CE410

Civil Engineering Design

ST - 6

Post Tensioned Concrete Water Reservoir

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INTRODUCTION

GENERAL INFORMATION ABOUT WATER RESERVOIRS

As Greek philosopher Thales said, "Water is the source of every creation." In day to day life one cannot live without water. Therefore water needs to be stored for daily use. Depending upon the location of the tank the tanks can be named as overhead, on ground or underground. The tanks can be made in different shapes usually circular and rectangular shapes are mostly used. The tanks can be made of RCC or even of steel.

Basing on the location, storage tanks can be classified into three categories. Those are:

- Underground tanks
- Tank resting on grounds
- Overhead tanks

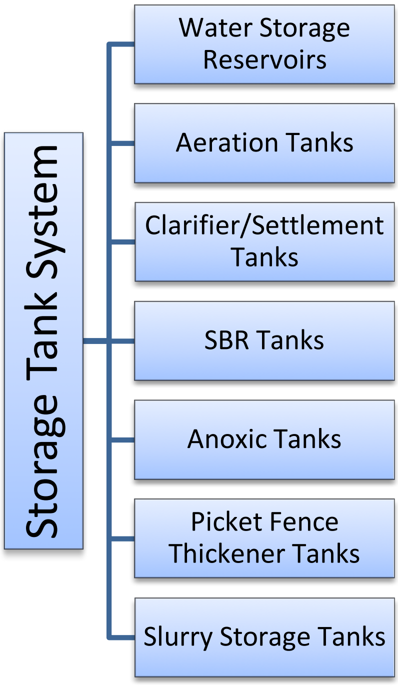
Figure 1: Overground Water Tanks

Figure 2: Underground Water Tanks

Figure3: Overhead Water Tanks

In most cases the underground and on ground tanks are circular or rectangular is shape but the shape of the overhead tanks are influenced by the aesthetical view of the surroundings and as well as the design of the construction. The overhead tanks are supported by the column which acts as stages. This column can be braced for increasing strength and as well as to improve the aesthetic views.

The Storage Tank system can be used for a wide range of applications.



BRIEF INFORMATION ABOUT THE PTRC WATER RESERVOIR

The water storage tank will be constructed in Gölbaşı, Ankara. It will have a storage capacity of 4250 tons water. Circular and post tensioned reinforced concrete over ground tank with the internal diameter of 15 meters and the height of 25 meters is requested by public authorities.

As observed in similar storage structures contemporary built in Anatolia, walls of the structure is decided to have thicknesses of 0,6 meters for first 10 meter from the ground level and 0,4 meters from 10 meters up to 25 meters.

For horizontal post tensioning application on the walls two-buttress containment is decided after an investigation on similar structures. In two-buttress containment, tendons run 180 or 360 degrees around the circumference. Conventional vertical and some horizontal reinforcement also used to prevent and control the cracks along the walls of reservoir.

For the foundation of the structure, due to inappropriate soil conditions piling, ground improvement and the raft foundation is examined. It is found that the piling operation will serve more safe structure but the cost increases drastically. Thus the foundation type of the water tank is going to be raft foundation and it is planned as a circular foundation with 25 m diameter and 1 meter depth due to ground characteristics.

The water reservoir will have a metal roof having a weight of 50 tons.

THEORY OF POST-TENSION CONCEPT

Post tensioning, as a concept, dates back to 1928 when France's Eugene Freyssinet developed a method for pre-stressing cast-in-place concrete.

Post tensioning is a method in which high strength steel reinforcement is tensioned after the concrete has set. The first serious use of this technique in the United States was for the Walnut Street Bridge in Philadelphia in 1949. Since then, there has been a steady and dramatic growth in its use for all types of concrete construction projects: office buildings, stadiums, parking structures, high-rise apartments, bridges of all sizes, and, of course, nuclear power plant containments.

Figure 4 : First Post-Tensioned Concrete Bridge

The popularity of post tensioning has been earned by the benefits it affords concrete construction.

- It permits the reduction of the structural depth of members.
- It virtually eliminates cracks in slabs, making them essentially water tight.
- It controls the deflection of structural members.
- It makes possible economical, longer spans.

Post tensioning enables the concrete to withstand high tensile forces. In a non-post tensioning vessel, internal pressures will cause cracks, since concrete has poor resistance to tensile forces. By reinforcing the vessel with post-tensioned steel tendons, a compressive stress is intentionally applied to the concrete. When internal pressures are applied to the post-tensioned concrete, they are offset by the previously applied pre-stressing forces, resulting in the desired stress condition.

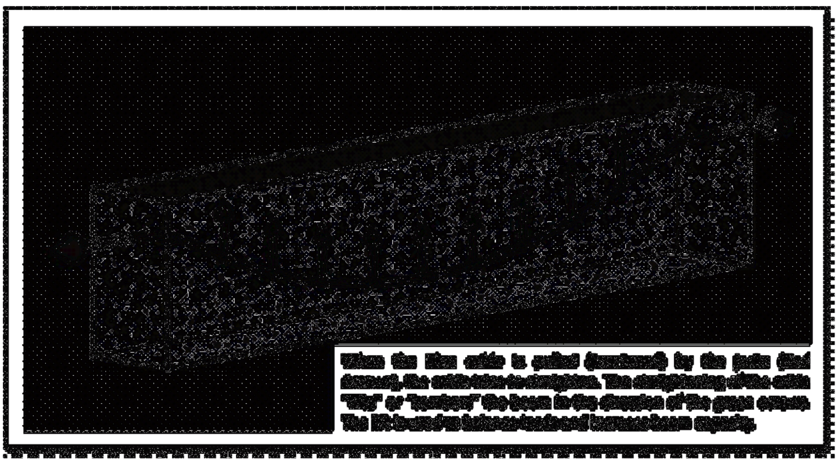


Figure 5: Post-Tensioning on a Beam

Historically, storage tanks were designed and constructed traditionally with very thick and heavily reinforced concrete wall sections. Even then, tanks designed and constructed in this way would frequently suffer wall cracking and leakage, leading to a reduced operational life; in addition, construction of such tanks was costly and slow, and the carbon footprint of structures of this type was very large. The key benefit of the innovative Post Tensioning approach to reservoir design is that it ensures that the tank walls are maintained in permanent horizontal compression under all load cases and for the whole design life (typically 80 - 100 years) of the structure, thereby guaranteeing leak-free construction. The use of post tensioning also allows much more economical wall thicknesses and reinforcement quantities, so that costs and build times are significantly reduced and the associated carbon footprint is dramatically lowered.

The horizontal tendons which encircle the containment structure are anchored at the buttresses. The number of buttresses may vary from six, at 60 degrees spacing, to two, at 180 degrees spacing.

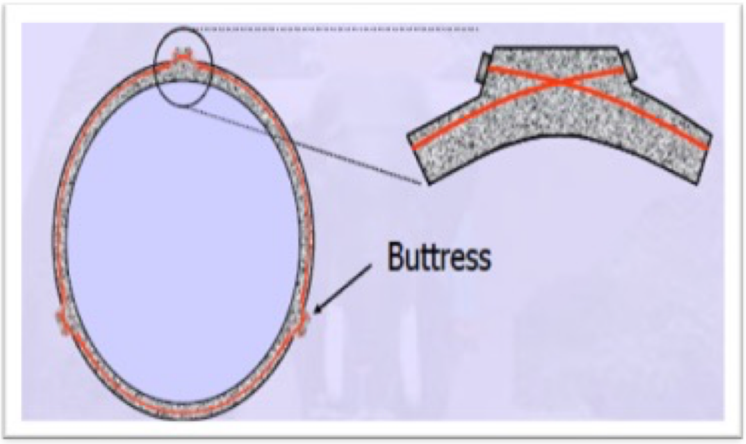


Figure 2: Representative Anchorage

METHOD OF CONSTRUCTION

After a brief investigation over "VSL CONCRETE STORAGE STRUCTURES - USE OF THE VSL SPECIAL CONSTRUCTION METHODS" we have decided to use slip forming system. The advantage of slip forming include the short construction time resulting from continuous working, monolithic construction without construction joints and of high dimensional accuracy and cost savings even where the height is moderate. The slip forms consist of 1 to 1.5 m high elements of steel. Thus pouring concrete of the tank will be completed as 17 or 18 levels.

Figure 7: Slip forming application

HOOP TENSION AND HOOP STRESS CONCEPT

The hoop stress is the force exerted circumferentially (perpendicular both to the axis and to the radius of the object) in both directions on every particle in the cylinder wall.

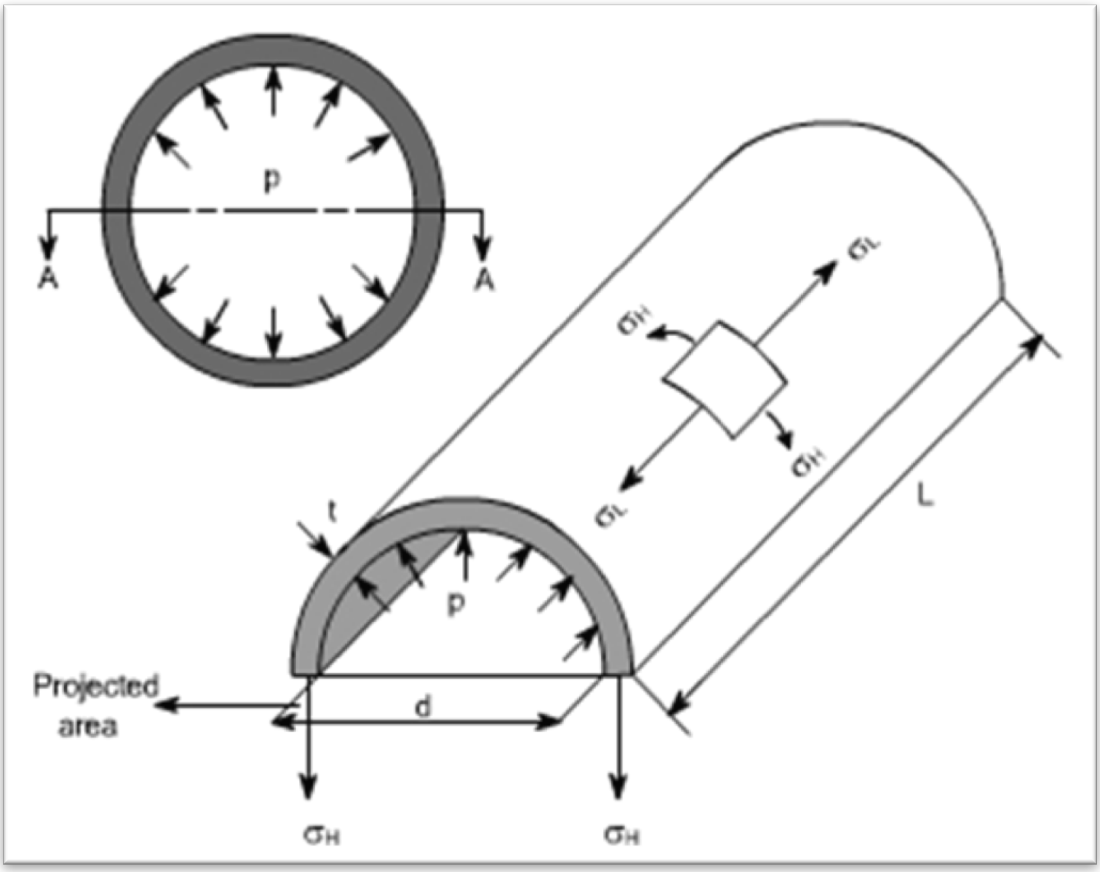


Figure 8: Hoop Stress

For the thin-walled assumption to be valid the structure must have a wall thickness of no more than about one-tenth of its radius. This allows for treating the wall as a surface, and subsequently using the Young–Laplace equation for estimating the hoop stress created by an internal pressure on a thin wall. Since wall thickness (thicker one) of our structure less than one tenth of its radius we can assume our structure as thin walled and we can use Young – Laplace equation in order to calculate hoop tension stress on the wall.

$$\text{Young} - \text{Laplace Equation: } \boldsymbol{\sigma_h} = \frac{P * r}{t} \text{ (for cyclinder)}$$

DESIGN CRITERIA

- **BS EN 1990:2002 -** Eurocode — Basis ofstructural design
- **EN 1991-1-1 -** General actions. Densities, self-weight, imposed loads for buildings
- **EN 1991-1 -3 -** General actions. Snow loads
- **EN 1991-1-4 -** General actions. Wind actions
- **EN 1991-1-5 -** General actions. Thermal Actions
- **EN 1991-4 –** Actions on silos and tanks
- **EN 1992-1-1** - General rules and rules for buildings
- **EN 1992-3** - Liquid retaining and containment structures
- **ACI 318 – 02** Building code requirements for structural concrete
- **TS 500** – Requirements for design and construction for reinforced concrete structures
- **ACI-350.3-01**- Seismic Design of Liquid-Containing Concrete Structures
- **Turkish Earthquake Code 2007**

LOAD COMBINATIONS

To obtain load combinations EN 1991-1-1 was used. The similar load combinations are taken out to avoid repeating. In order to get a result which considers all the effects on reservoir the load combinations are taken as listed below:

Dead Load (DL) : Dead weight of the silo + dead weight of roof, **EQ:** Earthquake Load

T: Temperature, **W:** Wind load, S: Snowload, **HS:** Hydrostatic water pressure

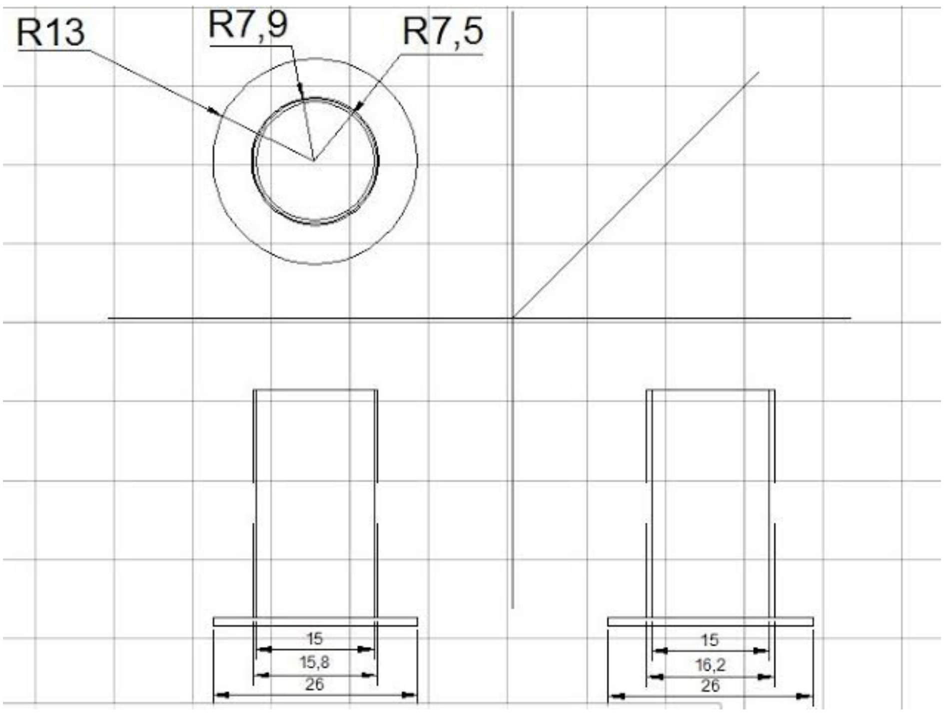
PCY: Convective hydrodynamic force during earthquake

PIY: Impulsive hydrodynamic force during earthquake

PHY: Loads on wall during earthquake…

COMB 1: 1,35 (DL + HS)	**COMB 7:** 1,35 (DL + HS) + 1,5 W + 0,75 S
COMB 2: 1,35 (DL + HS) + 1,5 S	**COMB 8 :** 1,0 (DL + HS)+ 1,0 (EQ + PCY + PIY + PHY)
COMB 3: 1,35 (DL + HS) + 1,5 W	**COMB 9:** 1,0 (DL + HS) + 1,5 S + 0,9 W
COMB 4: 1,0 (DL + HS) + 1,5 S	**COMB 10:** 1,0 (DL + HS) + 1,5 W + 0,75 S
COMB 5: 1,0 (DL + HS) + 1,5 W	**COMB 11:** 1,35 (DL+HS) + 1,5 T
COMB 6: 1,35 (DL + HS) + 1,5 S + 0,9 W	**COMB 12:** Envelope

STRUCTURAL ASPECTS OF WATER RESEVOIR



MATERIALS

Concrete

Type : C40 Type Concrete

Compressive Strength f_{ck} = 40 MPa

Specific Weight γ_c = 24 kN/m^3

Modulus of Elasticity E_c =35000 MPa

Poisson's ratio ν = 0.2

Thermal expansion coefficient α = 10^{-5} per °C

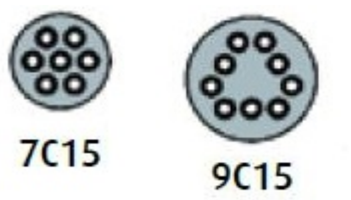
Post – Tensioning steel tendons

Type: 7C15 and 9C15 (Tendon is composed of 7 and 9 strands, each with 150 mm^2 cross-section area)

Ultimate Tensile Strength: 1860 MPa

Yield Strength: 1260 MPa

Initial Prestressing : 900 MPa



LOADS ACTING ON WATER TANK

DEAD LOADS

The weight of concrete for the tank walls, and the roof is considered as dead load.

- Weight of the tank walls : 14024 kN = 1402,4 ton
- Weight of the roof : 500 kN = 50 ton

SNOW LOADS

Gölbaşı has an altitude of 970 m and placed in region II in TS498 that corresponds a characteristic snow load of 1,05 kN/m^2. Snow load acting on the tank determined from EN1991-1-3.

$$s = \mu_i \, C_e \, C_t \, s_k$$

Where;

s = Snow load on the roof [kN/m2]

μ_i = Snow load Shape Coefficient

C_e = Exposure coefficient

C_t = Thermal coefficient

According to EN1991-1-3 Clause 5.2 – (7) and (8), C_e and C_t values are taken as 1. Snow load shape coefficient is taken as 0,8 assuming that the tank will have an conical roof.

$$s = 0{,}8 \; x \; 1{,}0 \; x \; 1{,}0 \; x \; 1{,}05 = 0{,}84 \; kN/m^2$$

WIND LOADS

Maximum basic wind velocity for the structure is given as 42 m/s for the buildings having a height between 21 meters to 100 meters in TS498. Rest of the calculations for wind action conducted according to EN1991-1-4 Wind Actions.

Zeminden Yükseklik m	Rüzgar Hızı v m/s
0 - 8	28
9 - 20	36
21 - 100	42

Mean Wind

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, v_b.

$$V_m(z) = C_r(z)\ C_0(z)\ V_b$$

Where :

C_r (z) is the roughness factor

C_0(z) is the orography factor taken as **1.0**

Terrain Roughness

The roughness factor, $c_r(z)$, accounts for the variability of the mean wind velocity at the site of the structure due to:

- the height above ground level
- the ground roughness of the terrain upwind of the structure in the wind direction considered

$$C_r(z) = k_r \ln\left(\frac{z}{z_0}\right)$$

where:

z_0 is the roughness length

k_r terrain factor depending on the roughness length z_0 calculated using

$$k_r = 0,19\left(\frac{z_0}{z_{0,II}}\right)^{0,07}$$

$z_{0,II}$ = 0,05 m

z_0 = 0,3 m (Terrain Cat. III)

From above equations,

$$k_r = 0{,}19\left(\frac{0{,}3}{0{,}05}\right)^{0{,}07} = 0{,}215$$

Since the roof will have an inclination %10, maximum height is taken as 25,75 meters.

$$C_r(25{,}75) = k_r \ln\left(\frac{z}{z_0}\right) = 0{,}215 * \ln\left(\frac{25{,}75}{0{,}3}\right) = 0{,}96$$

Wind Turbulance

The turbulence intensity **I_v(z)** at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity.

$$I_v(z) = \frac{\sigma_v}{V_m(z)} = \frac{k_I}{C_0(z)\ln\left(\frac{z}{z_0}\right)} \rightarrow I_v(25{,}75) = \frac{1{,}0}{4{,}45} = 0{,}225$$

Peak Velocity Pressure

The peak velocity pressure **q_p(z)** at height z, which includes mean and short-term velocity fluctuations, should be determined.

$$q_p(z) = [1 + 7\, I_v(z)]\, \frac{1}{2}\, \rho\, V_m^2(z)$$

where:

ρ is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms

$$q_p(25{,}75) = [1 + 7\, I_v(25{,}75)]\, \frac{1}{2}\, \rho\, V_m^2(25{,}75) = 2{,}62\ kPa$$

External Pressure Coefficients for Circular Cylinders

Pressure coefficients of sections depend upon the Reynolds numbers Re defined by

$$Re = \frac{b\, v(z_e)}{\vartheta} \qquad where;$$

b is the diameter

ϑ is the kinematic viscosity of the air (ϑ = $15 \cdot 10^{-6}$ m^2/s)

$v(z_e)$ is the peak wind velocity

$$v(z_e) = \sqrt{\frac{2\, q_p(z)}{\rho}}$$

The external pressure coefficients cpe of circular cylinders should be determined from ;

$$C_{pe} = C_{p,0}\varphi_{\lambda a}$$

Where

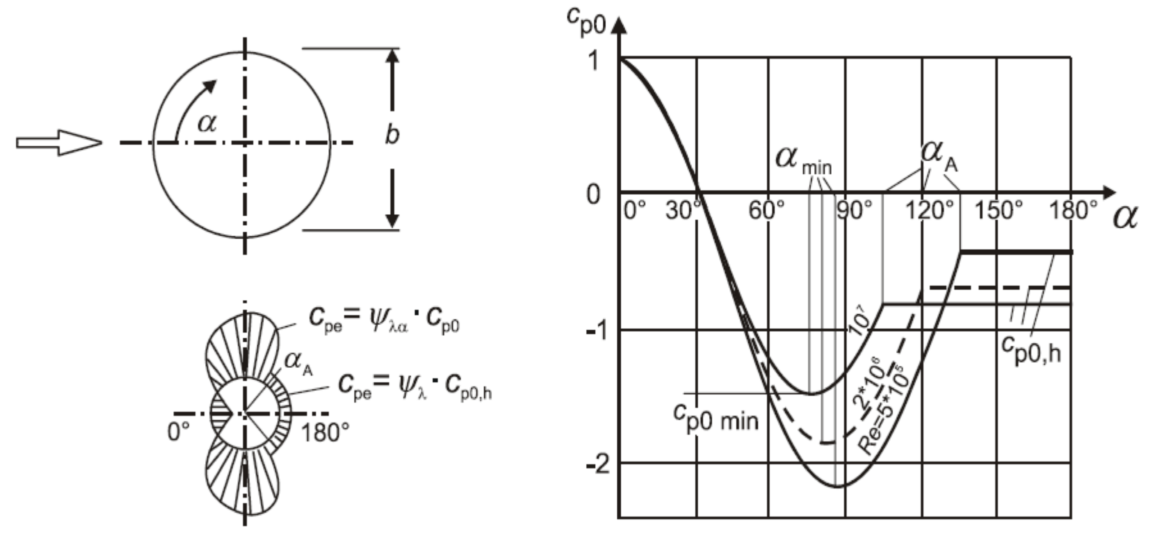
$C_{p,0}$ is the external pressure coefficient without free-end flow

$\psi_{\lambda\alpha}$ is the end-effect factor given by

$\psi_{\lambda\alpha} = 1$ for $0 \le \alpha \le \alpha_{min}$

$\psi_{\lambda\alpha} = \psi_{\alpha} + (1 - \psi_{\alpha}) \cos((\pi/2)*((\alpha - \alpha_{min})/(\alpha_A - \alpha_{min})))$ for $\alpha_{min} \le \alpha \le \alpha_A$

$\psi_{\lambda\alpha} = \psi_{\alpha}$ for $\alpha_A \le \alpha \le 180$



Results of peak velocity pressures are tabulated in excel;

Height(m)	$k_r(z)$	$c_r(z)$	vm(z) (m/s)	$I_v(z)$	$q_p(z)$ kPa	$v(z_e)$ m/s	Re
1	0,215	0,259	7,261	0,831	0,225	0,599	6,47E+05
2	0,215	0,409	11,441	0,527	0,384	0,784	8,46E+05
3	0,215	0,496	13,887	0,434	0,487	0,883	9,53E+05
4	0,215	0,558	15,622	0,386	0,565	0,951	1,03E+06
5	0,215	0,606	16,967	0,355	0,628	1,002	1,08E+06
6	0,215	0,645	18,067	0,334	0,681	1,044	1,13E+06

7	0,215	0,678	18,997	0,317	0,727	1,078	1,16E+06
8	0,215	0,707	19,802	0,305	0,768	1,108	1,20E+06
9	0,215	0,733	26,373	0,294	1,329	1,458	1,58E+06
10	0,215	0,755	27,190	0,285	1,384	1,488	1,61E+06
11	0,215	0,776	27,929	0,278	1,435	1,515	1,64E+06
12	0,215	0,795	28,604	0,271	1,482	1,540	1,66E+06
13	0,215	0,812	29,224	0,265	1,525	1,562	1,69E+06
14	0,215	0,828	29,799	0,260	1,566	1,583	1,71E+06
15	0,215	0,843	30,334	0,256	1,604	1,602	1,73E+06
16	0,215	0,857	30,834	0,251	1,640	1,620	1,75E+06
17	0,215	0,870	31,304	0,248	1,674	1,637	1,77E+06
18	0,215	0,882	31,748	0,244	1,707	1,653	1,78E+06
19	0,215	0,894	32,167	0,241	1,738	1,668	1,80E+06
20	0,215	0,905	32,565	0,238	1,767	1,682	1,82E+06
21	0,215	0,915	38,433	0,235	2,444	1,978	2,14E+06
22	0,215	0,925	38,854	0,233	2,481	1,993	2,15E+06
23	0,215	0,935	39,256	0,230	2,517	2,007	2,17E+06
24	0,215	0,944	39,641	0,228	2,551	2,020	2,18E+06
25	0,215	0,953	40,011	0,226	2,584	2,033	2,20E+06
25,75	0,215	0,959	40,278	0,225	2,608	2,043	2,21E+06

α	$C_{p,0}$	C_{pe}	α	$C_{p,0}$	C_{pe}	α	$C_{p,0}$	C_{pe}	α	$C_{p,0}$	C_{pe}
0	1,000	1,000	**46**	-0,668	-0,668	**92**	-1,540	-1,500	**138**	-0,700	-0,686
1	0,964	0,964	**47**	-0,704	-0,704	**93**	-1,510	-1,463	**139**	-0,700	-0,686
2	0,928	0,928	**48**	-0,740	-0,740	**94**	-1,480	-1,480	**140**	-0,700	-0,686
3	0,891	0,891	**49**	-0,776	-0,776	**95**	-1,450	-1,401	**141**	-0,700	-0,686
4	0,855	0,855	**50**	-0,813	-0,813	**96**	-1,420	-1,388	**142**	-0,700	-0,686
5	0,819	0,819	**51**	-0,849	-0,849	**97**	-1,390	-1,386	**143**	-0,700	-0,686
6	0,783	0,783	**52**	-0,885	-0,885	**98**	-1,360	-1,307	**144**	-0,700	-0,686
7	0,746	0,746	**53**	-0,921	-0,921	**99**	-1,330	-1,312	**145**	-0,700	-0,686
8	0,710	0,710	**54**	-0,958	-0,958	**100**	-1,300	-1,288	**146**	-0,700	-0,686
9	0,674	0,674	**55**	-0,994	-0,994	**101**	-1,270	-1,219	**147**	-0,700	-0,686
10	0,638	0,638	**56**	-1,030	-1,030	**102**	-1,240	-1,233	**148**	-0,700	-0,686
11	0,601	0,601	**57**	-1,066	-1,066	**103**	-1,210	-1,188	**149**	-0,700	-0,686
12	0,565	0,565	**58**	-1,103	-1,103	**104**	-1,180	-1,137	**150**	-0,700	-0,686
13	0,529	0,529	**59**	-1,139	-1,139	**105**	-1,150	-1,149	**151**	-0,700	-0,686
14	0,493	0,493	**60**	-1,175	-1,175	**106**	-1,120	-1,089	**152**	-0,700	-0,686
15	0,456	0,456	**61**	-1,211	-1,211	**107**	-1,090	-1,058	**153**	-0,700	-0,686
16	0,420	0,420	**62**	-1,248	-1,248	**108**	-1,060	-1,060	**154**	-0,700	-0,686
17	0,384	0,384	**63**	-1,284	-1,284	**109**	-1,030	-0,994	**155**	-0,700	-0,686
18	0,348	0,348	**64**	-1,320	-1,320	**110**	-1,000	-0,979	**156**	-0,700	-0,686
19	0,311	0,311	**65**	-1,356	-1,356	**111**	-0,970	-0,966	**157**	-0,700	-0,686
20	0,275	0,275	**66**	-1,393	-1,393	**112**	-0,940	-0,903	**158**	-0,700	-0,686

21	0,239	0,239	**67**	-1,429	-1,429	**113**	-0,910	-0,899	**159**	-0,700	-0,686
22	0,203	0,203	**68**	-1,465	-1,465	**114**	-0,880	-0,870	**160**	-0,700	-0,686
23	0,166	0,166	**69**	-1,501	-1,501	**115**	-0,850	-0,816	**161**	-0,700	-0,686
24	0,130	0,130	**70**	-1,538	-1,538	**116**	-0,820	-0,816	**162**	-0,700	-0,686
25	0,094	0,094	**71**	-1,574	-1,574	**117**	-0,790	-0,774	**163**	-0,700	-0,686
26	0,058	0,058	**72**	-1,610	-1,610	**118**	-0,760	-0,733	**164**	-0,700	-0,686
27	0,021	0,021	**73**	-1,646	-1,646	**119**	-0,730	-0,730	**165**	-0,700	-0,686
28	-0,015	-0,015	**74**	-1,683	-1,683	**120**	-0,700	-0,686	**166**	-0,700	-0,686
29	-0,051	-0,051	**75**	-1,719	-1,719	**121**	-0,700	-0,686	**167**	-0,700	-0,686
30	-0,087	-0,087	**76**	-1,755	-1,755	**122**	-0,700	-0,686	**168**	-0,700	-0,686
31	-0,124	-0,124	**77**	-1,791	-1,791	**123**	-0,700	-0,686	**169**	-0,700	-0,686
32	-0,160	-0,160	**78**	-1,828	-1,828	**124**	-0,700	-0,686	**170**	-0,700	-0,686
33	-0,196	-0,196	**79**	-1,864	-1,864	**125**	-0,700	-0,686	**171**	-0,700	-0,686
34	-0,233	-0,233	**80**	-1,900	-1,900	**126**	-0,700	-0,686	**172**	-0,700	-0,686
35	-0,269	-0,269	**81**	-1,870	-1,809	**127**	-0,700	-0,686	**173**	-0,700	-0,686
36	-0,305	-0,305	**82**	-1,840	-1,795	**128**	-0,700	-0,686	**174**	-0,700	-0,686
37	-0,341	-0,341	**83**	-1,810	-1,806	**129**	-0,700	-0,686	**175**	-0,700	-0,686
38	-0,378	-0,378	**84**	-1,780	-1,712	**130**	-0,700	-0,686	**176**	-0,700	-0,686
39	-0,414	-0,414	**85**	-1,750	-1,724	**131**	-0,700	-0,686	**177**	-0,700	-0,686
40	-0,450	-0,450	**86**	-1,720	-1,706	**132**	-0,700	-0,686	**178**	-0,700	-0,686
41	-0,486	-0,486	**87**	-1,690	-1,622	**133**	-0,700	-0,686	**179**	-0,700	-0,686
42	-0,523	-0,523	**88**	-1,660	-1,649	**134**	-0,700	-0,686	**180**	-0,700	-0,686
43	-0,559	-0,559	**89**	-1,630	-1,603	**135**	-0,700	-0,686			
44	-0,595	-0,595	**90**	-1,600	-1,540	**136**	-0,700	-0,686			
45	-0,631	-0,631	**91**	-1,570	-1,568	**137**	-0,700	-0,686			

External Wind Pressure for changing height and the angle,

	We (kPa)		
Height (m)	**0°**	**80°**	**120° – 180°**
1	0,225	-0,427	-0,157
2	0,384	-0,729	-0,269
3	0,487	-0,925	-0,341
4	0,565	-1,073	-0,395
5	0,628	-1,192	-0,439
6	0,681	-1,293	-0,476
7	0,727	-1,381	-0,509
8	0,768	-1,458	-0,537
9	1,329	-2,526	-0,931
10	1,384	-2,630	-0,969
11	1,435	-2,726	-0,984
12	1,482	-2,815	-1,016
13	1,525	-2,898	-1,046

14	1,566	-2,975	-1,074
15	1,604	-3,048	-1,100
16	1,640	-3,116	-1,125
17	1,674	-3,181	-1,149
18	1,707	-3,243	-1,171
19	1,738	-3,302	-1,192
20	1,767	-3,358	-1,213
21	2,444	-4,644	-1,677
22	2,481	-4,714	-1,702
23	2,517	-4,782	-1,727
24	2,551	-4,847	-1,750
25	2,584	-4,910	-1,773
25,75	2,608	-4,955	-1,789

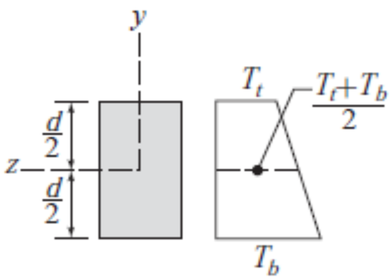
THERMAL ACTIONS

Temperature Difference Loads

Referring to Euro code 1991-1-5, effects arising from interaction between the structure and its contents during thermal changes

From structural analysis point of view, effects of thermal gradient on a beam can be calculated from following formulae;

$$M_T = EI * \alpha * \frac{T_b - T_t}{d}$$



Where;

T_t = Inside temperature

T_b = Outside temperature

d = depth of beam

α = coefficient of thermal conductivity of concrete

E = Elastic Section Modulus

I = Moment of Inertia of cross section (z-z) direction

By taking a temperature difference of 30^0 C

M_t = ± 140 kNm for (0,4 m wall) M_t = ± 315 kNm for (0,6 m wall)

Another approach given in A.Ghalis book, that takes the concrete property and the boundary conditions of the walls into account. Usual approximations in the theory of thin shells are considered valid. Moreover, the temperature variation (or shrinkage of concrete) is supposed to be constant along the height of the wall, symmetric to the axis of rotation and linear over the wall thickness. Vertical axis of rotation (symmetry) and constant wall thickness (stiffness) of the tank are also assumed. The elongation or shortening of the wall in the axial direction can be free to occur. The material of the tank is supposed to be linearly elastic. This approach strictly applies only before cracking of concrete.Assume that, comparing to a starting condition, the temperature variation of a circular cylindrical shell through the wall thickness can be expressed by

$$T(z) = \frac{t_i + t_0}{2} - \frac{t_i - t_0}{2} z = t - \Delta t z$$

where t_i and t_0 are the temperature rises of the inner and outer faces of the wall (*Fig. 1*), while $t = (t_i + t_0)/2$ and $\Delta t = (t_i - t_0)/2$ are characteristic values regarding the uniform and the linearly varying distribution of the temperature variation over the thickness of the wall, respectively.

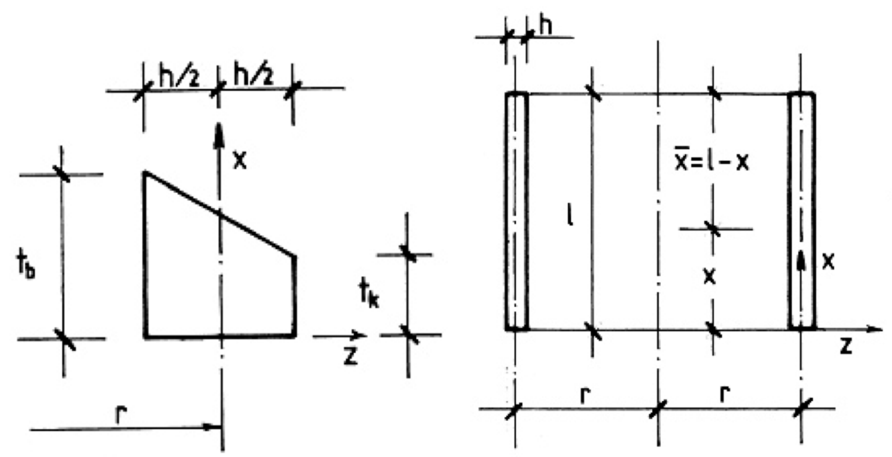


Fig. 1. Distribution of temperature variation over the wall thickness

Fig. 2. Geometrical data and coordinate system of analysis

$$t = \frac{(0+30)}{2} = 15 \qquad \Delta t = \frac{(0-30)}{2} = -15$$

$$\beta = \sqrt[4]{\frac{3(1-v^2)}{r^2 * h^2}} = \sqrt[4]{\frac{3(1-0.2^2)}{7.8^2 * 0.48^2}} = 0{,}673$$

$$\xi_1 = e^{-\beta x}\cos(\beta x) \qquad \xi_2 = e^{-\beta x}\sin(\beta x) \qquad \xi_3 = \xi_1 + \xi_2 \qquad \xi_4 = \xi_1 - \xi_2$$

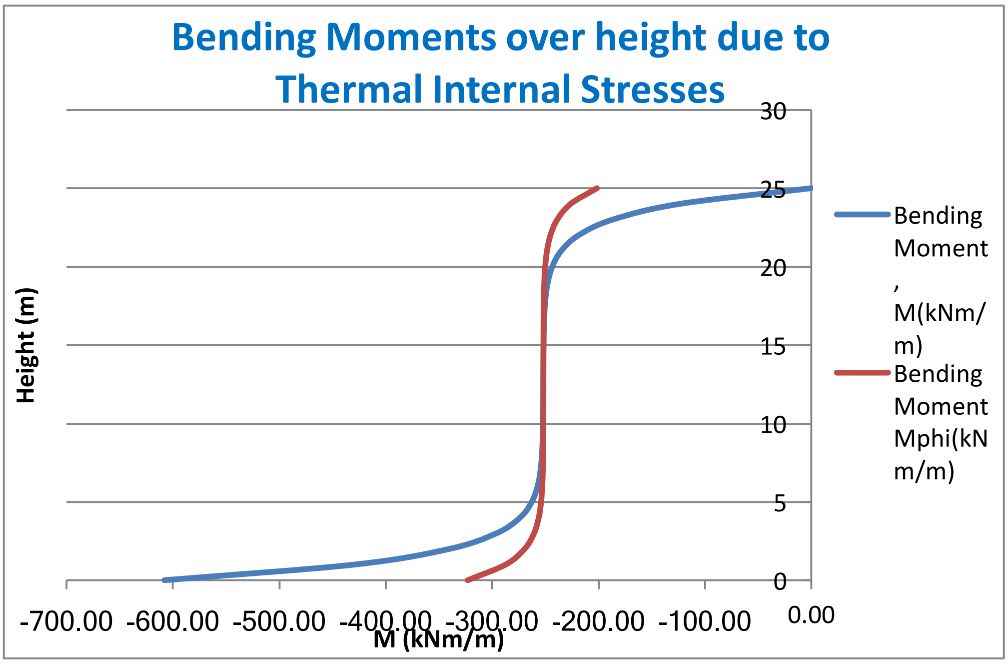
$$N = -E\alpha\left(ht\xi_3 - \frac{1+\vartheta}{r\beta^2}\Delta t\overline{\xi_4}\right)$$

$$M = \frac{Eh^2}{6}\alpha\left(\frac{rh\beta^2}{1-\vartheta^2}t\xi_4 - \frac{1}{1-\vartheta}\Delta t\left(1-\overline{\xi_3}\right)\right)$$

$$M_\emptyset = \frac{Eh^2}{6}\alpha\left(\vartheta\frac{rh\beta^2}{1-\vartheta^2}t\xi_4 - \frac{1}{1-\vartheta}\Delta t\left(1-\vartheta\overline{\xi_3}\right)\right)$$

x	**βx**	**ξ_3**	**ξ_2**	**ξ_4**	**ξ_1**	**βx**	**$\bar{\xi}_3$**	**$\bar{\xi}_2$**	**$\bar{\xi}_4$**	**$\bar{\xi}_1$**
0	0,000	1,000	0,000	1,000	1,000	17,165	0,000	0,000	0,000	0,000
1	0,687	0,509	0,006	0,497	0,503	16,478	0,000	0,000	0,000	0,000
2	1,373	0,259	0,006	0,247	0,253	15,792	0,000	0,000	0,000	0,000
3	2,060	0,132	0,005	0,123	0,127	15,105	0,000	0,000	0,000	0,000
4	2,746	0,067	0,003	0,061	0,064	14,418	0,000	0,000	0,000	0,000
5	3,433	0,034	0,002	0,030	0,032	13,732	0,000	0,000	0,000	0,000
6	4,120	0,017	0,001	0,015	0,016	13,045	0,000	0,000	0,000	0,000
7	4,806	0,009	0,001	0,007	0,008	12,359	0,000	0,000	0,000	0,000
8	5,493	0,004	0,000	0,004	0,004	11,672	0,000	0,000	0,000	0,000
9	6,179	0,002	0,000	0,002	0,002	10,985	0,000	0,000	0,000	0,000
10	6,866	0,001	0,000	0,001	0,001	10,299	0,000	0,000	0,000	0,000
11	7,552	0,001	0,000	0,000	0,001	9,612	0,000	0,000	0,000	0,000
12	8,239	0,000	0,000	0,000	0,000	8,926	0,000	0,000	0,000	0,000
13	8,926	0,000	0,000	0,000	0,000	8,239	0,000	0,000	0,000	0,000
14	9,612	0,000	0,000	0,000	0,000	7,552	0,000	0,000	0,000	0,000
15	10,299	0,000	0,000	0,000	0,000	6,866	0,001	0,000	0,001	0,001
16	10,985	0,000	0,000	0,000	0,000	6,179	0,002	0,000	0,001	0,002
17	11,672	0,000	0,000	0,000	0,000	5,493	0,004	0,000	0,003	0,004
18	12,359	0,000	0,000	0,000	0,000	4,806	0,008	0,001	0,007	0,008
19	13,045	0,000	0,000	0,000	0,000	4,120	0,017	0,001	0,014	0,016
20	13,732	0,000	0,000	0,000	0,000	3,433	0,034	0,002	0,030	0,032
21	14,418	0,000	0,000	0,000	0,000	2,746	0,067	0,003	0,061	0,064
22	15,105	0,000	0,000	0,000	0,000	2,060	0,132	0,005	0,123	0,127
23	15,792	0,000	0,000	0,000	0,000	1,373	0,259	0,006	0,247	0,253
24	16,478	0,000	0,000	0,000	0,000	0,687	0,509	0,006	0,497	0,503
25	17,165	0,000	0,000	0,000	0,000	0,000	1,000	0,000	1,000	1,000

x	Mϕ	M	N
0	-323,28	-608,38	-2520,00
1	-287,44	-429,20	-1283,40
2	-269,62	-340,08	-653,43
3	-260,75	-295,77	-332,60
4	-256,35	-273,74	-169,25
5	-254,16	-262,80	-86,10
6	-253,07	-257,36	-43,79
7	-252,53	-254,66	-22,26
8	-252,26	-253,32	-11,31
9	-252,13	-252,66	-5,73
10	-252,07	-252,33	-2,90
11	-252,03	-252,16	-1,46
12	-252,01	-252,07	-0,76
13	-252,00	-252,00	-0,49
14	-251,99	-251,93	-0,58
15	-251,96	-251,80	-1,14
16	-251,91	-251,54	-2,55
17	-251,80	-251,01	-5,67
18	-251,59	-249,93	-12,20
19	-251,16	-247,79	-25,62
20	-250,31	-243,55	-52,89
21	-248,64	-235,20	-107,86
22	-245,36	-218,81	-218,38
23	-238,93	-186,67	-440,30
24	-226,33	-123,66	-886,04
25	-201,60	0,00	-1781,91



Shrinkage, Creep and Heat of Hydration Effect

Early Age Shrinkage

Parameters used in Shrinkage Calculations

Age of concrete at time considered,		t = 7 days
Age of concrete at loading,		t_0 = 4 days
Age of concrete at start of drying		t = 1 days
Relative Humidity of environement		RH=60,3 %
Average Temperature		T=20 C
Type of Cement		=Class R
α_{ds1} = 6	α_{ds2} = 0,11	s = 0,2
Characteristic strenght of concrete		f_{ck}=40 Mpa
Mean compressive strenght		f_{cm}=48 MPa

$$f_{cm}(4) = f_{cm} * e^{0,2*(1-(\frac{28}{4})^{\frac{1}{2}})} = 48 * e^{0,2*(1-(\frac{28}{4})^{\frac{1}{2}})} = 31,809\ MPa$$

$$f_{cm0} = 10\ MPa$$

Drying shrinkage

$$\varepsilon_{cd}\,(t) = \beta_{ds}(t, t_s) * k_h * \varepsilon_{cd\,,0}$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0{,}04\sqrt{h_0^3}}$$

$$t - t_s = 7 - 1 = 6\ days$$

$$\beta_{RH} = 1{,}55 * (1 - (0{,}60)^3) = 1{,}2152$$

$$\varepsilon_{cd,0} = 0{,}85 * ((220 + 110 * \alpha_{ds1}) * \left(e^{\left(-\alpha_{ds2} * \frac{fcm}{fcm0}\right)}\right) * 10^{-6} * \beta_{RH}$$

$$\varepsilon_{cd,0} = 0{,}85 * ((220 + 110 * 6) * \left(e^{\left(-0{,}11 * \frac{48}{10}\right)}\right) * 10^{-6} * 1{,}2152 = 536{,}1 * 10^{-6}$$

For t_w = 60 cm , h_0 = 600 mm , k_h = 0,7

$$\beta_{ds}(7{,}1) = \frac{6}{6 + 0{,}04\sqrt{600^3}} = 10{,}10 * 10^{-3}$$

$$\varepsilon_{cd}\,(7) = 10{,}10 * 10^{-3} * 0{,}7 * 536{,}1 * 10^{-6} = 3{,}79 * 10^{-6}$$

For t_w = 40 cm, h_0 = 400 mm , k_h = 0,725

$$\beta_{ds}(7{,}1) = \frac{6}{6 + 0{,}04\sqrt{400^3}} = 18{,}40 * 10^{-3}$$

$$\varepsilon_{cd}\,(7) = 18{,}40 * 10^{-3} * 0{,}725 * 536{,}1 * 10^{-6} = 7{,}151 * 10^{-6}$$

Autogenous shrinkage

$$\varepsilon_{ca}\,(t) = \beta_{as}(t) * \varepsilon_{ca}(\infty)$$

$$\beta_{as}(7) = 1 - e^{-0{,}2*7^{0{,}5}} = 410{,}89 * 10^{-3}$$

$$\varepsilon_{ca}(\infty) = 75 * 10^{-6}$$

$$\varepsilon_{ca}\,(t) = 410{,}89 * 10^{-3} * 75 * 10^{-6} = 30{,}8171 * 10^{-6}$$

Total Shrinkage

$$\varepsilon_{cs} = 30{,}8171 * 10^{-6} + 3{,}79 * 10^{-6} = 34{,}607 * 10^{-6}$$

$$\varepsilon_{cs} = 30{,}8171 * 10^{-6} + 7{,}151 * 10^{-6} = 37{,}9681 * 10^{-6}$$

$$\Delta T = \frac{\varepsilon_{cs}}{\alpha} = 34{,}607 * \frac{10^{-6}}{10^{-5}} = 3{,}46°\mathrm{C}$$

$$\Delta T = \frac{\varepsilon_{cs}}{\alpha} = 37{,}9681 * \frac{10^{-6}}{10^{-5}} = 3{,}8°\mathrm{C}$$

When compared to with the temperature difference due to heat of hydration shrinkage has no significant effect. That is there will not be any additional reinforcement due to shrinkage.

Heat of Hydration Effects

Effect of hydration of cement can be calculated by the formulae

$$\varepsilon_T = \alpha\,\Delta T$$

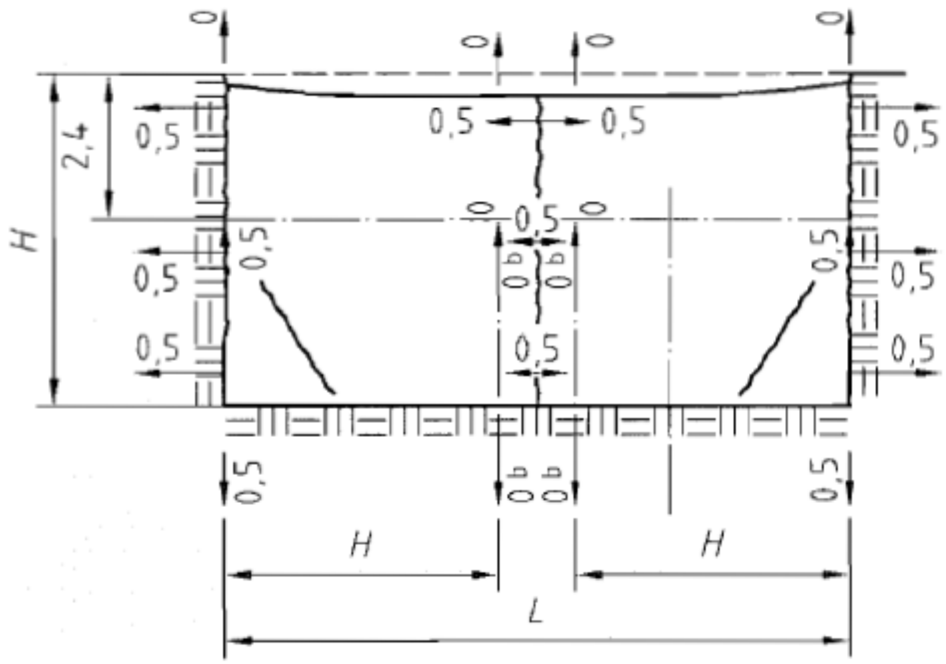
$$\varepsilon_T = 60 * 10^{-5} = 600 * 10^{-6}$$

$$\varepsilon_{Tot} = 634{,}607 * 10^{-6}$$

$$\varepsilon_{Tot} = 637{,}9681 * 10^{-6}$$

Restraint Effect

Since the whole concrete of the tank can not be poured at once, there will be restraint effect on thermal deformations due to the preceeding layer of concrete. This restraint effects decreases the imposed thermal deformations.



It is given in EC1992-3-3 as following formulae

$$\varepsilon_{az} = (1 - R_{ax})\varepsilon_{iav}$$

By taking as 0,5

$$\varepsilon_{az,Tot} = 317{,}3035 * 10^{-6}$$
$$\varepsilon_{az,Tot} = 318{,}9841 * 10^{-6}$$
$$\sigma_{az,Tot} = 63{,}46 MPa$$
$$\sigma_{az,Tot} = 63{,}80\ MPa$$

EARTHQUAKE

Geometrical Features of the Tank		
H_L	Design Height of Liquid	24,5 m
H_w	Circular Tank Wall Height	25 m
D	Internal Diameter of Tank	15 m
r	Internal Radius of Tank	7,5 m
t_w	Average Wall Thickness	0,48 m

Dynamic lateral forces

$\boldsymbol{W_e}$ = effective dynamic mass of the tank structure (walls and roof) ($\boldsymbol{W_e}$ = ($\mathbf{e}\boldsymbol{W_w}$ + $\boldsymbol{W_r}$)) (kN)

$\boldsymbol{W_w}$ = in a rectangular tank, the mass of one wall perpendicular to the direction of the earthquake force (kN)

$\boldsymbol{W_r}$ = mass of the tank roof, plus superimposed load, plus applicable portion of snow load considered as dead load (kN)

Ɛ = effective mass coefficient (ratio of equivalent dynamic mass of the tank shell to its actual total mass).

$$\boldsymbol{W_w} = \pi(8{,}1^2 - 7{,}5^2) * 10 * 24 + \pi(7{,}9^2 - 7{,}5^2) * 15 * 24 = 14024 \text{ kN}$$

$$\boldsymbol{W_r} = 50000 * \frac{9{,}81}{1000} + 0{,}544 * \pi * 8{,}3^2 = 608 \text{ kN}$$

$$\boldsymbol{\varepsilon} = \left[0{,}0151 * \left(\frac{15}{24{,}5}\right)^2 - 0{,}1908 * \left(\frac{15}{24{,}5}\right) + 1{,}021\right] = 0{,}9098 < 1.0 \text{ OK}$$

$$\boldsymbol{W_e} = 0{,}9098 * 14024 + 608 = 13368 \text{ kN}$$

$$\frac{\boldsymbol{D}}{\boldsymbol{H_L}} = \frac{15}{24{,}5} = 0{,}612$$

$$\frac{\boldsymbol{W_i}}{\boldsymbol{W_L}} = \frac{\tanh(0{,}866 * 0{,}612)}{0{,}866 * 0{,}612} = 0{,}9158$$

$$\frac{\boldsymbol{W_c}}{\boldsymbol{W_L}} = 0{,}230 * 0{,}612 * \tanh\left(\frac{3{,}68}{0{,}612}\right) = 0{,}1408$$

$\boldsymbol{W_L}$ = total mass of the stored liquid, (kN)

$$\boldsymbol{W_L} = \pi * 7{,}5^2 * 24{,}5 * 9{,}81 = 42473 \text{ kN}$$

$$\boldsymbol{W_i} = 42473 * 0{,}9158 = 38896 \text{ kN}$$

$$\boldsymbol{W_C} = 42473 * 0{,}1408 = 5980 \text{ kN}$$

the combined natural frequency of vibration, $\boldsymbol{\omega_i}$

$$\boldsymbol{\omega_i} = \frac{C_I}{\boldsymbol{H_L}}\sqrt{\frac{10^3 E_c}{\rho_c}}$$

C_I, C_w = coefficients for determining the fundamental frequency of the tank-liquid system.

H_L = design depth of stored liquid, (m)

E_c = modulus of elasticity of concrete (MPa)

ρc = mass density of concrete (2.40 kN.s2/m4) for standard-weight concrete

$$C_I = C_w\sqrt{\frac{t_w}{10R}}$$

t_w **:** average wall thickness (mm)

R = inside radius of circular tank,(m)

$$C_I = 0{,}143\sqrt{\frac{480}{75}} = 0{,}3618$$

$$\boldsymbol{\omega_i} = \frac{0{,}3618}{\mathbf{24,5}}\sqrt{\frac{10^3 35000}{2{,}4}} = 56{,}4\ rad/s$$

$$\boldsymbol{T_i} = \frac{2\pi}{\boldsymbol{\omega_i}} = 0{,}1114\ s$$

the frequency of the vibration $\boldsymbol{\omega_c}$,

$$\boldsymbol{\omega_c} = \frac{\lambda}{\sqrt{D}}$$

$$\lambda = \sqrt{3{,}68 * 9{,}81 * \tanh\left(\frac{3{,}68}{0.612}\right)} = 6{,}008$$

$$\boldsymbol{\omega_c} = \frac{6{,}008}{\sqrt{15}} = 1{,}5513$$

$$\boldsymbol{T_c} = \frac{2\pi}{\boldsymbol{\omega_c}} = 4{,}05\ s$$

Table of Seismic Coefficients Values		
C_i	Period-dependent spectral amplification factor for the horizontal motion of the impulsive component	2,29
C_c	Period-dependent spectral amplification factor for the horizontal motion of the convective component	0,37
C_v	Period-dependent spectral amplification factor for vertical motion of the contained liquid	2,29
C_l	Coefficient for determining the fundamental frequency of the tank-liquid system	0,36
C_w	Coefficient for determining the fundamental frequency of the tank-liquid system	0,14
ω_i	Circular frequency of the impulsive mode of vibration, rad/s	56,40
ω_c	Circular frequency of the convective mode of vibration, rad/s	1,55
T_i	Fundamental period of oscillation of the tank (plus the impulsive component of the contents), s	0,11
T_c	Natural period of the first (convective) mode of sloshing, s	4,05
T_v	Natural period of vibration of vertical liquid motion, s	0,10
S	Site profile coefficient representing the soil characteristics as they pertain to the structure	1,20

Importance factor I

Tanks that are intended to remain usable for emergency purposes after an earthquake, or tanks that are part of lifeline systems.

I = 1,25

Soil profile coefficient S

TYPE B: A soil profile with predominantly medium-dense to dense or medium-stiff to stiff soil conditions, where the soil depth exceeds (60 960 mm).

S= 1,2

Spectral amplification factors Ci and Cc

For ***Ti*** **≤** 0.31 s,

$$C_i = \frac{2{,}75}{S} = \frac{2{,}75}{1{,}2} = 2{,}2917$$

For ***Tc*>** 2.4 s,

$$C_c = \frac{6{,}0}{{T_c}^2} = \frac{6{,}0}{4{,}05^2} = 0{,}3658$$

Seismic zone factor Z

Ankara has an effective peak acceleration (EPA) corresponding to a ground motion of 2.4 m/s^2.

According to ACI categorization

Zone 3 = 0.3 g = 2.943 m/s^2.

Zone 2B=0.2 g = 1.962 m/s^2.

Thus Ankara can be put in the zone 3 which has a seismic zone factor of **Z=0.3**.

Response modification factor Rw

Fixed- or hinged-base tanks

R_{wi}= 2.75

R_c= 1.0

Seismic Design Parameters		
I	Importance factor	1,25
R_{wi}	Response modification factor - Impulsive component	2,75
R_{wc}	Response modification factor - Convective component	1,00

$$P_w = ZSIC_i \frac{\varepsilon W_w}{R_{wi}} = 0,3 * 1,2 * 1,25 * 2,2917 * \frac{0,9098 * 14024}{2,75} = 4785\ kN$$

$$P_r = ZSIC_i \frac{W_r}{R_{wi}} = 0,3 * 1,2 * 1,25 * 2,2917 * \frac{608}{2,75} = 228\ kN$$

$$P_i = ZSIC_i \frac{W_i}{R_{wi}} = 0,3 * 1,2 * 1,25 * 2,2917 * \frac{38896}{2,75} = 14586\ kN$$

$$P_c = ZSIC_c \frac{W_c}{R_{wc}} = 0,3 * 1,2 * 1,25 * 0,3658 * \frac{5980}{1,0} = 985\ kN$$

Total base shear

$$\mathrm{V} = \sqrt{(P_w + P_r + P_i)^2 + P_c^2} = 19624\ kN$$

Dynamic Properties of Water Tank		
W_W	Mass of the tank wall (shell),(kN)	13777
W_L	Total mass of the stored liquid,(kN)	42460
W_i	Equivalent mass of the impulsive component of the stored liquid,(kN)	38883
W_C	Equivalent mass of the convective component of the stored liquid, (kN)	5979
ε	Effective mass coefficient	0,91
h_i	Height above the base of the wall to the center of gravity of the impulsive lateral force, (m)	10,84
h_C	Height above the base of the wall to the center of gravity of the convective lateral force, (m)	20,45
P_i	Total lateral impulsive force , (kN)	14581
P_C	Total lateral convective force, (kN)	985

Pressure distribution

Shear transfer

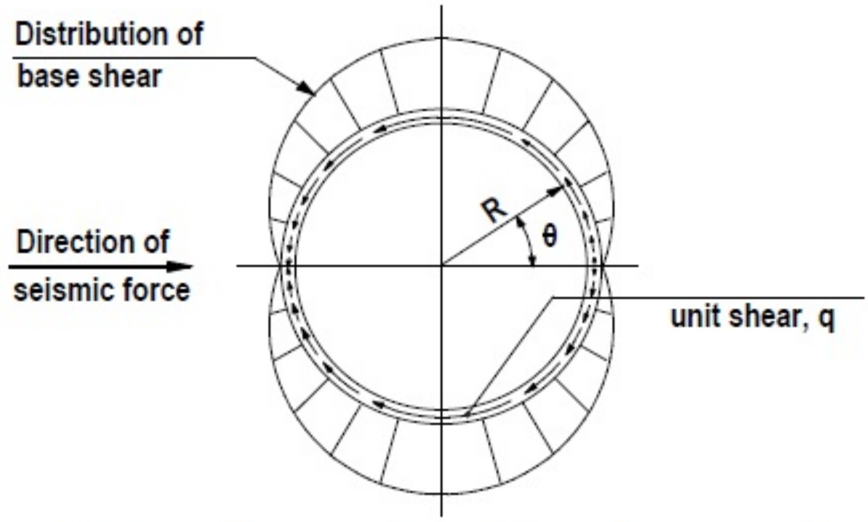
The horizontal earthquake force ***V*** generates shear forces between the wall and footing, and the wall and roof.

Circular tanks

The wall-to-footing and wall-to-roof joints shall be designed for the earthquake shear forces.

In fixed- and hinged-base circular tanks (Types 2.1 and 2.2), the earthquake base shear is transmitted partially by membrane (tangential) shear and the rest by radial shear that causes vertical bending. For a tank with a height-to-diameter ratio of 1:4 (***D/HL* = 4.0**), approximately 20% of the earthquake shear force is transmitted by the radial base reaction to vertical bending. The remaining 80% is transmitted by tangential shear transfer ***Q***. To transmit this tangential shear, ***Q***, a distributed shear force, ***q***, is required at the wall/footing interface, where

$$q = \frac{Q}{\pi R} sin\theta$$

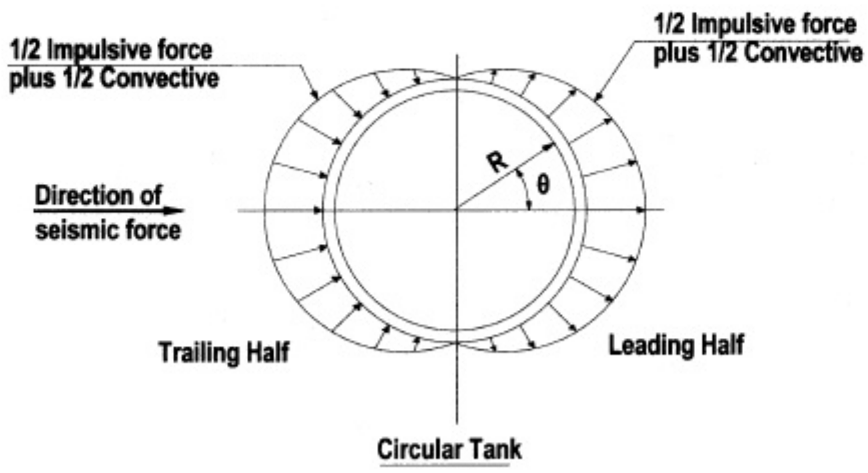


-Membrane shear transfer at the base of circular tanks

The maximum tangential shear occurs at a point on the tank wall oriented 90 degrees to the design earthquake direction being evaluated, and is given by

$$q_{max} = \frac{Q}{\pi R} = \frac{0.8V}{\pi R}$$

The radial shear is created by the flexural response of the wall near the base, and is therefore proportional to the hydrodynamic forces shown in figure below. The radial shear attains its maximum value at points on the tank wall oriented 0 and 180 degrees to the ground motion and should be determined using cylindrical shell theory and the tank dimensions. The design of the wall-footing interface should take the radial shear into account. In general, the wall-footing interface should have reinforcement designed to transmit these shears through the joint. Alternatively, the wall may be located in a preformed slot in the ring beam footing.



-Hydrodynamic pressure distribution in tank walls

Dynamic force distribution above base

Circular tanks

The cylindrical walls of circular tanks shall be loaded by

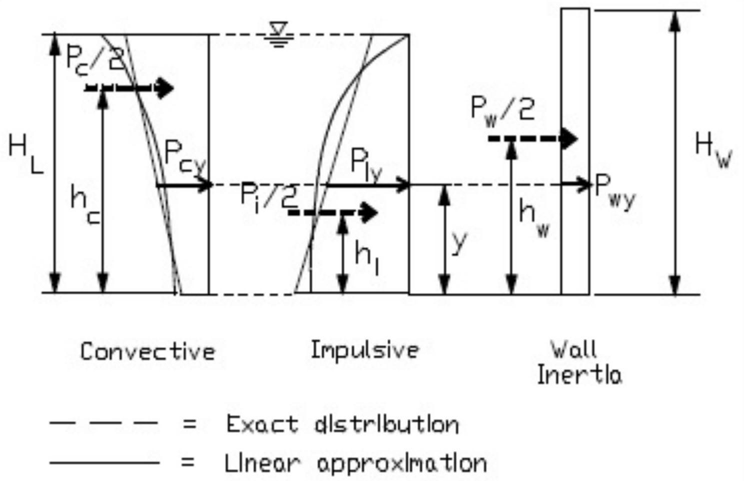
(a) the wall's own inertia force distributed uniformly around the entire circumference;

(b) one-half the impulsive force, P_i applied symmetrical about **Ɵ = 0** and acting outward on one half of the wall, and one-half P_i symmetrically about **Ɵ =** π and acting inward on the opposite half of the wall;

(c) one-half the convective force, P_c, acting on one-half of the wall symmetrical about **Ɵ = 0** and one-half P_c symmetrical about **Ɵ =** π and acting inward on the opposite half of the wall; and

(d) the dynamic earth and ground water pressure against the trailing half of the buried portion of the wall. Superimposed on these lateral unbalanced forces shall be the axisymmetric lateral hydrodynamic force resulting from the hydrodynamic pressure p_{hy} acting on the tank wall.

The vertical distribution, per foot of wall height, of the dynamic forces acting on one half of the wall may be assumed as shown below



$P_{wy} = \dfrac{P_w}{2H_w}$ (for constant-thickness wall. For tapered wall, modify accordingly)

$$P_{iy} = \frac{\frac{P_i}{2}\left[4H_L - 6h_i - (6H_L - 12h_i) \times \left(\frac{y}{H_L}\right)\right]}{H_L^2}$$

$$P_{cy} = \frac{\frac{P_c}{2}\left[4H_L - 6h_c - (6H_L - 12h_c) \times \left(\frac{y}{H_L}\right)\right]}{H_L^2}$$

–Vertical force distribution: circular tanks.

The horizontal distribution of the dynamic pressure across the tank diameter ***D*** may be assumed as follows:

$$\mathrm{p}_{wy} = \frac{P_{wy}}{\pi R} \qquad \mathrm{p}_{cy} = \frac{16P_{cy}}{9\pi R}cos\theta$$

$$\mathrm{p}_{iy} = \frac{2P_{iy}}{\pi R}cos\theta \qquad \mathrm{p}_{hy} = \ddot{u}_v q_{hy}$$

$\boldsymbol{P_{wy}}$ = lateral inertia force due to $\boldsymbol{W_w}$, per unit height of the tank wall, occurring at level $\boldsymbol{y}$ above the tank base(kN/m)

$$P_{wy} = \frac{P_w}{2H_w} = \frac{4785}{50} = 95.7\ kN/m$$

$\boldsymbol{h_c}$ (EBP), $\boldsymbol{h_{c'}}$ (IBP)= height above the base of the wall to the center of gravity of the convective lateral force, (m)

h_i(EBP),***$h_{i'}$*** (IBP)= height above the base of the wall to the center of gravity of the impulsive lateral force, (m)

For tanks with $\frac{D}{H_L} < 1.333$,

$$\frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{D}{H_L}\right)$$

For all tanks,

$$\frac{h_c}{H_L} = 1 - \frac{\cosh\left[3.68\left(\frac{H_L}{D}\right)\right] - 1}{3.68\left(\frac{H_L}{D}\right) \times \sinh\left[3.68\left(\frac{H_L}{D}\right)\right]}$$

$$\frac{\mathrm{D}}{\mathrm{H_L}} = \frac{15}{24{,}5} = 0{,}612$$

$$\frac{\mathrm{h_i}}{\mathrm{H_L}} = 0{,}5 - 0{,}09375 * 0{,}612 = 0{,}4426 \rightarrow \mathrm{h_i} = 10{,}8443 \text{ m}$$

$$\frac{\mathrm{h_c}}{\mathrm{H_L}} = 1 - \frac{\cosh\left[\frac{3{,}68}{0{,}612}\right] - 1}{\frac{3{,}68}{0{,}612}\mathrm{xsinh}\left[\frac{3{,}68}{0{,}612}\right]} = 0{,}8345 \ \rightarrow \mathrm{h_c} = 20{,}4454 \text{ m}$$

P_{cy} = lateral convective force due to ***W_c***, per unit height of the tank wall, occurring at liquid level ***y***,(kN/m)

$$P_{cy} = \frac{\frac{P_c}{2}\left[4H_L - 6h_c - (6H_L - 12h_c)\frac{y}{H_L}\right]}{H_L^2}$$

At bottom of tank y = 0 m

$$P_{cy} = \frac{\frac{985}{2}[4 * 24{,}5 - 6 * 20{,}4454]}{24{,}5^2} = -20{,}24\ kN/m$$

$$\mathrm{p}_{cy} = -1.5274 * cos\theta\ kN/m^2$$

At bottom of top = 25 m

$$P_{cy} = \frac{\frac{985}{2}\left[4 * 24{,}5 - 6 * 20{,}4454 - \frac{(6 * 24{,}5 - 12 * 20{,}4454) * 25}{24{,}5}\right]}{24{,}5^2} = -62{,}09\ kN/m$$

$$\mathrm{p}_{cy} = 4{,}6851 * cos\theta\ kN/m^2$$

P_{iy} = lateral impulsive force due to ***W_i***, per unit height of the tank wall, occurring at liquid level ***y***,(kN/m)

$$P_{iy} = \frac{\frac{P_i}{2}\left[4H_L - 6h_i - (6H_L - 12h_i)\frac{y}{H_L}\right]}{H_L^2}$$

At bottom of tank y = 0 m

$$P_{iy} = \frac{\frac{14586}{2}[4 * 24,5 - 6 * 10,8443]}{24,5^2} = 400.15\ kN/m$$

$$\text{p}_{iy} = \frac{2P_{iy}}{\pi R}cos\theta = 33,9657 * cos\theta\ kN/m^2$$

At bottom of top = 25 m

$$P_{iy} = \frac{\frac{14586}{2}\left[4 * 24,5 - 6 * 10,8443 - \frac{(6 * 24,5 - 12 * 10,8443) * 25}{24,5}\right]}{24,5^2} = 191,02\ kN/m$$

$$\text{p}_{iy} = 16,2139 * cos\theta\ kN/m^2$$

q_{hy} = unit hydrostatic pressure at liquid level ***y*** above the tank base [***q_{hy}*** =**γ_L**(***H_L* – *y***)] (kPa)

$\ddot{u}_v$ = effective spectral acceleration from an inelastic vertical response spectrum that is derived by scaling from an elastic horizontal response

$$\text{p}_{hy} = \ddot{\text{u}}_v q_{hy}$$

$$\ddot{\text{u}}_v = ZSC_vI\frac{b}{R_{wi}}$$

$$\text{T}_v = 2\pi\sqrt{\frac{\gamma_L D H_L^2}{2gt_wE_c}} = 2\pi\sqrt{\frac{9,81 * 15 * 24,5^2}{2 * 9,81 * 480 * 35000}} = 0,1029$$

$$\text{C}_v = \frac{1,25}{T_v^{\frac{2}{3}}} = 5,692 > 2,2917 \rightarrow \ \text{C}_v = 2,2917$$

b = ratio of vertical to horizontal design acceleration (2/3)

$$\ddot{\text{u}}_v = ZSC_vI\frac{b}{R_{wi}}$$

$$\ddot{\text{u}}_v = 0,3 * 1,2 * 2,2917 * 1,25 * \frac{0,667}{2,75} = 0,25$$

Unit hydrostatic pressure at bottom y = 0

$$\text{q}_{hy} = 9,81 * (24,5 - 0) = 240,345\ kPa$$

lateral hydrostatic force at Bottom

$$p_{hy} = 0{,}25 * 240{,}345 = 60{,}09\ kPa$$

lateral hydrostatic force at top

$$p_{hy} = 0$$

Height (m)	P_{cy}	P_{iy}	P_{hy}
0	-20,24	400,15	245,25
1	-16,95	391,78	235,44
2	-13,66	383,42	225,63
3	-10,36	375,05	215,82
4	-7,07	366,69	206,01
5	-3,78	358,32	196,2
6	-0,48	349,96	186,39
7	2,81	341,59	176,58
8	6,10	333,23	166,77
9	9,40	324,86	156,96
10	12,69	316,50	147,15
11	15,99	308,13	137,34
12	19,28	299,76	127,53
13	22,57	291,40	117,72
14	25,87	283,03	107,91
15	29,16	274,67	98,1
16	32,45	266,30	88,29
17	35,75	257,94	78,48
18	39,04	249,57	68,67
19	42,33	241,21	58,86
20	45,63	232,84	49,05
21	48,92	224,48	39,24
22	52,21	216,11	29,43
23	55,51	207,75	19,62
24	58,80	199,38	9,81
25	62,09	191,02	0

Overturning Moments

h_w	11,25
h_r	25,15
h_i	10,84
h_c	20,44

For tanks with $\frac{D}{H_L} < 0.75$: $\frac{h_i'}{H_L} = 0.45$

h_i'= 11,025 m

$$\frac{h_c'}{H_L} = 1 - \frac{\cosh\left[3.68\left(\frac{H_L}{D}\right)\right] - 2.01}{3.68\left(\frac{H_L}{D}\right) \times \sinh\left[3.68\left(\frac{H_L}{D}\right)\right]}$$

h_c' = 20,66 m

Bending moment on the entire tank cross section just above the base of the tank wall (EBP):

$$M_w = P_w \times h_w$$

$$M_r = P_r \times h_r$$

$$M_i = P_i \times h_i$$

$$M_c = P_c \times h_c$$

$$M_b = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

$$M_w = P_w * h_w = 4785 * 11{,}25 = 53831{,}25\ kNm$$
$$M_r = P_r * h_r = 228 * 25{,}15 = 5734{,}2\ kNm$$
$$M_i = P_i * h_i = 14586 * 10{,}8443 = 158175\ kNm$$
$$M_c = P_c * h_c = 985 * 20{,}4454 = 20138{,}7\ kNm$$

$$M_b = \sqrt{(M_w + M_r + M_i)^2 + M_c^2} = 218670\ kNm$$

Overturning moment at the base of the tank, including the tank bottom and supporting structure (IBP):

$$M_w = P_w \times h_w$$

$$M_r = P_r \times h_r$$

$$M_i' = P_i \times h_i'$$

$$M_c' = P_c \times h_c'$$

$$M_o = \sqrt{(M_i' + M_w + M_r)^2 + M_c'^2}$$

$$M_i' = P_i * h_i' = 14586 * 11{,}025 = 160810{,}65\ kNm$$

$$M_c' = P_c * h_c' = 985 * 20{,}66 = 20350{,}1\ kNm$$

$$M_o = \sqrt{(M_w + M_r + M_i')^2 + M_c'^2} = 221314\ kNm$$

POST TENSIONING OF WATER TANK

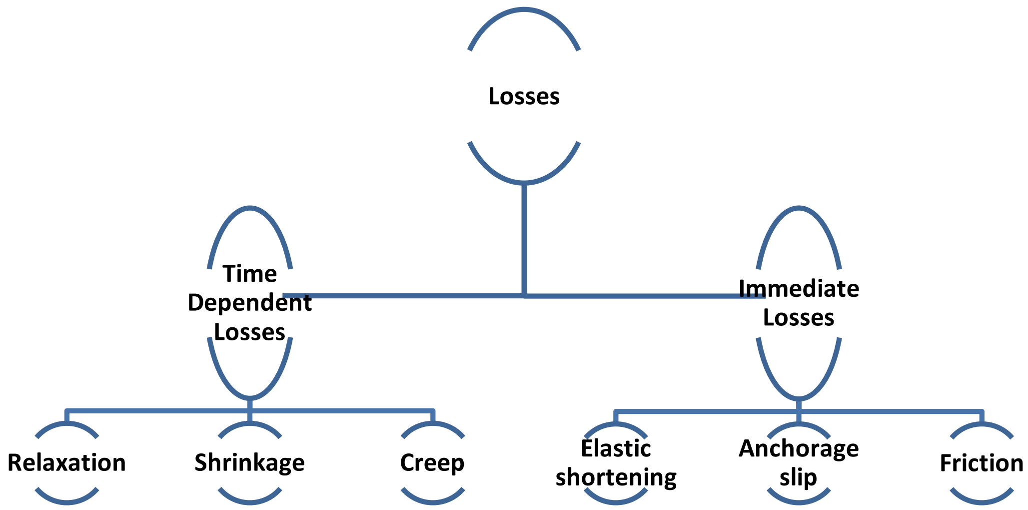
LOSSES IN WATER STORAGE TANK

In structural elements, provided strength of materials may reduce during their service life. Time dependent wearing of materials and the deleterious effects of atmosphere are the prominent reasons for those strength losses. In post tension applications, it was observed the initial force created by post – tension application reduces with time also. Since the initial post – tension stress will decrease in time, in order to overcome that decrease initial post – tension stress must be greater than the design stress at an amount of predicted decrease. That is;

$$\boldsymbol{\sigma_0 = \sigma_t + \Delta\sigma}$$

(**Δσ :** decrease in stress, **σ_t :** design stress , **σ_0 :** initial stress)

Since in post tension applications, there is various number of reasons that lead the decrease in strength, strength loss calculations should be done in more detailed. Those reasons are tabulated in a schematic form below.



Immediate Losses

Elastic Shortening

During tensioning the concrete is subjected to compression and a corresponding shortening. If there are several tendons which can not be tensioned at the same time, the force in tendons already tensioned will decrease each time another tendon is tensioned.

The average loss can be related to half the total prestress. The concrete shortening $c and the corresponding loss of prestress $\Delta\sigma_{cp}$ is then:

$$\varepsilon_c \approx 0{,}5P\,/\,(A_cE_c) = 0{,}5\sigma_c/E_c \qquad \Delta\sigma_p = E_s\cdot\varepsilon_c = 0{,}5(E_s/E_c)\cdot\sigma_c \approx 3\sigma_c$$

P **=** total prestressing force

A_c **=** concrete area,

σ_c= P/Ac (Taken as 10,5 MPa from Freyssinet Catalogue)

E_c **and** E_s = E-modulus of concrete and steel respectively (200 GPa and 35 GPa respectively)

$$\Delta\sigma_p = 0{,}5 * \left(\frac{200}{35}\right) * 10{,}5 = 30\ MPa$$

$$Percent\ Loss\ due\ to\ Elastic\ Shortening = \frac{30}{1488} * 100 \cong 2\%$$

Friction

The prestressing force decreases with increased distance from the active end due to friction.

$$\Delta P_{\mu}(x) = P_{max}[1 - e^{-\mu(\theta + kx)}]$$

where

θ is the sum of the anguler displacements over a distance x (**θ=s/r)**

μ is the coefficient of friction between the tendon and its duct

k is an unintentional angular displacement for internal tendons (per unit length)

x is the distance along the tendon from the point where the prestressing force is equal to P_{max} (the force at the active end during tensioning)

μ and k values are taken as μ=0.18 (1/rad) and k=0.005 (1/m). Angle change along the tendon length has taken as π/2, because in the design we are using 2 anchorage sections in one elevation. Jacking stresses are taken as 0,8 fpk according to the Freyssinet pre-stressing catalogue(P_{max} = 1860 *0,8 = 1488 MPa).

$$\theta = \frac{\left[\frac{2\pi r}{4}\right]}{r} = \frac{\pi}{2}$$

$$x = \left[\frac{2\pi 16{,}2}{4}\right] = 12{,}72\ m$$

$$\Delta P_{\mu}(12{,}72) = 1488 * \left[1 - e^{-0{,}18\left(\frac{\pi}{2} + 0{,}005 * 12{,}72\right)}\right] = 379{,}24\ MPa$$

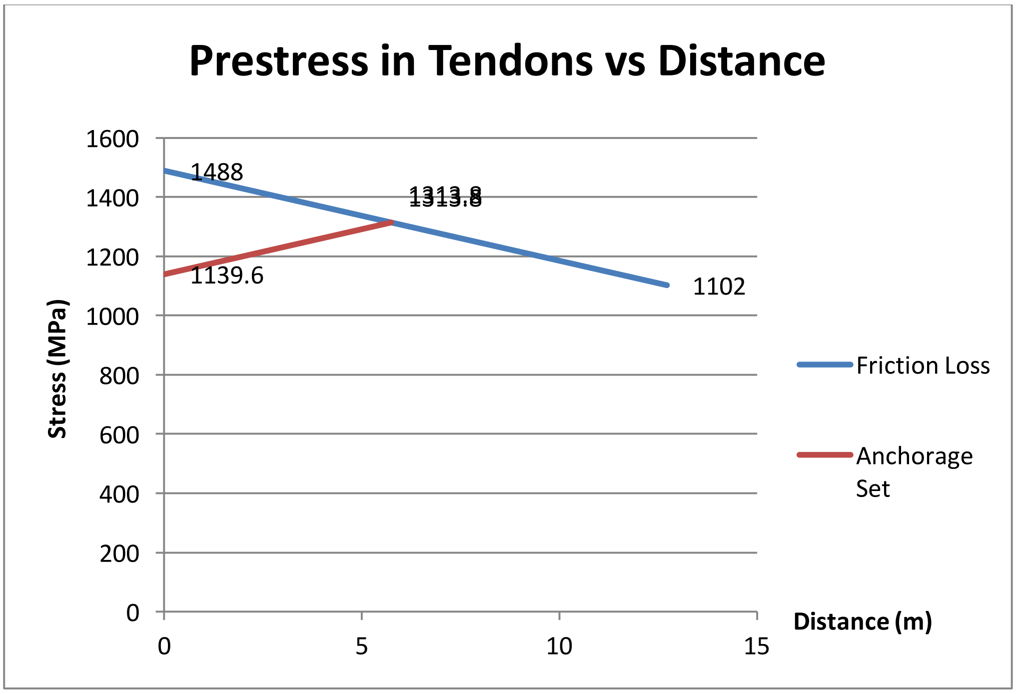
$$Percent\ Loss\ due\ to\ Friction = \frac{379{,}24}{1488} * 100 \cong 25{,}5\%$$

Anchorage Losses

Both bonded and un-bonded tendons are typically anchored with two-piece, conical wedges. When the tension applied by the jack is released, the strand retracts pulling the wedges into anchorage device and locks the strand in place. The loss in elongation is small. It depends on the wedges, the jack and the jacking procedure but is approximately 5 mm. This loss in elongation is resisted by friction just as the initial elongation is resisted by friction. To calculate pre-stress loss following formulae can be used.

$$x = \sqrt{\frac{0,005 * 200000}{386} 12,72} = 5,74m$$

$$\Delta\sigma_p = 2 * \frac{386}{12,72} * \sqrt{\frac{0,005 * 200000 * 12,72}{386}} = 348,4\ MPa$$



$$\sigma_{avg} = \frac{[\frac{(1139,58 + 1313,8)}{2} * 5,74 + \frac{(1313,8 + 1102)}{2} * 6,98]}{12,72} = 1216\ MPa$$

$$Percent\ Loss\ due\ to\ Friction\ and\ Anchorage\ Set = \frac{1488 - 1216}{1488} * 100 \cong 18,3\%$$

Time dependent Losses

The prestress will decrease with time due to shrinkage and creep in the concrete, plus relaxation of the tendons. Different expressions for the time-dependent loss can be found in codes. the basic expression is always

$$\Delta\sigma_p = E_S\varepsilon_{cs} + E_S\varphi\frac{\sigma_c}{E_c} + \chi\sigma_{sp}$$

Where

E_s and E_c; E- Modulus of steel and modulus of concrete respectively

ε_{cs} : shrinkage of concrete

φ : creep coefficient of concrete (creep = strain increase under constant stress)

σ_{cp} : concrete compressive stress at level of tendons for quasi-permanent load

χ : relative relaxation loss (relaxation = stress decrease under constant strain)

σ_{sp} : stress in tendons

The physical meaning of the basic expression is simple: the first two terms express the stress decrease due to concrete shortening from shrinkage and creep respectively, the third term expresses the stress decrease due to prestress steel relaxation, given by the coefficient)χ.

The concrete stress σ_{cp} should be evaluated for the quasi-permanent (long-term) load combination together with prestress.

$$\Delta\sigma_p = 200000 * 483{,}25 * 10^{-6} + 200000 * 0{,}00027945 + 0{,}03 * 1216 = 189\ MPa$$

$$Percent\ Loss\ due\ to\ time\ dependent\ losses = \frac{189}{1216} * 100 \cong 15{,}54\%$$

Summary

Friction and Anchorage Setting → 18,3 %

Elastic Shortening → 2 %

Time dependent Losses → 15,54 %

Total Losses = 34 %

$$Final\ Stress\ in\ tendons\ = 1488 * \left(\frac{100-34}{100}\right) = 982\ MPa$$

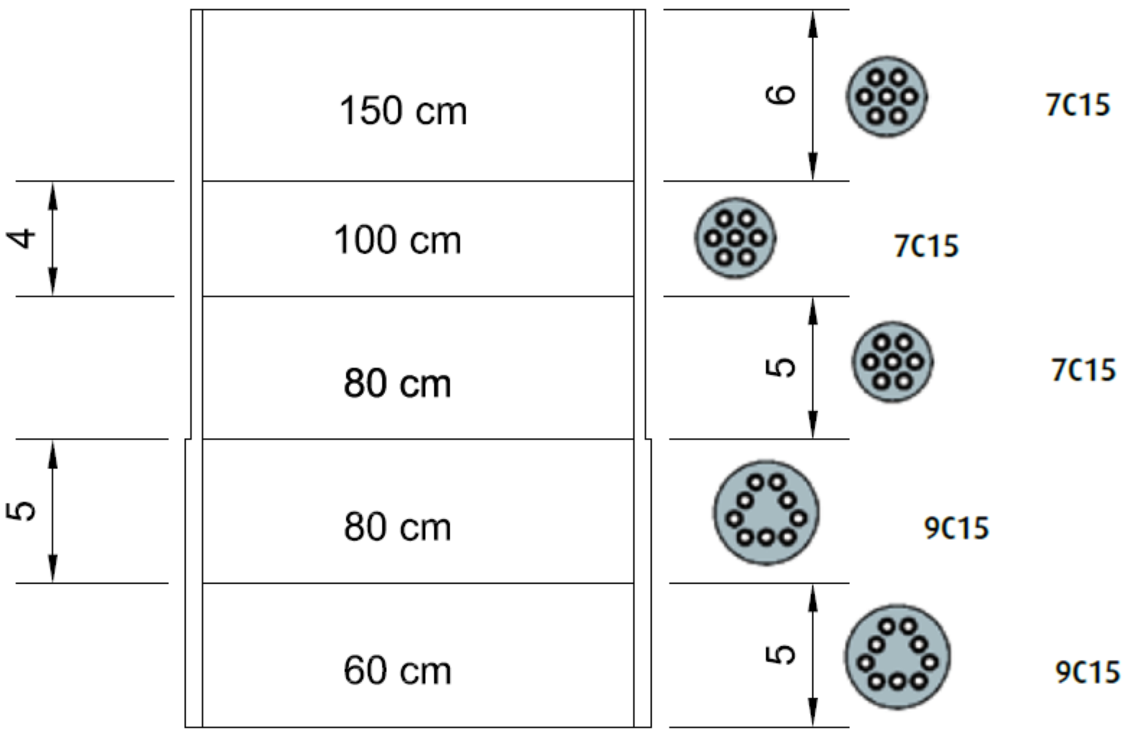
$\rightarrow$ *Close to value that we have taken into consideration in Preliminary Design Stage.*

POST TENSION TENDONS

We designed our post tension cables places and types according hoop stress distribution due to hydrostatic distributed force. To calculate spacing, horizontal pressure is multiplied with the radius to obtain tension force at each level. Then the minimum required amount of cable is computed for each level and spacing was adjusted according to these forces and losses of post - tension cables. Since there are losses due to elastic shortening, creep and relaxation also we predict that the total losses as approximately %50 to be conservative in this stage and decided design strength of post- tensioned cable as 982 MPa. Calculated hydrostatic pressures, hoop tension forces, required steel area and spacing are tabulated below.

Height (m)	V (m)	Hydrostatic Pressure (kN/m^2)	F (kN/m)	Required Reinforcement (cm^2 / m)	Max Spacing (m)	
25	0	0	0	-	-	
24	1	9,81	73,575	0,8175	12,84404	150 cm 7C15
23	2	19,62	147,15	1,635	6,422018	
22	3	29,43	220,725	2,4525	4,281346	
21	4	39,24	294,3	3,27	3,211009	
20	5	49,05	367,875	4,0875	2,568807	
19	6	58,86	441,45	4,905	2,140673	
18	7	68,67	515,025	5,7225	1,834862	100 cm 7C15
17	8	78,48	588,6	6,54	1,605505	
16	9	88,29	662,175	7,3575	1,427115	
15	10	98,1	735,75	8,175	1,284404	
14	11	107,91	809,325	8,9925	1,16764	80 cm 7C15
13	12	117,72	882,9	9,81	1,070336	
12	13	127,53	956,475	10,6275	0,988003	
11	14	137,34	1030,05	11,445	0,917431	
10	15	147,15	1103,625	12,2625	0,856269	
9	16	156,96	1177,2	13,08	1,03211	80 cm 9C15
8	17	166,77	1250,775	13,8975	0,971398	
7	18	176,58	1324,35	14,715	0,917431	
6	19	186,39	1397,925	15,5325	0,869145	
5	20	196,2	1471,5	16,35	0,825688	
4	21	206,01	1545,075	17,1675	0,78637	60 cm 9C15
3	22	215,82	1618,65	17,985	0,750626	
2	23	225,63	1692,225	18,8025	0,71799	
1	24	235,44	1765,8	19,62	0,688073	
0	25	245,25	1839,375	20,4375	0,66055	

Hoop tension, Steel area and Spacings

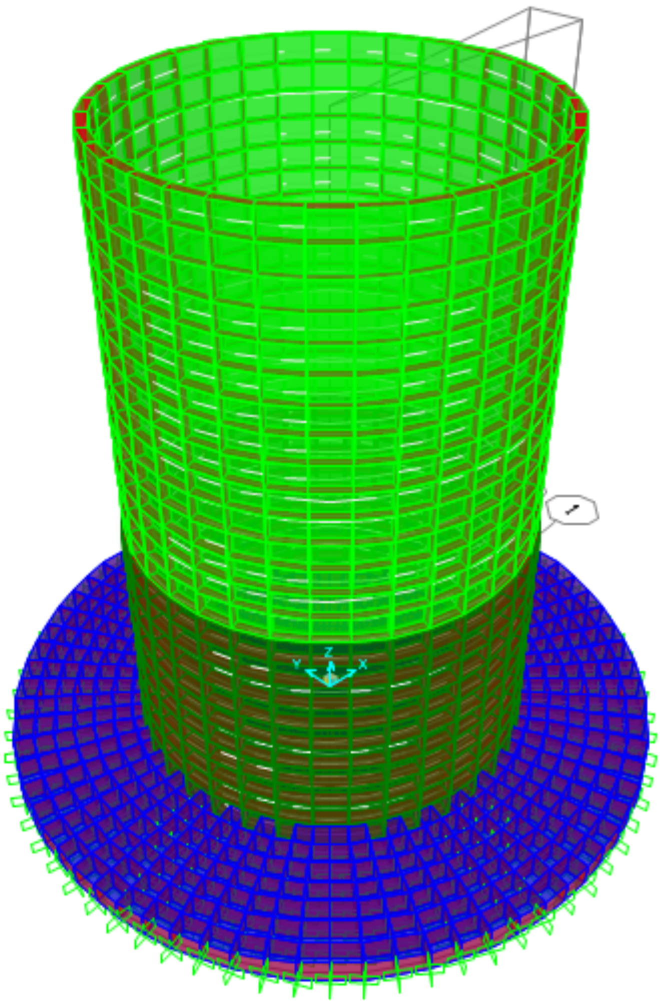


Tendon spacing and type along the height

SAP 2000 ANALYSIS OF THE WATER TANK

GENERAL INFORMATION ABOUT THE MODEL

The model was created with defining a cylindrical structure with constant inner radius of 7,5 m along 25 meters height. The bottom 10 meters of the silo thickness was decided as 0,6 m and the upper 15 meters of the silo wall was adjusted as 0,4 m. The concrete material used in design is EN C40/50 concrete. Model consists of 2088 shell thick elements and 2169 joints. The model looks like below:



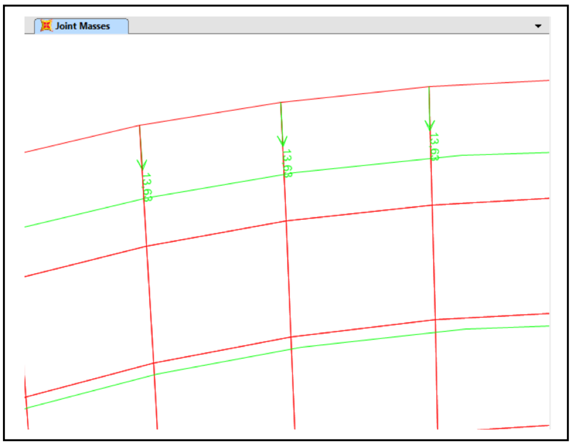
Meshing Information

The model was auto-meshed except outer foundation part. Walls of water tank was meshed at each 1 meter elevation, and in radial direction model was divided into 36 part, each part 10^0. The idea behind meshing is to model structure better.

LOAD ASSIGNMENT TO MODEL

ROOF LOAD

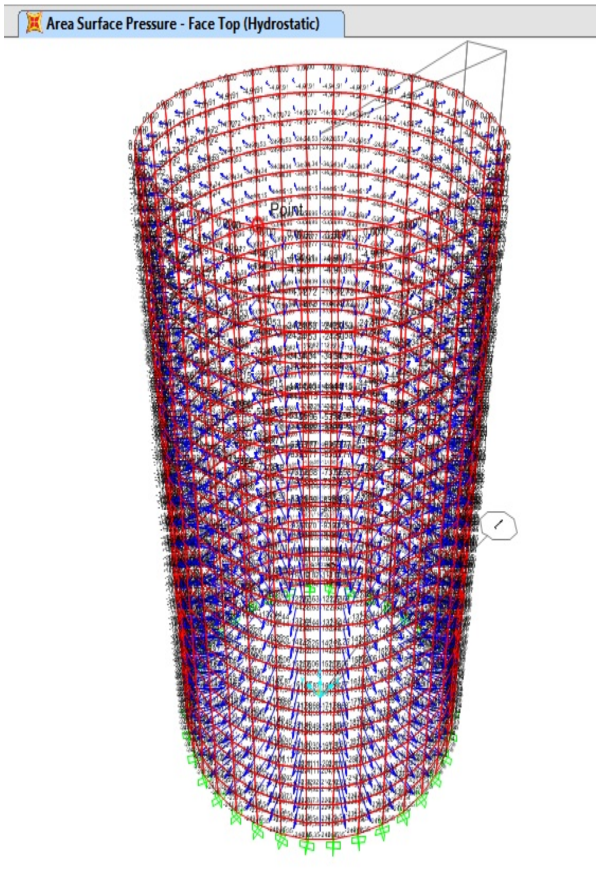
Dead weight of the roof was added to model as joint masses at +25 m elevation. Since the estimated weight of the roof is taken as 50 ton (500 kN) it is divided to 36 joints and assigned respectively.

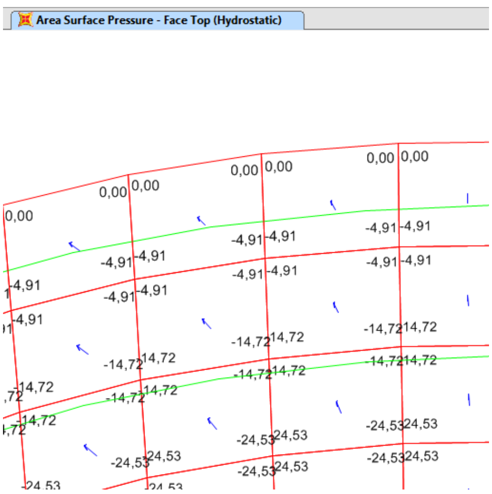


HYDROSTATIC LOAD

Dead weight of the water over the foundation was added as uniform area load by taking the operating level as 24,5 meters.

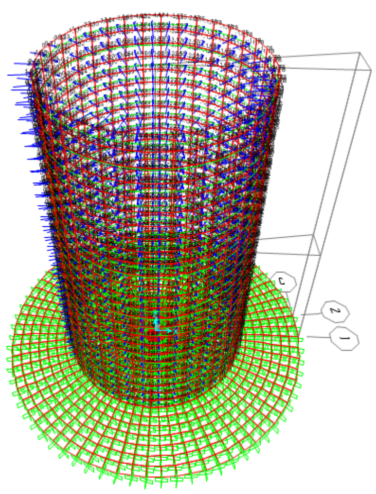
The horizontal hydrostatic pressure acting outwards to walls was defined as joint patterns, then added as surface pressures as shown in figure. Some values of hydrostatic pressure over walls can be seen closely from the figure below.

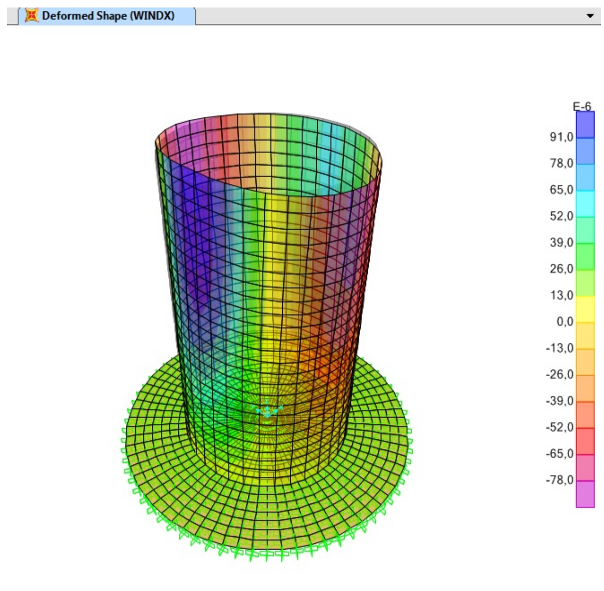




WIND LOAD

Since the wind load calculations are done according to we have tried to load our tank in accordance to SAP 2000 automated wind load pattern. However, it requires the diaphragm definition to analyze structure which causes the unstressed post tension cables. We just created the wind load joint pattern and then applied it as the surface pressure along +x direction. Since the structure has a cylindrical symmetric shape effect of wind load at directions –x, +y and –y also same in magnitude. Loading condition and the analyzed model is shown below.





End results of the analysis is verified by comparing the section cut forces and the hand calculations. To illustrate ;

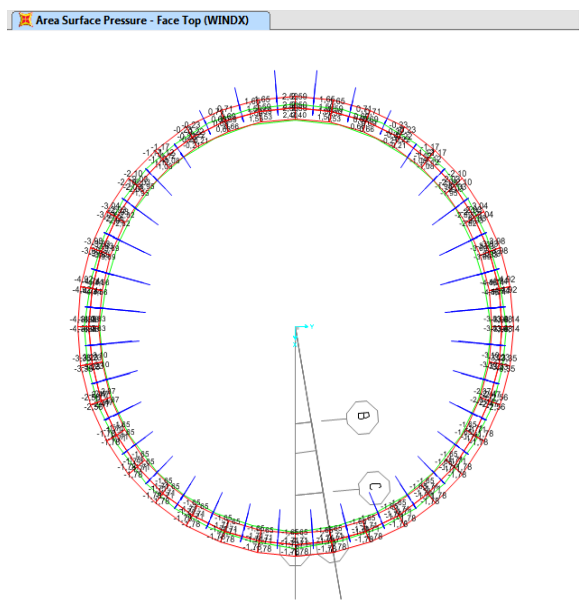
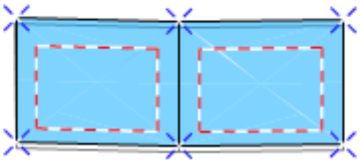


TABLE: Section Cut Forces - Analysis			
SectionCut	**F1**	**F2**	**F3**
Text	KN	KN	KN
0 m 0 deg	-0,441	-8,9E-15	4,26E-14
0 m 180 deg	-0,495	-1,4E-14	5,68E-14
0 m 270 deg	0,007745	0,972	-7,1E-14
0 m 90 deg	0,007745	-0,972	4,26E-14
10 m 0 deg	-2,149	5,17E-14	-1,7E-13
10 m 180 deg	-2,549	-2E-13	2,56E-13
10 m 270 deg	0,041	4,973	0
10 m 90 deg	0,038	-4,573	-1,7E-13
20 m 0 deg	-3,858	1,42E-13	-1,5E-13
20 m 180 deg	-4,603	-5,2E-14	-9,9E-14
20 m 270 deg	0,075	8,975	-6,4E-14
20 m 90 deg	0,078	-9,375	1,97E-13

At 10 m height and 180^0 external pressure found as follows, detailed calculations shown in wind loads part of the report

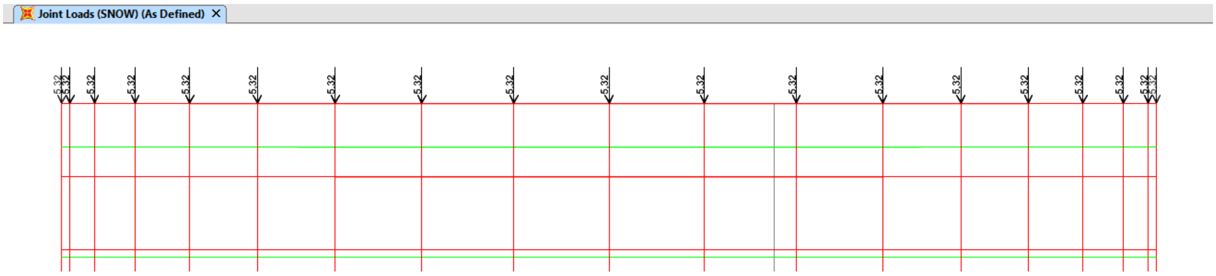
10	1,384	-2,630	-0,969



Since we divided the structure 36 strips radially, total area at section cut is 2,618 m^2 thus the total force acting on the section found as 2,536 kN. Sections are taken as 2 shell elements and 1 joint.

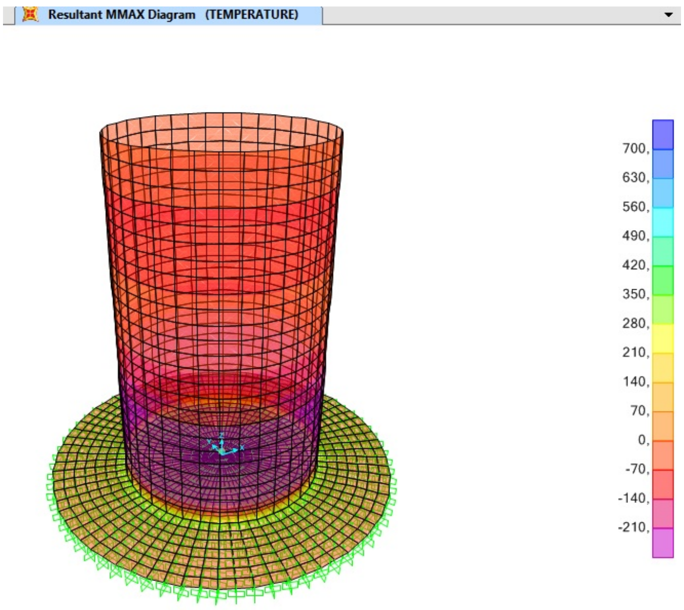
SNOW LOAD

The total value of snow loads was obtained previously. This value was assigned to the structure as joint loads at +25 m. Basically founded value of snow load is divided into 36 and added to joints in direction of gravity.

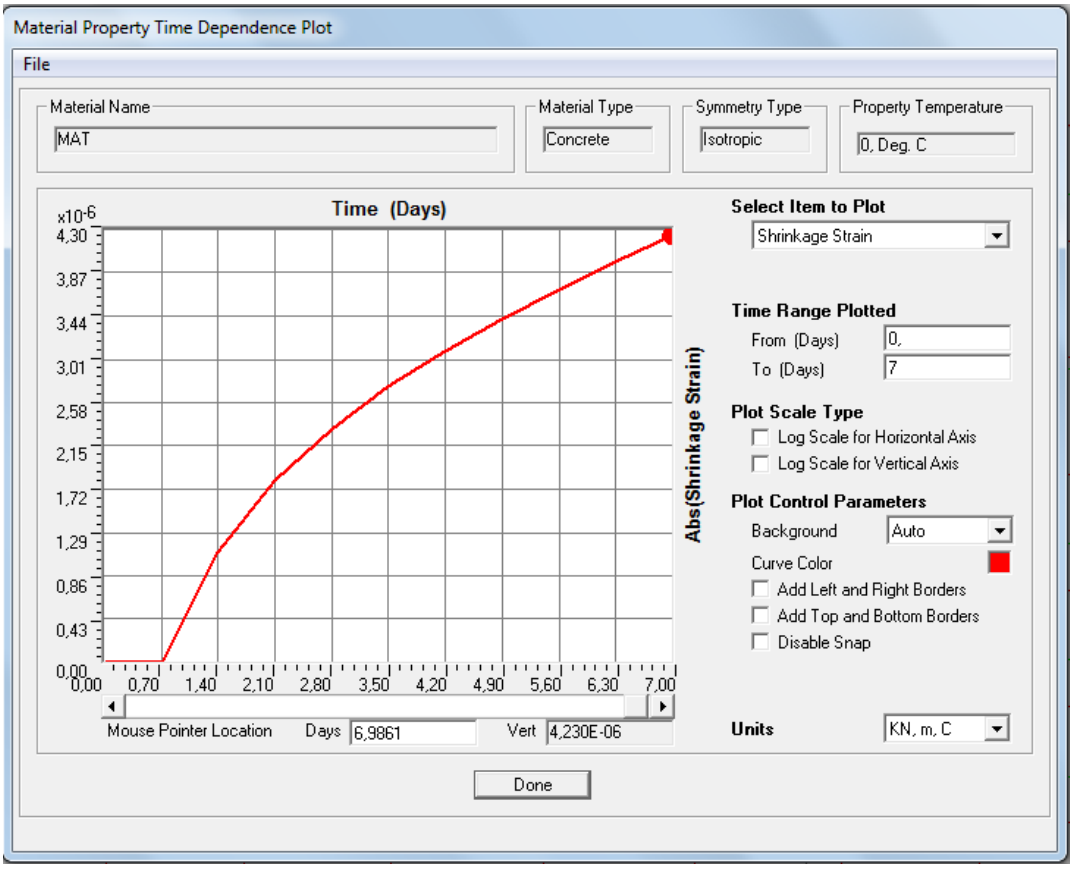


TEMPERATURE DIFFERENCE LOADS

Firstly the 30^0 C uniform temperature applied the walls of the tank then a temperature gradient with -30^0C /m added to temperature loads. When we compare the results of temperature gradient analysis with our hand calculations, the results was similar.



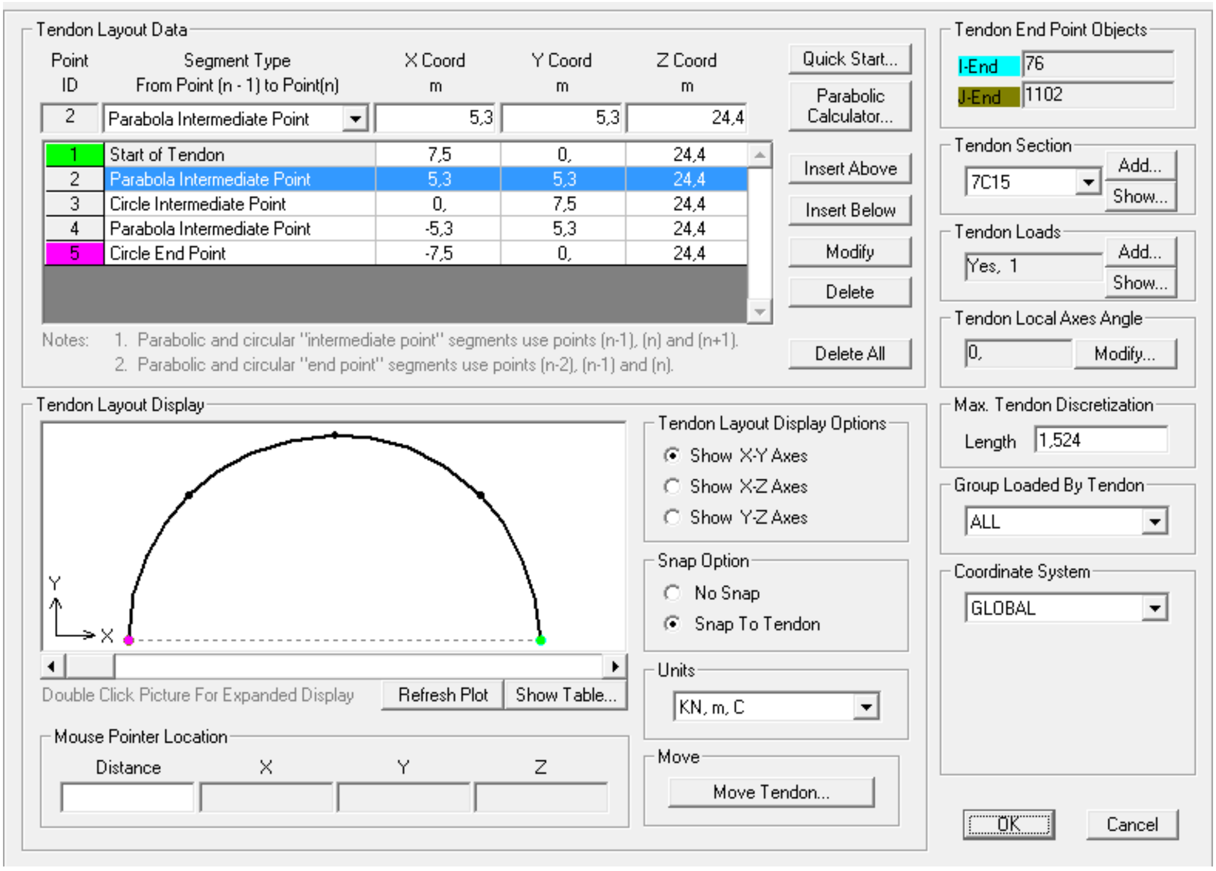
Moreover, the effects of the shrinkage over the walls added the model as concrete's time dependent property.

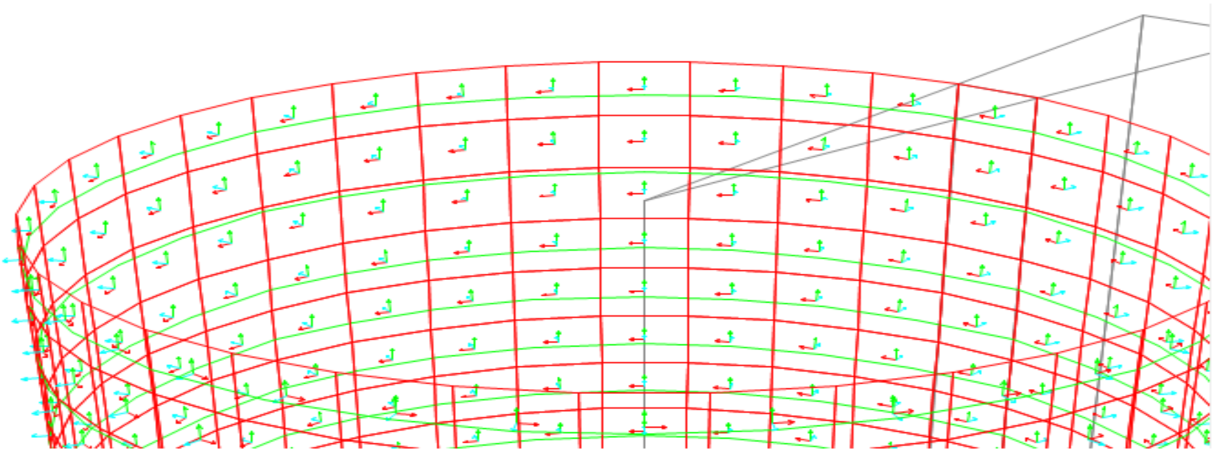


Again the values corresponding to 7 day shrinkage is close to the values what we found by our hand calculations. In addition to those, heat of hydration was also added to model as temperature loads with an effect of 60^0C.

POST TENSION TENDONS

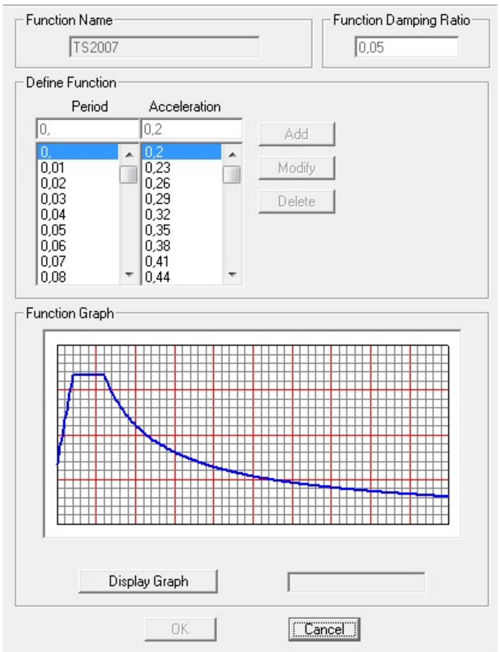
Post tension cables are placed over the silo body, in accordance to the spacing that we calculated at the preliminary design stage. Tendons are modeled as loads in material definition. Moreover while placing the tendons post tension stress is taken as the final stress on tendons after the losses.

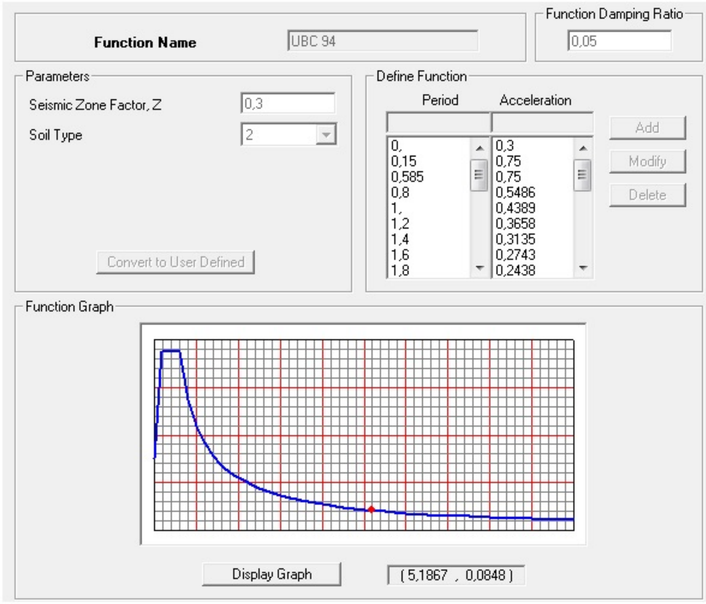




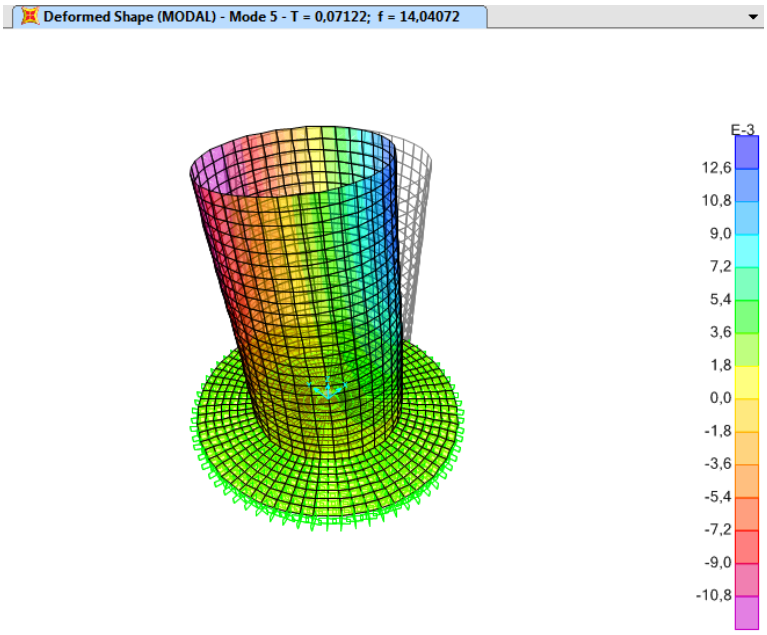
EARTHQUAKE LOADS

Since the earthquake forces are calculated according to ACI standard which is in compliance with the UBC standard earthquake load defined as the automated UBC response spectrum in SAP2000. Also TS 2007 response spectrum was also applied to the structure but the values obtained from the analysis was smaller than the UBC spectrum. Thus the analysis results of UBC 94 taken into consideration. UBC 94 Spectrum and TS 2007 Spectrum is given below;





In analysis of structure, 60 modes is used to reach +90% mass participation and fundamental period of the water tank is found at 5th mode as T=0,07122 sec.



As result of the analysis base reactions and the maximum moments found as ;

TABLE: Base Reactions							
Output Case	**Step Type**	**Global FX**	**Global FY**	**Global FZ**	**Global MX**	**Global MY**	**Global MZ**
Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
ENVELOPE	Max	1390,835	1390,834	92747,878	23984,4992	23984,501	0,0247
ENVELOPE	Min	-20607,864	-1390,834	-0,001608	-23984,4992	-244859,48	-0,0247

COMPRESSION CHECK OF TANK WALLS

Although post-tensioning provides many advantages for tanks and silos, tank wall should be checked compression of post-tensioning cables for empty case. Most critical section is the bottom.

$$Concrete\ Design\ Strenght = \frac{40}{1,5} = 26,67\ MPa$$

$$for\ the\ wall\ having\ width\ of\ 0,6\ m\ has\ an\ area\ of\ 0,6\ m^2$$

$$Post\ Tension\ Stress = 985\ MPa\ , Post\ Tensioning\ Force = 1330\ kN$$

$$\frac{1330}{0,6} = 2216,67\ kPa = 2,22\ MPa \ll 26,67\ MPa$$

$$for\ the\ wall\ having\ width\ of\ 0,4\ m\ has\ an\ area\ of\ 0,4\ m^2$$

$$Post\ Tension\ Stress = 985\ MPa\ , Post\ Tensioning\ Force = 1034\ kN$$

$$\frac{1034}{0,4} = 2585,63\ kPa = 2,56\ MPa \ll 26,67\ MPa$$

HORIZONTAL REINFORCEMENT OF THE SILO

SectionCut	Output Case	Step Type	F1	F2	F3	M1	M2	M3
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
0 m 0 deg	ENVELOPE	Max	502,67	63,65	1672,85	5,91	505,90	25,99
0 m 90 deg	ENVELOPE	Max	152,47	458,69	804,49	1266,21	12,04	287,82
0 m 270 deg	ENVELOPE	Max	152,47	**2503,64**	804,49	490,87	12,04	25,99

Between 0 and 3 meters

Minimum reinforcement area for the thickness 'h=60 cm'

$h = 60\ cm \qquad d = 55\ cm$

$\rho_{min} = 0.002 \quad b = 100cm$

$$A_{s,min} = \rho_{min}\ x\ b\ x\ d = 0.002\ x\ 100\ x\ 55 = 11\ \frac{cm^2}{m}$$

$$Length\ of\ shell = \frac{10}{360}\ x\ 2\ x\ \pi\ x\ 7.5\ x\ 2 = 2.618\ m$$

$N_{design} = 956\ kN/m$

$N_{design} = \emptyset \, x \, f_y \, x \, A_s$

$956 = 0.9 \, x \, 420000 \, x \, A_s \quad \rightarrow \quad A_s = 2530 \, mm^2/m$

$$2530 \; mm^2 \quad \rightarrow \quad \left(2 * \left(\frac{1000}{200}\right) * 18^2 * \frac{\pi}{4}\right) = 2545 \, mm^2$$

For horizontal reinforcement for both faces between elevations 0.00 m. and 3.00 m ϕ18/200 is adequate.

Between 3 meter and 25 meters

Minimum reinforcement area for the thickness ‘h=60 cm’

$h = 60 \, cm \qquad d = 55 \, cm$

$\rho_{min} = 0.002 \quad b = 100cm$

$$A_{s,min} = \rho_{min} \, x \, b \, x \, d = 0.002 \, x \, 100 \, x \, 55 = 11 \, \frac{cm^2}{m}$$

$Length \; of \; shell = \frac{10}{360} \, x \, 2 \, x \, \pi \, x \, 7.5 \, x \, 2 = 2.618 \, m$

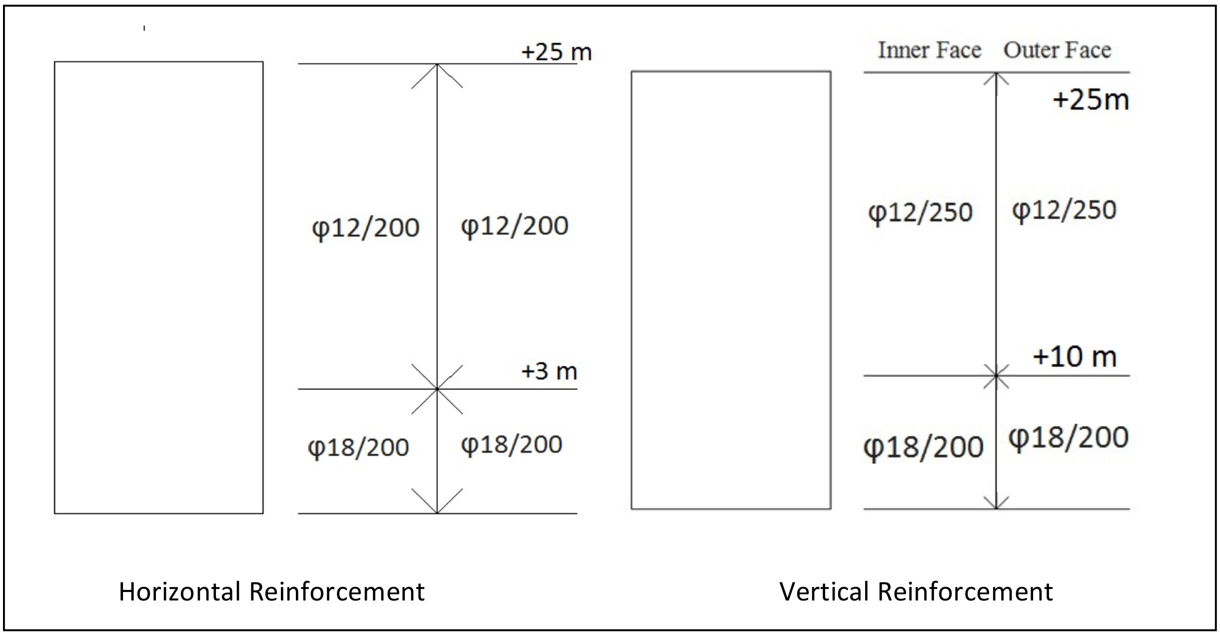
$N_{design} = 163{,}5 \, kN/m$

$N_{design} = \emptyset \, x \, f_y \, x \, A_s$

$164 = 0.9 \, x \, 420000 \, x \, A_s \quad \rightarrow \quad A_s = 433 \frac{mm^2}{m} < \; 1100 \, mm^2$

$$1100 \; mm^2 \quad \rightarrow \quad \left(2 * \left(\frac{1000}{200}\right) * 12^2 * \frac{\pi}{4}\right) = 1131 \, mm^2$$

For horizontal reinforcement for both faces between elevations 3.00 m. and 25.00 m ϕ12/200 is adequate.



VERTICAL REINFORCEMENT OF SILO

The calculations of vertical reinforcement design are made according to ACI 318-02 Building Code Requirements for Structural Concrete.

TABLE: Section Cut Forces - Analysis								
SectionCut	**Output Case**	**Step Type**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
0 m 0 deg	ENVELOPE	Max	502,67	63,65	1672,85	5,91	505,90	25,99
0 m 90 deg	ENVELOPE	Max	152,47	458,69	804,49	1266,21	12,04	287,82
0 m 270 deg	ENVELOPE	Max	152,47	2503,64	804,49	490,87	12,04	25,99
10 m 0 deg	ENVELOPE	Max	60,26	57,27	759,63	0,93	233,82	16,45
10 m 90 deg	ENVELOPE	Max	60,14	57,75	419,84	22,48	2,54	202,38
10 m 270 deg	ENVELOPE	Max	62,37	201,19	381,72	233,58	0,93	16,45
0 m 180 deg	ENVELOPE	Max	2559,22	63,65	376,51	5,91	1282,58	25,99
10 m 180 deg	ENVELOPE	Max	179,05	57,27	261,10	0,93	10,95	16,45
20 m 0 deg	ENVELOPE	Max	0,21	25,51	150,09	0,13	123,61	6,23
20 m 180 deg	ENVELOPE	Max	118,17	25,51	108,91	0,13	21,06	6,23
20 m 270 deg	ENVELOPE	Max	25,51	166,04	103,59	124,83	1,36	6,23
20 m 90 deg	ENVELOPE	Max	20,72	1,69	81,16	16,03	1,32	42,57
25 m 180 deg	ENVELOPE	Max	76,23	4,58	25,86	0,20	0,67	1,29
25 m 90 deg	ENVELOPE	Max	8,38	45,24	22,81	1,43	1,21	11,49
25 m 270 deg	ENVELOPE	Max	8,38	101,87	22,81	53,57	1,21	1,35
25 m 0 deg	ENVELOPE	Max	45,24	4,58	17,67	0,20	53,64	1,29
25 m 180 deg	ENVELOPE	Min	-45,24	-4,58	-1,97	-0,20	-53,31	-1,29
25 m 90 deg	ENVELOPE	Min	-37,39	-101,87	-2,14	-53,57	-0,20	-1,35
25 m 270 deg	ENVELOPE	Min	-37,39	-45,24	-2,14	-1,43	-0,20	-11,49
25 m 0 deg	ENVELOPE	Min	-128,57	-4,58	-5,37	-0,20	-6,81	-1,29
20 m 90 deg	ENVELOPE	Min	-137,71	-190,58	-9,27	-128,02	-0,28	-5,05
20 m 270 deg	ENVELOPE	Min	-177,78	-1,84	-13,54	-16,92	-0,64	-53,22
20 m 0 deg	ENVELOPE	Min	-215,73	-25,51	-14,26	-0,13	-19,02	-6,23
20 m 180 deg	ENVELOPE	Min	-6,11	-25,51	-46,66	-0,13	-125,55	-6,23
10 m 0 deg	ENVELOPE	Min	-210,41	-57,27	-81,44	-0,93	-10,30	-16,45
10 m 270 deg	ENVELOPE	Min	-639,92	-60,26	-83,08	-12,18	-9,00	-167,24
10 m 90 deg	ENVELOPE	Min	-690,85	-252,71	-92,15	-317,17	-0,20	-18,90
0 m 90 deg	ENVELOPE	Min	-966,12	-2503,64	-175,45	-490,87	-85,56	-25,99
0 m 270 deg	ENVELOPE	Min	-966,12	-458,69	-175,45	-1266,21	-85,56	-287,82
0 m 0 deg	ENVELOPE	Min	-2444,26	-63,65	-175,45	-5,91	-1245,93	-25,99
10 m 180 deg	ENVELOPE	Min	-60,26	-57,27	-440,35	-0,93	-233,17	-16,45
0 m 180 deg	ENVELOPE	Min	-417,52	-63,65	-1331,60	-5,91	-478,75	-25,99

Minimum reinforcement area for the thickness 'h=60 cm'

$h = 60\ cm \qquad d = 55\ cm$

$\rho_{min} = 0.002 \quad b = 100cm$

$$A_{s,min} = \rho_{min}\ x\ b\ x\ d = 0.002\ x\ 100\ x\ 55 = 11\ \frac{cm^2}{m}$$

$Length\ of\ shell = \frac{10}{360}\ x\ 2\ x\ \pi\ x\ 7.5\ x\ 2 = 2.618\ m$

$N_{max} = 1672.845/2.618 = 639\ kN/m$

$N_{min} = -1331.6/2.618 = -509\ kN/m$

$N_{d-compression} = 639\ kN/m$

$N_{d-tension} = 509\ kN/m$

$N_{d-tension} = \emptyset\ x\ f_y\ x\ A_s$

$509 = 0.9\ x\ 420000\ x\ A_s \quad \rightarrow \quad A_s = 1.35\ x\ 10^{-3}\ m^2/m$

$A_s = 1346.56\ mm^2/m$

$P_{n,max} = 0.8\ x\ \emptyset\ x\ [0.85\ x\ f_c\ x\ (A_g - A_{st}) + f_y\ x\ A_{st}]$

$\boldsymbol{P_{n,max} = 8573\ kN/m\ \ > 509\ kN/m}$

Using $A_s = 1350\ mm^2$ would be sufficient. Thus,

$1350\ \ mm^2 \quad \rightarrow \quad \left(2 * \left(\frac{1000}{300}\right) * 18^2 * \frac{\pi}{4}\right) = 1696\ mm^2$

For vertical reinforcement for both faces between elevations 0 and 10 meters ϕ18/200 would be adequate.

Minimum reinforcement area for the thickness 'h=40 cm'

$h = 40\ cm \qquad d = 35\ cm$

$\rho_{min} = 0.002 \quad b = 100cm$

$$A_{s,min} = \rho_{min}\ x\ b\ x\ d = 0.002\ x\ 100\ x\ 35 = 7\ cm^2/m$$

$Length\ of\ shell = \frac{10}{360}\ x\ 2\ x\ \pi\ x\ 7.5\ x\ 2 = 2.618\ m$

$N_{max} = 760/2.618 = 290\ kN/m$

$N_{min} = -440/2.618 = -168\ kN/m$

$N_{d-compression} = 290\ kN/m$

$N_{d-tension} = 168\ kN/m$

$N_{d-tension} = \emptyset\ x\ f_y\ x\ A_s$

$168 = 0.9\ x\ 420000\ x\ A_s \quad \rightarrow \quad A_s = 4.44\ x\ 10^{-4}\ m^2/m$

$A_s = 444.44\ mm^2/m$

$P_{n,max} = 0.8\ x\ \emptyset\ x\ [0.85\ x\ f_c\ x\ (A_g - A_{st}) + f_y\ x\ A_{st}]$

$\boldsymbol{P_{n,max} = 5637.5\ kN/m\ > 168\ kN/m}$

$444\ mm^2 < A_{s,min}$ so use $A_{s,min} = 7\ cm^2/m$ for reinforcement design

$$700\ mm^2 \quad \rightarrow \quad \left(2 * \left(\frac{1000}{250}\right) * \pi * \frac{12^2}{4}\right) = 905\ mm^2$$

For vertical reinforcement for both faces between elevations 10 and 20 meters ϕ12/250 would be adequate.

FOUNDATION DESIGN

Introduction and Soil Studies

The geotechnical report which is available was conducted by ARGEM Geotechnical Engineering Company. The field which was investigated is on Gölbaşı, Ankara. Geotechnical report consists of soil and site investigation, laboratory results which are water content, sieve analysis, Atterberg limit tests, and undrained uniaxial triaxial test and boring logs.

In geotechnical design part of the water tank, these topics are included:

- General characteristics of water tank
- Site investigation and idealized soil profile
- Characteristics of soil layers
- Determining foundation type
    - Bearing capacity aspect
    - Settlement aspect

General Characteristics of Water Tank

- Inner diameter of water tank: 15 meters & Height of water tank: 25 meters elevation
- Thickness of wall:
  - First 10 meters from foundation: 0.60 meters thickness
  - Between 10 meters and 25 meters: 0.40 meters thickness
- Roof = 50 tons= 500 kN
- Volume and mass of water tank wall:

$$V = [(8.1^2 - 7.5^2)\times\pi\times10] + [(7.9^2 - 7.5^2)\times\pi\times15] \rightarrow V = 584.34\, m^3$$

$$Unit\ weight\ of\ concrete\ \gamma_c = 24\, kN/m^3$$

$$Mass = 584.34\times24 = 14024.06\, kN = 1402.41\, tons$$

- Volume and mass of water storing in the water tank (Free board distance is not considered at preliminary stage)

$$V = \pi\times7.5^2\times25 \rightarrow V = 4417.86\, m^3$$

$$Unit\ weight\ of\ water\ \gamma_w = 9.806\, kN/m^3$$

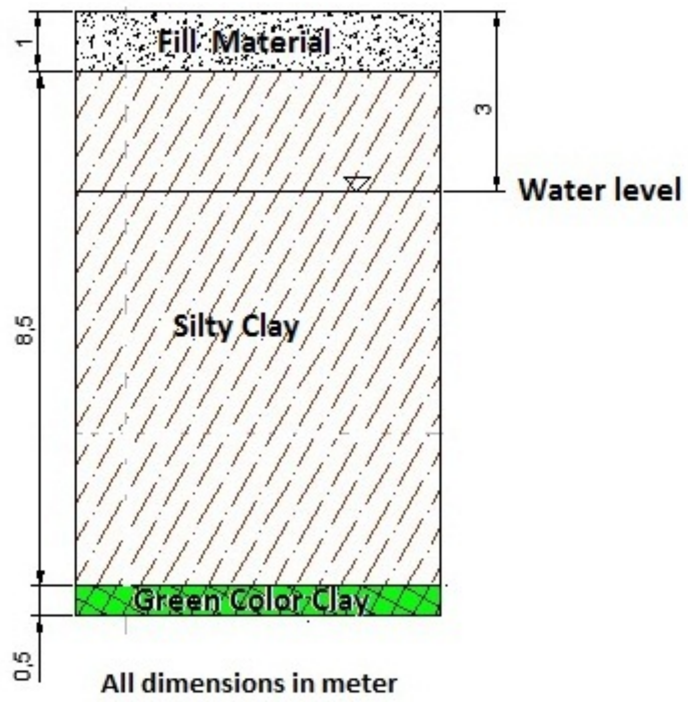
$$Mass = 4417.86\times9.806 = 43321.6\, kN = 4332.16\, tons$$

Site Investigation and Idealized Soil Profile

There are eight borings at the field. Six of them have 30 meters depth and depth of other two boring is 50 meters. In addition to these 8 borings, there are also seismic breaking measurements taken at 2 different points of field. Due to the flat topography, boring elevations change from 978.5 to 979.5.

The site is at 4th degree of earthquake zone. According to 2007 Turkish Earthquake Specifications, the soil profile is Z2 local ground class.

In the guidance with site investigation works, soil profiles are placed homogenous in both vertical and horizontal direction. There are 3 different types of soil at the ground profile. First layer is material fill which is 1 meter depth. Second layer which is 8.5 meters thickness is brown silty clay with little gravels. Last soil layer is green color clay.



Characteristics of soil layers

Fill Material: The thickness of the fill material is 1 meter. Since this layer is shown as homogenous characteristics, it should be excavated and therefore, bearing capacity and settlement calculations at foundation design step is not affected due to this layer.

Silty Clay with little gravel: This soil layer has high plasticity and brown color silty clay. It starts from 1 meter depth to 9.5 meters depth. This layer can be grouped as CH (clay of high plasticity). Soil layer properties are tabulated as below:

Green Color Clay: All boring works indicate that below the 9.5 meters depth from the ground, green color clay exists. SPT values on this layer are generally range in between 38 and 48. Above 50 SPT N value is ignored and N=43 is accepted remaining safe side at the geotechnical report.

		Brown Clay	Green Clay
Atterberg Limit Test	**PI**	30%	35%
Triaxial UU Test	$\emptyset_u$ **(friction angle)**	3-4^0	3-9^0
	C_u (undrained shear strength)	65-70 MPa	50-70 MPa
N (Average number of SPT)		28	43
$N_{60} = N \times \frac{E.R}{0.6} \times C_b \times C_s \times C_R$ **(E.R=0.6, C_b=C_s=1, C_R=0.75)**		21	32.25
Accepted Values during Calculations			
C_u (undrained shear strength) (KPa)		85	120
$\emptyset_u$ **(friction angle)**		0	0
γ		17	17
m_v **(from Stroud(1989))(m^2/MN)**		0.1	0.07
E_u (from Stroud(1989))(E'=(0.7-0.9)xN_{60} for stiff clays)(MPa)		13.44	20.64

Note that

C_u result was found low value in UU Triaxial Test due to the fact that inevitable disturbing the soil sample leads to decreasing undrained shear strength.

E_u is calculated from $E'/E_u = (1+\vartheta)/(1+\vartheta_u)$ where

Soil Type	Poisson's Ratio (ϑ)
Clays(undrained)	0.5
Clays(Stiff,undrained)	0.1-0.2

(From CE366 Foundation Engineering Lecture Notes)

According to Stroud (1989)

E'=(0.7-0.9)xN_{60} for plastic, I_p=50% and less plastic, I_p=15% clays)

E' is taken as 0.8xN_{60}

Recall

At the first preliminary design, 20x20x1 raft foundation calculations was done. After calculations,

- **Bearing capacity consideration:**

  $q_{all} = 179.06\ kPa = 17.91\ t/m^2$

  Structure pressure on soil ($q^{'}$) = 16.74 t/m^2

  Therefore, bearing capacity is not a problem for foundation.

- **Settlement consideration:**

  - **Immediate Settlement:** $S_i = 0.0247\ m$
  - **Consolidation Settlement:** $S_C = 0.1078$
  - **Total Settlement:** $= 0.1325$ m

Since 0.1325 m settlement is not tolerable for clay, settlement consideration is problematic for this foundation.

Soil Charachterization

After this point, soil characterization is focused on and some soil properties are changed.

For brown color clay:

Soil charachterization:

$E'/E_u = (1+\vartheta)/(1+\vartheta_u)\ where\ E' = (0.7-0.9)xN_{60}$

$E' = 0.9xN_{60} \rightarrow 0.9\ is\ taken\ due\ to\ stiff\ clay$

$E' = 0.9x21 = 18.9\ MPa$

$18.9/E_u = (1+0{,}5)/(1+0{,}2) \rightarrow E_u = 15.12\ MPa$

For Green Color Clay:

Soil charachterization:

$E'/E_u = (1+\vartheta)/(1+\vartheta_u)\ where\ E' = (0.7-0.9)xN_{60}$

$E' = 0.9xN_{60} \rightarrow 0.9\ is\ taken\ due\ to\ stiff\ clay$

$E' = 0.9x32 = 28.8\ MPa$

$28.8/E_u = (1+0{,}5)/(1+0{,}2) \rightarrow E_u = 23.04\ MPa$

		Brown Clay		Green Clay	
		Before	**Now**	**Before**	**Now**
Atterberg Limit Test	**PI***	30%	30%	35%	35%
Triaxial UU Test	$\emptyset_u$ **(friction angle)***	3-4^0	3-4^0	3-9^0	3-9^0
	C_u (undrained shear strength)*	65-70 MPa	65-70 MPa	50-70 MPa	50-70 MPa
N (Average number of SPT)*		28	28	43	43
$N_{60} = N \times \frac{E.R}{0.6} \times C_b \times C_s \times C_R$ **(E.R=0.6, C_b=C_s=1, C_R=0.75)***		28	21	32.25	32.25
Accepted Values during Calculations					
C_u (undrained shear strength) (KPa)*		85	85	120	120
$\emptyset_u$ **(friction angle)***		0	0	0	0
γ*		17	17	17	17
m_v **(from Stroud(1989))(m^2/MN)**		0.1	0.1	0.07	0.07
E_u (from Stroud(1989))(E'=(0.7-0.9)xN_{60} for stiff clays)(MPa)		13.44	15.12	20.64	23.04

*These values are taken from Argem Geo Eng Report from Gölbaşı

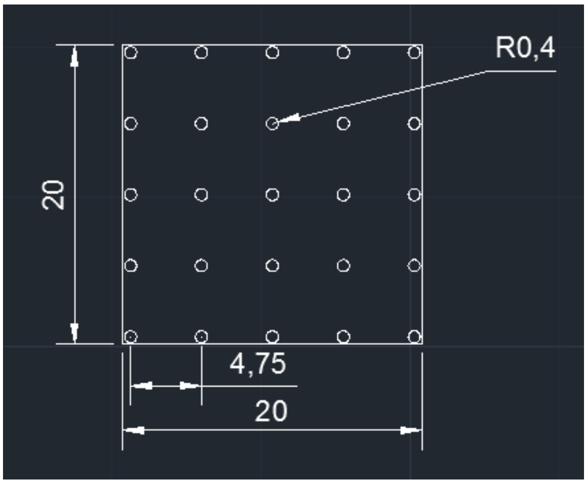
Note that the only change in soil characterization is undrained elastic modulus.

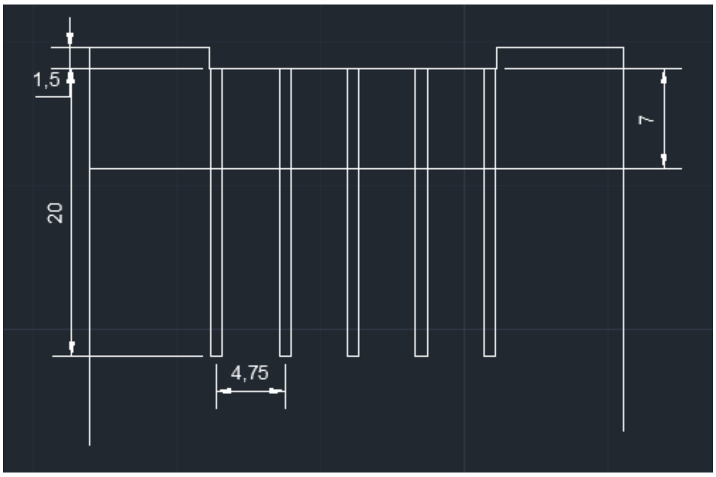
Foundation Design

Pile Foundation

Since raft foundation of 20x20 was not adequate, pile foundation is designed. Different piles length and order types are designed and determined which one is suitable in the view of foundation design criteria.

Assuming 25 piles having 80 cm diameter under the 20x20 m plate:





- **Bearing Capacity Consideration:**
  - **End Bearing:**

$Q_p = 9xC_u xA_p$

$Q_p = 9x120x\pi x(0.8)^2/4$

$Q_p = 542.87\ kN$

$- Q = 5784.6\ tons\ without\ foundation$
$- Foundation\ thickness = 0.8\ m$
$Q_f = 20x20x0.8x24 = 7680\ kN = 768\ tons$
$- Q_t = 6552.6\ tons = 65526\ kN$

- **Skin Friction:**

$\alpha_1 = 1 - 00615x(85 - 25) = \mathbf{0.631}\ for\ brown\ clay$

$\boldsymbol{\alpha_2} = \mathbf{0.516}\ interpolation\ of\ coefficient\ of\ adhesion\ table\ from\ CE366\ Lecture\ Notes)$

$Q_{s1} = \alpha x C_{ui} x P x \Delta L$

$Q_{s1} = 0.631x85x\pi x0.8x7 = 943.6\ kN$ & $Q_{s2} = 0.516x120x\pi x0.8x13 = 2023.1\ kN$

$Q_{ult} = \sum Q_s + Q_p = 943.6 + 2023.1 + 542.87 = 3509.6\ kN$

$Q_{all} = \frac{Q_{ult}}{F.S} = \frac{3509.6}{3} = 1169.8\ kN \rightarrow\ Q_{all} < Q_t$

Group Action Reduction

- **Converse – Labarre**

$E = 1 - \theta \left[\frac{(n-1)\ x\ m + (m-1) x\ n}{90\ x\ m\ x\ n}\right]\ where$

m = # of rows, n= # of columns, $\theta = \arctan(D/S)\ \ where\ D: diameter, S: spacing$

$\theta = \arctan(0.8/4.75) = 9.56^0$

$E = 1 - 9.56 \left[\frac{(5-1)\ x\ 5 + (5-1) x\ 5}{90\ x\ 5\ x5}\right] = 0.83$

$Q_{all} = 1169.8\ x\ 0.83 = 970.98\ kN$

- **Terzaghi – Peck**

  $A = 19.4x19.4 = 376.36\ m^2$

  $P = 77.6\ m$

  $Q_g = 77.6x7x85 + 77.6x13x120 + 120x376.36x9 = 573696.8\ kN$

  $$Q_{all} = \frac{573696.8x1}{25x3} = 7649.3\ kN > 1169.8\ kN$$

  No group action reduction. Allowable bearing value of pile is 1169.8 kN < 65526/25=2621 kN

Therefore bearing capacity is problematic for this pile foundation system.

Optimization works with respect to bearing capacity was done

- **m = 5, n = 5, # of piles = 25, F.S = 3**

D of pile	Length of pile	End bearing capacity	Skin friction	Qult	Qall	Converse-Labarre	Terzaghi - Peck	Qa(group)	Qall	Q'	Qall/Q'
0.8	20	542.8672	2966.681	3509.549	1169.85	971.0251	573696.8	7649.291	1169.85	2621.04	0.44633
1	37	848.23	7015.318	7863.548	2621.183	2067.185	737880	9838.4	2621.183	2621.04	1.000054
1.2	30	1221.451	6784.351	8005.802	2668.601	1989.105	666816.8	8890.891	2668.601	2621.04	1.018146

- **m = 6, n = 6, # of piles = 36, F.S = 3**

D of pile	length of pile	End bearing capacity	Skin friction	Qult	Qall	Converse-Labarre	Terzaghi - Peck	Qa(group)	Qall	Q'	Qall/Q'
0.8	33	542.8672	4989.766	5532.634	1844.211	1438.189	694752.8	6432.896	1844.211	1820.167	1.01321
1	25	848.23	4680.989	5529.219	1843.073	1339.861	625560	5792.222	1843.073	1820.167	1.012585
1.2	20	1221.451	4450.022	5671.473	1890.491	1270.815	573696.8	5312.007	1890.491	1820.167	1.038636

- **m = 7, n = 7, # of piles = 49, F.S = 3**

D of pile	length of pile	End bearing capacity	Skin friction	Qult	Qall	Converse-Labarre	Terzaghi - Peck	Qa(group)	Qall	Q'	Qall/Q'
0.8	24	542.8672	3589.169	4132.036	1377.345	1005.382	610944.8	4156.087	1377.345	1337.265	1.029972
1	18	848.23	3319.297	4167.527	1389.176	925.4405	560040	3809.796	1389.176	1337.265	1.038818
1.2	13	1221.451	2815.992	4037.443	1345.814	808.6222	508512.8	3459.271	1345.814	1337.265	1.006393

- **m = 8, n = 8, # of piles = 64, F.S = 3**

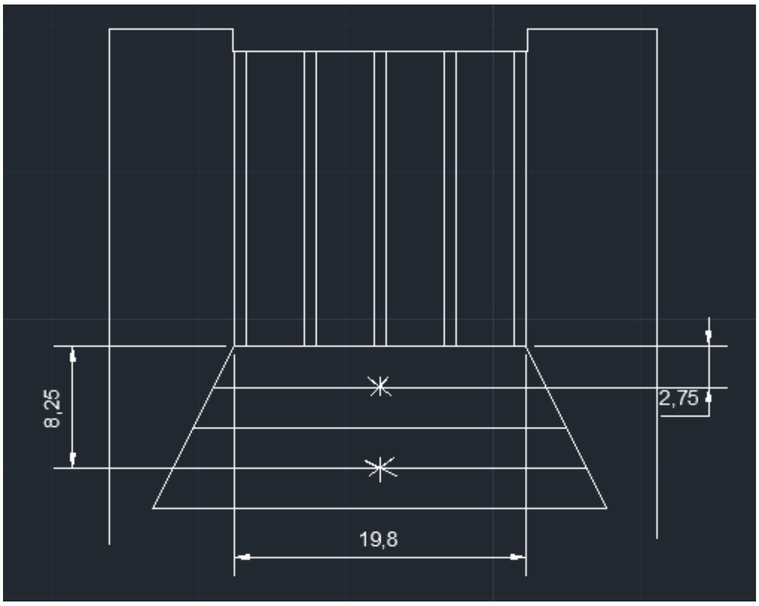
D of pile	length of pile	End bearing capacity	Skin friction	Qult	Qall	Converse-Labarre	Terzaghi - Peck	Qa(group)	Qall	Q'	Qall/Q'
0.8	18	542.8672	2655.437	3198.305	1066.102	725.6737	555072.8	2891.004	1066.102	1023.844	1.041274
1	13	848.23	2346.66	3194.89	1064.963	646.1546	513240	2673.125	1064.963	1023.844	1.040162
1.2	9	1221.451	1882.26	3103.711	1034.57	550.2521	471264.8	2454.504	1034.57	1023.844	1.010477

36 piles having 20 meter depth and 1.2 meter diameter is selected. Bearing capacity consideration is suitable due to Q_{all}= 1890.491 > Q'= 1820.167 kN

- **Settlement Consideration**

  Maximum depth taking into consideration is calculated from De Beer's Rule:

$$\frac{65526}{19.5 + x} = \frac{(17\ x\ 1.5)(7x)}{10} \rightarrow x = 29.61\ m \rightarrow x = 31.11\ m\ from\ foundation\ level$$



Stress calculation at midpoint of layer 1 and layer 2:

$$\Delta\sigma_1 = \frac{65526}{(19.8 + 2.75)^2} = 128.86\ kN/m^2$$

$$\Delta\sigma_2 = \frac{65526}{(19.8 + 8.25)^2} = 83.28\frac{kN}{m^2}$$

$$-\boldsymbol{s = m_v\ x\ H\ x\ \Delta\sigma}$$
$$s_1 = 0.07\ x\ 10^{-3}\ x\ 5.5\ x\ 128.86 = 0.045\ m$$
$$s_2 = 0.07\ x\ 10^{-3}\ x\ 5.5\ x\ 83.28 = 0.032\ m$$

$$S_{oed} = 0.077\ m \rightarrow S_c = 0.75 * 0.077$$

$$S_c = 0.058\ m$$

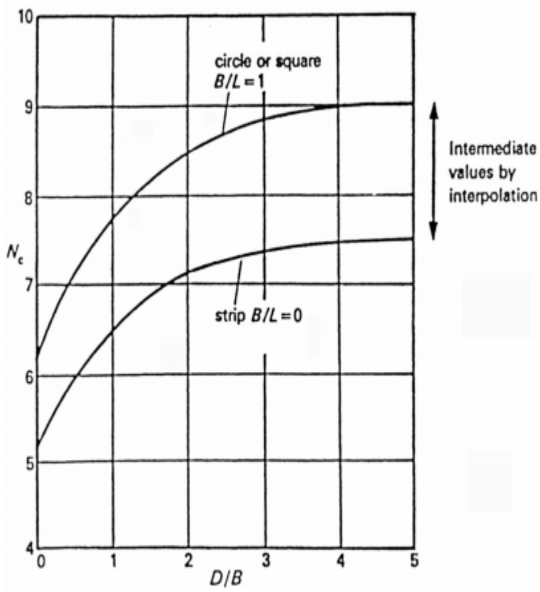
Both bearing capacity and settlement consideration is not problematic for 36 piles foundation having 1.2 meter diameter and 20 meter length.

Raft Foundation

Second option is to increase raft foundation dimensions. It is going to be 26 meter diameter raft foundation having 1 thickness and 2 m excavation.

Bearing Capacity of Foundation:

The bearing capacity formula is simplified in case of $\emptyset_u = 0$ (apparent) assumption:



$q_f = C_u \times N_c + \gamma D$ where N_c = Skempton's N_c value

$$q_{nf} = C_u \times N_c + \gamma D - \gamma D = C_u \times N_c$$

$$D/B = 2/26 = 0.077 \rightarrow N_c = 6.34 \rightarrow q_{nf} = C_u \times N_c \rightarrow q_{nf} = 85 \times 6.37 = 541.45\ kPa$$

Factor of safety is taken 3.0

$$q_{all} = \frac{q_{nf}}{F.S} \rightarrow q_{all} = \frac{541.45}{3.0} \rightarrow q_{all} = 180.48\ kPa$$

$$q_{all} = 179.06\ kPa = 17.91\ t/m^2$$

- ***Stresses due to structure***

Structure pressure on soil (q'): 7058.8/530.9= 13.3 t/m²

$- Q = 5784.6\ tons\ without\ foundation$

$Q_f = \pi\ x\ 26^2/4\ x 24 = 12742\ kN = 1274.2\ tons$

$- Q_t = 7058.8\ tons = 70588\ kN$

Since $q_{all} > q'$ condition, bearing capacity condition for 26 m diameter foundation with 1 meter thickness is OK.

Settlement Aspect

Settlement consists of immediate settlement and consolidation settlement. Finally sum of them gives total settlement due to the net pressure on the soil layer.

Immediate Settlement

Calculation procedure for immediate settlement of foundations on clay is proposed by Janbu, Bjerrum and Kjearnsli(1956) which is modified by Christian and Carrier(1978) Immediate average settlement is calculated by

$S_i = \mu_0 \times \mu_1 \times \frac{q \times B}{E}$ where

- q is net foundation pressure without pressure due to mass of water(due to immediate settlement, net foundation pressure just after construction meaning no water inside the water tank)
- $\mu_0 and\ \mu_1\ are\ the\ emprical\ factors$
- $B\ is\ width, E\ is\ undrained\ modulus\ of\ elasticity$

Calculation depth is calculated from De Beer's Rule: $\Delta\sigma_v{}' = (1/10)x\Delta\sigma_0$

$$\frac{69627}{(26+x)^2} = \frac{1}{10}(17+7x) \rightarrow x = 29\ m\ below\ water\ table$$

$Calculation\ depth = 29 + 1 = 30\ m\ is\ taken\ during\ the\ calculations\ below\ foundation\ depth$

Immediate settlement is calculated by some assumption. First layer is assumed to be brown color clay having 9.5 meter depth and immediate settlement of this layer is called as s_1. Second layer is assumed to be green color clay under the foundation through 30 meter depth and immediate settlement of this layer is called as s_{2a}. Third layer is assumed to be green color clay distance between foundation bottom level to 9.5 meter depth and called as $s_{2b.}$ Immediate settlement = s_1+s_{2a}-s_{2b}

$Q_t = 7058.8\ tons = 70588\ kN\ \&\ weight\ due\ to\ the\ water = \ 43321.6\ kN$

$q = (70588 - 43321.6)/530.9 = 51.36\ \ kN/m^2$

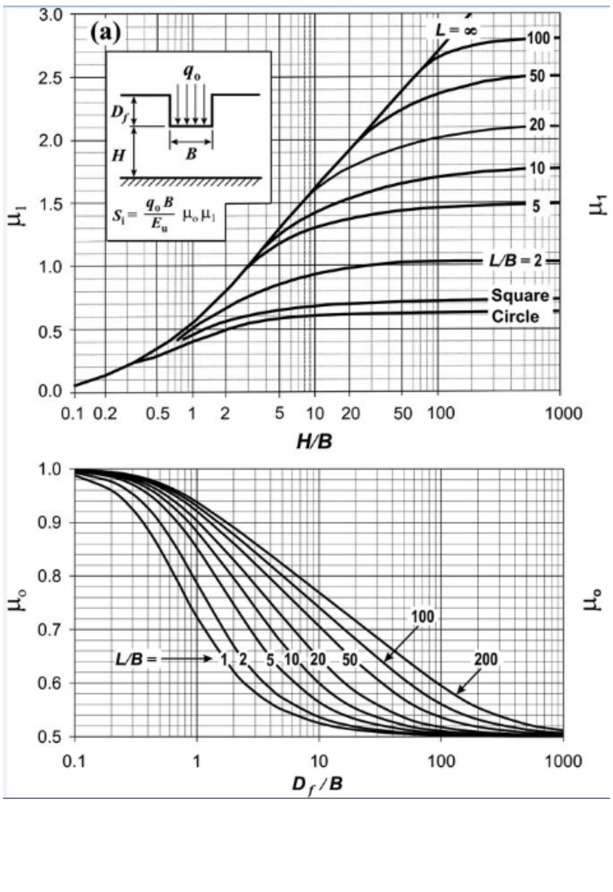
Immediate settlement of silty clay:

➔ Between 2 m depth and 9.5 m depth

(L/B=1, D/B=2/26=0.077, H/B=7.5/26=0.29)

$$S_i = \mu_0 \times \mu_1 \times \frac{q \times B}{E} = 1 \times 0.045 \times \frac{51.36 \times 25}{15120}$$

$$S_{i1} = 0.0038\ m$$



Immediate settlement of green color clay:

➔ Between 2 m depth and 30 m depth

(L/B=1, D/B=2/26=0.077, H/B=30/26=1.15)

$$S_i = \mu_0 \times \mu_1 \times \frac{q \times B}{E} = 1 \times 0.363 \times \frac{51.36 \times 25}{23040}$$

$S_{i2a} = 0.0202\,m$

➔ Between 2 m depth and 9.5 m depth

(L/B=1, D/B=2/26=0.077, H/B=7.5/26=0.29)

For silty clay:

$E_u = 15.12\ MPa$
$m_v = 0.1\ m^2/MN$

For green color clay:

$E_u = 20.64\ MPa$
$m_v = 0.07\ m^2/MN$

$$S_i = \mu_0 \times \mu_1 \times \frac{q \times B}{E} = 1 \times 0.045 \times \frac{51.36 \times 25}{23040}$$

$S_{i2b} = 0.0025\,m$

$S_{i2} = 0.0211 - 0.0025 = 0.0177\,m \rightarrow S_i = S_{i1} + S_{i2} = 0.004 + 0.0177 = 0.0217\,m$

Consolidation Settlement

$S_c = \Delta\sigma' \times H \times m_v$ and 2:1 approximation is used in order to find $\Delta\sigma'$ (vertical effective stress)

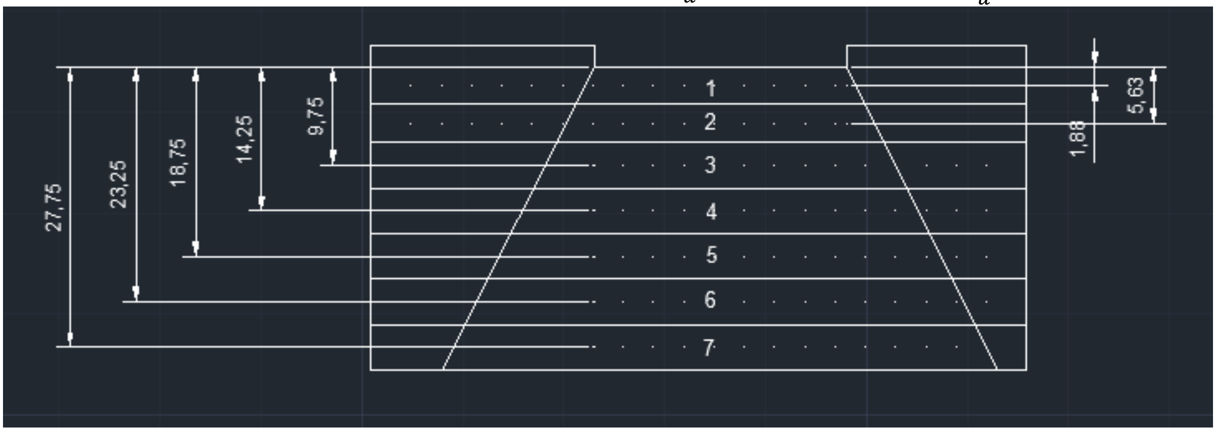
$q_{net} = (70588/530.9) - 1x17 = 115.96\ kPa$

For green color clay:

$E_u = 23.04\ MPa$

For silty clay:

$E_u = 15.20\ MPa$



$$-\ \Delta\sigma'_1 = \frac{115.96 \times 26 \times 26}{27.875 \times 27.875} = 100.88\ kPa$$

$$S_{c1} = \Delta\sigma' \times H \times m_v = 100.88 \times 3.75 \times 10^{-4} = 0.0378\,m$$

$$-\ \Delta\sigma'_2 = \frac{115.96 \times 26 \times 26}{31.625\ x\ 31.625} = 78.38\ kPa$$

$$S_{c2} = \Delta\sigma' \times H \times m_v = 70.38 \times 3.75 \times 10^{-4} = 0.0294\,m$$

$$-\Delta\sigma'_3 = \frac{115.96 \times 26 \times 26}{35.75\ x\ 35.75} = 61.33\ kPa$$

$$S_{c3} = \Delta\sigma' \times H \times m_v = 61.33 \times 4.5 \times 7 \times 10^{-5} = 0.0193\ m$$

$$-\Delta\sigma'_4 = \frac{115.96 \times 26 \times 26}{40.25\ x\ 40.25} = 48.39\ kPa$$

$$S_{c4} = \Delta\sigma' \times H \times m_v = 48.39 \times 4.5 \times 7 \times 10^{-5} = 0.0152\ m$$

$$-\Delta\sigma'_5 = \frac{115.96 \times 26 \times 26}{44.75\ x\ 44.75} = 39.14\ kPa$$

$$S_{c5} = \Delta\sigma' \times H \times m_v = 39.14 \times 4.5 \times 7 \times 10^{-5} = 0.0123\ m$$

$$-\Delta\sigma'_6 = \frac{115.96 \times 26 \times 26}{49.25\ x\ 49.25} = 32.32\ kPa$$

$$S_{c5} = \Delta\sigma' \times H \times m_v = 32.32 \times 4.5 \times 7 \times 10^{-5} = 0.0102\ m$$

$$-\Delta\sigma'_7 = \frac{115.96 \times 26 \times 26}{53.75\ x\ 53.75} = 27.13\ kPa$$

$$S_{c7} = \Delta\sigma' \times H \times m_v = 27.13 \times 4.5 \times 7 \times 10^{-5} = 0.0085\ m$$

$$S_{oed} = 0.0378 + 0.0294 + 0.0193 + 0.0152 + 0.0123 + 0.0102 + 0.0085 = 0.1327\ m$$

By Skempton-Bjerrum correction factor $\mu = 0.6$ for normally consolidated clay

$$S_C = S_{oed} \times \mu \rightarrow S_C = 0.1327 \times 0.6 = 0.0796$$

$$\boldsymbol{Total\ Settlement = S_i + S_c = 0.0217 + 0.0796 = 0.1013\ m}$$

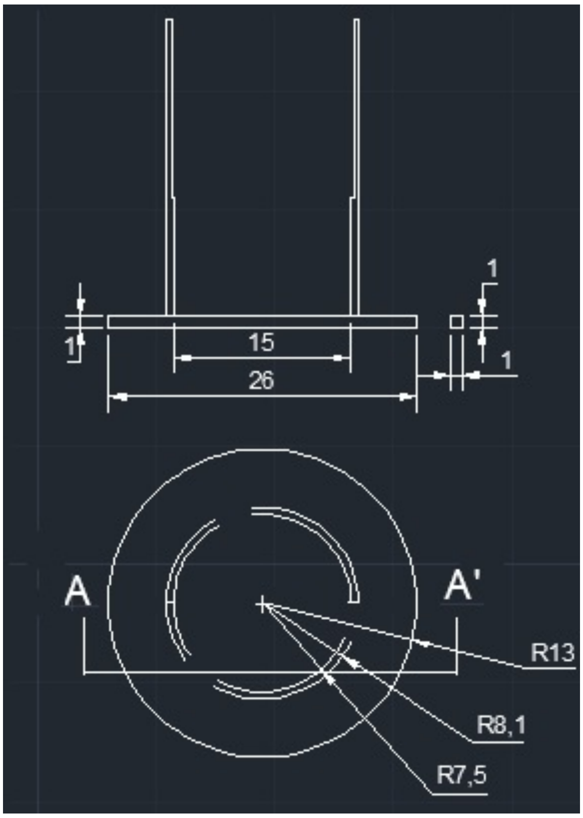
10 cm is tolarable for raft foundations when clay is existing beyond the foundation

Ground Improvement

Due to the stiff clay existing beyond the foundation, it is found that the extra ground improvement work is not practical and economic solution.

Since raft foundation is more economic and less construction time than pile foundation, 26 meter diameter having 1 meter thickness raft foundation is prefered rather than pile foundation.

Flexural Design of Foundation



A-A'section

Depth is one meter, length is one meter and it is assumed to be square cross area section. . (Normally, the cross section is going to be rounded shape due to the circle foundation type) A-A' cross section is not taken from the middle of the water tank. We select representative area which is the half of the area at the middle

Materials

C30 & S420 K_l = 247

Water pressure: 24 x 9.81 x 1= 235.44 kN/m -> 253.44 x22.5/26=219.3 kN/m(representative)

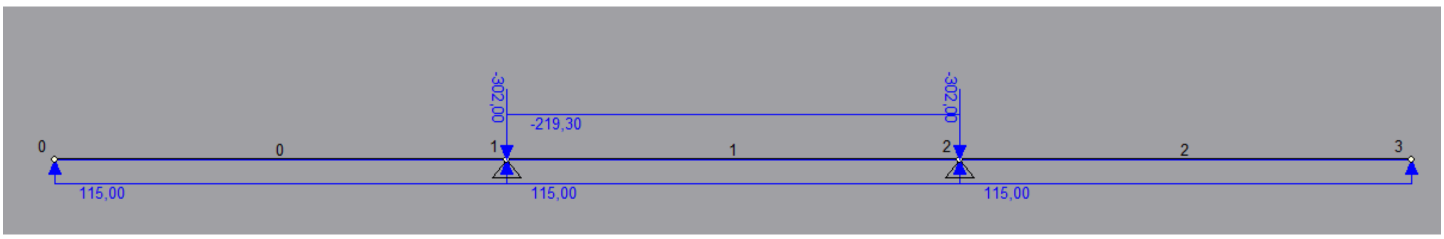
Wall load due to the weight:

Total weight without foundation: 57846 kN, **Water weight:** 43321.6 kN, **Perimeter:** 48.07 m

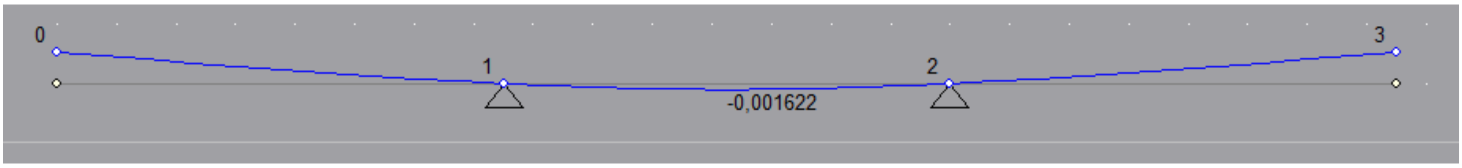
(57846 – 43321.6)/48.07 = 302.15 kN

Foundation pressure: 70588/(area of the foundation) x 1=132.95 kN/m

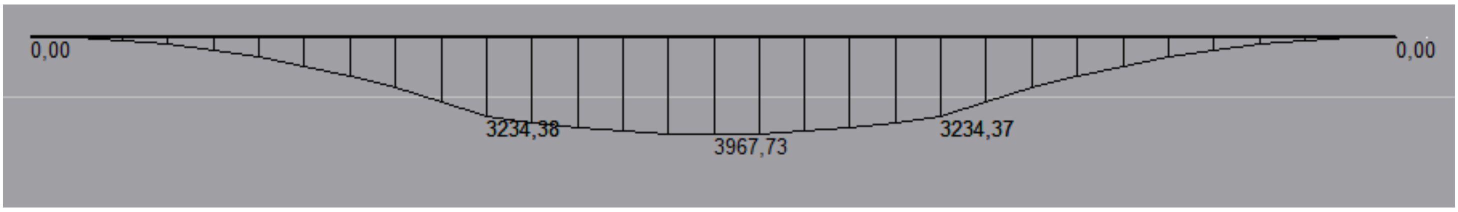
->132.85 x 22.5/26=115 kN/m (representative)



Free Body Diagram



Deflected Shape



Moment Diagram

<u>At the support of the foundation (critical at the middle of foundation) : Bottom Reinforcement</u>

$\mathrm{M} = 3967.73\ \mathrm{kN.m}\ \&\ \mathrm{d} = 950\ \mathrm{mm}\ \&\ \mathrm{clear\ cover} = 50\ \mathrm{mm}$

$$\mathrm{K} = \frac{b_w\ x\ d^2}{M_d} = \frac{1000\ x\ 950^2}{3967730} = 227 > K_l$$

$$j_l = 0.86 \rightarrow A_s = \frac{M_d}{f_{yd} x j_l x d} = \frac{3967730}{0.365\ x\ 0.86\ x\ 950} = 13305\ mm^2$$

With a 50 mm distance of two lines, $\mathbf{2\emptyset 40/150}$ steel reinforcement is used. They are placed horizontally in x and y directions.

$$A_s = \frac{40^2 x\pi}{4} x12 = 15079\ mm^2 > 13305\ mm^2\ for\ 1\ meter$$

Crack Control

$$\rho_{min} = 0.8x\frac{f_{ctd}}{f_{yd}}\ where\ f_{ctd} = \frac{0.35\times\sqrt{40}}{1.5} = 1.47\ MPa = 1475.73\ kPa$$

$$\rho_{min} = 0.8x\frac{1475.73}{365217} = 0.0032$$

$$\rho = \rho_{min} = \frac{A_s}{b_w\ x\ d} = 0.0032 \rightarrow A_s = 0.0032\ x\ 1000\ x\ 950 = 3040\ mm^2$$

Therefore, $\emptyset\,\mathbf{20/200}\ (\mathbf{3141}\ \boldsymbol{mm^2})$ cracking steel reinforcement is used for per 300 mm. (top and middle portion of the cross section)

FOUNDATION DESIGN CHECKS

Sliding Check of the foundation

Vertical Forces:

Total vertical load on the foundation = $\sum N = 70588\ kN$

Horizontal Forces:

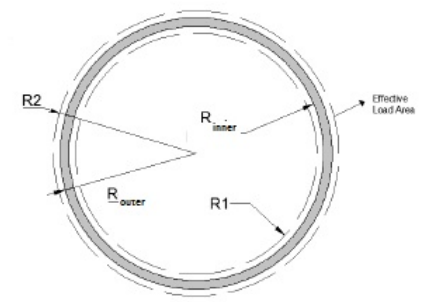
Design base shear (EQ) = $19684\ kN$

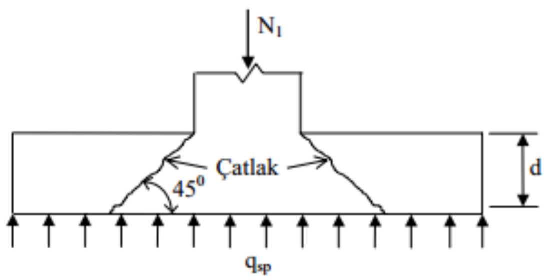
$$\text{F.S.} = \mu \frac{\sum N}{\sum H} = 0.5\ x\ \frac{70588}{19624} = 1.8 > 1.5 \quad \text{check for sliding OK!!}$$

Punching Shear check for foundation thickness = 1 meter

According to TS500 Specifications, punching shear condition is

$V_{pr} > V_{pd}$ where V_{pr} : punching shear resistance and V_{pd}: punching design resistance





$R_{outer} = 16.2\ meters\ \&\ R_{inner} = 15\ meters$

$R_1 = R_{inner} - D/2 = 15 - 0.5 = 14.5\ meters$

$R_2 = R_{outer} + D/2 = 16.2 + 0.5 = 16.7\ meters$

$u_p = \pi(R_1 + R_2) = 98.017\ \ meter$

$f_{ctk} = 0.35\times\sqrt{40} = 2.21\ MPa = 2213\ kPa$

$f_{ctd} = f_{ctk}/1.5 = 1475.73\ kPa$

$\gamma = 1.0\ for\ axial\ load$

$Punching\ Stress\ V_{pr} = \gamma\times f_{ctd}\times u_p\times D$

$V_{pr} = 1\times1475.73\times98.017\times1 = 144646.6\ KN$

$V_{pr} = 14464.7\ tons$

- $Q = 5784.6\ tons\ without\ foundation$
- $Foundation\ thickness = 1\ m$

$Q_f = \pi\ x\ 26^2/4\ x24 = 12742\ kN = 1274.2\ tons$

- $Q_t = 7058.8\ tons = 70588\ kN$

$Area\ of\ foundation: A = \pi x26^2/4 = 530.9\ m^2$

$Punching\ Area\ A_p = \pi\times(R_1 + R_2)\times D = 98.017\ m^2$

$N = 7058.8\ tons$

$q_{sb} = N/A = 7058.8/530.9 = 13.29\ t/m^2$

$F_a = q_{sb}\times A_p = 13.29\times98.017 = 1303.2\ tons$

$Punching\ design\ shear\ strength\ V_{pd} = N - F_a$

$V_{pd} = 7058.8 - 1303.2 = 5755.6\ tons$

$$V_{pr} > V_{pd}\,.\textit{With depth of } 1 \textit{ meter thick, punching shear is OK}$$

Overturning Check

Overturning moment affected by the moment coming from earthquake is more important than moment due to the wind load.

- $\textbf{Overturning moments} = \textbf{earthquake moment} + \textbf{base shear x depth of foundation}$

$$\text{Overturning moments} = M_o = 208672 + 19624 \text{ x } 1 = 228296 \text{ kN.m}$$

- $\textbf{Normal stress at foundation} = \sigma = \frac{705887}{\pi \text{ x } 13^2} = 133 \text{ kPa}$

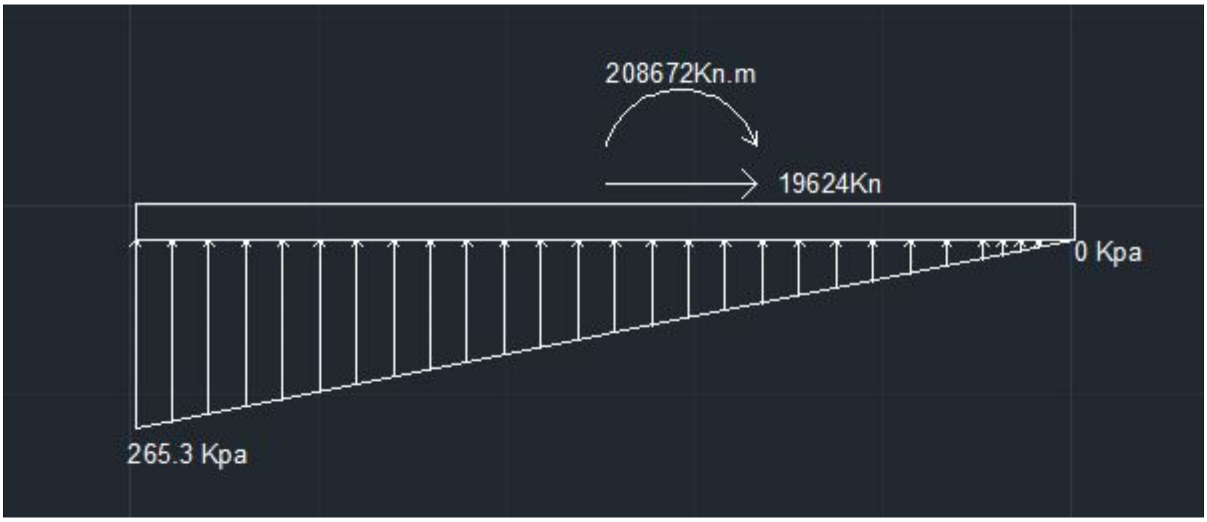
- $\textbf{Maximum strees at foundation} = \sigma + \frac{M \text{ x } c}{I}$

$$\text{Max. stress} = 133 + \frac{228296 \text{ x } 13}{\pi \text{ x } 0.25 \text{ x } 13^4} = 265.3 \text{ kPa} \quad \text{(compression)}$$

- $\textbf{Minimum strees at foundation} = \sigma - \frac{M \text{ x } c}{I}$

$$\text{Min. stress} = 133 - \frac{228296 \text{ x } 13}{\pi \text{ x } 0.25 \text{ x } 13^4} = 0.69 \text{ kPa} \quad \text{(compression)}$$

All stresses are compression, so there is no overturning problem.



COST ESTIMATION

Material	Unit Price	Quantitiy	Price(TL)
Concrete (C40) (for wall)	160 (TL/m^3)	584.34 m^3	93494.4
Concrete (C40) (for foundation)	160 (TL/m^3)	530.93 m^3	84948.8
Reinforcement (S420)	1,167 (TL/kg)	98 tons	114400
Post-tension cable	4 (€/kg)	14.160 tons	169920
Anchorage	2700(TL/tendon)	60 tendons	161142
Steel Roof	-	50 tons	~50000

- **Post-tensioning cables:**

Length: In each level whole perimeter divided into 2 parts (2 cables and 4 anchorages). Each cable has a length of 26 (24+2) m. in this length calculation. 2 m. wasted length is added to actual length because of hydraulic-jacking process.

There are 30 post-tensioning layers in the design of silo which means 1500(25*2*30) meters of post-tensioning cable is used.

Since we use 9C15 and 7C15 cable which has an average weight of 10.62 kg/m, total weight of cables is 14,16 tons.

Total cost of post-tensioning cables is 170000 TL

- **Concrete:**

Amount of concrete for wall is 584.34 m^3 and total cost is 93494.4 TL

Amount of concrete for foundation is 530.93 m3 and total cost is 84948.8 TL

- **Steel:**

Amount of reinforcement for foundation is 98 tons and total cost is 114400 TL

Total Material Cost = 673,843.2 TL

According to the past projects the cost of materials are only 1/2.75 of the total cost so we are predicting a total cost of 2.75 * material cost;

Total Cost = 1,853,070 TL