School of Engineering

7022 Advanced steel structures
Semester II 2006

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Assignment:

Design of Water Tank Tower

Submitted to:

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Contents

Chapter 1 Design of Water Tank in Toowoomba

H. C. Andersen has ordered a Water Tank for his farm in Toowoomba. This report deals with the structural design of the supporting tower for this water tank.

The results are 4 drawings in appendix, cf. Table 1.1.

Drawing no.:	View:	Scale:
1	Front elevation	1:100
2	Side elevation	1:100
3	Top view	1:100
4	Front elevation	1:20

Table 1.1: *List of drawings.*

1.1 General Description of the Construction

The water tank is placed 20 m above ground and is 6 m tall. The diameter of the water tank is 10m. It is estimated that the water tank has a life time of 20 years. It is chosen to use a steel frame to support the water tank. Since, there is only sand in the soil under the water tank it is possible to use direct reinforced concrete foundation. The water tank is made by plastic material. In order to services the water tank, there is constructed a service platform around the tank.

1.2 Description of Construction Parts

In the following, the parts of the tower are described from the upper parts and down. Firstly the service platform is all around the tank and has a small railing. The tank is placed on a metal plat which is 10mm thick. This is supported by 4 welded beams since their main purpose are to carry the moment from the water tank. The 4 beams spans 12 m and are supported by 2 other welded beams. The 6 beams and the metal plat are called the upper construction. The upper construction is supported by a square hollow section in each corner. It is chosen to use a square hollow section because it will be exposed to the same load about both directions. In order to reduce the column length there is a square hollow section placed in the middle of the column. All horizontal loads are transferred to the foundation using a wind girder. The construction is illustrated in Figure 1-1.

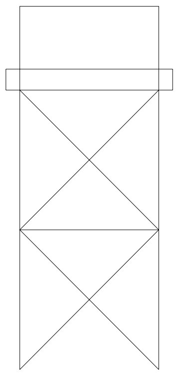

Figure 1-1: *Elevation of the construction.*

1.3 Constructional Principal

Vertical loads are lead through the beams to the columns and then to the foundations. From the foundations, the loads are distributed to the sub soil.

The horizontal loads are mainly wind load which act on the tank it self and the steel frame. The load on the tank is transferred through the beams and from there it becomes compression in the columns and tension in the upper bracings. The bracings are not designed to take compression but only tension. From the upper bracing the load passes to the middle connection of the frame structure. Here, the load is transformed to the horizontal column as compression and to the vertical column as tension. Furthermore, the wind load from the frame is added and led to the other side of the frame. The load is then transferred as tension in the lower bracing and compression in the column. The next step is the foundation.

All connections are design as pinned joint connections. In this assignment, only the connection between the bracing and column is designed.

Chapter 2 Actions

In the following section, the actions are determined. The actions are wind action, imposed action the service platform, action from the stored water and dead load. All actions are determined according to HB 2.2 – 2003. Furthermore, robustness is discussed and the relevant combinations of actions are considered.

2.1 Wind Action

In the following section, the wind pressure is calculated according to HB 2.2 -2003,

$$p = (0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn} \qquad (.1)$$

Where,

P = design wind preasure acting normal to the surface in , Pascal's

= p_e, p_i or p_n where sign is given by the C_p values used to evaluate C_{fig}

Note: Pressures are taken as positive, indicating pressures above ambient and negative indicating pressure below ambient.

ρ_{air} = Density of air which shall be taken as 1.2Kg/m^3

$V_{des,\theta}$ = building orthogonal design wind speeds (usually, θ= 0°. 90°, 180° and 270°)

C_{fig} = aerodynamic shape factor

C_{dyn} = dynamic response factor.

And the site wind speed is calculate according to equation 2.2 from HB 2.2-2003.

$$V_{sit,\beta} = V_R M_d (M_{Z,cat} M_s M_t) \qquad (.2)$$

Where

V_R = Regional 3s gust wind speed, in meters per second, for annual probability of exceedance of 1/R.

M_d = Wind directional multipliers for the 8 cardinal directions (β)

$M_{Z,cat}$ = terrain / height multipliers

M_s = Shielding multiplier

M_t = topographic multipliers

As the orientation of the building is not known th regional wind speed is assumed to act from any cardinal direction (i.e. M_d =1.0 for all directions).

The water tank is situated in Toowoomba on a farm that means that it is placed in region A4. The water tank is placed in open terrain and therefore it is assessed to be a category 1 structure. Since, the structures estimated lifetime is 20 years the wind speed is 37m/s (table 3.1, HB 2.2 – 2003).

From equation (.2)

$$V_{sit,\beta} = V_R M_d (M_{Z,cat} M_s M_t)$$

V_R = 37 m/s Region A4 and regional wind speed for 20 years. (From table 3.1 HB 2.2 – 2003).

M_d = 1 (For any direction) (Table 3.2 HB 2.2 – 2003).

$M_{Z,cat}$ = $\left[\frac{1.22-1.19}{30-20}\right]\left(26-20\right)-1.19$) by linear interpolation between 20 m and 30 m as the water tank height is 26 m (from table 4.1(A) HB 2.2 – 2003).

M_s = 1, since no shielding of the building is required (from table 4.3 HB 2.2 – 2003).

M_t = M_h = 1 since no hills (from Clause 4.4.1 HB 2.2 – 2003).

$$\therefore V_{sit,\beta} = 37\, {}^{m}\!/\!_{s} \times 1 \times (1.21 \times 1 \times 1)$$

$$= 44.77\, {}^{m}\!/\!_{s}$$

Aerodynamic shape factor; C_{fig} for encosed buildings

$$C_{fig} = C_{pe} \times ka \times kc \times kl \times kp \tag{.3}$$

Where

C_{pe} = External pressure coefficient for rectangular enclosed buildings windward wall =0.8 for elevated buildings above ground from clause 5.2 B HB 2.2 2003

ka = Area reduction factor for side walls = 20 percent of area being considered for 100 m sq the reduction factor = 0.9333 from clause 5.4.2 HB 2.2 2003

kc = Combination Factor = 0.8/ ka = 0.8/0.9333 = 0.857 from clause 5.4.3 HB 2.2 2003

kl = Local pressure factor = 1 for wind forces contribution increases beyond the structure till water tank from clause 5.4.4 HB 2.2 2003

kp = Permeable cladding reduction factor depends on soliditry ratio = 0.7 from clause 5.4.2 HB 2.2 2003

C_{fig} For frictional drag forces shall be taken as 0.63 (for calculating external pressures on the walls of bins, silos and tanks of circular cross section elevated above ground). Appendix C ……………… 2.2.1

C_{fig} for open plane in a single plane

$$C_{fig} = 1.2 + 0.26(1 - \delta_e)$$

Where

δ_e = solidarity ratio of the structure which is the ratio of solid area to total area of the structure which is assumed as 20 % = 0.2

$$=1.2+0.26(1-0.2)$$

$$C_{fig}=1.408 \qquad \ldots\ldots\ldots\ldots\ldots \qquad 2.2.2$$

C_{dyn} = 1 from section 6 clause 5.4.2 HB 2.2 2003

Therefore;

Pressure on circular water tank elevated above ground.

$$p=(0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn} \qquad C_{fig} \text{ from 2.2.1}$$

$$p=(0.5\times1.2)[44.57]^2\times0.63\times1$$

= 751 Pascal

Force on water tank :

Periphery of the water tank = $P=\pi DH$

$$=\pi\times10\times6$$

$$=188.49m^2$$

Force = Pressure in Pascal $\times$Periphery of water tank

$$=751\times188.49$$

$$=141563.5N$$

$$=141.56KN\cong141.6KN$$

Pressure on open frames in single plane

$$p=(0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn} \qquad C_{fig} \text{ from 2.2.2}$$

$$p=(0.5\times1.2)[44.57]^2\times1.408\times1$$

= 1678.18 Pascal

Force on open frame

$$\text{Force} = \text{Pressure in Pascal} \times \text{Periphery of frame}$$

$$\text{Periphery of frame structure} = 10\times 20\ \times 0.2$$

$$= 40\text{m}^2$$

Note: The solidarity area of the water tank frame is taken as 20 percent

$$p = 1678\times 10\times 20\times 0.2$$

$$= 67.12KN \cong 67.2KN$$

2.2 Imposed Action

The imposed action consists of people on the service platform repairing the tank. It is estimated that R2 (ii) covers this type of activities. The action is found in table 3.2 which yields Q equal to 1.8/A + 0.12 kPa. This value must not be less than 0.25 kPa. The imposed action is applied to one way slabs and therefore the reduction factor is equal to 1.0. The area is 44m^2 and therefore 0.25kPa is used.

2.3 Water Load

The liquid density is well defined since it is water and the liquid height cannot be exceeded. Therefore, the self weight of the liquid must be treated as permanent action. One important issue concerning the water is that the tank can either be full of water or empty or somewhere in between. In the following, it is estimated that the water tank must be design for the case where the tank is either empty or full. The water load is given as (.4) or (.5).

$$G_w = \pi\left(5m\right)^2 6m\cdot 998\,{}^{kg}\!/_{m^3}\cdot 9.82\,{}^{m}\!/_{s^2} = 4618.3kN \qquad (\ .4)$$

$$G_w = 0kN \qquad (\ .5)$$

Since the water tank is constructed of plastic, it is assumed that the weight of the water tank it self is about the same as the water. The diameter of the water tank is the outer diameter and therefore no additional weight is taken into account.

2.4 Dead Load

For standard elements, the weight per meter is usual given when dealing with steel members. If the weight is not given the unit weight of steel is 76.9 kN/m^3.

The dead load is determined for each member due to the actual size of the member. The live load is defined with a G according to Australian Standards.

All the pumps is placed on the ground. This means that they give no load to the construction.

2.5 Load Combination

In this project, only the ultimate limit state is used and only for strength. The following is the worst assumed load combinations, given in Table 2.1.

1	1.35G	permanent action only
2	1.2G + 1.5Q	permanent and imposed
3	1.2G + Wu + ΨcQ	Permanent, wind and imposed

Table 2.1: *Load combination.*

Chapter 3 Design

In the following chapter, the members are designed according to HB 2.2 – 2003.

3.1 Spacing

In this section, the spacing between the beams supporting the water tank is determined. It is chosen to have 4 beams supporting the water tank. Since there is need for some space around the tank for servicing the tank, it is chosen to extend the platform dimension to 12mx12m. The aim is to have an equal load on each of the four beams. Therefore, the area of the water tank is divided into 4 areas which are about the same size, cf. Figure 3-1 where only half the plan of the water tank is shown. The upper beams are numbered with numbers, while the lower beams have letters.

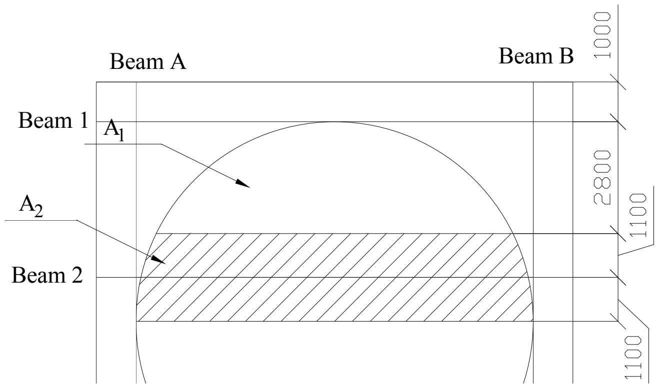

Figure 3-1: *Spacing of beams, all measures in mm.*

The relevant measures are given in Table 3.1.

	Area [m^2]	Dist. From center [m]
Beam 1 and 4	18	1.1
Beam 2 and 3	19.3	5

Table 3.1: *Data for the beams.*

This solution is safe, according to the lower bound theorem, since it distributes the load is a static possible way.

3.2 Design of Beam 1, 2, 3 and 4

As shown in Figure 3-1, the load from the water tank on beam 1 and 4 can be assumed as a point load in the middle of the beam and the load from the water tank on beam 2 and 3 can be assumed to be uniform distributed. The plate supporting the water tank is assumed to be 1cm thick. The beam is chosen as a 900WB257, grade 300, with the dimensions given in Table 3.2.

b_f [mm]	400
d_t [mm]	916
t_w [mm]	12
t_f [mm]	28
I_{xx} [mm^4]	5050 million

Table 3.2: *Cross section data. (steelweb, 2006)*

It can be shown that beam 1 and 4 are subject to the largest moment. Therefore is it only nessessary to determine the bearing capacity for these beams. The characteristic loads on the beam are given in Table 3.3.

	Load
Plate [kN/m]	4.6
Water tank [kN]	1058
Imposed load [kN/m]	0.25
Beam [kN/m]	2.6

Table 3.3: *Characteristic loads on beam 1 and 4.*

Using the load combination given in Table 2.1 the following moments and shear forces are determined in Table 3.4.

Load combination	M_{max} [kNm]	V_{Mmax} [kN]
1	2767	0
2	2758	0
3	2755	0

Table 3.4: *Moments and shear force in beam 1 and 4.*

Since the compression flange is attached to the metal plat it is assumed that the flange can not buckle and the effective length is therefore 0m. It is only possible for the beam to failure by bending. The factor ø is 0.9 (AS4100 Table 3.4).

In order to verify the bearing capacity of the member, a moment capacity Table is used, given in Figure 3-2. The moment capacity is read to be just above 3000kNm which is larger than the load given in Table 3.4. The 900WB257, grade 300, is therefore sufficient for beam 1, 2, 3 and 4.

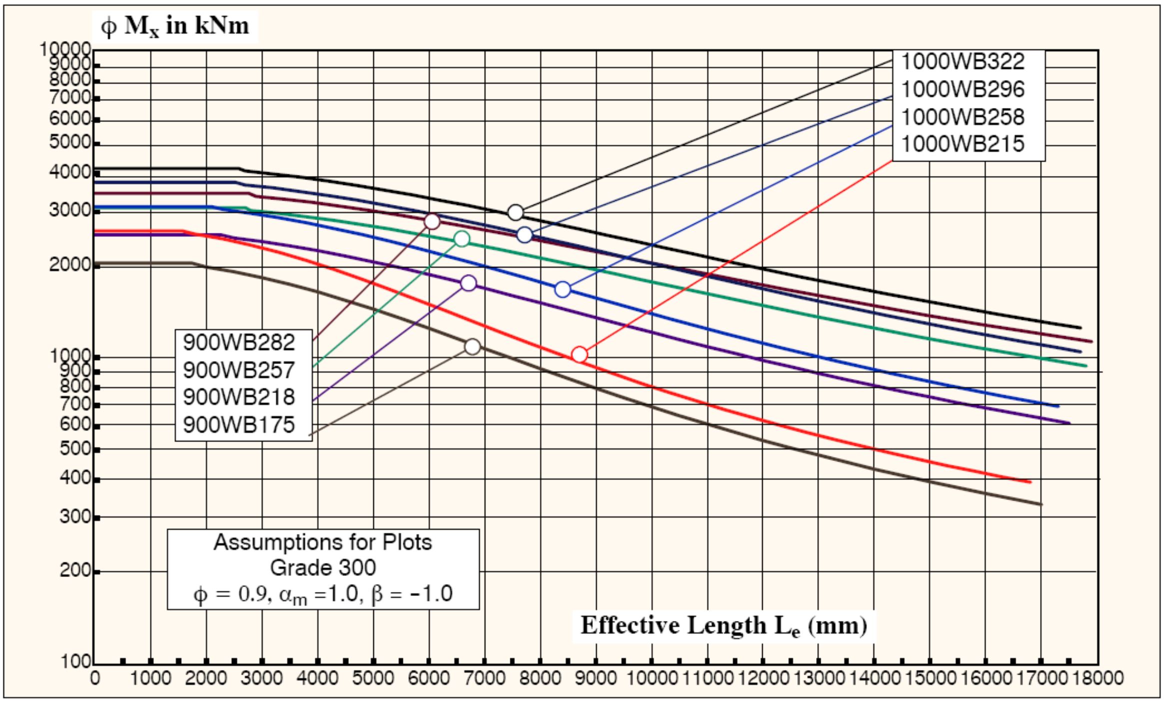

Figure 3-2: *Design member Moment capacity øM$_b$ [kNm]. (Doh, 2006)*

3.3 Design of Beam A and B

The loads effecting beam A and B are from beam 2 and 3 and the dead load from the beams. It is assumed that the load from beam 1 and 4 can run straight through beam A and B to the columns. It is noted that it might be necessary to reinforce the web between the columns and beam 1 and 4 because of the load transferred through the web.

The loads are therefore two point loads on the beams and the dead load from the beam it self. It is chosen to use a 1200WB392, grade 400, with the dimensions given in Table 3.5.

b_f [mm]	500
d_t [mm]	1184
t_w [mm]	16
t_f [mm]	32
I_{xx} [mm^4]	12500 million

Table 3.5: ***Cross section data. (steelweb, 2006)***

Using the load combination given in Table 2.1 the following moments and shear forces are determined in Table 3.6.

Load combination	M_{max} [kNm]	V_{Mmax} [kN]
1	3582,6	0
2	3189,2	0
3	3186,4	0

Table 3.6: *Moments and shear force in beam 1 and 4.*

In the following, the effect of attaching a fully restrain on each of the beams from beam 2 and 3 is investigated or none at all. This means that the effective length either is 3.9m or 10m.

Again, a design member moment capacity table is used given in Figure 3-3. It is clear that by using an effective length of 3.9m it is possible to chose the 1200WB278, grade 400, which is over 100kg lighter.

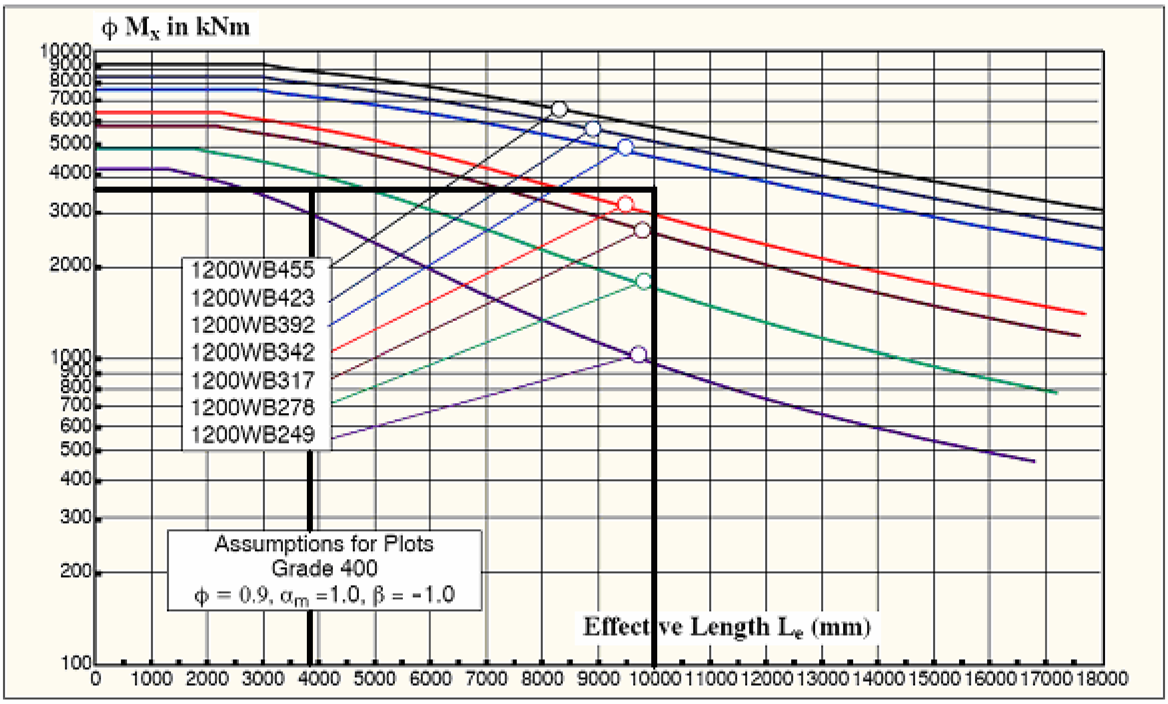

Figure 3-3: *Design member Moment capacity øM$_b$ [kNm]. (Doh, 2006)*

Furthermore, if it is chosen to use the fully restrained connections α_m=1.75 (AS4100 Table 5.6.1) which means that by chosen 1200WB249, grade 400, with $øM_b$= 3000kNm the section moment capacity can be calculated by

$$øM_s = ø\alpha_m M_b = 1.75 \cdot 3000kNm = 5250kNm \quad (.6)$$

Since

$$M^* = 3582.6kNm \leq ø\alpha_m M_b = 5250kNm \quad (.7)$$

It is possible to choose a 1200WB249, grade 400, for the beam A and B

3.4 Design of Column

Since the heights of the two beams are 2.1m together the length of the column is reduced to 17.9m. Because of the restrain in the middle of the column it is not possible to move horizontal at this point but it can rotate. The effective length of the column is therefore L_e=8.95m. It is chosen to use a square hollow section 250x250x9, cold formed, grade 350 with f_y=350MPa.

According to AS 4100 – 1998 section 6.1 a compression member shall satisfy both

$$N^* \leq øN_s \tag{.8}$$

$$N^* \leq øN_c \tag{.9}$$

where

$$N_s = k_f A_n f_y \tag{.10}$$

$$N_c = \alpha_c N_s \leq N_s \tag{.11}$$

where

k_f	is a form factor which is equal to 1 if (.12) is fulfilled
A_n	is the net area of the cross section
α_c	is the member slenderness reduction factor

Effect of plate buckling is investigated by

$$\lambda_{ey} > \lambda_e \tag{.12}$$

where

λ_{ey}	is the yield slenderness limit which is 40 for cold formed members with both ends supported (AS 4100 Table 6.2.4)
λ_e	is the plate slenderness which is given by (.13)

$$\lambda_e = \frac{b}{t}\sqrt{\frac{f_y}{250}} = \frac{250}{9}\sqrt{\frac{350}{250}} = 33 \tag{.13}$$

This results stats that k_f is equal to 1.

In order to determine α_c, Table 6.3.3(3) from AS 4100 is used. To use the table, the following values are determined

$$\lambda_n = \frac{L_e}{r}\sqrt{k_f}\sqrt{\frac{f_y}{250}} = \frac{8950}{89.5}\sqrt{1}\sqrt{\frac{350}{250}} = 31.8 \qquad (\ .14)$$

The second value is α_b which is -0.5 according to Table 6.3.3(1) for cold formed sections. By using interpolation, the α_c is read as 0.962.

The bearing capacity is then found by using (.8) and (.9)

$$øN_s = 0.9\cdot 1\cdot 8767\cdot 350 = 2762kN \qquad (\ .15)$$

$$øN_c = 0.9\cdot 0.962\cdot N_s = 2657kN \qquad (\ .16)$$

The load is then calculated according to the three load combinations. This is done for load combination 3, according to

$$\begin{aligned}P=&1.2\cdot\frac{\text{water}+\left(\text{beam1}\cdot 4+\text{beamA}\cdot 2\right)\cdot 12\text{m}+\text{plate}+\text{column}}{4}\\&+0.6\cdot\frac{\text{imposed}}{4}+1\cdot\left(\frac{\text{windtop}}{2}\cdot\frac{23\text{m}}{10\text{m}}+\frac{\text{windmiddle}}{2}\cdot\frac{8.98\text{m}}{10\text{m}}\right)\end{aligned} \qquad (\ .17)$$

$$\begin{aligned}P=&1.2\frac{4618.3\text{kN}+\left(2.5\text{kN/m}\cdot 4+2.4\text{kN/m}\cdot 2\right)\cdot 12\text{m}+110.7\text{kN}+0.7\text{kN/m}\cdot 17.9\text{m}}{4}\\&+0.6\frac{0.25\text{kN/m}^2\cdot 4\cdot 12\text{m}}{4}+1\left(\frac{141.5\text{kN}}{2}\frac{23\text{m}}{10\text{m}}+\frac{67.2\text{kN}}{2}\frac{8.95\text{m}}{10\text{m}}\right)\\&=1671kN\end{aligned} \qquad (\ .18)$$

This is also done for the 2 other load combinations and the results are given in Table 3.7.

LC 1	1661kN
LC 2	1481kN
LC 3	1671kN

Table 3.7: *Design loads on columns.*

By comparing Table 3.7 and equation (.15) is can be seen that the bearing capacity is ok. However, it seams as though the dimension of the column can be reduced but this is not the case when using standard element because this will affect the form factor k_f.

3.5 Design of Bracing

It is chosen only to use the bracings for tension. This means that the horizontal forces are lead to the foundation trough tension in the bracings as described in section 1.3. The bracing which carry the largest force is the bottom bracing. It carries the wind load from both the water tank at 23m above ground and the force from the truss 8.95m above ground. By taking horizontal equilibrium and projection the force in the angle of the bracing the force is determined

$$N^* = \frac{\left(\frac{141.6kN}{2} \cdot 23m + \frac{67.2kN}{2} \cdot 8.95m\right)}{cos\left(41.8^\circ\right)} = 140.1kN \tag{.19}$$

According to AS 4100 – 1998 section 7, the nominal section capacity of a tension member must be the lesser of

$$N_t = A_g f_y \tag{.20}$$

$$N_t = 0.85 k_t A_n f_u \tag{.21}$$

where

A_g is the gross area of the cross-section

k_t is the correction factor

It is chosen to use an equal angle 65x65x6 grade 250LO with the data given in Table 3.8.

f_y	260MPa	
f_u	410MPa	
A_g	748mm	
A_n	(748-18)mm= 730mm	
k_t	0.85	Table 7.3.2

Table 3.8: *Data for Equal angle.*

Note that there are subtracted 18mm from A_n, which is due to the bolted connection, see section 3.7.

The bearing capacity is then validated

$$N^* = 140.1kN \le 0.9 \cdot A_g f_y = 0.9 \cdot 748mm^2 \cdot 260MPa = 175kN \tag{.22}$$

$$N^* = 140.1kN \le 0.9 \cdot 0.85 \cdot k_t A_n f_u = 0.9 \cdot 0.85 \cdot 0.85 \cdot 730mm^2 \cdot 410MPa = 195kN \tag{.23}$$

The selected member is therefore able to carry the load.

3.6 Design of horizontal Column

The same procedure as in section 3.4 is used in the following section. The force in the horizontal column is from the wind load. The force is therefore equal to half the wind load since there is a column in both sides of the frame. The design wind load is given by

$$N^* = \frac{(141.6 + 67.2)kN}{2} = 104.4kN \quad (.24)$$

It is chosen to use a SHS with the dimension and properties in Table 3.9, Grade C350.

h and b	75mm
t	3mm
A	864mm^2
I	747792mm^4
r	29.4mm
L	10m

Table 3.9: *Dimensions and properties.*

The check for (.12) where λ_{ey} is equal to 40 for cold formed when both ends supported (AS 4100 Table 6.2.4)

$$\lambda_e = \frac{b}{t}\sqrt{\frac{f_y}{250}} = \frac{75}{3}\sqrt{\frac{350}{250}} = 29.5 \quad (.25)$$

Since $\lambda_{ey} > \lambda_e$ k_f is equal to 1.

In order to determine α_c, Table 6.3.3(3) from AS 4100 is used. To use the table, the following values are determined

$$\lambda_n = \frac{L_e}{r}\sqrt{k_f}\sqrt{\frac{f_y}{250}} = \frac{10000}{29.4}\sqrt{1}\sqrt{\frac{350}{250}} = 119 \quad (.26)$$

The second value is α_b which is -0.5 according to Table 6.3.3(1) for cold formed sections. By using interpolation, the α_c is interpolated to 0.469.

The bearing capacity is then found by using (.8) and (.9)

$$øN_s = 0.9 \cdot 1 \cdot 864 \cdot 350 = 272.2kN \quad (.27)$$

$$øN_c = 0.9 \cdot 0.469 \cdot 302.4 = 127.6kN \quad (.28)$$

Since both (.27) and (.28) are higher than the actual load given by (.24) the horizontal column is able to carry the load.

3.7 Design of Connection

It is chosen to design the connection of the bracing to the columns. The force will be different in the top and bottom pair of bracings but it is chosen to use the same size of connection anyway due to simplicity. The design load is the same as the load in the bracing given by (.19) and is equal to 140.1kN.

The design of the connection is divided into 4 steps, where the first is reduction of the cross section area of the bracing, the next is design of the bolts, followed by design of the plate from the bracing to the column and finally the design of the weld to the column. The connection is shown in Figure 3-4.

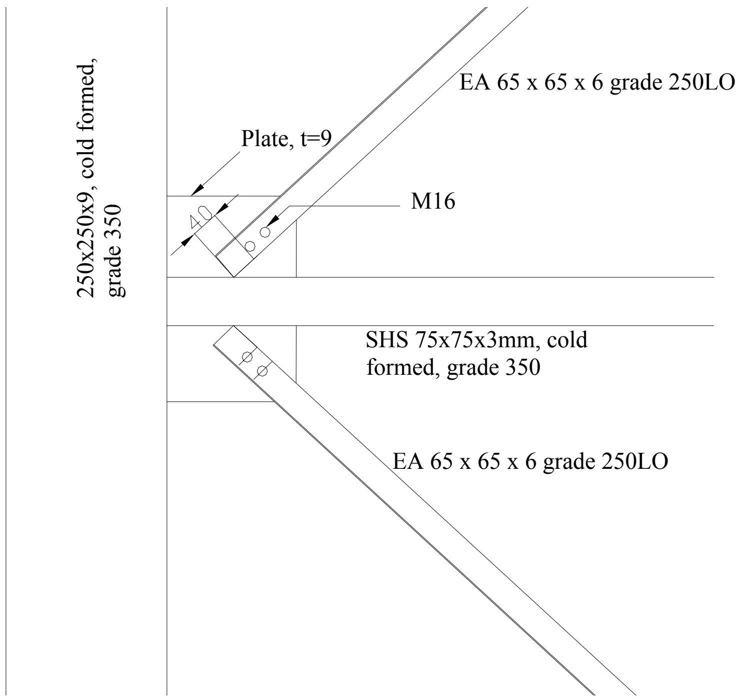

Figure 3-4: *view of the connection.*

As the figure shows, it is chosen to 2 bolts M16 and the must have a tolerance of 2mm. The cross section of the bracing must therefore be reduced with 18mm. The design of the reduced area is carried out in section 3.5.

The next step is the design of the bolts that must fulfill

$$V_f^* \le ø \cdot 0.62 f_{uf} k_r \left(n_n A_c + n_x A_o\right) \qquad (.29)$$

$$V_b^* \le ø \cdot V_b \qquad (.30)$$

where

V_f^*	is the design shear force
f_{uf}	is the minimum tensile strength of the bolt
k_r	is the reduction factor given in Table 9.3.2.1 and is 1 in this case
n_n	is the number of shear planes with threads intercepting the shear plane
n_o	is the number of shear planes without threads intercepting the shear plane
A_c	is the minor area of the bolt as defined in AS 1275
A_o	is the nominal plain area of the bolt
V_b^*	is the design bearing force, the design bearing capacity of one bolt
V_b	is the nominal bearing capacity of the ply

The nominal bearing capacity of the ply is given by

$$V_b = 3.2 \cdot d_f t_p f_{up} \qquad (.31)$$

$$V_b = a_e t_p f_{up} \qquad (.32)$$

where

d_f is the diameter of the bolt

t_p is the thickness of the ply

f_{up} is the tensile strength of the ply

a_e is the minimum distance form the edge of a hole to the edge of the ply, measured in the direction of the force

The parameters for the bolt connection are given in Table 3.10, where the bolts are M16 Grade 8.8.

$Ø_{bolt}$	0.8, Table 3.4
f_{uf}	830MPa
n_n	1
n_o	0
A_c	201mm^2
A_o	201 mm^2
d_f	20mm
t_p	6mm
f_{up}	420MPa
a_e	40mm
t_{plate}	9mm

Table 3.10: *Parameters for the bolt connection.*

Since there are two bolts, the shear force is distributed equally to both bolts. The bearing capacity of the bolt is given by (.29) and yields

$$V_f^* = 64.4kN \leq 0.8 \cdot 0.62 \cdot 830MPa \cdot 1 \cdot 201mm = 82.7kN \qquad (.33)$$

The nominal bearing capacity is given by

$$V_b^* = 82.7kN \leq V_b = 3.2 \cdot 16mm \cdot 6mm \cdot 410MPa = 126kN \qquad (.34)$$

The nominal bearing capacity is therefore ok. The tearing capacity is given by

$$V_b^* = 82.7kN \leq V_b = 40mm \cdot 6mm \cdot 410MPa = 98kN \qquad (.35)$$

This means that the bolted connections bearing capacity is sufficient.

The next step is to prove that the bearing capacity of the steel gussate plat between the column and bracing is ok. Since this is a tension member, the approach is similar to the one in section 3.5. The design parameters for the plate are given in Table 3.11 where the chosen steel is 300L15 for a thickness between 8 and 12mm. Note that the steel gussate plate on Figure 3-4 is larger than designed.

f_y	31Pa	
f_u	43MPa	
Size	65mm x 9mm	
A_g	585	
A_n	(585-18)mm= 567mm	
k_t	0.85	Table 7.3.2

Table 3.11: *Design parameters for the plate.*

The load must be smaller than (.20) and (.21)

$$N^* = 140.1kN \leq 0.9 \cdot A_g f_y = 0.9 \cdot 585mm^2 \cdot 310MPa = 163kN \qquad (.36)$$

$$N^* = 140.1kN \leq 0.9 \cdot 0.85 \cdot k_t A_n f_u = 0.9 \cdot 0.85 \cdot 0.85 \cdot 567mm^2 \cdot 430MPa = 159kN \qquad (.37)$$

The plate element is therefore sufficient to transfer the load.

The length of the weld the length of the plat cut in the angle of the bracing which is 41.8 deg. The length is found by

$$L_w = \frac{65mm}{cos(41.8)} = 87mm \qquad (.38)$$

The weld is on both sides of the plat leaving the total length at 174mm. The weld is chosen as a fillet weld with a thickness of 4mm. The nominel capacity per length of a fillet weld is given by

$$v_w^* \leq øv_w \tag{.39}$$

For this weld, ø is equal to 0.9 and v_W is given by

$$v_w = 0.6 \cdot f_{uv} t_t k_r \tag{.40}$$

where

f_{uw}	is the nominal tensile strength
t_t	is the design throat thickness
k_r	is the reduction factor given in Table 9.7.3.10(2), here equal to 1

It is chosen to use a E48XX with a nominal tensile strength at 480MPa. The bearing capacity of the weld is then given by

$$v_w^* = 140.1kN \leq øv_w = 0.9 \cdot 0.6 \cdot 480MPa \cdot 4mm \cdot 174mm \cdot 1 = 200kN \tag{.41}$$

The bearing capacity is hereby ok.

3.8 Chosen Elements

This section gives a brief summary of the results are given in Table 3.12.

Element:	Cross section and steel grade:	Numbers	Length per element [m]
Beam 1, 2, 3 and 4	The 900WB257, grade 300	4	12
Beam A and B	1200WB249, grade 400	2	12
Column (vertical)	250x250x9, cold formed, grade 350	4	17.9
Bracing	Equal angle 65x65x6 grade 250LO	16	13.420
Column (horizontal)	SHS 75x75x3mm, cold formed, grade 350	4	10

Table 3.12: *Summary of results.*

Chapter 4 References

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