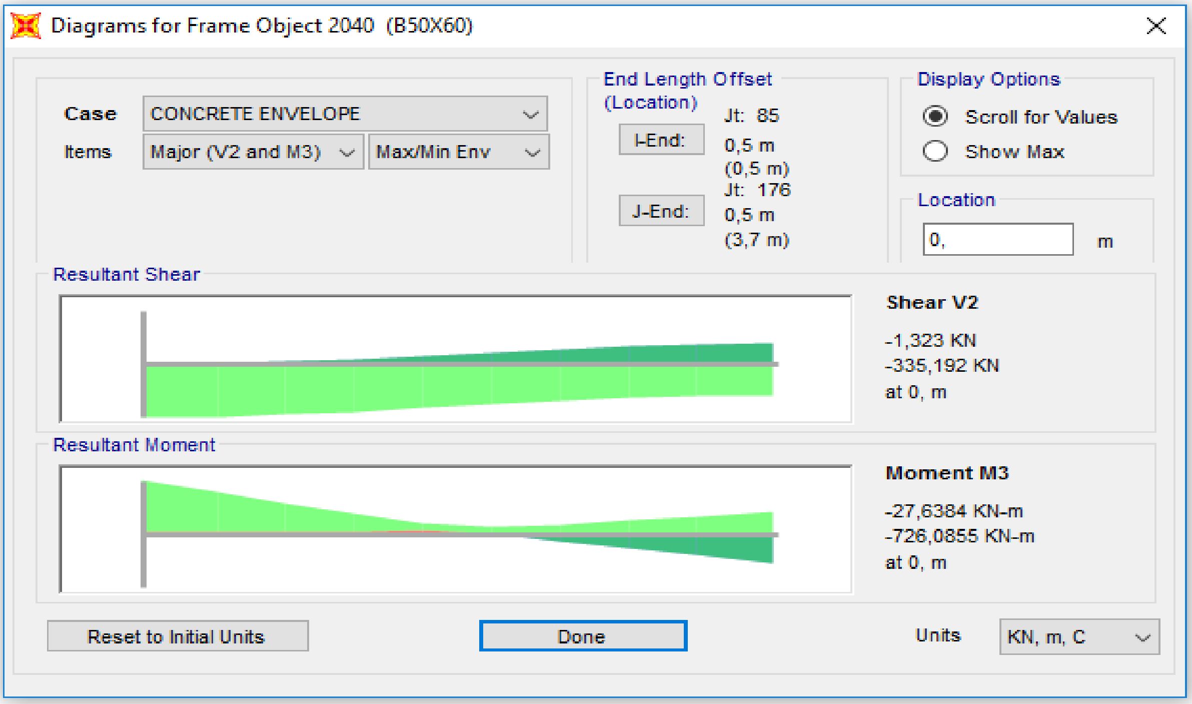


Figure 49: Support Moment Diagram for Critical Beam taken from SAP2000

$$K = \frac{b_w * d^2}{M_d} = \frac{500 * 560^2}{726{,}09 * 10^3} = 216\frac{mm^2}{kN} \leq K_l = 247\frac{mm^2}{kN}$$

There is need to use compression reinforcements.

$$M_1 = \frac{b_w * d^2}{K} = \frac{500 * 560^2}{247000} = 635\ kNm.$$

M_2 = 726,09 - 635 = 91,18 kNm

$$A_{s1} = \frac{M_d}{f_{yd} * j_l * d} = \frac{635*10^6}{365*0,86*560} = \mathbf{3612,4}\ \text{mm}^2 \geq (A_s)_{min} = 784{,}3\text{mm}^2 \quad \text{for tension zone.}$$

$$A_{s2} = \frac{M_d}{f_{yd} * j_l * d} = \frac{91{,}18*10^6}{365*0,86*560} = \mathbf{518,7}\ \text{mm}^2 \text{ for compression zone.}$$

Use **6 φ28** = 3695 > 3612,4 for tension zone

Use **4 φ 16** = 804 > 518,7 for compression

Use **2 φ 14** at middle zone(gövde donatısı)

Shear Reinforcement

$V_r = V_c + V_w$ where $V_c = 0{,}8*V_{cr}$

$V_{cr} = 0{,}65*f_{ctd}*b_w*d = 0{,}65 * 1{,}278 * 500 * 560 *10^{-3} = 232{,}6$ kN

V_c = 0,8*232,6 = 186,08 kN and V_r = 335,19 kN

V_w = 149,11 kN

$V_w = \frac{Asw}{s} * fywd * d$

$\frac{Asw}{s} = 0{,}73$

Use **φ14 / s=420 mm (it is not possible)**

TS-500 criteria's:

$s \leq d/4 = s \leq 560/4 = 140$

$s \leq 8\varphi = s \leq 8*14 = 112$

$s \leq 150$

Use φ14 / s=100 mm

50cm/60cm beams cross section at spans can be seen in Figure 44.

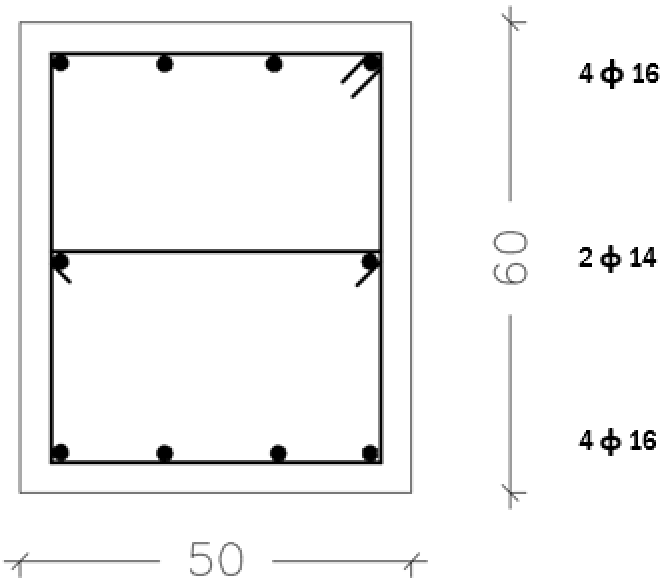


Figure 50: Typical Cross Section of 50x60 cm beams at the span

50cm/60cm beams cross section at supports can be seen in Figure 44.

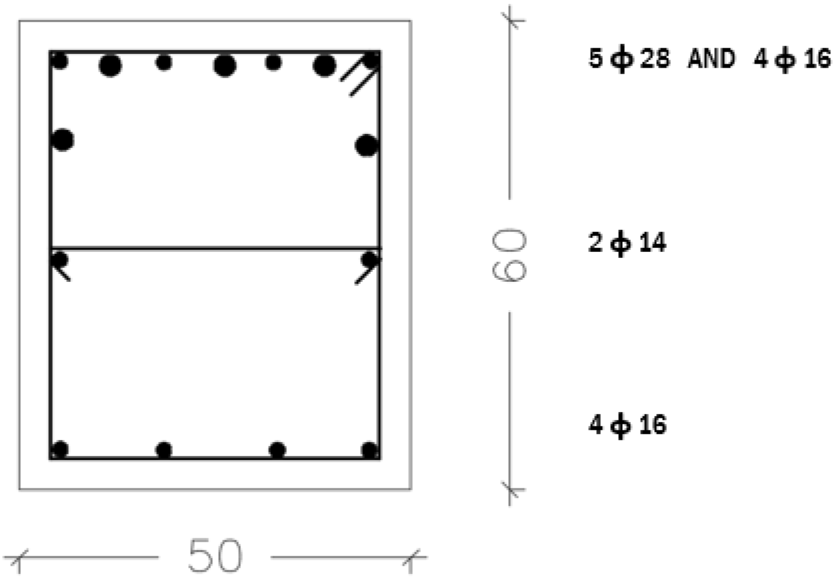


Figure 51: Typical Cross Section of 50x60 cm beams at Supports

30cm/138cm Beams

According to SAP2000 analysis results, #1281 beam is critical under 1,4G+1,6Q load combination.

Minimum longitudinal reinforcements can be found following formula:

$$(A_s)_{min} = 0{,}8 * \frac{f_{ctd}}{f_{yd}} * b_w * d = 0{,}8 * \frac{1{,}278}{365} * 300 * 1340 = 1126\ \text{mm}^2$$

Span Design

Span moment and shear force is **682,56 kNm** and **479,48 kN** according to SAP2000 results for #1281 beam.

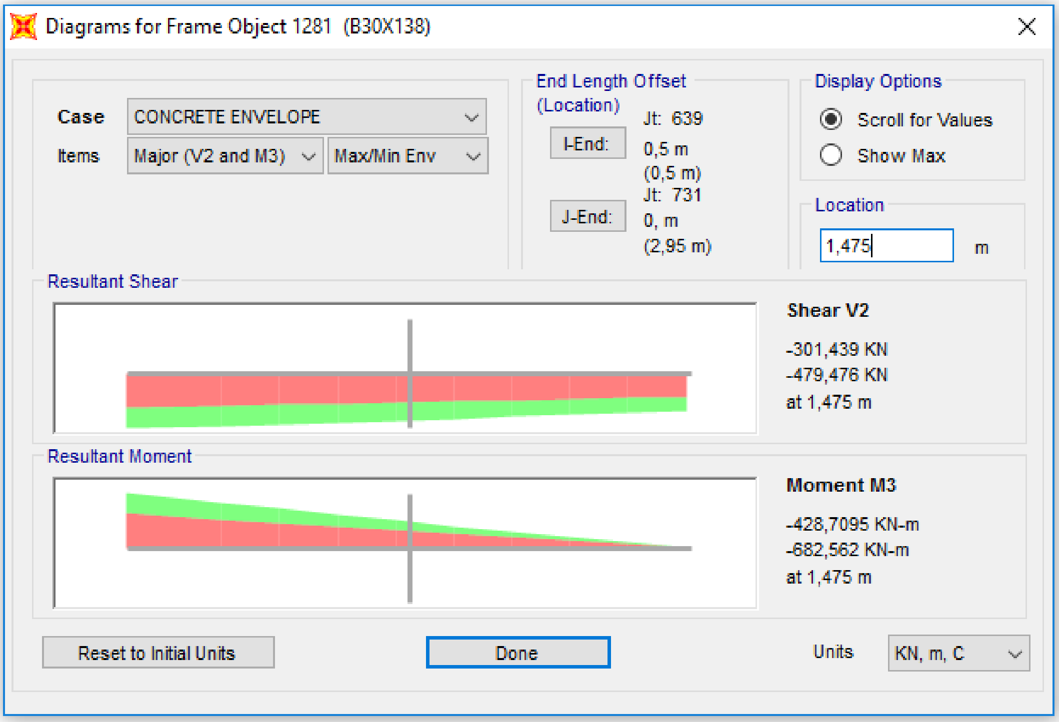


Figure 52: Span Moment and Shear Forces for 1281 Beam

$$K = \frac{b_w * d^2}{M_d} = \frac{300 * 1340^2}{682{,}56 * 10^3} = 789 \frac{mm^2}{kN} \geq K_l = 247 \frac{mm^2}{kN}$$

There is no need to use compression reinforcements.

$$A_s = \frac{M_d}{f_{yd} * j_l * d} = \frac{682{,}56 * 10^6}{365 * 0{,}86 * 1340} = 1622{,}7\ mm^2 \geq (A_s)_{min} = 1126\ mm^2$$

Use **3φ28** = 1847,3 mm^2 > 2459,8 mm^2 for tension zone.

Use **3φ14** for compression zone (due to shear reinforcement)

According to TS-500, there are necessary longitudinal reinforcements at ‘gövde bölgesi’ that can be found by using A_{si} = 0,001*b_w*d formula

A_{si} = 0,001*300*1340 = 402 mm^2

Use **4φ14** = 615 mm^2 > 402 for middle zone

Shear reinforcement

$V_r = V_c + V_w$ where $V_c = 0{,}8*V_{cr}$

$V_{cr} = 0{,}65*f_{ctd}*b_w*d = 0{,}65 * 1{,}278 * 300 * 1340 *10^{-3} = 333{,}9$ kN

$V_c = 0{,}8*333{,}9 = 267{,}15$ kN and $V_r = 479{,}48$ kN

$V_w = 479{,}48-267{,}15 = 212{,}33$ kN

$V_w = \frac{Asw}{s} * fywd * d$

$\frac{Asw}{s} = 0{,}434$

Use **φ14 / s=200 mm**

Support Design

Span moment and shear force of the beam is 1453,37 kNm and 566,25 kN (see Figure .

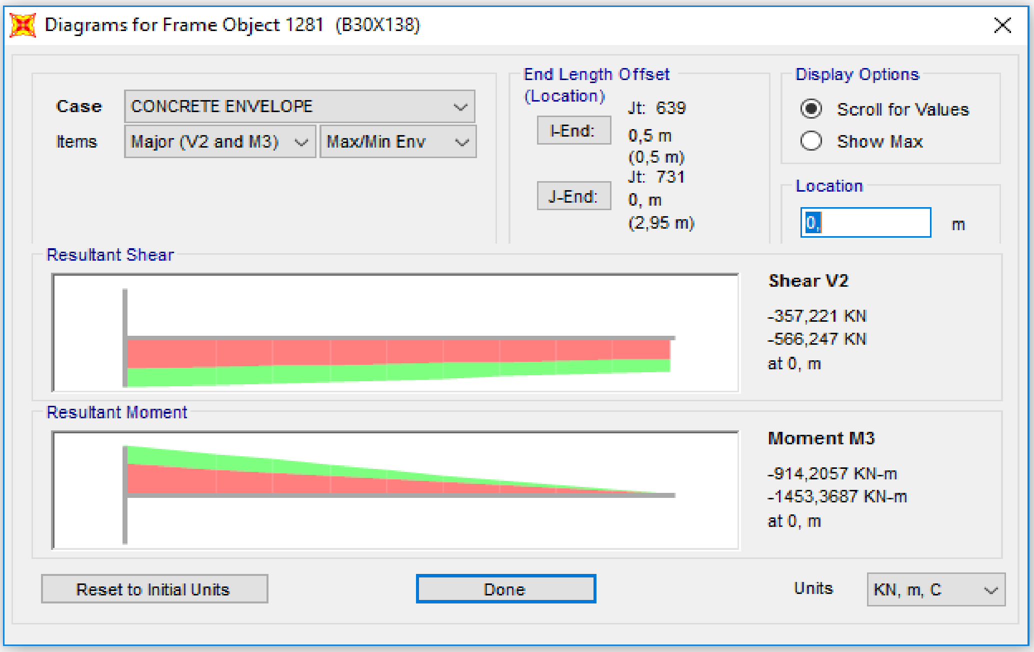


Figure 53: Support Moment and Shear Diagrams for 1281 Beam

$$K = \frac{b_w * d^2}{M_d} = \frac{300 * 1340^2}{1453{,}37 * 10^3} = 370\frac{mm^2}{kN} \geq K_l = 247\frac{mm^2}{kN}$$

There is no need to use compression reinforcements.

$$A_s = \frac{M_d}{f_{yd} * j_l * d} = \frac{1453,37 * 10^6}{365 * 0,86 * 1340} = 3455,3\ mm^2 \geq (A_s)_{min} = 1126\ mm^2$$

Use **5φ28 +3φ14** =3540,6 mm^2 > 3455,3 mm^2 for tension zone.

Use **3φ28** for compression zone (due to shear reinforcement)

According to TS-500, there are necessary longitudinal reinforcements at 'gövde bölgesi' that can be found by using $A_{si} = 0,001*b_w*d$ formula

$A_{si} = 0,001*300*1340 = 402\ mm^2$

Use **4φ14** = 615 mm^2 > 402 for middle zone

Shear reinforcement

$V_r = V_c + V_w$ where $V_c = 0,8*V_{cr}$

$V_{cr} = 0,65*f_{ctd}*b_w*d = 0,65 * 1,278 * 300 * 1340 *10^{-3} = 333,9$ kN

$V_c = 0,8*333,9 = 267,15$ kN and $V_r = 566,25$ kN

$V_w = 566,25-267,15 = 299,1$ kN

$V_w = \frac{Asw}{s} * fywd * d$

$\frac{Asw}{s} = 0,612$

Use **φ14 / s=150mm**

Cross section of 30cm/138cm Beam at span zone can be seen in Figure:

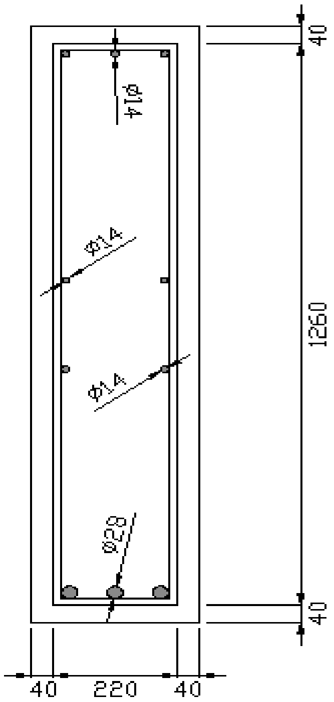


Figure 54: Typical Cross Section of 30/138 cm Beams at Span

Cross section of 30cm/138cm Beam at support zone can be seen in Figure:

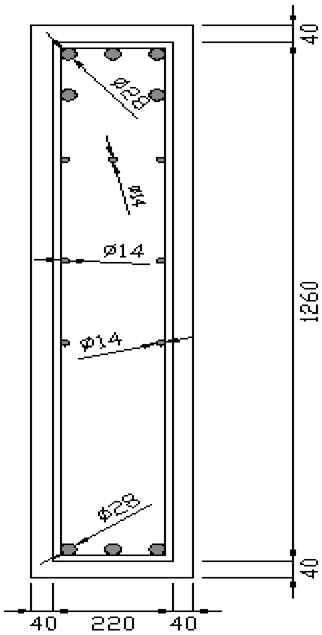


Figure 55: Typical Cross Section of 30/138 Beams at Supports

L-SHAPED BEAM DESIGN

60cm/80cm Beams

According to analysis results, #2013 beam which is 8 meter is critical under 1,4G+1,6Q load combination. Minimum longitudinal reinforcements can be found following formula:

$$(A_s)_{min} = 0{,}8 * \frac{f_{ctd}}{f_{yd}} * b_w * d = 0{,}8 * \frac{1{,}278}{365} * 600 * 760 = 1277{,}3 \text{ mm}^2$$

Span Design

Span moment and shear force is **586,82 kNm** and **80,09 kN** according to SAP2000 results for #2013 beam.

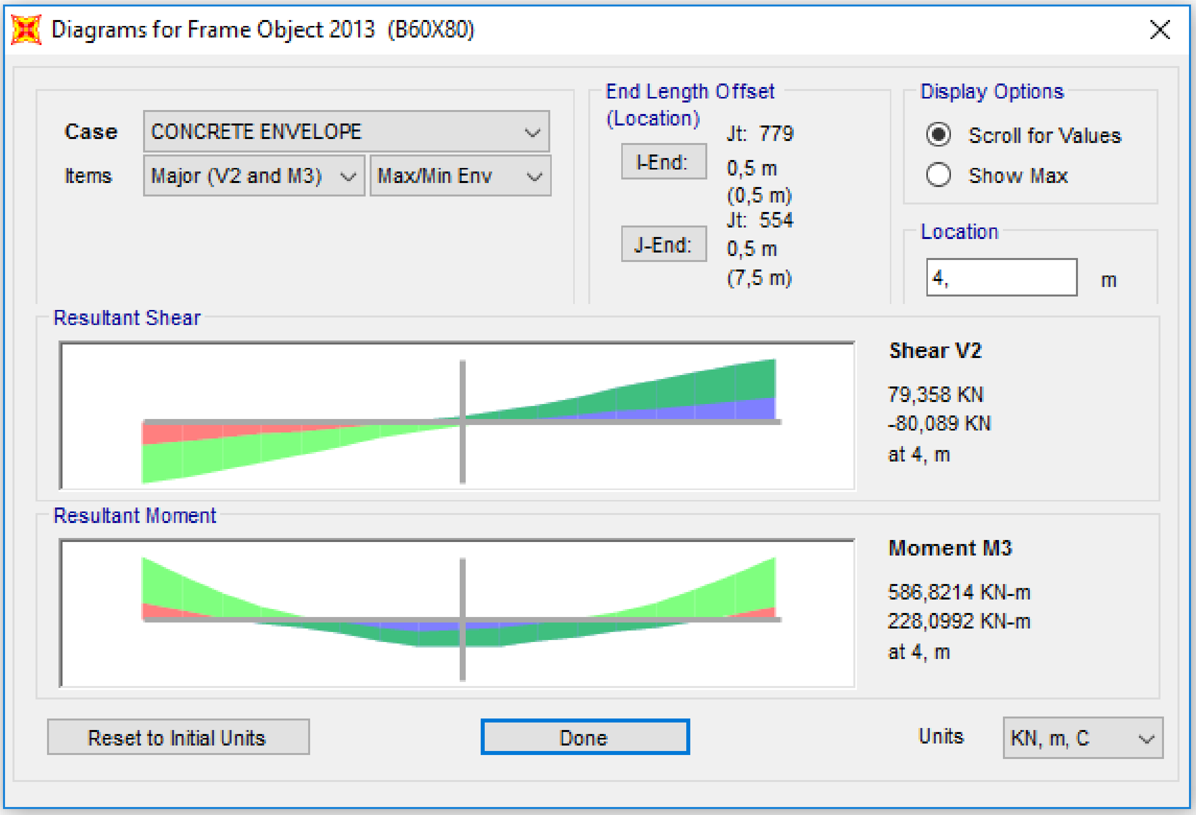


Figure 56: Critical Moment Diagram for L-Shaped Beam at the Span

$$K = \frac{b_w * d^2}{M_d} = \frac{600 * 760^2}{586{,}82 * 10^3} = 591\frac{mm^2}{kN} \geq K_l = 247\frac{mm^2}{kN}$$

There is no need to use compression reinforcements.

$$A_s = \frac{M_d}{f_{yd} * j_l * d} = \frac{586{,}82 * 10^6}{365 * 0{,}86 * 760} = 2459{,}8\ mm^2 \geq (A_s)_{min} = 1277{,}3mm^2$$

Use **4φ28** = 2463 mm^2 > 2459,8 mm^2 for tension zone.

Use **3φ14** for compression zone (due to shear reinforcement)

According to TS-500, there are necessary longitudinal reinforcements at 'gövde bölgesi' that can be found by using A_{si} = 0,001*b_w*d formula

A_{si} = 0,001*600*760 = 456 mm^2

Use **4φ14** = 615 mm^2 > 456 for middle zone

60cm/80cm beams have extension 25 cm/30cm. According to TS-500, these beams can be considered as short console. There is additional reinforcement from this situation.

$A_s = \frac{Vd*av}{0,8fyd*d} =$

$A_s = \frac{80,09*1000*250}{0,8*365*760}$ =90,2 mm^2

Use **1φ14** = 154 mm^2 > 90,2 mm^2

Shear reinforcement

$V_r = V_c + V_w$ where $V_c = 0,8*V_{cr}$

$V_{cr} = 0,65*f_{ctd}*b_w*d$ = 0,65 * 1,278 * 600 * 760 *10^{-3} = 378,8 kN

V_c = 0,8*378,8 = 303,04 kN and V_r = 80,09 kN

It is possible to say that minimum shear reinforcement ratio can be taken due to $V_c > V_r$ or concrete capacity can be neglected.

$V_w = \frac{Asw}{s} * fywd * d$

$\frac{Asw}{s}$ = 0,288

Use **φ14/s=200 mm**

Support Design

Span moment and shear force of the beam is 1298,9 kNm and 907,7 kN

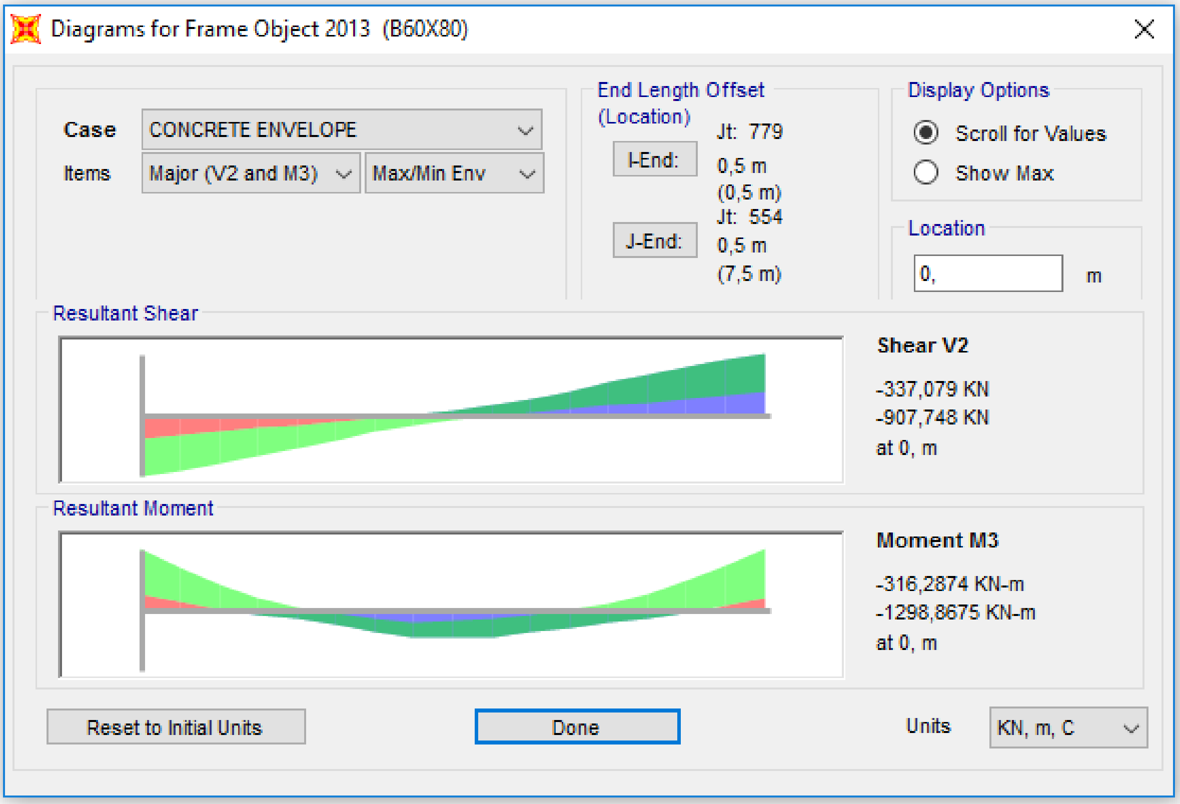


Figure 57: Critical Moment Diagram for L-Shaped Beam at Supports

$$K = \frac{b_w * d^2}{M_d} = \frac{600 * 760^2}{1298{,}9 * 10^3} = 266{,}8 \frac{mm^2}{kN} \geq K_l = 247 \frac{mm^2}{kN}$$

There is no need to use compression reinforcements.

$$A_s = \frac{M_d}{f_{yd} * j_l * d} = \frac{1298{,}9 * 10^6}{365 * 0{,}86 * 760} = 5443{,}4\, mm^2 \geq (A_s)_{min} = 1277{,}3 mm^2$$

Use **8φ28** = 4926 mm^2 > 1277,3 mm^2 for tension zone.

Use **4φ28** = 2463 mm^2 > 2459,8 mm^2 for compression zone. (due to shear reinforcement)

According to TS-500, there are necessary longitudinal reinforcements at ‘gövde bölgesi’ that can be found by using A_{si} = 0,001*b_w*d formula

A_{si} = 0,001*600*760 = 456 mm^2

Use **4φ14** = 615 mm^2 > 456 for middle zone

60cm/80cm beams have extension 25 cm/30cm. According to TS-500, these beams can be considered as short console. There is additional reinforcement from this situation.

$A_s = \frac{Vd*av}{0{,}8fyd*d} =$

$A_s = \frac{907{,}75*1000*250}{0{,}8*365*760}$ =1022,6 mm^2

Use **2φ28** = 1231 mm^2 > 1022,6 mm^2

Shear reinforcement

$V_r = V_c + V_w$ where $V_c = 0{,}8*V_{cr}$

$V_{cr} = 0{,}65*f_{ctd}*b_w*d$ = 0,65 * 1,278 * 600 * 760 *10^{-3} = 378,8 kN

V_c = 0,8*378,8 = 303,04 kN and V_r = 907,75 kN

V_w = 907,75-303,04 = 604,71 kN

$V_w = \frac{Asw}{s} * fywd * d$

$\frac{Asw}{s}$ = 2,18

Use **φ14/s=140 m**

Cross section of 60cm/80cm beam at span zones can be seen in Figure.

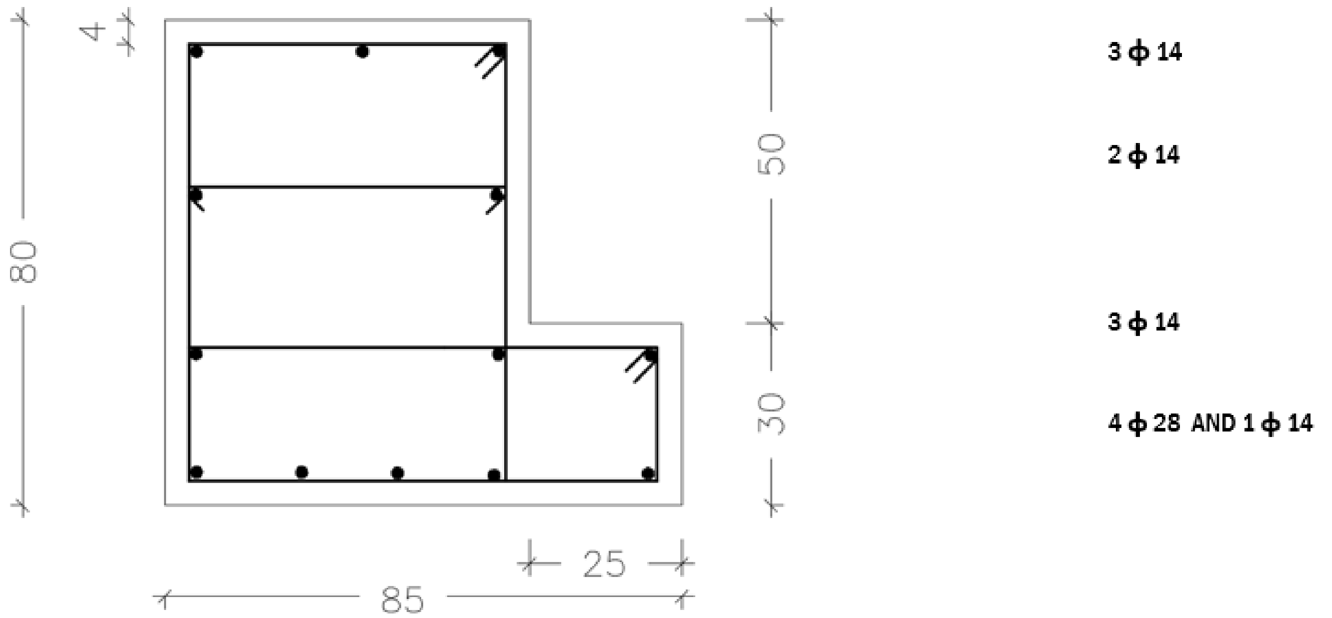


Figure 58: Typical Cross Section for L-Shaped Beam at Spans

Cross section of 60cm/80cm beam at support zones can be seen in Figure.

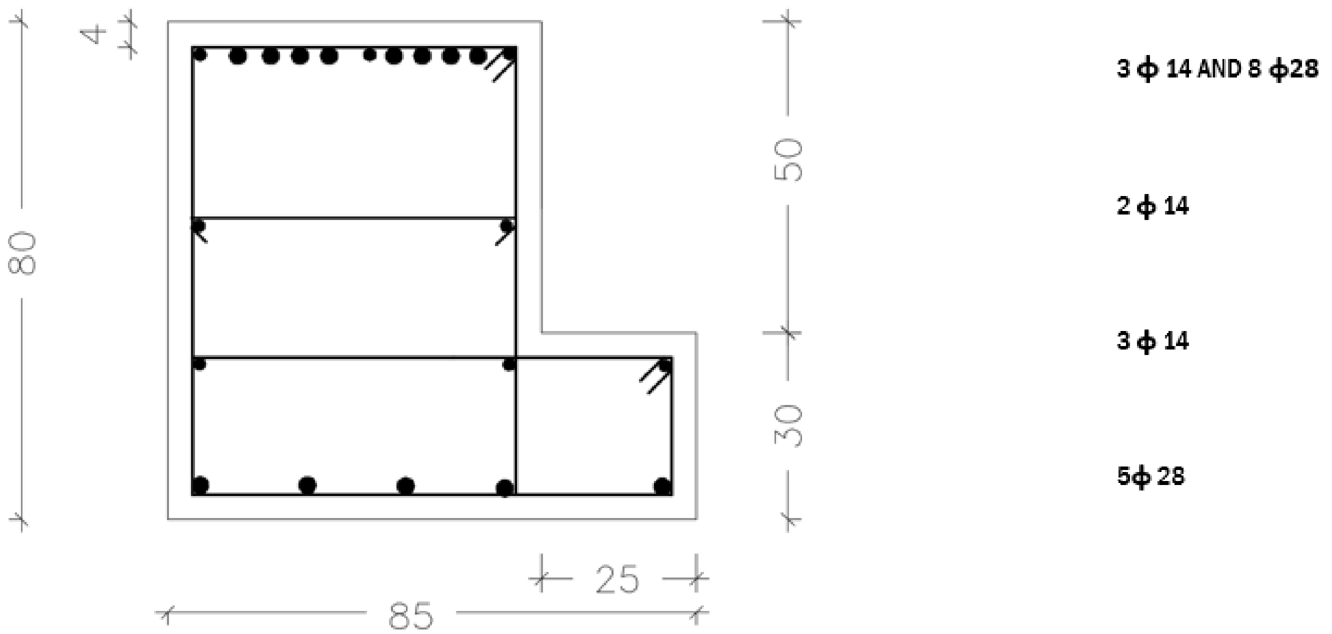


Figure 59: Typical Cross Section of L-Shaped Beams at Supports

COLUMN DESIGN

At preliminary design stage, all ground floor columns are chosen as 1-meter diameter circular reinforced concrete and first floor columns may be chosen as 0,60-meter diameter circular reinforced concrete or steel members. At advance design stage, ground floor columns are 1-meter diameter circular reinforced concrete and after some thinking, all first floor columns are chosen as 0,8-meter circular reinforced concrete. According to these cross section properties and acting on the forces, columns' longitudinal and shear reinforcements are calculated

LONGITUDINAL REINFORCEMENTS

Material properties are C30 and S420 so f_{ck} =30 MPa f_{cd}=30/1.5 =20 MPa f_{yk}=420 MPa f_{yd}=420/1,15 = 365 MPa.

Ground Floor Columns

According to Sap2000 results, columns shown in Figure 57 below is critical due to 1.0G+1.0Q+1.0EY-0.3EX combination and concrete envelope so forces acting on the column are following.

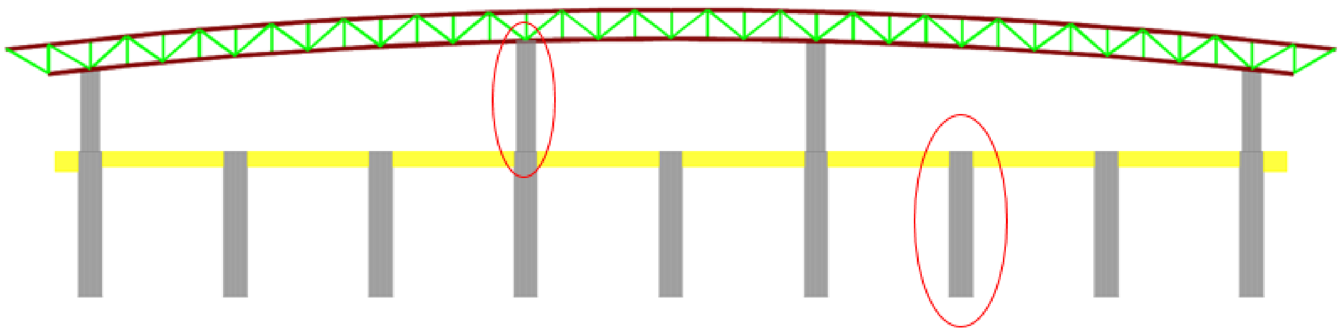


Figure 60: Critical Columns at y=0 Axis taken from SAP2000

P=1788 kN

M_2 = 1409,74 kNm

M_3 = 317,79 kNm

According to Uğur Ersoy 'Taşıma Gücü El Kitabı', circular columns which are under two axis moment can be calculated as one axis moment by considering their equivalent moment. Therefore, M_d = $\sqrt{1409.74^2 + 317.79^2}$ =1445.15 kNm.

$A_c = \Pi\ 500^2 = 785398\ mm^2$

$N_d \leq 0{,}9 f_{cd} A_c$ ……… (TS-500 - 7.7)

$1788 \text{ kN} \leq 0{,}9 \text{ x } 20 \text{ x } 785398 \text{ x } 10^{-3} = 14137 \text{ kN}$ …ok

$N_d \leq 0{,}5 f_{ck} A_c$ ……… (Turkish Seismic Code - 3.3.1.2)

$1788 \text{ kN} \leq 0{,}5 \text{ x } 30 \text{ x } 785398 \text{ x } 10^{-3} = 11781 \text{ kN}$ …ok

$\frac{d''}{h} = \frac{920}{1000} = 0{,}92$ and $m = \frac{fyd}{fcd} = \frac{365}{20} = 18{,}25$

$\frac{N}{Ac\ fcd} = \frac{1788\ 000}{785398x20} = \mathbf{0{,}114}$

$\frac{M}{Ac\ h\ fcd} = \frac{1788x10\text{^}6}{785398x1000x20} = \mathbf{0{,}092}$

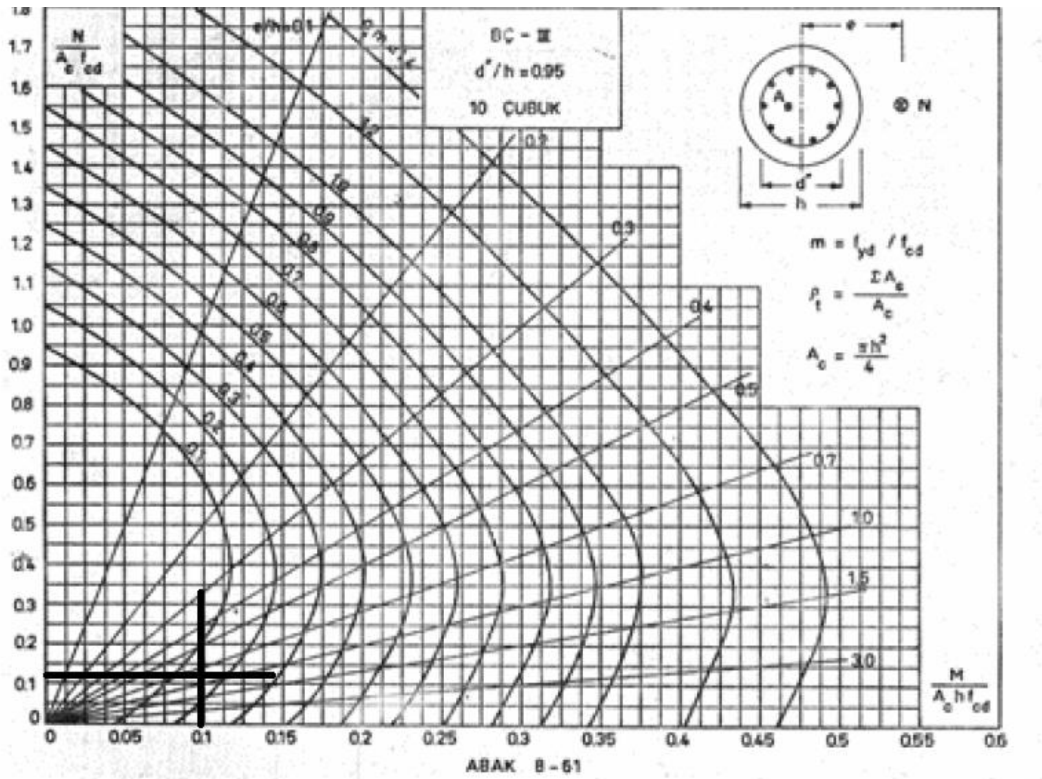


Figure 61: Column Reinforcement Ratios

$\varrho_t m = 0{,}15$ as it can be seen in ABAK B-61.

$\varrho_t = 0{,}15/18{,}25 = 0{,}0082$ … not ok.

According to TS500, minimum longitudinal reinforcement ratio is 0,01 and maximum longitudinal reinforcement ratio is 0,04. Therefore, ground floor columns longitudinal reinforcement ratio is 0,01.

$A_{st} = 0{,}01 \text{ x } 785398 = 7854 \text{ mm}^2$

Use 13Φ28 = 8004 > 7854 ok.

First Floor Columns

According to Sap2000 results, columns shown in Figure 59 below is critical and forces acting on the column are as follows:

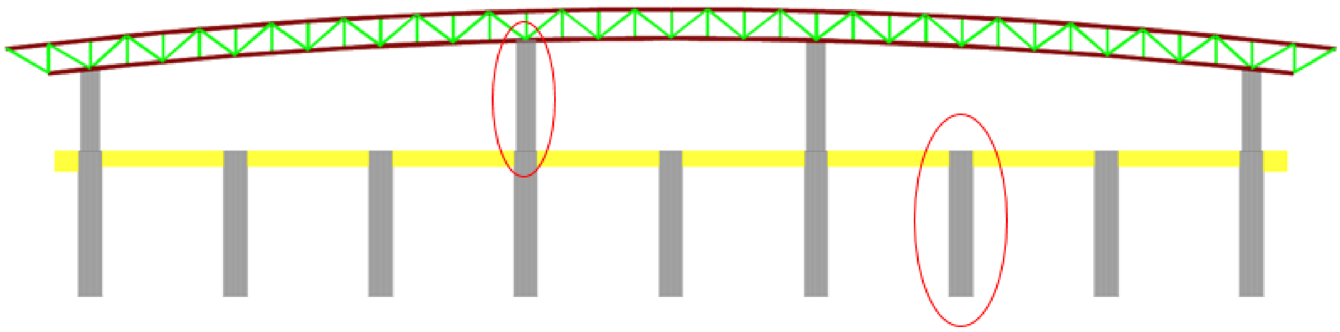


Figure 62: Critical Column at y=0 Axis taken from SAP2000

P_1 = 387,85 kN (compression)

P_2 = -93,73 kN (tension)

M_2 = 473,66 kNm

M_3 = 88,25 kNm

$M_d = \sqrt{473{,}66^2 + 88{,}25^2}$ = 481,81 kNm

A_c = Π 400^2 = 502655 mm^2

$N_d \leq 0{,}9 f_{cd}\, A_c$ ……… (TS-500 - 7.7)

387,85 kN ≤ 0,9 x 20 x 502655 x 10^{-3} = 9048,8 kN …ok

$N_d \leq 0{,}5 f_{ck}\, A_c$ ……… (Turkish Seismic Code - 3.3.1.2)

1788 kN ≤ 0,5 x 30 x 502655 x 10^{-3} = 7539,8 kN …ok

$\frac{d''}{h} = \frac{720}{800} = 0{,}9$ and $m = \frac{fyd}{fcd} = \frac{365}{20} = 18{,}25$

$\frac{N}{Ac\ fcd} = \frac{387850}{502655x20} = \mathbf{0{,}038}$

$\frac{M}{Ac\ h\ fcd} = \frac{481{,}81x10^{\wedge}6}{502655x800x20} = \mathbf{0{,}06}$

ϱ_t m = 0,1 as it can be seen in ABAK B-61.

ϱ_t = 0,1/18,25 = 0,0054 < 0,01… not ok.

Longitudinal reinforcement ratio of first floor columns is 0,01.

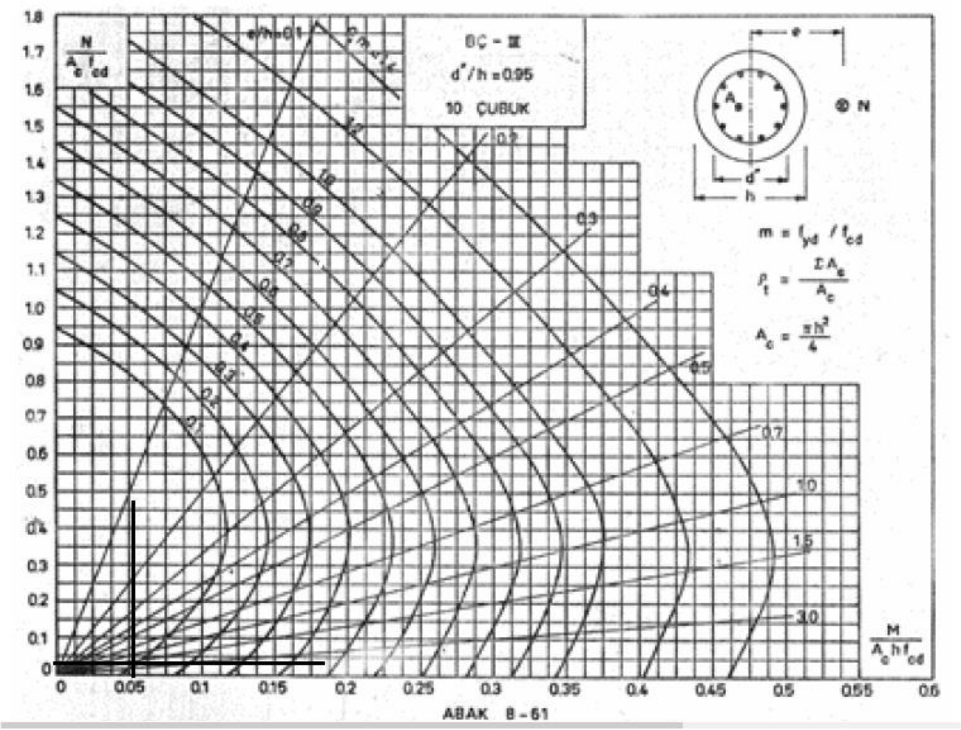


Figure 63: Column Reinforcement Ratios

A_{st} = 0,01 x 502655 = 5027 mm^2

Use 9Φ28 = 5542 > 5027… ok.

Tension:

N = A_c x δ_{ct} + A_{st} x δ_s

93,73 kN < (502655 x 1,92 + 9 x Π x 14^2 x 420) x10^{-3} = 3293 kN …ok. There is no need additional reinforcement for tension.

SHEAR REINFORCEMENTS

Confinement of the columns is supplied by helping of spiral shear reinforcements.

Ground Floor Columns

Before starting to calculate necessary shear reinforcement for columns, it has to be proved that columns are stronger than beams at the beam-column joints. If the beams are stronger than columns, plastic hinge will occur at the column ends. This causes columns reach their moment capacity before beams crack and structure can collapse. Therefore, there is a relation in point of moment capacity of columns and beams between them. Columns end zone moment capacity should be 20% higher than beams end zone moment capacity.

$(M_{ra} + M_{rü}) \geq 1.2(M_{ri} + M_{rj})$ … (Turkish Seismic Code 3.3.5)

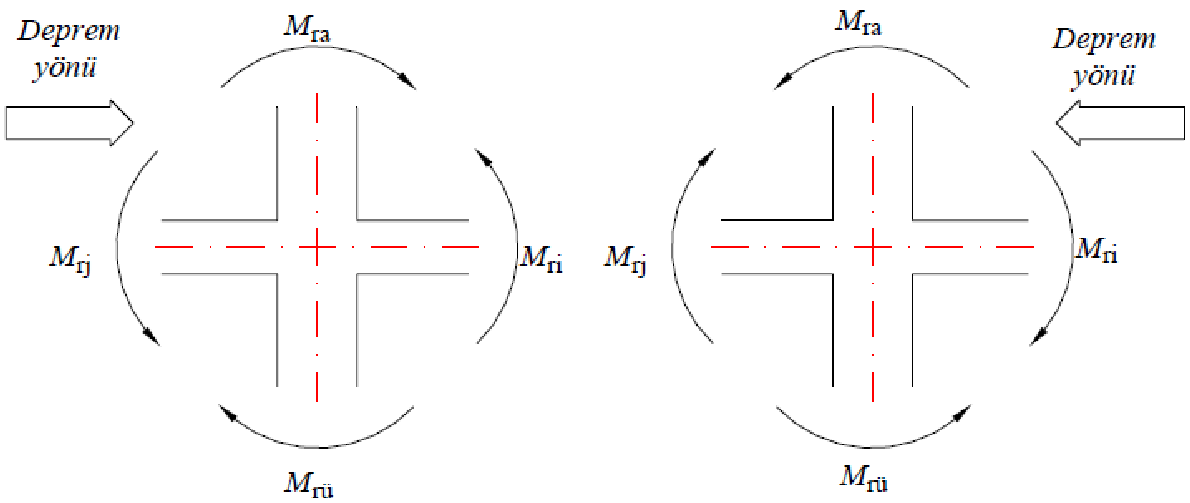


Figure 64: Strong Column- Weak Beam Condition

For number 108 column and number 115 and 116 beams joint, it will be checked.

Mrj = 359,7 kNm for #115 beam

Mri = 348 kNm for #116 beam

Mrü = 0 (there is no column)

Mra = 1445 kNm

Therefore, 1445 + 0 ≥ 1,2(359,7+348) = 849,24 kNm … ok.

Minimum shear reinforcements can be found by using following formulas:

$\varrho_s = 0{,}45 x \frac{fc}{fyw} x (\frac{Ac}{Ack} - 1)$

$\varrho_s = 0{,}12 x \frac{fc}{fyw}$ Turkish Seismic Code (3.3.4.1-c).

According to these formulas, minimum shear reinforcement ratio is 0,0236.

In columns, the shear reinforcement will be calculated using following formula.

$\frac{Asw}{s} = \frac{Ve - Vc}{fywd \times d}$ where Ve = $\frac{(Ma + Mü)}{ln}$ and Vc = 0,52 fctd x b x d

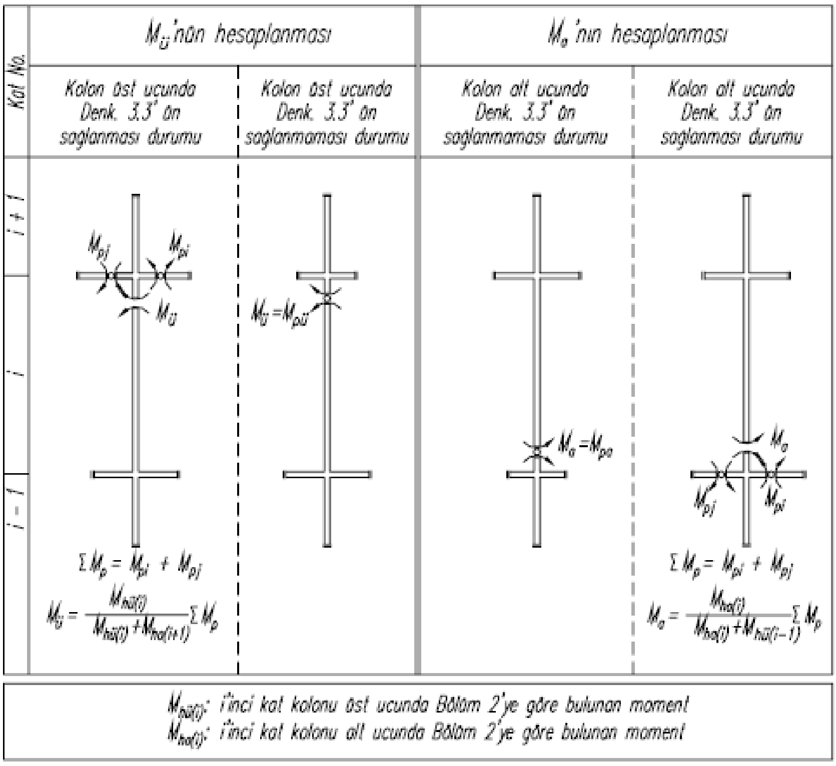


Figure 65: Moment Calculation at Column Ends

$M_{hü}$ = 1610 kNm

$M_{ha(i+1)}$ = 0 (no column)

$\sum M_p = M_{pi} + M_{pj} = 1{,}4 x M_{ri} + 1{,}4 x M_{rj}$ = 1,4 x 359,7 + 1,4 x 348 = 990,78 kNm

Mü = $\frac{1610}{1610}$ x 990,78 = 990,78 kNm

Ma = 0

Ve = 990,78 / 5,35 = 198,2 kN

Turkish Seismic Code says that Ve should be higher than Vd. On the other hand, number 108 columns Vd = 441,7 kN from SAP2000 results which is considered under gravity loads and earthquake loads. Therefore, Ve can be taken as 441,7 kN. Also, if Ve is smaller than Vc, it is possible to say that minimum shear reinforced ratio can be used.

ϱ_s =0,0236

$\frac{Ao}{Ds} = \frac{\varrho s}{4}$ then $\frac{Ao}{s}$ = 2,714

Use Φ14/70 at end zones.

First Floor Columns

In first floor, there is a joint which consists of columns 2 and 160, beams 112 and 113. Strong column-weak beam principle will be checked.

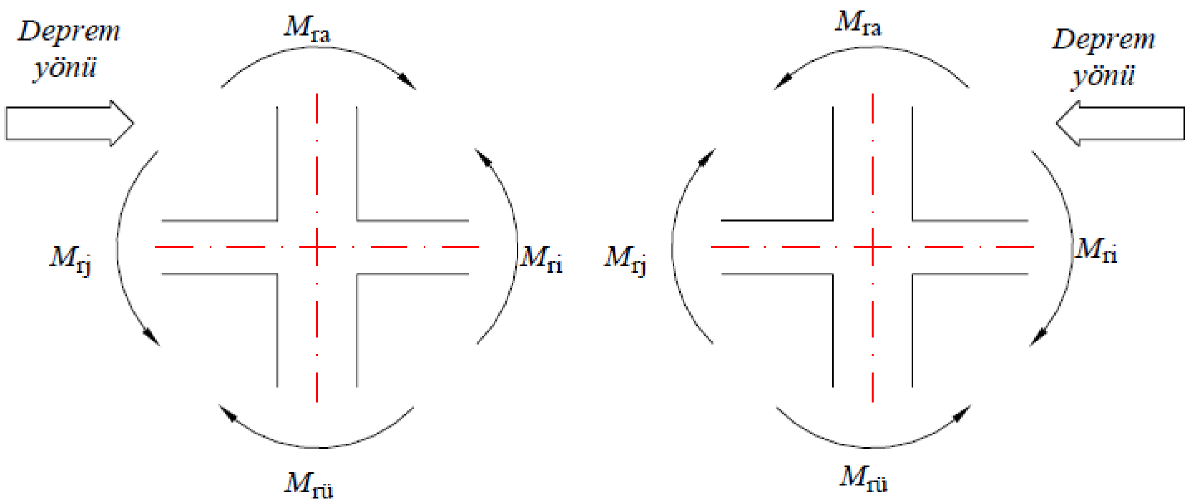


Figure 66: Strong Column-Weak Beam Condition

Mrj = 355,6 kNm for #112 beam

Mri = 359,6kNm for #113 beam

Mrü = 1445 kNm for #2 column

Mra = 481,81 kNm for #160 column

$(M_{ra} + M_{rü}) \geq 1.2(M_{ri} + M_{rj})$ … (Turkish Seismic Code 3.3.5)

Therefore, 1445 + 481,81=**2408,62** ≥ 1,2 (355,6 + 359,6) = **858,24** kNm … ok.

$\varrho_s = 0{,}45 x \frac{fc}{fyw} x (\frac{Ac}{Ack} - 1)$

$\varrho_s = 0{,}12 x \frac{fc}{fyw}$ Turkish Seismic Code (3.3.4.1-c).

According to these formulas, minimum shear reinforcement ratio is 0,0075.

$\frac{Asw}{s} = \frac{Ve - Vc}{fywd\ x\ d}$ where Ve = $\frac{(Ma + Mü)}{ln}$ and Vc = 0,52 fctd x b x d

$M_{hü}$ = 1283 kNm

$M_{ha(i+1)}$ = 528,4 kNm

$\sum M_p = M_{pi} + M_{pj} = 1{,}4xM_{ri} + 1{,}4xM_{rj}$ = 1,4 x 359,3+ 1,4 x 359,6= 1006,5 kNm

Mü = $\frac{1283}{761283+528{,}4}$ x 1006,5 = 712,9 kNm

Ma = 0

Ve = 990,78 / 5,35 = 133,25 kN

Vd = 116,7 kN (Sap2000). Ve>Vd …ok.

Vc = 0,52 x 1,278 x 800 x 720 x 10^{-3} = 382,8 kN.

Similarly, Vc > Ve . Minimum shear reinforcement ratio can be used.

ϱ_s =0,0075

$\frac{Ao}{Ds} = \frac{\varrho s}{4}$ then $\frac{Ao}{s}$ = 1,35

Use Φ14/100 at end zones.

Also, Use Φ14/200 at middle zone.

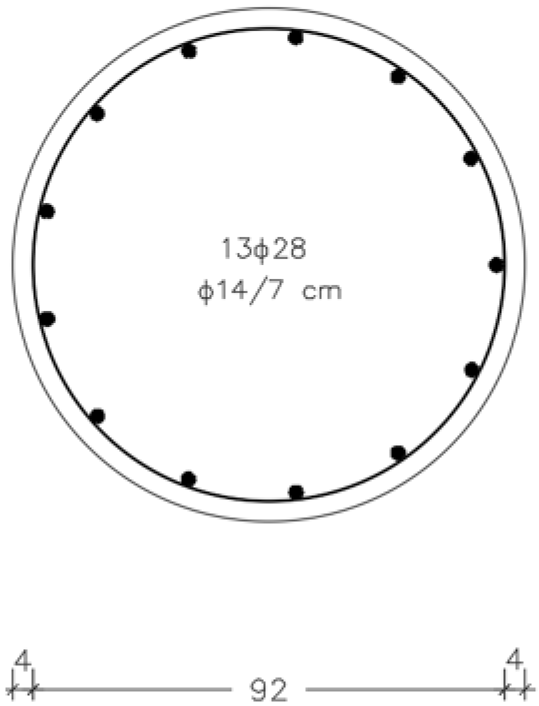


Figure 67: Typical Column Cross Section at Ground Level

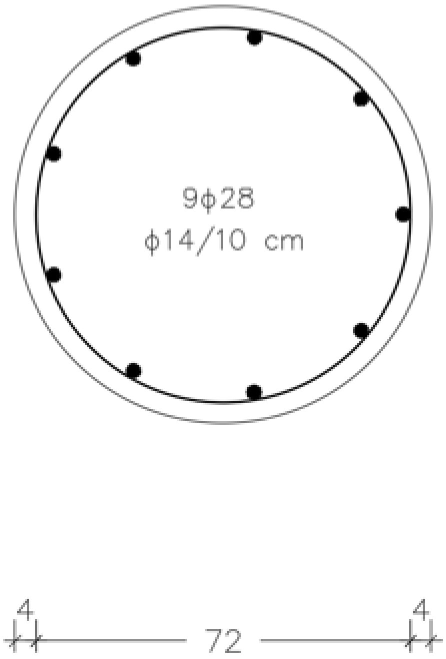


Figure 68: Typical Column Cross Section at First Floor

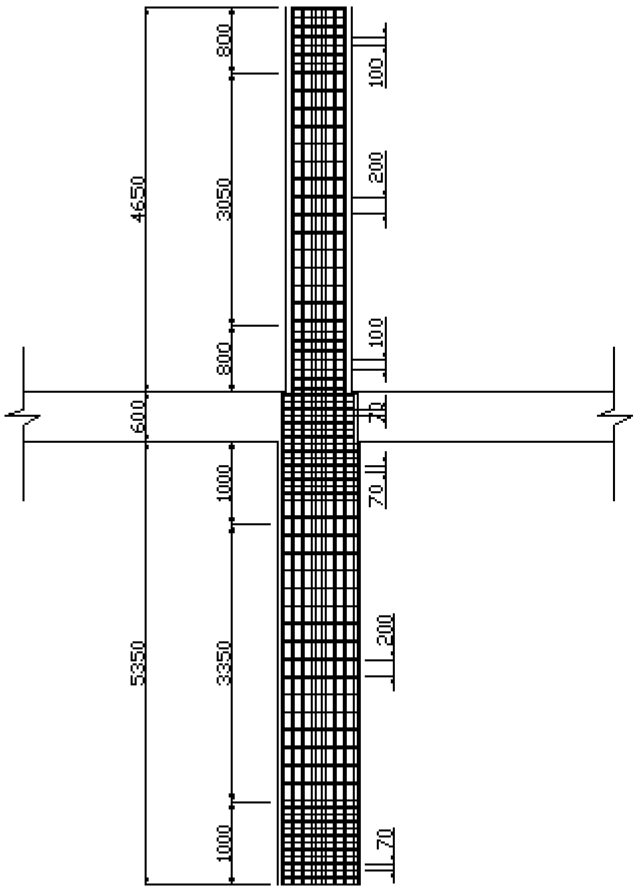


Figure 69: Longitudinal Reinforcement for Ground Level and First Floor Columns

Also, strong column-weak beam principle checked at critical beams-column joints.

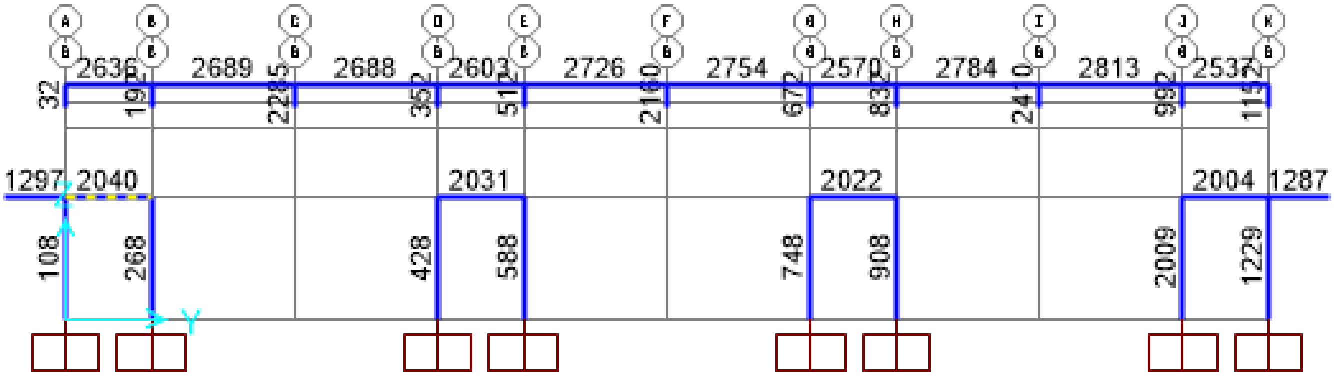


Figure 70: Side View of Mathematical Model

Column 108 or 268 does not continue to first floor, there is no need to check this principle (DBYBHY-3.3.5.4-b).

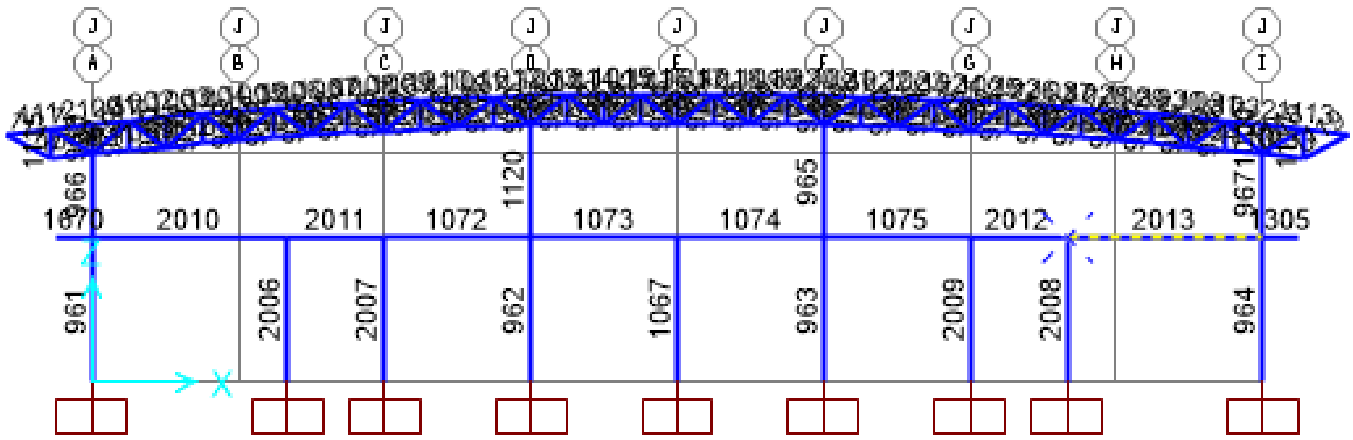


Figure 71: Front View of Mathematical Model

Since Column 2008 does not continue to first floor, there is no need to check strong column weak beam principle at this joint. (DBYBHY-3.3.5.4-b). On the other hand, Column 964 continues to first floor so it has to be checked at this joint.

$(M_{ra} + M_{rü}) \geq 1.2\,(M_{ri} + M_{rj})$ … (Turkish Seismic Code 3.3.5)

M_{ra} = 1539,77 kNm

$M_{rü}$ = 847.25 kNm

M_{ri} = 1298,9 kNm

M_{rj} = 380,2 kNm

$1539,77 + 847,25 = 2387 \geq 1,2(1202,8 + 378,4) = 2014,9$ kNm

Columns are stronger than beams so plastic hinge forms at beam ends first.

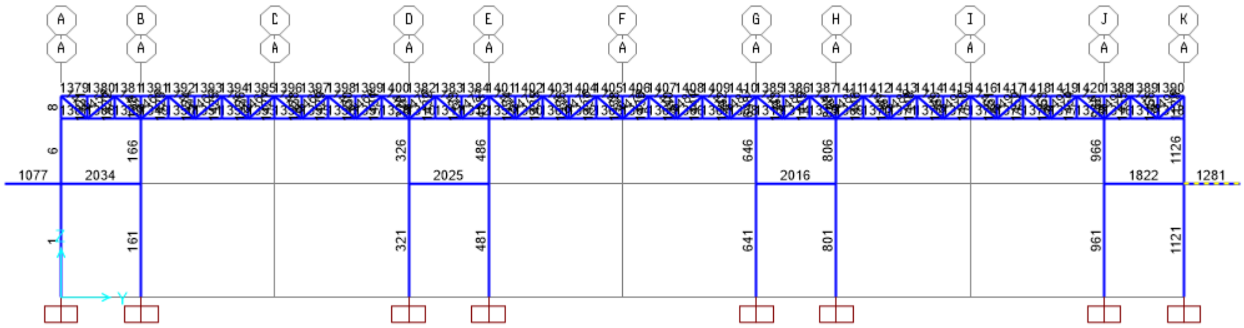


Figure 72: Location of Column 1121

Column 1121 continues to first floor, strong column weak beam principle has to be checked.

$(M_{ra} + M_{rü}) \geq 1.2\,(M_{ri} + M_{rj})$ … (Turkish Seismic Code 3.3.5)

M_{ra} = 1539,77 kNm

$M_{rü}$ = 847.25 kNm

M_{ri} = 531, 2 kNm

M_{rj} = 1453,4 kNm

$1539,77 + 847,25 = 2387 \geq 1,2(1453,4+531,2) = 2381,5$ kNm ….ok.

Moment curvature diagrams are provided in the following 2 figures below.

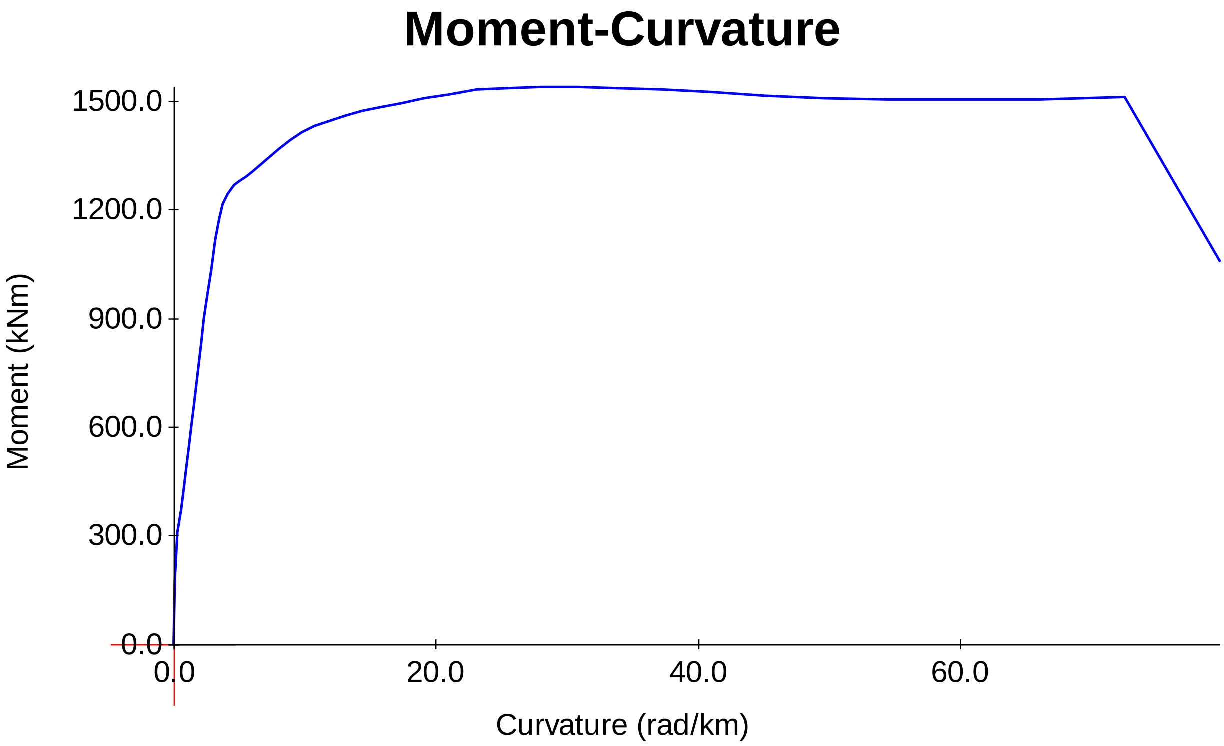


Figure 73: Ground Floor Moment Curvature Diagram

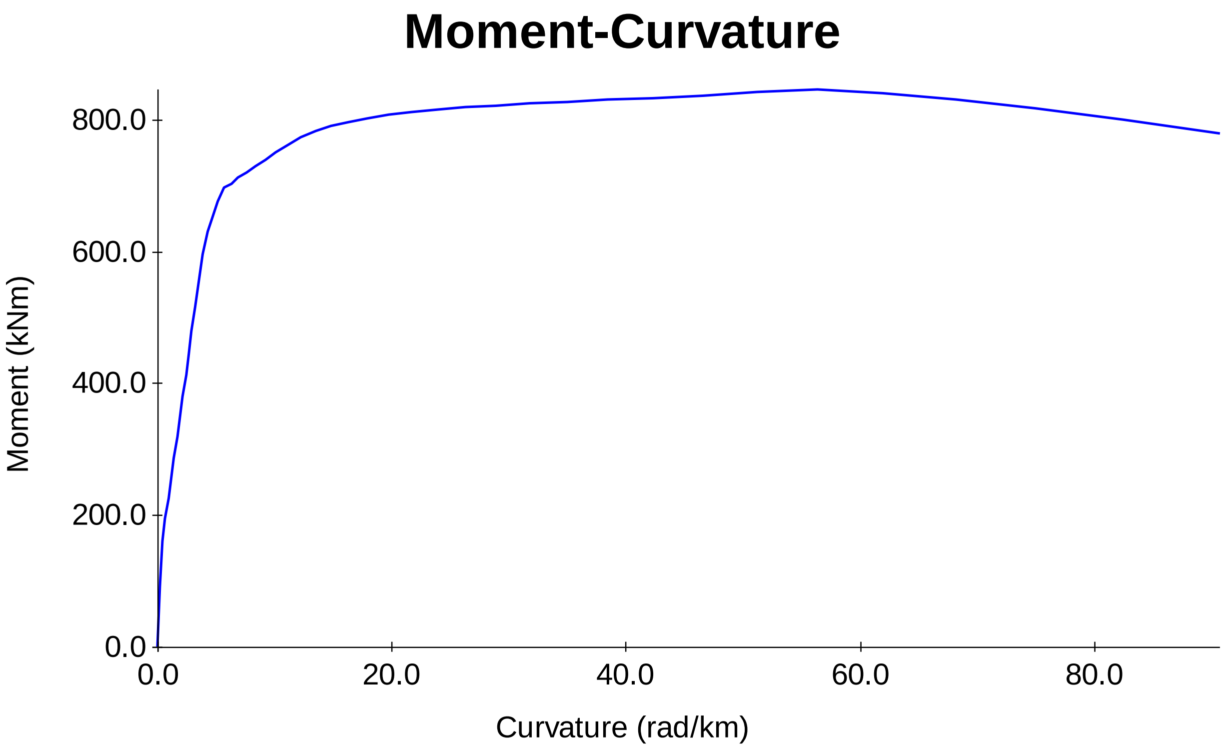


Figure 74: First Floor Moment Curvature Diagram

FOUNDATION DESIGN

GEOTECHNICAL PROPERTIES

In this part of report, there are information about geotechnical properties and preliminary design of foundation. For our Marmaray Station Project, a soil investigation report of a dwelling house project in Ankara is given us. According to investigation of borehole logs, field and laboratory tests, an idealized soil profile with estimated and correlated soil parameters is determined. Moreover, the dimensions and type of foundation is estimated based on estimated settlement and structure loads.

Site Investigation and Idealized Soil Profile

There are 9 different boreholes with the total depth of 235.00 m in field work. The depth and location of each borehole log is given below.

Table 18: Depth of Borehole Logs

No. Of Borehole (BH)	Depth (m)
BH-1	30.00
BH-2	25.00
BH-3	30.00
BH-4	25.00
BH-5	30.00
BH-6	25.00
BH-7	25.00
BH-8	30.00
BH-9	15.00

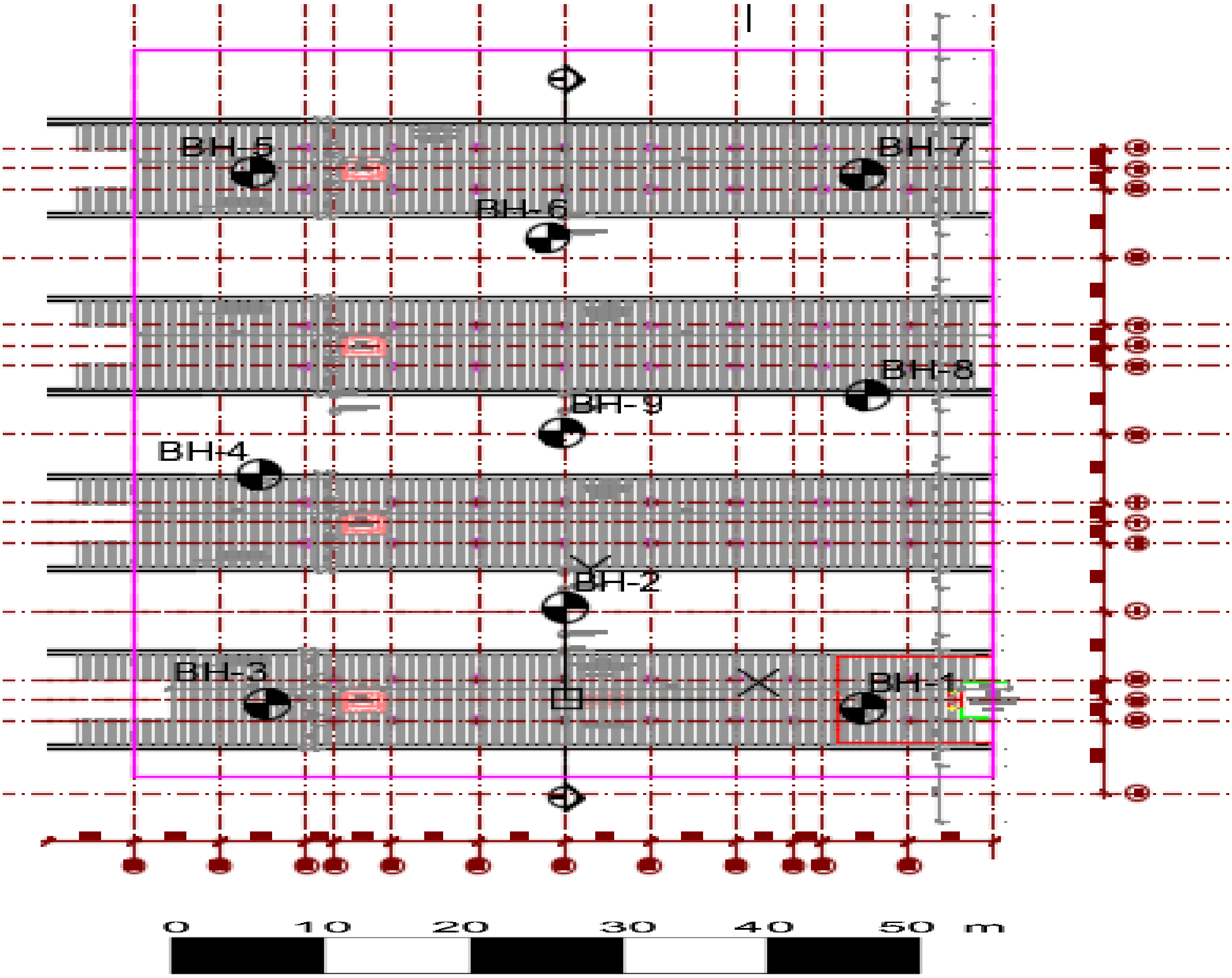


Figure 75: Location of Boreholes

In accordance with land survey, the most dominant soil type is silty clay for soil under our project. In order to estimate the soil parameters, all laboratory test results and borehole data are used. One thing to note is that, as a common geotechnical engineering practice, depth of a borehole should be taken as 3*B for strip foundation. We have 4 platform foundations and their width is 10.0 m so, our borehole depth is computed as 30 m. Maximum borehole depth is given as 25 m in the corresponding geotechnical site report for the station. Since given borehole depth is not enough, calculations are done accordingly, if it is necessary to examine greater depths, it is convenient to make assumptions by stating them in the report as well.

Obtained soil parameters are explained in the following:

Fine and coarse content are determined from sieve analysis test results.

Liquid Limit, Plastic Limit, and Plasticity index values are taken from Atterberg Test Results.

Water content and unit weight of soil are taken from laboratory test results. The obtained

results of above information are tabulated in Table 2.

Table 19: Some Soil Parameters Obtained from Laboratory Test Results

Group Symbol	Clay (CH, CL) & Silt (MH, ML)
Fine Material Ratio	F(%) = 50.8 – 96.3 => Estimated Value of F(%) = 80.0
Coarse Material Ratio	C(%) = 0.0 – 12.6 => Estimated Value of C(%) = 1.0
Water Content	w(%) = 15.2 – 40.9 => Estimated Value of w(%) = 26.0
Liquid Limit	LL(%) = 50.8 – 96.3 => Estimated Value of F(%) = 50.0
Plastic Limit	PL(%) = 50.8 – 96.3 => Estimated Value of F(%) = 27.0
Plasticity Index	PI(%) = 50.8 – 96.3 => Estimated Value of F(%) = 23.0
Unit Weight (kN/m^3)	ɣ(%) = 16.07–20.30 => Estimated Value of ɣ(%) = 18.5 kN/m^3

After determining some soil parameter from laboratory test results, the other needed soil parameter are computed and correlated based on geotechnical design criteria.

Firstly, since the given SPT N values are N_{45}, they are corrected based on the Energy Ratio to find N_{60}. In other words, given SPT-N values are multiplied with 0,45/0,6= 0,75 (N_{60} = 0.75*N) and SPT N_{60} numbers are obtained accordingly.

According to laboratory test results (Unconfined undrained triaxial test & unconfined compression test), undrained shear strength (c_u) of soil is calculated in the following Table 19:

Table 20: Undrained Shear Strength Base on Laboratory Tests

Description	cu (kPa) (Lab)	qu (kPa) (Lab)	cu (kPa) (qu/2)
Silty Clay	85.0- 350.0 (ave. 185)	158.7 – 375.4 (ave. 235)	120

Then, undrained shear strength (c_u) of soil is calculated in according to recommendations of Kulhawy and Mayne (1990) and Stroud (1974). Kulhawy and Mayne (1990) stated that relation between (c_u/p_a) and SPT-N given in Table 4 can approximately be represented by (c_u/p_a) = 0.06*N.

SPT-N	*Consistency*	*Approximate c_u/p_a ratio*
0 – 2	Very soft	< 1/8
2 – 4	Soft	1/8 – 1/4
4 – 8	Medium stiff	1/4 – 1/2
8 – 15	Stiff	1/2 – 1
15 – 30	Very stiff	1 – 2
> 30	Hard	> 2

p_a: atmospheric pressure ≅ 100 kN/m^2

Figure 76: Relation Between SPT N-C_u (Terzaghi and Peck, 1967)

$$(cu/pa) = 0.06*N$$

Table 21: Undrained Shear Strength According to Kulhawy and Mayne (1990)

Description	**N_{ave}**	**(cu/pa) ratio**	**pa (kN/m^2)**	**cu (kPa)**
Silty Clay	50	> 2	100	300

Also, Stroud (1974) proposed a correlation between f1 (is a factor depending on plasticity index) and PI as shown in Figure 73 and cu (kN/m^2) = f1*N60 .

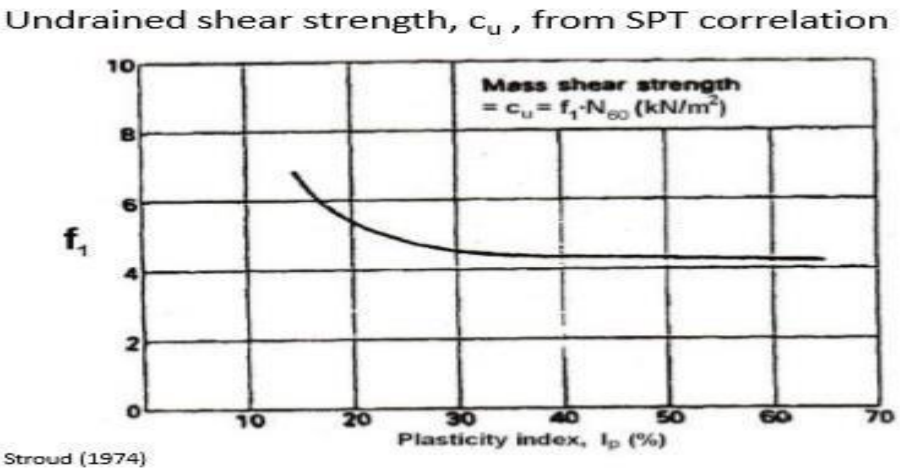


Figure 77: Relation Between SPT-N60-C_u-PI (Stroud, 1974)

Table 22: Undrained Shear Strength According to Stroud (1974)

Description	N_{ave}	PI (%)	f1	cu (kPa) (Stroud)
Silty Clay	50	23	5	250

For long term parameters of silty clay soil, effective angle of shearing resistance can be estimated from Figure 74 or Figure 75. These figures include some correlations between plasticity index and effective friction angle which are recommended by Terzaghi, Peck, & Mesri (1996) and Gibson (1953).

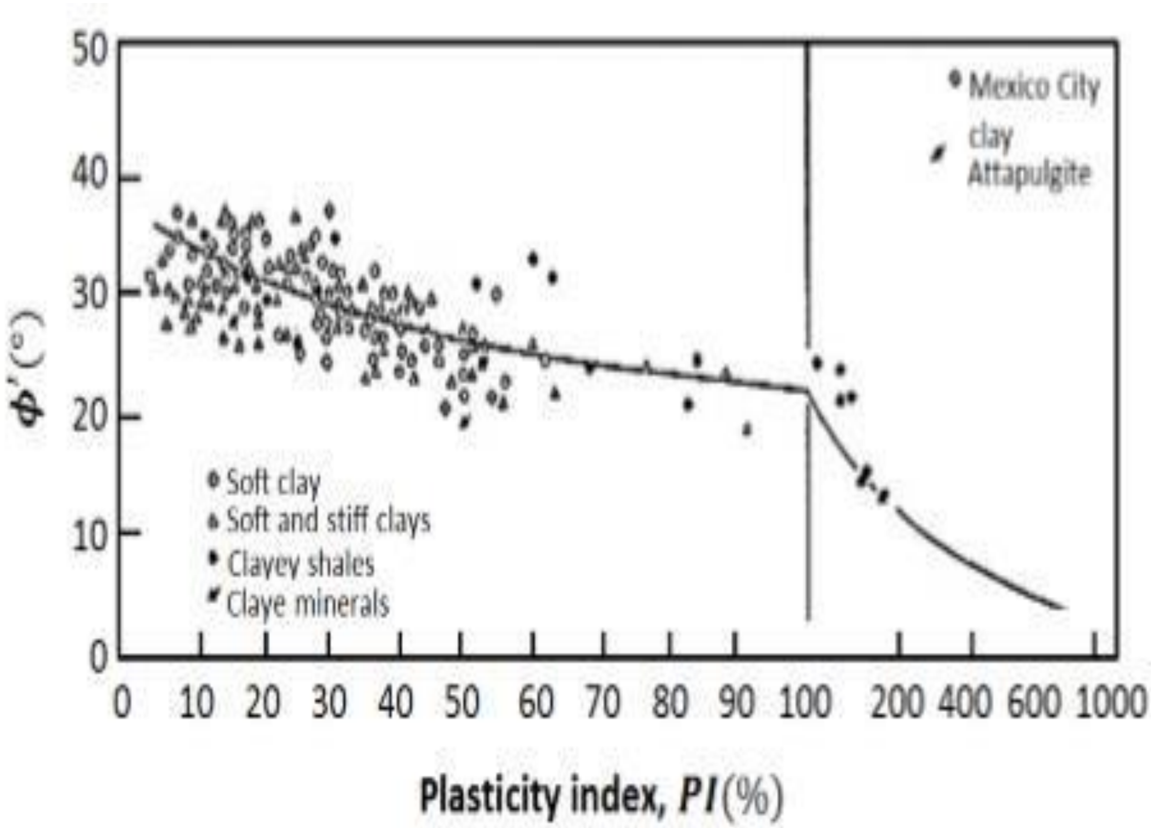


Figure 78: Relation between Plasticity Index-(Terzaghi, Peck and Mesri, 1996)

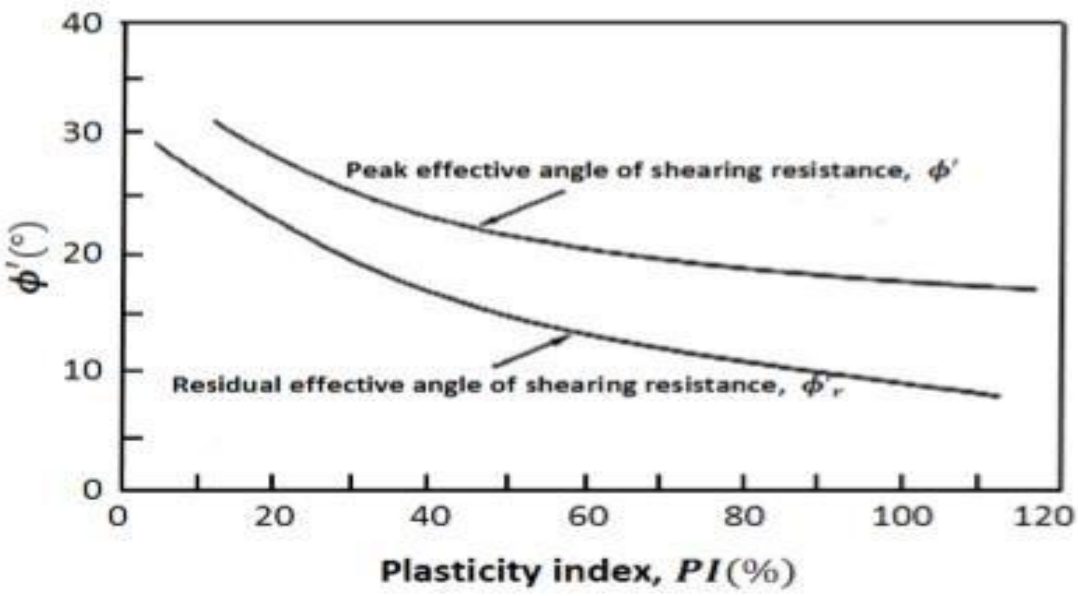


Figure 79: Relation Between Plasticity Index-(Gibson, 1995)

Table 23: Effective Friction Angle of Shearing Resistance

Description	PI (%)	Φ' (^{0})	
		Terzaghi, Peck, Mesri (1996)	Gibson (1953)
Silty Clay	23	30	28

In according to above calculation, for long and short term shear strength parameters of silty clay soil can be taken as the following:

Table 24: Shear Strength Parameter for Drained and Undrained Conditions

Description	cu (kPa)	Φu	c' (kPa)	Φ'	ɣ (kN/m^3)
Silty Clay	200	0	15	29	18.5

The coefficient of compressibility (m_v) can be obtained from Stroud (1974) chart:

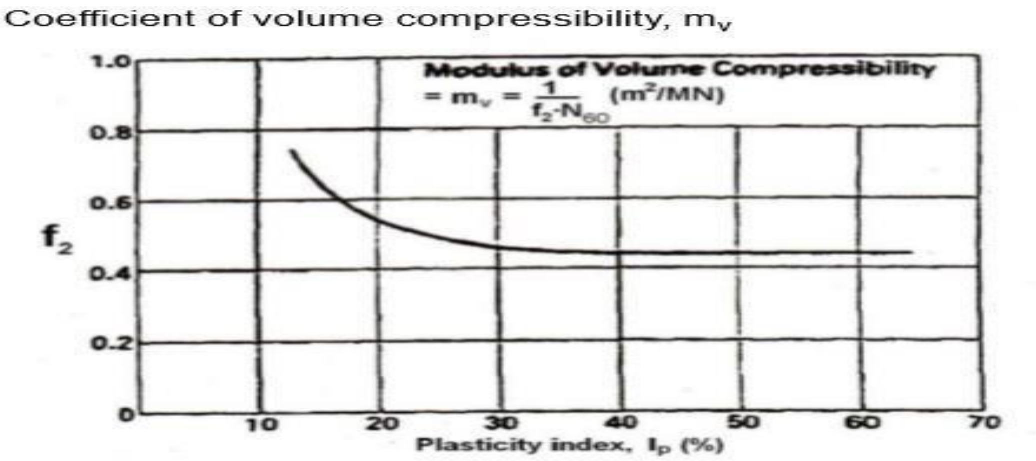


Figure 80: Estimation of mv Based on Relation Between f_2-PI-SPT N

Table 25: Obtained Coefficient of Compressibility Based on Stroud (1974) Recommendation

Description	**N_{ave}**	**PI (%)**	**f2**	**mv (m^2/MN) (Stroud)**
Silty Clay	50	23	0.5	0.04

Undrained deformation modulus (E_u) can be estimated based on undrained shear strength or average SPT N value;

- Duncan&Buchignani (1976) suggested that $E_u = 100 – 300c_u$ and,
- Butler (1975) proposed that $E_u/N_{60} = (1 – 1.2)$ (MN/ m^2)

Table 26: Obtained Fu Based on Undrained Shear Strength or Average SPT Value

Description	c_u (kPa)	N_{ave}	E_u (kPa)	
			Duncan& Buchignani (1976)	Butler (1975)
Silty Clay	200	50	20000 - 60000	50000 - 60000

All estimated soil parameters for all layers for long and short term conditions are shown below:

Table 27: Soil Parameters of Silty Clay Soil

Description	c_u (kPa)	Φ_u	c' (kPa)	Φ'	γ (kN/m^3)	mv (m^2/kN)	E_u (kPa)
Silty Clay	200	0	15	28	18.5	$0.40*10^{-4}$	45000

In the light of test results which are taken from boring holes, idealized soil profile is drawn below figures 77.

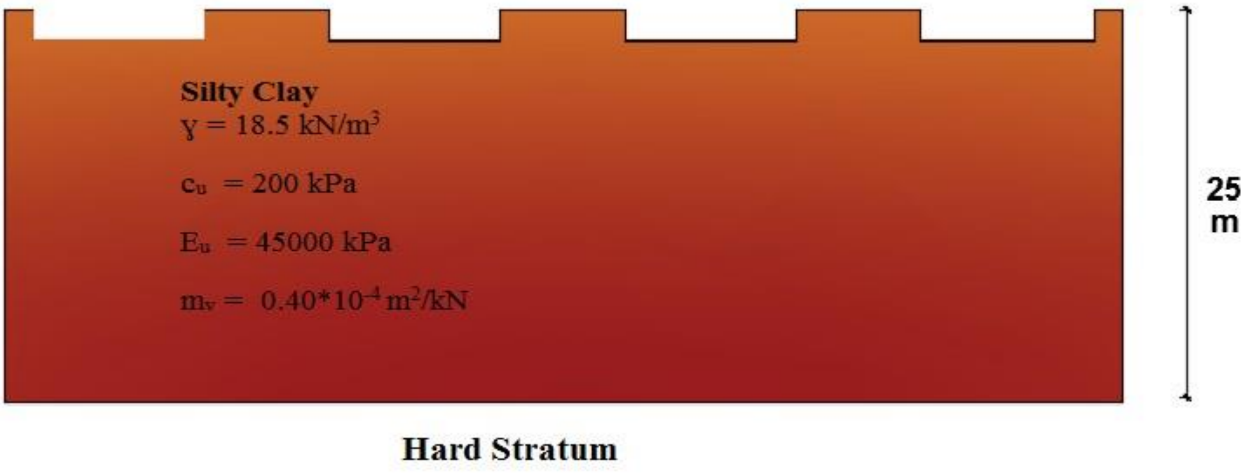


Figure 81: Idealized Soil Profile

Foundation Type and Characteristics

There are 4 platforms in this project and the dimensions of each platform are 10.00 m*59.80 m. Depth of foundation is considered as 2 m since the depth of frost zone is 1.5 m. A continuous foundation will be design for each platform and the thickness of foundation is assumed as 50 cm.

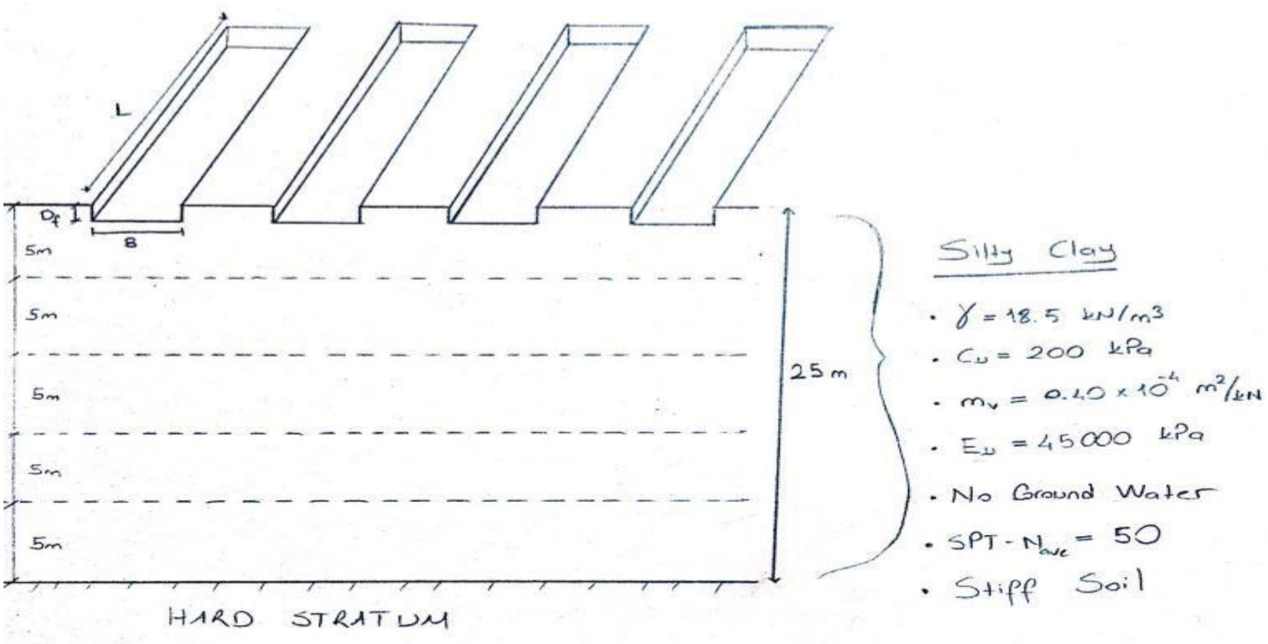


Figure 82: Approximate Drawing of Expected Foundation's Locations and Divided Layers for Settlement Calculation

Bearing Capacity

The bearing capacity formula of foundation on clays is written as;

$$q_f = C_u * N_c + \gamma * D$$

Where, for N_C value Skempton (1951) proposed a chart for design of foundations on clay (See Figure 8).

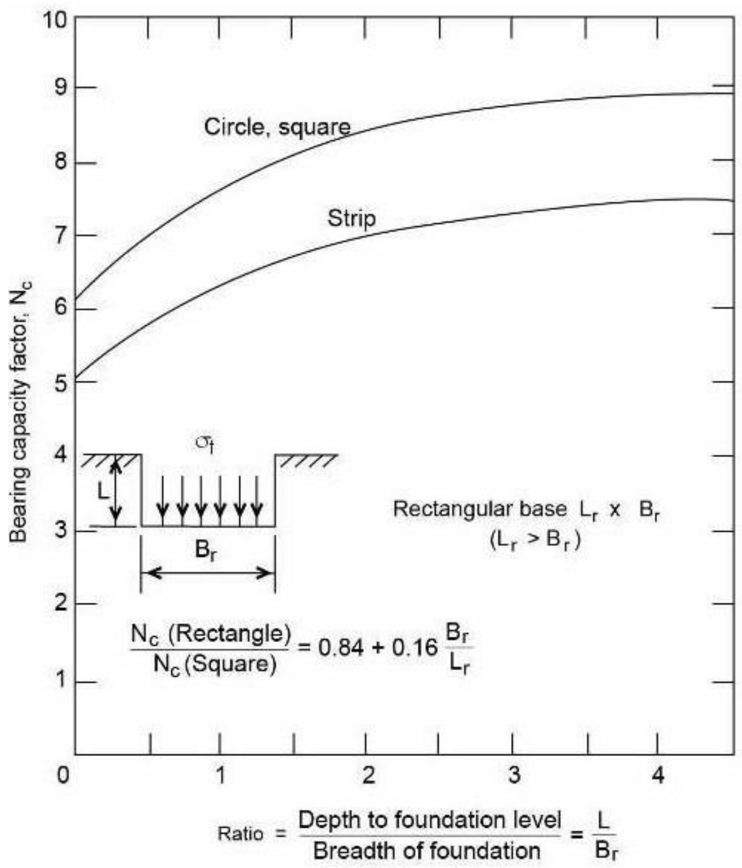


Figure 83: Bearing Capacity Factors for Foundations in Clay (F=0) After Skempton 1951

- D_f = 2 m (Depth of Foundation)
- Factor of Safety = 3.0
- Total Ultimate Bearing Capacity : $q_f = C_u * N_c + \gamma * D = 200*5.2 + 18.5*2 = 1077$ kPa
- Net bearing Capacity : $(q_{net})_{ultimate} = q_f - \gamma * D = C_u * N_c = 1040$ kPa
- Net safe Bearing Capacity : $(q_{net})_{safe} = q_{net} / F.S. = (1040/3) = 347$ kPa
- Total Safe Bearing Capacity : $(q_{total})_{safe} = (q_{net})_{safe} + \gamma * D_f = 347 + 18.5*2 = 384$ kPa

Table 28: Bearing Capacity for Foundation of One Platform

Structure	Type of Foundation	B*L (m)	Nc	cu(kPa)	(qnet)safe (kPa)	ɣ*Df	(qtotal)safe
Marmaray Station (on 4 platforms)	Continuous Foundation	10.0* 59.80	5.2	200	347	37	384

NET FOUNDATION PRESSURE

The loads of the building were determined based on dead and live load calculations of the structural system. In addition to loads of the building, foundation characteristics and load can also be seen in Table 12 and also there is a backfill pressure.

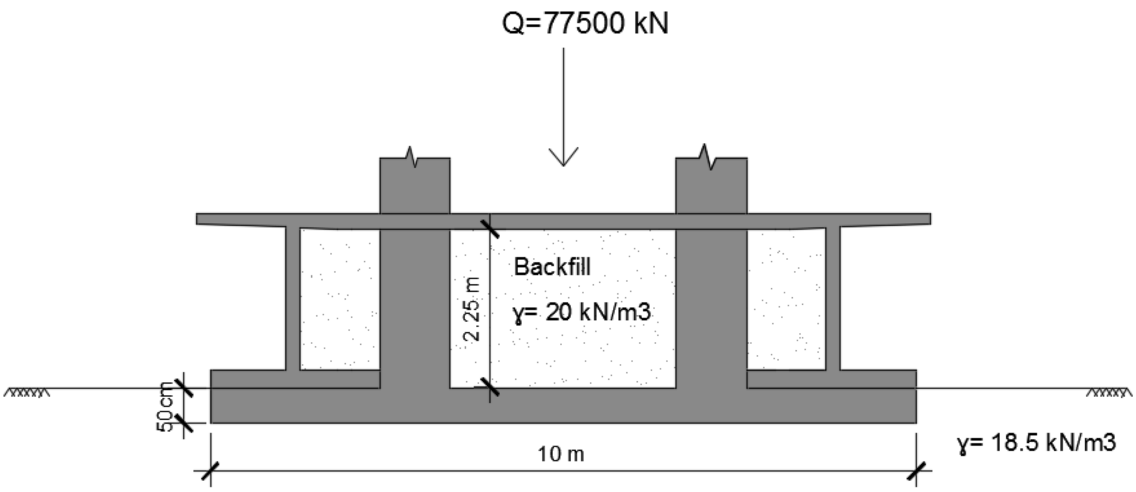


Figure 80: Cross-Section of the Middle Platform

Building Load = 7750 tons, Q = 7750*10 = 77500 kN

There are 4 platforms. The middle two platforms take 2Q load and the out two platforms take Q load. 6Q = 77500 kN => Q= 12916.67 kN and 2Q = 25833.3 kN

Area of one platform, A = 10*51.2 = 512 m^2

Pressure on the foundation of middle platform, q = (25833.3)/ (512) = 50.45 kPa

Table 29: Foundation Characteristics of One Platform

Type	Continuous Foundation
Width	10.0 m
Thickness	0.50 m
Length	51.20 m
Area	10*51.20 = 512 m^2
Volume	512*0.5 = 266 m^3
Concrete Unit Weight	25 kN/m^3
Weight	266*25 = 6400 kN
Pressure	6400/ 512 = 12.5 kPa

Backfill pressure; $q_{backfill}$ = 2.25*20 = 45 kPa

qgross = 50.45 + 12.5 + 45= 107.95 kPa

qswelling = 17.5 kPa

qsoil = 18.5*2 = 37 kPa

qnet = qgross – qswelling – qsoil = 107.95 – 37- 17.5 = 53.45 kPa

(qult)net = 1040 kPa > qnet = 53.45 so it is safe.

(qtotal)safe = 384 kPa > qgross =107.95kPa OK.

SETTELMENT CALCULATIONS

Immediate Settlement

A procedure that gives average vertical settlement under a flexible area carrying a uniform pressure q is followed for immediate settlement of foundations on clay layer.

$$Si = \mu 0 * \mu 1 * \frac{q * B}{E|}$$

Where, μ0, μ1= empirical factors dependent on the foundation geometry B=Width of the foundation

E=Deformation Modulus

the empirical factors can be estimated using the charts given in Figure 81 below.

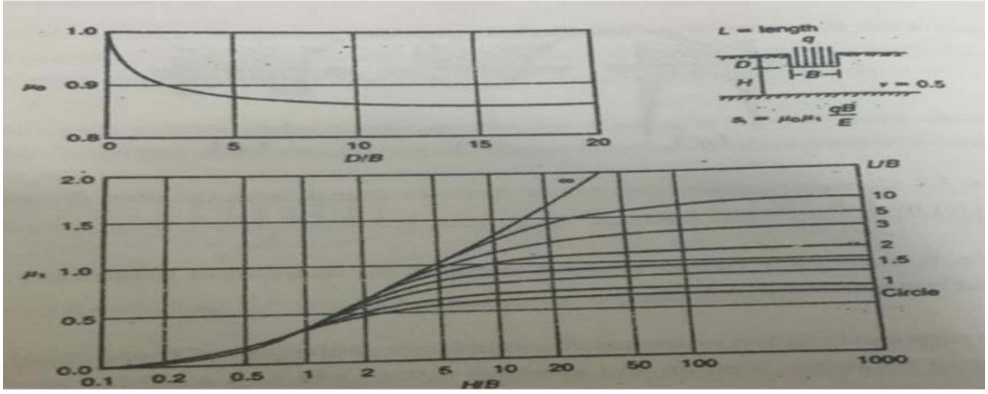


Figure 81: Empirical correlation factors for immediate settlement calculation

In our case D/B=2/10 = 0.5, so from chart μ_0 can be taken as 1.0. In the case of swelling pressure, the thickness of compressible layer, H can be taken as the length of width, B= 10.0 m. Hence, L/B = 59.80/10.0 = 5.98 and H/B = 1. Then, from chart μ_1=0.4 Undrained deformation modulus had been calculated as 45000 kPa before. And the net pressure on the foundation was calculated as 53.45 kPa. Assuming single soil layer under the foundation, the problem can be simplified into the following Figure 10.

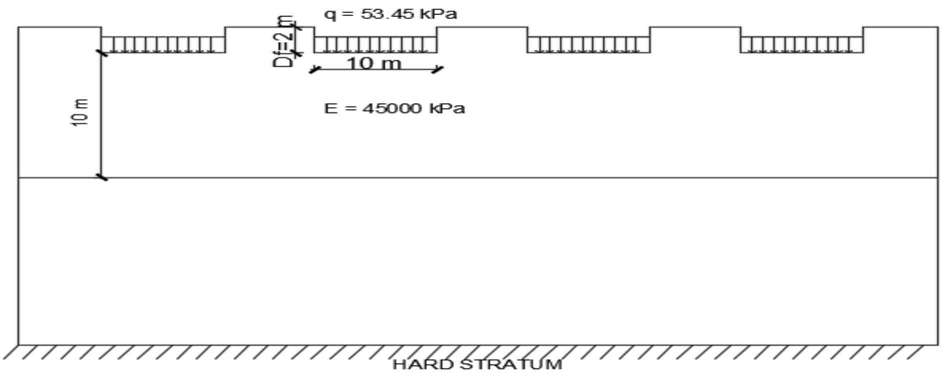


Figure 82: Representation of Immediate Settlement Case

Then;

$$Si = 1.0 * 0.4 * \frac{53.5*10}{45000} = 0.0048 \text{ m}$$

Consolidation Settlement

There swelling pressure in foundation, since the distance between two platforms is great enough (8.20 m), there is no need to calculate the stress increasing due to overlapping. Only the settlement calculation due to net foundation pressure will be enough. In case of strip foundation, the thickness of settlement layer can be taken as H = 2B = 20 m. And to calculate the settlement properly, layer should be divided to two layers. (See Figure 11)

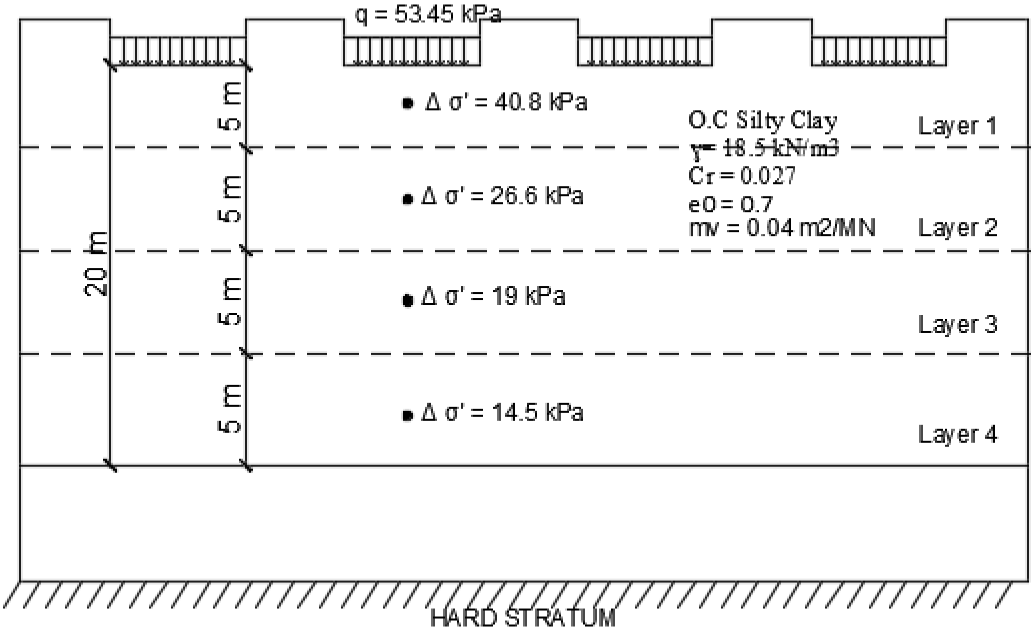


Figure 83: Representation of the Consolidation Settlement Case

In order to calculate the consolidation settlement, the following formula can be used;

Soed = H*mv* Δ σ'

For layer 1:

Stress increment due to foundation is calculated from Boussinesq 2V:1H approximation rule

$\Delta\sigma' = 40.8$ kPa

For layer 2:

Stress increment due to foundation is calculated from Boussinesq 2V:1H approximation rule

$\Delta\sigma' = 26.6$ kPa

For layer 3:

Stress increment due to foundation is calculated from Boussinesq 2V:1H approximation rule

$\Delta\sigma' = 19$ kPa

For layer 4:

Stress increment due to foundation is calculated from Boussinesq 2V:1H approximation rule

$\Delta\sigma' = 14.5$ kPa

Soed=$\sum(H * mv * \Delta\sigma') = 5 * 0.04 * 10^{-3} * (40.8 + 26.6 + 14.5) = 0.02\ m$

Skempton-Bjerrrum correction (μ) should be applied to calcualte settlement due to actual case.

This correction factor is abtained from Table-13. The foundation soil is overconcolidated clays, so averaged as 0.6.

$S_c = \mu * S_{oed} = 0.6 * 0.02 = 0.012\ m$

Then, the final settlement will be the summation of intial and consolidation settlement.

$S_{total} = Si + Sc = 0.0048 + 0.0121 = 0.0169\ m = 1.69\ cm\ and\ it\ is\ acceptable$

FOUNDATION DESIGN

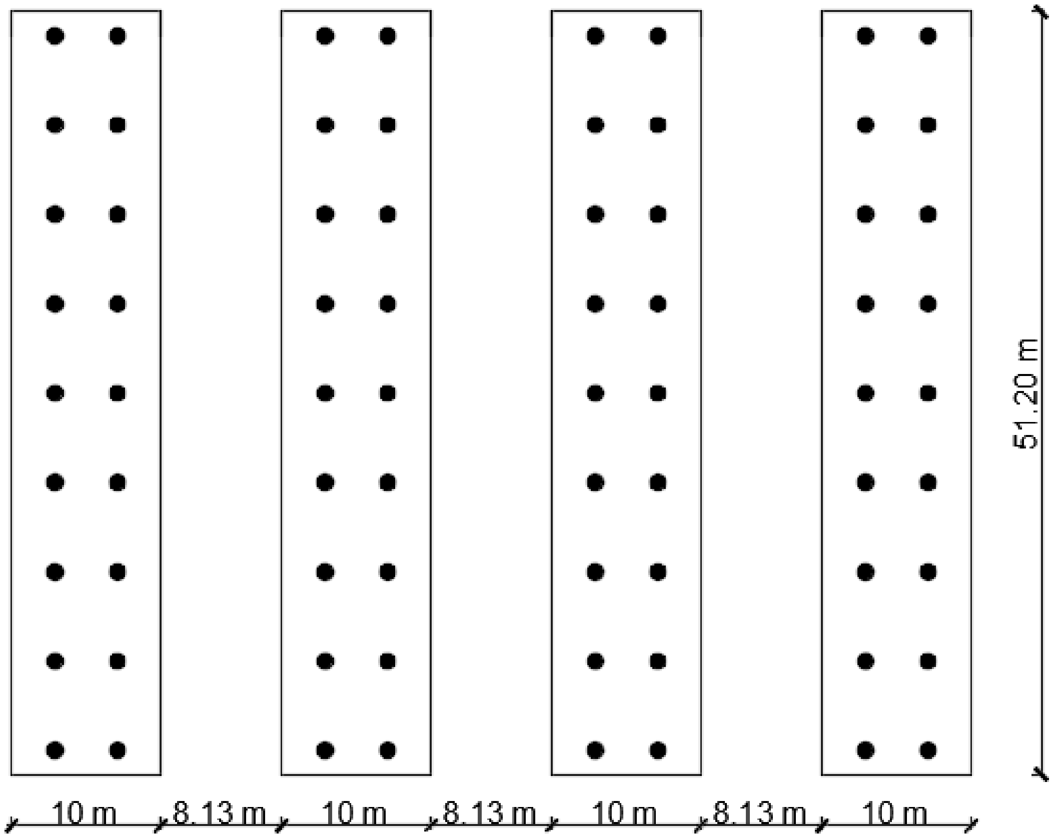


Figure 84: Platform Plan View

Coefficient of subgrade reaction:

$$k = \frac{E_{soil}}{B(1-\upsilon^2)} = \frac{45000}{10*(1-0.15^2)} = 4603.58 \ {}^{kN}/_{m^3}$$

where $\upsilon = 0.1 - 0.2 \ for \ undrained \ stiff \ clay, E_{soil} = 45000 \ kPa, B = 10 \ m$

According to spring tributary area:

k = 4603.58*2.1= 9667.5 kN/m, $k = 9667.5 \ {}^{kN}/_{m}$

Soil Pressure:

$q = \frac{\sum Q}{B*L} * (1 \pm \frac{6e_1}{L} \pm \frac{6e_2}{B})$

q_{max} = 168 kPa and q_{min} =90 kPa

Bearing Capacity Check; $(q_{net})_{safe}$ = 347 kPa > q_{max} = 168 kPa OK.

Punching Check

Edge:

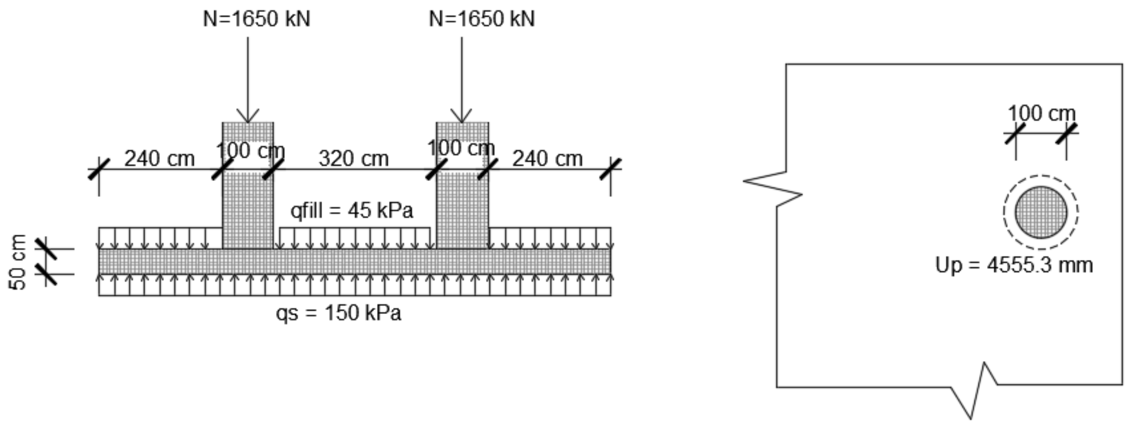


Figure 85: Representation of Punching Shear Check

h =50 cm, d=45 cm $U_p = \pi * (1000 + 450) = 4555.3$ mm

$$A_p = \frac{\pi * 1.45^2}{4} = 1.65\ m^2$$

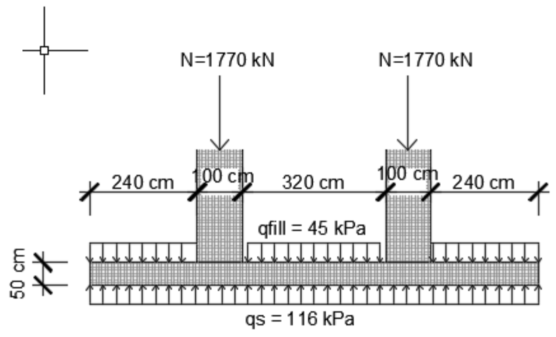
$$V_{pd} = 1650 - (150 - 45) * 1.65 = 1476.75\ kN$$

$f_{ctd} = \frac{0.35\sqrt{30}}{1.5}$ = 1.28 Mpa

$$V_{pc} = \gamma f_{ctd} U_p d = 1 * 1.28 * 4555.3 * 450 = 2624\ kN$$

$$V_{pc} > V_{pd}\ \ OK\ for\ punching$$

In the middle column for N= 1770 kN



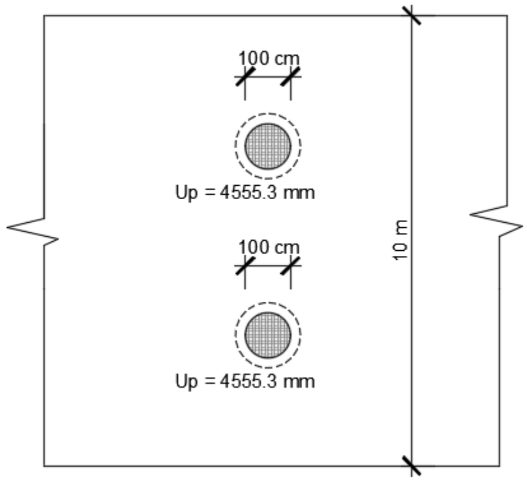


Figure 86: Representation of Punching Check

$$U_p = \pi * (1000 + 450) = 4555.3 \text{ mm}$$

$$A_p = \frac{\pi * 1.45^2}{4} = 1.65\, m^2$$

$$V_{pd} = 1770 - (116 - 45) * 1.65 = 1652.85\, kN$$

$$V_{pc} = \gamma f_{ctd} U_p d = 1 * 1.28 * 4555.3 * 450 = 2624\, kN$$

$$V_{pc} > V_{pd} \;\; OK\ for\ punching$$

FOUNDATION REINFORCEMENT DETAIL

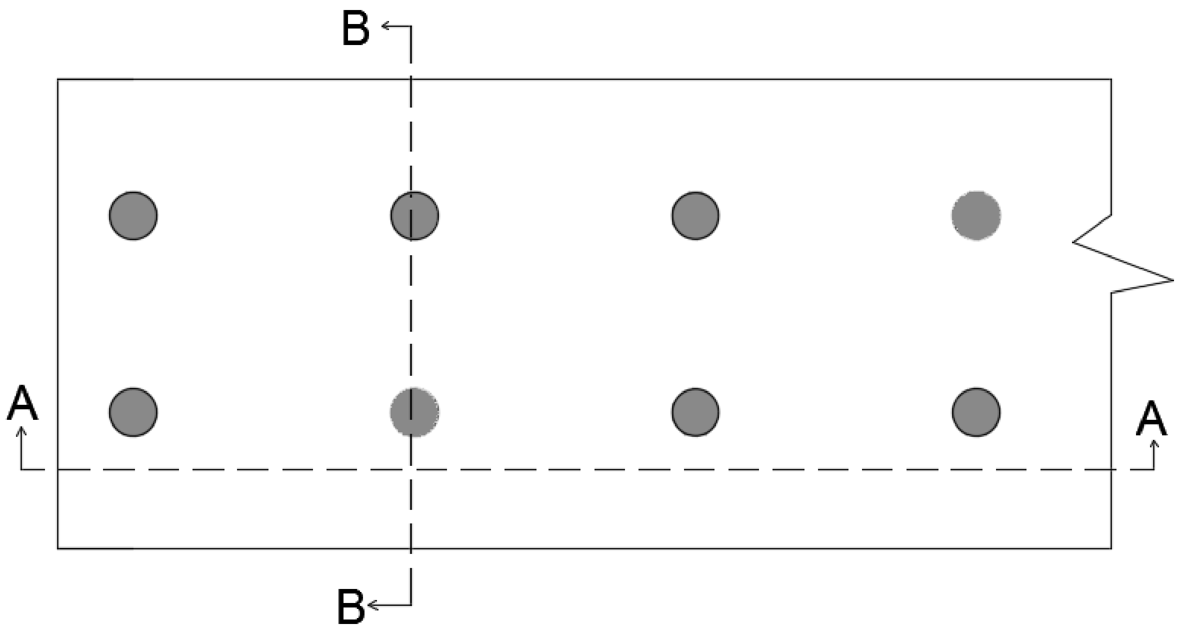


Figure 87: Cross Section of One Platform

B-B Section

Support (bottom) reinforcement

M_{dy} = 430 kN.m/m and K_l = 247 mm^2/ kN for C30& S420

$$K = \frac{b_w d^2}{M_{dy}} = \frac{1000 * 450^2}{430 * 10^6} = 470 \, {}^{mm^2}/_{kN} > K_l$$

$$A_s = \frac{M_d}{f_{yd} * j * d} = \frac{430 * 10^6}{365 * 0.86 * 450} = 3044 mm^2 \;\; (bottom\ reinforcement)$$

270Φ24/15 cm to bottom

Span (top) reinforcement

M_{dy} = -156 kN.m/m and Kl = 247 mm^2/ kN for C30& S420

$$K = \frac{b_w d^2}{M_{dy}} = \frac{1000 * 450^2}{156 * 10^6} = 1298 \, {}^{mm^2}/_{kN} > K_l$$

$$A_s = \frac{M_d}{f_{yd} * j * d} = \frac{156 * 10^6}{365 * 0.86 * 450} = 1104 \, mm^2 \;\; (top\ reinforcement)$$

170Φ24/25 cm top

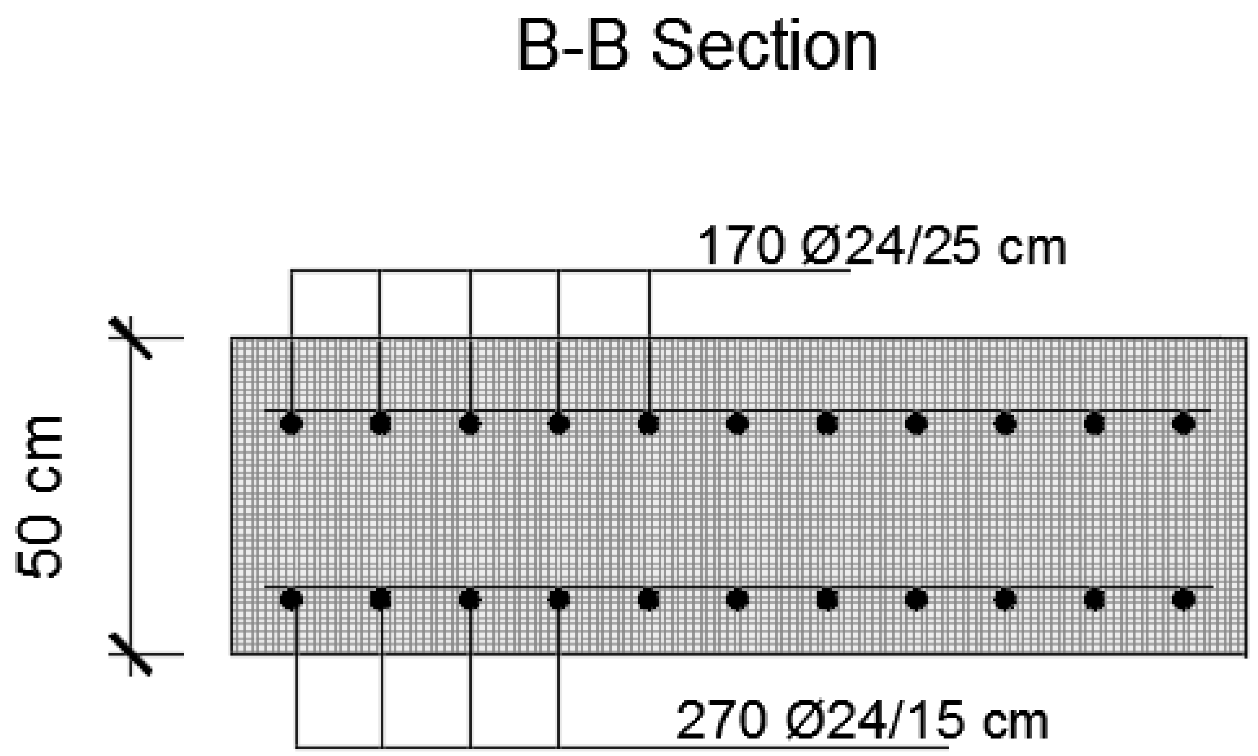


Figure 88: Reinforcement on B-B Section

A-A Section

Support (bottom) reinforcement:

M_{dx} = 475 kN.m/m and Kl = 247 mm^2/ kN for C30& S420

$$K = \frac{b_w d^2}{M_{dy}} = \frac{1000 * 450^2}{475 * 10^6} = 426 \, {}^{mm^2}/_{kN} > K_l$$

$$A_s = \frac{M_d}{f_{yd} * j * d} = \frac{475 * 10^6}{365 * 0.86 * 450} = 3363 \, mm^2 \;\; (bottom \;\; reinforcement)$$

340Φ24/15 cm to bottom

Span (top) reinforcement

M_{dx} = -291 kN.m/m and Kl = 247 mm^2/ kN for C30& S420

$$K = \frac{b_w d^2}{M_{dy}} = \frac{1000 * 450^2}{291 * 10^6} = 696 \, {}^{mm^2}/_{kN} > K_l$$

$$A_s = \frac{M_d}{f_{yd} * j * d} = \frac{291 * 10^6}{365 * 0.86 * 450} = 2060 \, mm^2 \;\; (top \;\; reinforcement)$$

205Φ24/25 cm to top

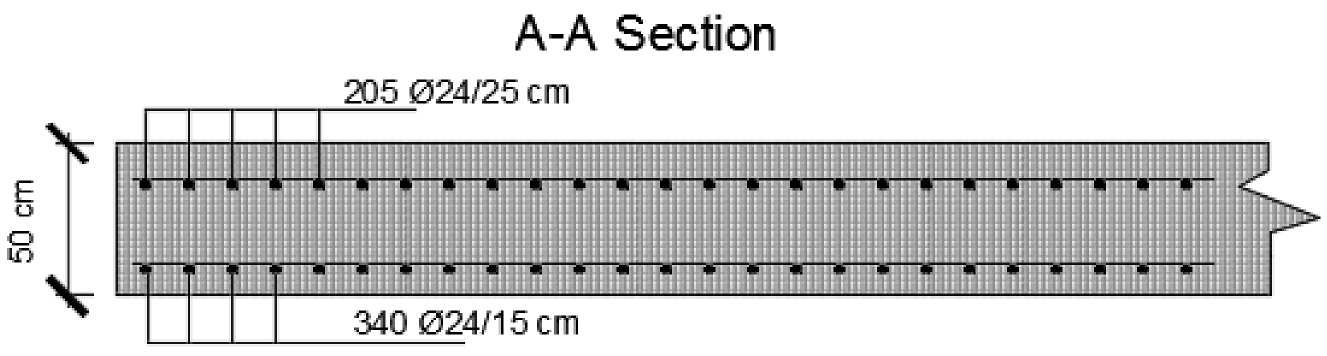


Figure 89: Reinforcement on Section A-A

COST ESTIMATION

In cost estimation part, unit prices which is taken from Ministry of environment and urbanization are used.

What we actually did is:

Cost of 1 m^3 concrete (in-situ casting) is taken as 150 TL and 1m^3 precast is taken as 375 TL

Cost of diameter of 24 mm steel is given as 3.551 TL for kg/m in group energy (without cost of contingencies) by taking into consideration transport (10%), KDV (value-added-tax, 18%) and losses (3%):

$$3.551 + \left(\frac{3.551 * 18}{100}\right) + \left(\frac{3.551 * 10}{100}\right) + \left(\frac{3.551 * 3}{100}\right) = 4.65\ TL$$

This calculation is done for different diameter of reinforcement in the same way.

COST OF FOUNDATION

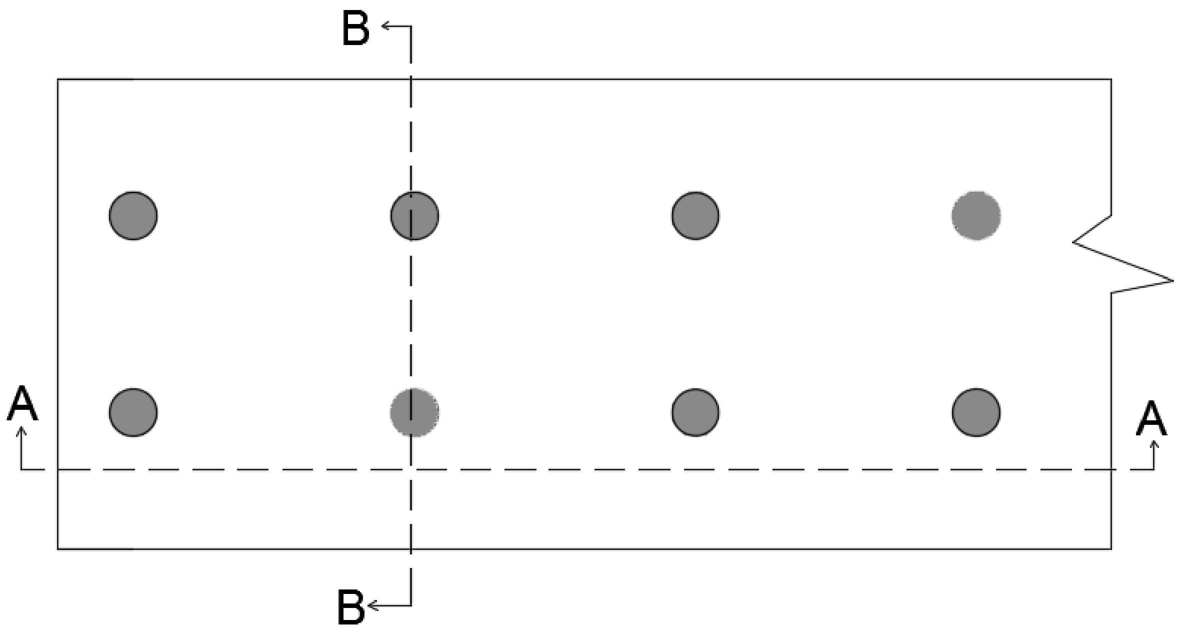


Figure 90: Platform Level Plan View

Cost of concrete part of foundation:

There are 4 platforms

4*51.2*10*0.5(depth of foundation) = 1024 m^3

1024 m^3 * 150 = 153600 TL

Cost of reinforcement for A-A section:

Support part:

Number* length*mass (kg/m).TL= price (TL)

270*51.2*4.65 = 64282 TL

Span part:

Number* length*mass (kg/m).TL= price (TL)

170*51.2*4.65 = 40474 TL

Cost of reinforcement for B-B section:

Support part:

Number* length*mass (kg/m).TL= price (TL)

340*10*4.65 = 15810 TL

Span part:

Number* length*mass (kg/m).TL= price (TL)

205*10*4.65 = 9533 TL

Total cost of foundation: 283699 TL take it 283700 TL

Cost of slabs:

1. Cost of platform level:

Concrete (4 platforms):

4*59.90*10.06*0.2(20 cm thickness of concrete) = 482 m^3

Cost= 482* 150 = 72300 TL

COST OF CONCOURSE LEVEL

Precast of Marmora Feneryolu Station has properties as following:

Length of one precast element: 13.5 m

Width of one precast element: 1.2 m

Height of one precast element: 0.46 m

Volume of one precast element: 13.5*1.2*0.46 = 7.5 m^3

In concourse floor 42 elements are used for one part, as it can be seen from formwork plan there are three parts which makes 42*3= 126 precast element,

Total volume = 126*7.5 = 945 m^3

Price of precast = 945*375 = 354375 TL

In situ concrete:

2(2 of the platforms have same geometry)*9.83*51.60*0.2(20 cm thickness of concrete) = 203 m^3

203*150 = 30450 TL

2(remaining 2 platforms have same geometry)*9.20*51.60*0.2 = 190 m^3

190*150= 28500 TL

Reinforcement

Φ10/15 cm in both ways

51.6 m/ 0.15 m =344, 65.18 m/0.15 m= 435   totally, 780

344*0.808*65.18 = 18117 TL, 435*0.808*51.6 = 1813,6 TL

COST OF COLUMNS

Columns of ground floor

Concrete:

Volume of columns:

72(number of columns)*((π*1^2)/4*5.04) = 285 m^3

285* 150 = 42750 TL

Vertical bars:

Number* length*mass (kg/m).TL= price (TL)

For one column 13*5.35*6.32 = 440 TL

Since there are 72 columns in ground floor,

72*440= 31680 TL

Stirrups:

Length of middle stirrups = (2*π*r)*number of stirrups

Length of middle stirrups = (2* π*0.46)*16 =46 m

Length* mass (kg/m).TL = 46*1.58 = 73 TL

Length of end stirrups = (2*π*r)*number of stirrups

Length of end stirrups = (2* π*0.46)*29 = 84 m

Length* mass (kg/m).TL = 84*1.58 = 133 TL

Cost of stirrups = 73+133=206 TL

Columns of first floor

Concrete:

Volume of columns:

16(number of columns)*(π*0.8^2)/4*3.29) = 26.5 m^3

16(number of columns)*(π*0.8^2)/4*4.46) =36 m^3

62.5*150 = 9375 TL

Vertical bars:

Number* length*mass (kg/m).TL= price (TL)

9*4.94*1.58= 70 TL

Stirrups:

Length of middle stirrups = (2*π*r)*number of stirrups

Length of middle stirrups = (2* π*0.36)*16 =36 m

Length* mass (kg/m).TL = 36*1.58 = 57 TL

Length of end stirrups = (2*π*r)*number of stirrups

Length of end stirrups = (2* π*0.36)*29 = 66 m

Length* mass (kg/m).TL = 66*1.58 = 104 TL

Cost of stirrups = 57+104 = 161 TL

Cost of beams:

1. 50 cm/60 cm beams:

Concrete:

Volume: 0.5*0.6*4.2= 1.26 m^3

Cost: 1.26*150= 189 TL

Reinforcement:

33*4*4.2*2.07 = 1148 TL,   33*4*4.2*1.58 =876 TL

2. 60cm/ 80 cm beams:

Concrete:

Volume: 0.6*0.8*8= 3.84 m^3

Cost: 3.84*150 = 576 TL

Reinforcement:

60*4*6.34*8= 12173 TL, 60*3*1.58*8= 2275 TL

3. 30 cm/138 cm beams:

Concrete:

Volume: 0.3*13.8*6= 24.8 m^3

Cost: 24.8*150 = 3726 TL

Reinforcement:

10*3*6.34*6 = 1141 TL, 10*7*1.58*6 = 664 TL

Up to roof total cost =880000 TL

COST OF ROOF TRUSS SYSTEM

Weight of the roof taken from SAP2000 as shown:

Table 30: Types of Steel Profiles in Roof Design

TABLE: Material List 2 - By Section Property				
Section	**Object Type**	**Unit pieces**	**Total Length**	**Total Weight**
Text	Text	Unitless	m	KN
UPN200	Frame	306	1777.62	449.189
2UPN140	Frame	560	854.78191	273.77
2UPN200	Frame	96	144.26466	73.147
2L70X10	Frame	1059	1661.07395	339.025
TUBO-D114.3X3.6	Frame	176	1283.98027	126.192
2UPN180	Frame	416	615.17124	270.816
				1532.139

Since 1 kN =0.10197 ton

Weight of the roof is 1532.139 kN = 156.24 Ton=156240 kg

From a factory prices if price is taken as 1.8 dollar/kg

1 Dollar =3.54 TL,   so it becomes 6.37 TL/kg

Cost of roof = 6.37*156240 = 995249 TL

Total material cost = 1875249 TL

Material cost is approximately 30% of the total cost so that:

Total cost = 6250830 TL

Area of the station is: 51.6*65.18 = 3363.29 m^2

6250830/ 3363.29 = 1859 TL (for 1 m^2 of the station)

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