



job.-no.: 13033N1B

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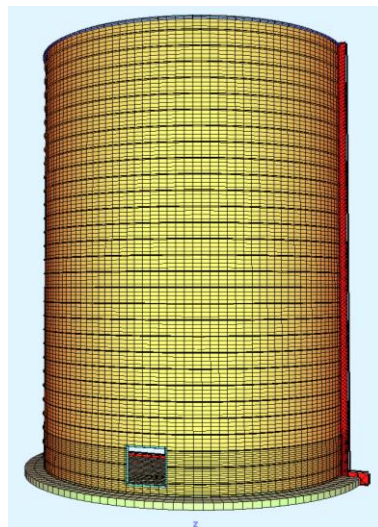
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STATICAL CALCULATION



- Subject:** Construction of a spiral folded steel tank with a membrane roof and a floor slab
- Tank designation:** Storage tank
- Site:** Vermont Technical College
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- Client:** Lipp GmbH
Industriestraße 27 in 73497 Tannhausen
- Engineer:** E H S beratende Ingenieure für Bauwesen GmbH
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Job no.: 13033N1B **Project:** Construction of a spiral folded steel tank with a membrane roof and a floor slab **Position:** Revisions **Date:** 06.08.2013 **Page:** II.1

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06.08.2013	I.1, II.1, III.1 – 3, 1 – 21, V.1	PDF, 1x	-



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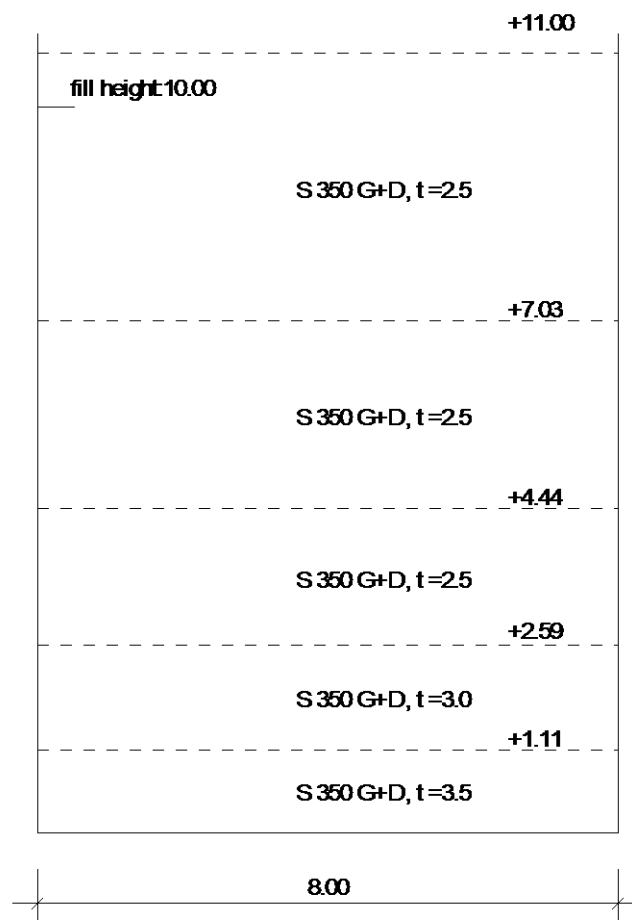
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1 Overview

1.1 Plate thickness and material

Upper edge: Membrane roof with 2x Standard Lipp profile



Picture 1: Plate thickness and material

Anchors:

IPE 80, a = 350 mm (plus: in man hole one additional anchor)

Plates between anchors: 300 x 50 x 10 – S 235 (with tack welds)

Machine:

t ≤ 3.0 mm, then SM 30, else SM 40 (up to max. t = 4.0 mm)



1.2 Load combinations

Only decisive load combinations are considered. Due to the range of experience of the consulting engineer elastic buckling due to wind loads can be treated as neglectable. Also, due to earth quake, statical plastic buckling does not need to be considered. It is important to have the buckling of the upper edge checked since the membrane roof induces higher normal forces into the Lipp rolled profile. When the LBA/MNA combination is successfully proved the tank automatically is not subjected to elastic global buckling.

LComb	factors
<u>Static material</u>	
1000	1.20 LC1 + 1.60 LC4 + 0.50 LC3
1100	1.20 LC1 + 1.00 LC3 + 1.00 LC10 + 0.50 LC4
<u>Earthquake</u>	
2000 to 2900	1.00 LC1 + 1.00 LC1010 + 0.20 LC4

1.3 Collapse load factors of MNA and MNA+LBA analysis

LComb	max. LC	Load factor	Buckling LC	Eigenvalue
1000	1005	1.52	1051	3.70
1100	1106	2.56	1171	10.92

1.4 Collapse load factors of elastic-plastic earthquake buckling analysis (GMNIA)

LComb	Imp. LC	w _k	Max. LC	Load factor
2900	none	none	2906	1.656
2100	2906	0.0116	2106	1.594
2200	2011	0.0116	2205	1.625
2300	100	0.0116	2305	1.094
2400	101	0.0116	2406	1.148
2500	102	0.0116	2505	1.625
2600	103	0.0116	2608	1.100



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2 Specifications

With this static and earthquake analysis a steel tank of the company Lipp is proved. The tank is covered by a membrane roof. The tank is made of Verinox, which is a structural steel plated with stainless steel. The structural steel is used as S 235 and S 355, the stainless steel has the material number 1.4571. The wall thickness can be varied from 2.0 mm up to 4.0 mm. At the bottom the tank is anchored in a floor slab of 25 cm thickness with steel profiles.

For purposes of foundation engineering a bedding modulus of 8000 kN/m³ (tangential 800 kN/m³) is assumed. The ground pressure resulting from the loads should not exceed 150 kN/m².

These assumptions have to be carefully checked on site and have to be confirmed before the commencement of the construction.

Tank geometry:	diameter D:	8.00 m
	tank height H:	11.00 m (max fill height H _f = 10.00 m)
	stored medium:	slurry, $\gamma = 10 \text{ kN/m}^3$
Loads:	snow	→ $s = 50 \text{ psf} = 2.40 \text{ kN/m}^2$
	wind	→ $v = 115 \text{ mph}$, $w = 1.63 \text{ kN/m}^2$
	earthquake	→ PGA: $a_g = 0.147 \text{ g} = 1.44 \text{ m/s}^2$
	risk category	→ II (importance factors are all 1.00)
Material properties:	Steel S 235 (t ≤ 40 mm)	yield strength = 235 N/mm ² tensile strength = 360 N/mm ²
	Steel S 355 (t ≤ 40 mm)	yield strength = 355 N/mm ² tensile strength = 490 N/mm ²
	Steel S 500SB	yield strength = 500 N/mm ² max. strain = 50 ‰
	Concrete C 25/30	max. cylindrical compression = 25 N/mm ² max. cubic compression = 30 N/mm ²

3 Literature

- ECCS: "Buckling of Steel Shells, European Design Recommendations" (5th Edition)
- Dr. Stefan Wirth: "Beulsicherheitsnachweise für schalenförmige Bauteile nach EN 1993-1-6: Kritische Analyse der praktischen Anwendbarkeit anhand zweier Fallstudien mit experimentellem Hintergrund"
- Ernst und Sohn: "Der Stahlbau" Heft 6, Jahrgang 2006 (S. 409)
- Gehring, Hans: "Vereinfachte Bemessung flüssigkeitsgefüllter, verankerter Kreiszyinderschalen unter Erdbebenbelastung"
- Meskouris, Konstantin: "Baudynamik", chapter 9
- Meskouris, Hinzen, Butenweg, Mistler: „Bauwerke und Erdbeben“, 3rd Edition (2011), chapter 7
- "Stahlbaukalender 2009" Kapitel 5: Stabilität stählerner Schalentragwerke
- "Stahlbaukalender 2009" Kapitel 6: Einwirkungen auf Silos aus Metallwerkstoffen
- Schneider: "Bautabellen", 18. Auflage
- B. Eng. Andreas Jäger: "Tragfähigkeitsuntersuchungen an spiralgefalteten Flüssigkeitsbehältern aus Stahl"
- ASCE/SEI: „Minimum Design Loads for Buildings and Other Structures“, ASCE (2010)

4 Standards

- DIN EN 1991-1-4: "Einwirkungen auf Tragwerke: Teil 1-4: Allgemeine Einwirkungen, Windlasten"
- DIN EN 1993-1-6: "Bemessung und Konstruktion von Stahlbauten – Teil 1-6: Festigkeit und Stabilität von Schalen"
- DIN EN 1993-4-1: "Bemessung und Konstruktion von Stahlbauten –Teil 4-1: Silos"
- DIN EN 1993-4-2: "Bemessung und Konstruktion von Stahlbauten – Teil 4-2: Tankbauwerke"
- DIN EN 1993-3-2: "Bemessung und Konstruktion von Stahlbauten –Teil 3-2: Türme, Maste und Schornsteine – Schornsteine"
- DIN EN 1998-1: "Auslegung von Bauwerken gegen Erdbeben - Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für Hochbauten"
- DIN EN 1998-4: "Auslegung von Bauwerken gegen Erdbeben - Teil 4: Silos, Tankbauwerke und Rohrleitungen"

5 Programs

Sofistik 2012 versions, servicepack 06/2012

6 Documents by Lipp

- general type approval no. Z-14.3-15
- statical calculation for the man hole from 09/2007 (by Benz GmbH (No. 07092))

7 Legal notice

It is the responsibility of the construction company to make sure that all materials used have the correct strength and quality. It is claimed that construction stages that do not need analytical investigations have to be carefully considered by the construction company as well. Material quality has to be conforming to common Eurocode or DIN standards.

8 Membrane roof

8.1 Loads on the membrane roof

8.1.1 Permanent loads

LC 12: Dead weight

The dead weight of the membrane is considered by the software.

$g = 1.00 \text{ kN/m}^2$ (assumed additional load for insulation and gravel)

$P = 1.00 \text{ kN/m}$ (pre-stress in X and Y direction)

prescribed deflection in middle of membrane: $f = 250 \text{ mm}$

8.1.2 Varying loads

LC 2: Snow

ground snow load: $p_g = 50 \text{ psf}$ Exposure factor: $C_e \leq 1.00$

Thermal factor: $C_t = 1.20$ Roof slope factor: $C_s = 0.75$

Flat roof snow load: $p_f = 0.70 * C_e * C_t * I_s * p_g = 42.0 \text{ psf}$

Sloped roof snow load: $p_s = C_s * p_f = 31.5 \text{ psf} = 1.51 \text{ kN/m}^2$

LC 3: Wind pressure

Basic wind speed: $V_0 = 115 \text{ mph} = 51 \text{ m/s}$ Directionality factor: $K_d = 0.95$

Topographic factor: $K_{zt} = (1 + k_1 * k_2 * k_3)^2 = (1 + 0.21 * 1.0 * 1.0)^2 = 1.46$ (Expos. class C)

Height level factor: $K_z = 2.01 * (11.0 / 274.32)^{2/9.5} = 1.02$ (top of roof, Exposure class C)

Velocity pressure: $q_z = 0.613 * K_z * K_{zt} * K_d * V_0^2 = 2256 \text{ N/m}^2 = 2.26 \text{ kN/m}^2$

Pressure factor: $\max q_w = 1.45 * q_z$

LC 5: Maximum pressure

$q_{k,max} = 0.30 \text{ kN/m}^2$

LC 6: Minimum pressure

$q_{k,min} = -0.30 \text{ kN/m}^2$

8.1.3 Earthquake loads

Since the roof's dead weight is almost neglectable, earthquake is not an issue for the roof design.

8.1.4 Load combinations

- LComb 11: $1.20 LC1 + 1.20 LC2 + 0.50 LC3$
LComb 12: $1.20 LC1 + 1.60 LC2 + 1.00 LC5$
LComb 13: $1.20 LC1 + 1.00 LC3 + 1.00 LC5 + 0.50 LC2$
LComb 14: $1.00 LC1 + 1.50 LC5 + 0.50 LC2$

9 Membrane

9.1 Model and calculation

The membrane is modeled as a shell structure (no bending possible → membrane elements) with a thickness of 0.8 mm and stainless steel material,

The analysis is carried out using the following steps:

1. System generation and load generation,
2. Form finding with zero stiffness (load = dead weight),
3. Compensation with full stiffness (load = dead weight),
4. System update → Save coordinates from form finding and use it as new system coordinates,
5. Geometric and material nonlinear calculation of the load cases.

9.2 Result

The maximum derived tension is 91 N/mm² (LComb 12).

This is smaller than the design strength of 214 N/mm².

10 Tank

10.1.1 Permanent loads

LC 1: Dead weight

The dead weight of the tank is considered by the software.

A load of 0.30 kN/m² is assumed for the weights of insulation and mixers connected to the tank.

Bearing forces from the roof are taken into account (LC 1).

10.1.2 Variable loads

LC 2: Hydrostatic load

$$h_{f,max} = 10.00 \text{ m} \quad \gamma = 10 \text{ kN/m}^3$$

$$q_{max} = 10.0 \text{ kN/m}^2 \text{ (at top of floor slab)}$$

LC 3: Wind from +X, tanks in seriation

$$\text{Height level factor: } K_z = 2.01 * (11.0 / 274.32)^{2/9.5} = 1.02 \text{ (top of tank, Exposure class C)}$$

$$\text{Velocity pressure: } q_z = 0.613 * K_z * K_{zt} * K_d * V_0^2 = 2256 \text{ N/m}^2 = 2.26 \text{ kN/m}^2$$

Bearing forces from the roof are taken into account (LC 3). The loads are shown in LC7 of the tank.

See annex A for detailed information of the load distribution.

LC 4: Snow load

Bearing forces from the roof are taken into account (LC 2).

LC 9: Minimum pressure

$$q_{k,min} = -0.30 \text{ kN/m}^2$$

Bearing forces from the roof are taken into account (LC 6).

LC 10: Maximum pressure

$$q_{k,max} = 0.30 \text{ kN/m}^2$$

Bearing forces from the roof are taken into account (LC 5).

Temperature loads

LC 11: Constant warming of 28 K

LC 12: Constant cooling of 28 K

LC 13: Circumferential distributed warming of 10 K



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10.1.3 Earthquake loads

Earthquake loads are determined according to the procedure recommended by Meskouris in 2011 (see point 3). This procedure is close to the Eurocode but differs in details. Since this is a deviation from the Eurocode a second check based on the procedure developed earlier by Meskouris is performed.

The ground acceleration parameters are derived using the appropriate design codes of the USA.

Parameters:

$$\begin{aligned}
 \text{site class: } & C \quad \rightarrow \quad \text{PGA} = 1.20 * 0.122 = 0.147 \text{ g} = 1.44 \text{ m/s}^2 \\
 S_s = 0.246 \text{ g} \quad F_a = 1.20 \quad & S_{MS} = 0.246 \text{ g} * 1.20 = 0.296 \text{ g} = 2.94 \text{ m/s}^2 \\
 S_1 = 0.087 \text{ g} \quad F_v = 1.70 \quad & S_{M1} = 0.087 \text{ g} * 1.70 = 0.147 \text{ g} = 1.45 \text{ m/s}^2 \\
 T_0 = 0.10 \quad S_{M1} / S_{MS} = 0.500 \quad & T_s = S_{M1} / S_{MS} = 0.497 \text{ s} \quad T_L = 6.000 \text{ s}
 \end{aligned}$$

These values are used for the earthquake analysis without further reduction because the plastic behavior of the structure is taken into account while performing the analysis.

These values are used for the earthquake analysis without further reduction because the plastic behavior of the structure is taken into account while performing the analysis.

The following values of the base shear are calculated:

Subject	Meskouris 2011	DIN EN 1998-4, A 3.3.2
Base shear Q	1253 kN	1242 kN

Table 1: Loads due to earthquake compared between Meskouris 2011 and EC8-4

$$C_1 = 6.56 \quad k_i = M_i / M = 0.81 \quad C_{con} = 1.48 \quad k_c = M_c / M = 1 - (M_i / M)$$

Base shear force according to EC 8-4, annex A 3.3.2:

design factor: $q_0 = 1.0$ (plastic analysis)

$$H/R = 10.00 / 4.00 = 2.50$$

$$M = \pi * R^2 * H * \rho = 502 \text{ t}$$

$$T_{imp} = C_1 * \rho^{0.5} * H / ((t_{mean} / R)^{0.5} * E^{0.5}) = 0.17 \text{ s}$$

$$S_{e,imp} = S_{MS}$$

$$Q_{imp,d} = S_{e,imp} * (k_i * M) = 1195 \text{ kN}$$

$$T_{con} = C_{con} * R^{0.5} = 2.96 \text{ s}$$

$$S_{e,con} = S_{M1} / T = 0.49 \text{ m/s}^2$$

$$Q_{con,d} = S_{e,con} * (k_c * M) = 47 \text{ kN}$$

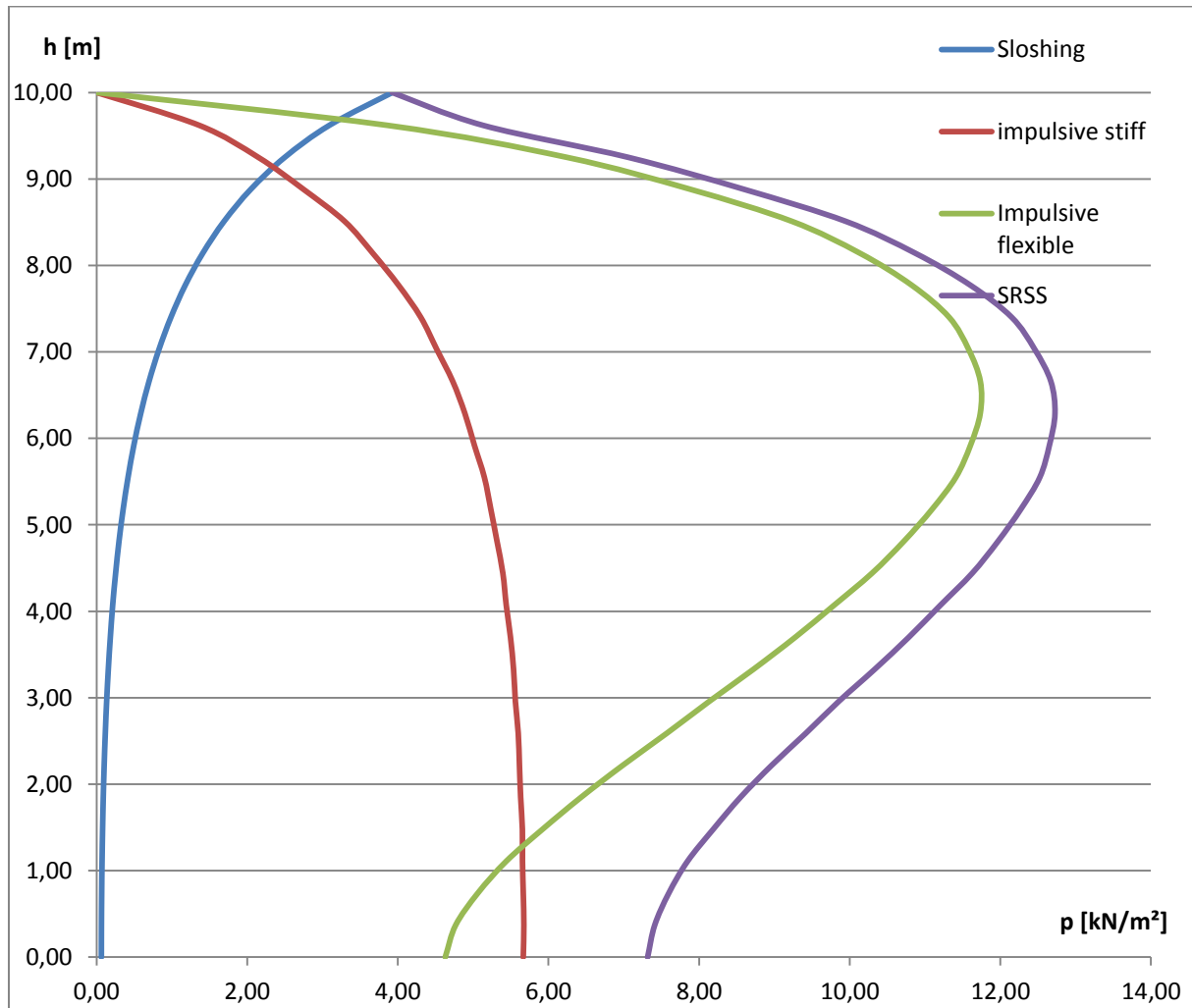
$$Q_d = Q_{con,d} + Q_{imp,d} = 1242 \text{ kN}$$



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Comparing the results of both procedures the values calculated based on Meskouris' 2011 procedure are a good estimate.

All three load distributions (sloshing mode, impulsive mode 1, impulsive mode 2 and the design pressure calculated with the SRSS-rule) are illustrated below based on $a_{g,d}$.



Picture 2: Design load distribution p_0 (SRSS) (h-p-diagram)



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fill height	sloshing	impulsive rigid	impulsive flexible	SRSS
0.000	0.064	5.666	4.630	7.317
0.370	0.065	5.674	4.768	7.411
0.740	0.068	5.666	5.062	7.598
1.110	0.073	5.655	5.441	7.848
1.480	0.080	5.654	5.923	8.189
1.850	0.089	5.631	6.435	8.551
2.220	0.101	5.615	6.989	8.966
2.590	0.116	5.599	7.573	9.419
2.960	0.134	5.561	8.139	9.858
3.330	0.156	5.537	8.723	10.333
3.700	0.183	5.496	9.278	10.785
4.070	0.214	5.440	9.793	11.204
4.440	0.253	5.395	10.296	11.627
4.810	0.298	5.317	10.720	11.970
5.180	0.353	5.235	11.094	12.272
5.550	0.418	5.149	11.410	12.525
5.920	0.495	5.017	11.604	12.652
6.290	0.587	4.890	11.734	12.726
6.660	0.697	4.729	11.735	12.671
7.030	0.829	4.517	11.579	12.457
7.400	0.987	4.305	11.321	12.152
7.770	1.177	4.011	10.826	11.605
8.140	1.409	3.669	10.139	10.874
8.510	1.693	3.294	9.233	9.948
8.880	2.047	2.748	7.886	8.598
9.250	2.498	2.148	6.246	7.061
9.620	3.090	1.365	3.865	5.133
10.000	3.927	0.000	0.000	3.927

Table 2: Derived pressures due to earthquake



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The circumferential distribution of these loads is assumed to be $p = p_0 \cdot \cos \theta$. The pressure is the pressure occurring in tank layer ($\xi = r/R = 1$).

According to EC8-4 the design pressure is applied in a way that the difference between the hydrostatic load and the earthquake load in every point is greater than 0 kN/m² (the earthquake load doesn't imply compressional circumferential membrane forces into the tank wall). The redistribution of the earthquake load is carried out iteratively regarding to the total base shear force being nearly the same as calculated with Meskouris' 2011 procedure (see table 1).

– Generated load cases

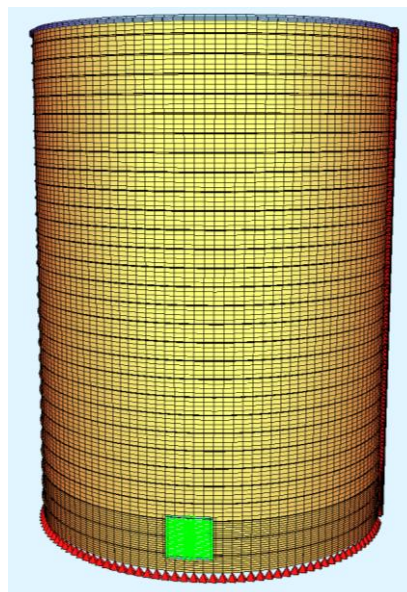
LC 1000	redistributed earthquake pressure (according to picture 2)
LC 1001	earthquake load resulting from the dead weight of the tank
LC 1004	reference earthquake pressure distribution (according to picture 2)
LC 1010	redistributed earthquake pressure + earthquake load from tank dead weight

Wind loads do not have to be considered together with the earthquake load ($\Psi_{2,Wind} = 0$).

10.1.4 Load combinations

A summary of the load combinations used for buckling checks is given within point 1.

10.2 Model



Picture 3: Overview of the model

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The tank is modeled as a whole system including the stepping of the wall thicknesses according to point 1. Load and geometrical symmetry are taken into account.

Despite the approval, the seams are thought to take tension and compression resulting from different load conditions according to their cross-section area of approximately 144t. This is proved by several German research studies including the doctoral thesis of Dr. Stefan Wirth.

The angle of the seam is neglected. The model is generated in sections with a height of 370 mm each (Machine SM40). The dimensions of the FEA-model elements are taken as $l/h = 2/1$ with 4 elements for each section ($l = 18.5$ mm, $h = 9.3$ mm). The bottom part (first plate thickness section) is modeled with $h \leq 4.7$ mm and $l/h > 2$. The bottom edge of the tank is assumed not to be displaceable and fully jointed (for earthquake analysis the floor slab is taken into account within the FEM-model discretely). The upper edge is elastically prevented from moving and fully jointed.

Cutouts such as the manhole are part of the model when they are at least for nozzles greater than $d = 250$ mm. All smaller nozzles have a negligible effect on the structural behavior of the shell. The man hole is modeled with springs (horizontal stiffness $CP = 2.1 \cdot 10^8 \cdot 0.012 = 2.52 \cdot 10^6$ kN/m; vertical stiffness $CQ = CP/10$ (assumed); yielding load $F_y = 1171$ kN * t / 4mm * $f_{y,Rk} / 360$ [kN/m]).

10.3 Procedure

The decisive load combinations are calculated based on the recommendations suggested by the ASCE book, chapter 2.

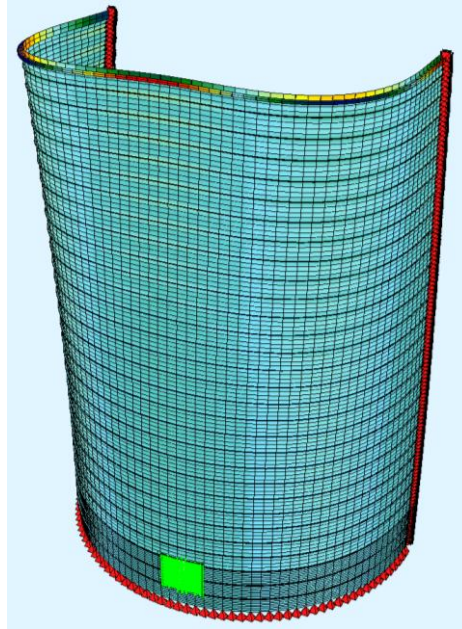
For this tank the limit states LS2 (cyclic plastification) and LS4 (fatigue) are not decisive and therefore not further investigated.

All determined limit load factors are summarized in point 1.

10.3.1 Limit state LS 3: Buckling

a) Buckling of the upper edge

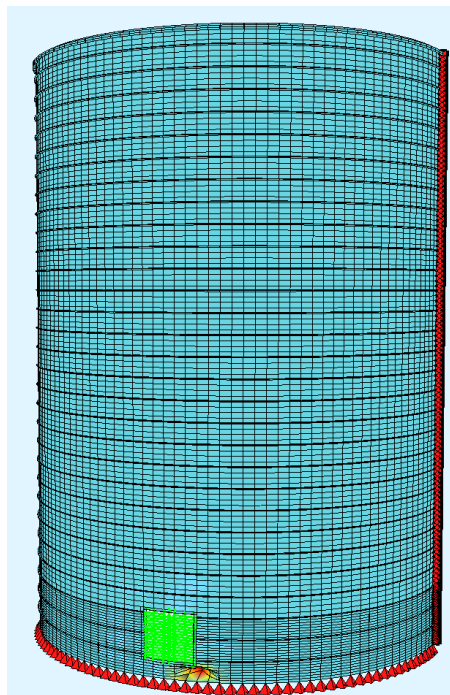
The upper edge is no subject to buckling since the roof induces tensional forces into the ring at the top of tank. If there is any kind of buckling to consider, the buckling calculations of the tank will automatically consider this. Local bending moments are proved by the nonlinear calculation of the load combinations as well.



Picture 4: 1st Eigenform for LComb 1000 (LC1051), $r_{Rcr} = 3.70$

Parameters: $\bar{\lambda}_0 = 0.40$ $\beta = 0.60$ $\eta = 1.00$ $Q = 16$ $\alpha = 0.50$

LK1000: $r_{Rpl,d} = 1.52$ $\lambda_\theta = (r_{Rpl,d} / r_{Rcr,d})^{0.5} = 0.64 < 1.12 \rightarrow$ partly plastical buckling
 $\chi_p = 1 - \beta ((\lambda_\theta - \lambda_{\theta,0}) / (\lambda_p - \lambda_{\theta,0}))^\eta = 0.80$ $r_{Rd} = r_{Rpl,d} * \chi = 1.22 > 1$



Picture 5: 1st Eigenform for LComb 1100 (LC1151), $r_{Rcr} = 3.15$



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b) Buckling of the cylindrical shell

The buckling proof of the cylindrical shell is carried out using imperfections:

Decisive measuring length $l_{g,x} = 4 \sqrt{rt} = 4 * \sqrt{(4000 * 3.5)} = 473 \text{ mm}$

Decisive measuring length $l_{g,\theta} = 2.3 (l^2 * r * t)^{0.25} = 2.3 * (11000^2 * 4000 * 3.5)^{0.25} = 2525 \text{ mm}$

According to DIN EN 1993-1-6, eq. 8.29/30 the amplitude of the imperfection $\Delta w_{0,eq}$ is:

$$\Delta w_{eq,x} = \max (l_{g,x} * 0.025 ; 4 * 25 * 0.025) = 11.83 \text{ mm}$$

$$\Delta w_{eq,\theta} = \max (l_{g,\theta} * 0.025 ; 4 * 25 * 0.025) = 63.12 \text{ mm}$$

This amplitude is assumed to be situated between two seams. The maximum difference between the perfect and the imperfect structure depends on the buckling mode. If there is a global buckling mode the amplitude is thought to be $\Delta w_{eq,\theta}$ else $\Delta w_{eq,x}$.

The buckling proof is accomplished by a geometrically and physically non-linear calculation applied on the imperfect system (GMNIA). The derived load factor is the limit load factor. If it is above 1.0 the system is capable of bearing the applied loads. The material strength f_{yd} is assumed to be $f_{yk} / 1.10$, when earthquake is taken into account $f_{yd} = f_{yk}$.

Local plastic buckling (elastic-plastic ring buckling = Elephant's foot buckling) is investigated for the case of an earthquake.

10.3.2 Limit state of Serviceability

Serviceability load cases are calculated material and geometrically nonlinear.

LComb 998 → LC 998: GMNA 1.00 LC1 + 1.00 LC 2

LComb 999 → LC 999: GMNA 1.00 LC1 + 1.00 LC 3

10.4 Results

Decisive proof of buckling (static): $r_{Rd} = 1.21 > 1.0$ (LComb 1000)

Decisive proof of buckling (earthquake): $r_{Rd} = 1.09 > 1.0$ (LComb 2300)

The results of the calculations can be seen in annex D (LS 3: LComb 1000) and annex E (LS 3: LComb 2300).

11 Structural details

11.1 Cutouts

Cutouts have to be carried out as given by the construction plans of Lipp. If the force derived from the integration of the van-Mise-stress exceeds the limit of the nozzle construction (static proof deliverable by Lipp) the responsible structural engineer shall be informed in order to get further instructions on how to solve this problem.

These forces can be taken from annex D.

11.2 Coil joints

All coil joints have to be produced according to the construction plans of Lipp (refer to annex 2). Between two coils there has to be a butt weld (HV weld). If steel type S 235 is welded with S 355 the weld has to have the strength of the S 355 material. In any other case, the weld strength has to be the same as the one of the plates welded.

The joint plates have to be at least the same thickness the tank itself has. 21 bolts, M8-8.8 situated as shown in annex 2 make sure that the forces can be completely transferred.

11.3 Connection of the membrane with the tank

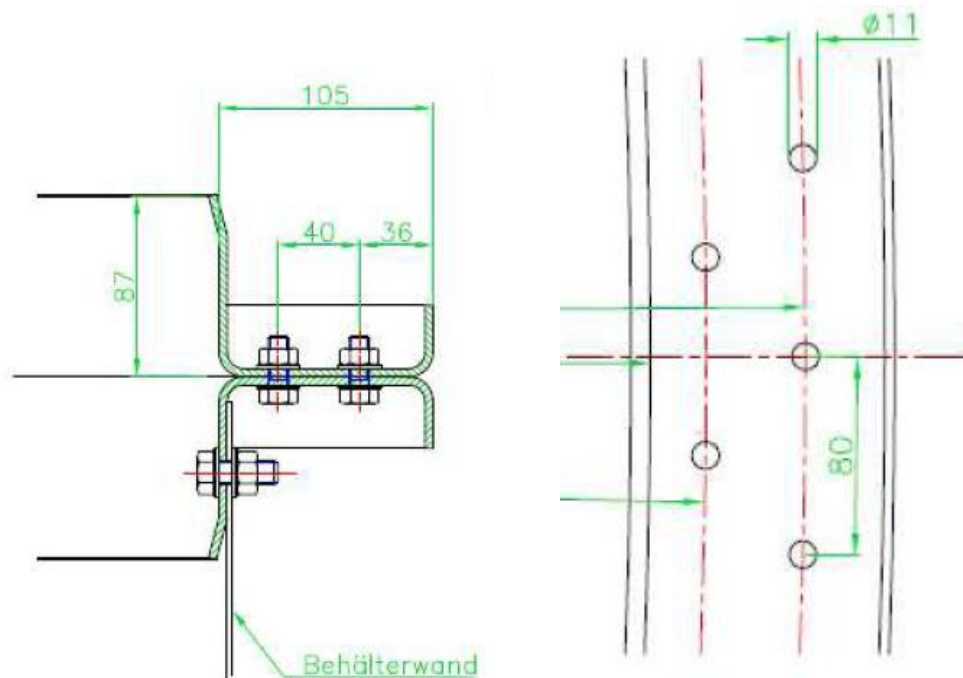


Bild 1: Sketch of the connection

decisive: LC12 of the mebrane

membrane forces: $H_d = 65 \text{ kN/m}$

chosen bolts: M 12-8.8 (tensile strength $f_u = 800 \text{ N/mm}^2$), distance less than 100 mm

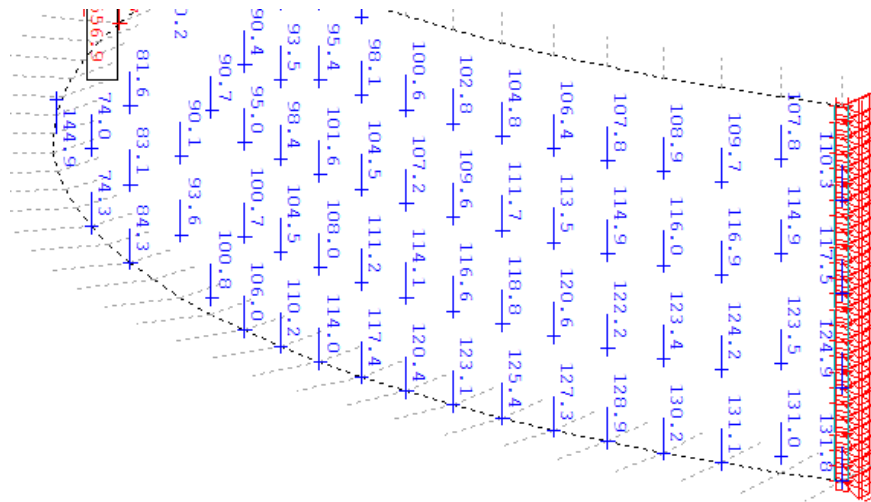
Pre-tension of bolt: 47.2 kN $\rightarrow F_{s,R,d} = 1.0 * 0.2 * 47.2 / 1.25 = 7.55 \text{ kN}$

minimum distance off he bolts: $a_{cal} \leq 1 / (H_d / F_{s,R,d}) = 0.11 \text{ m}$

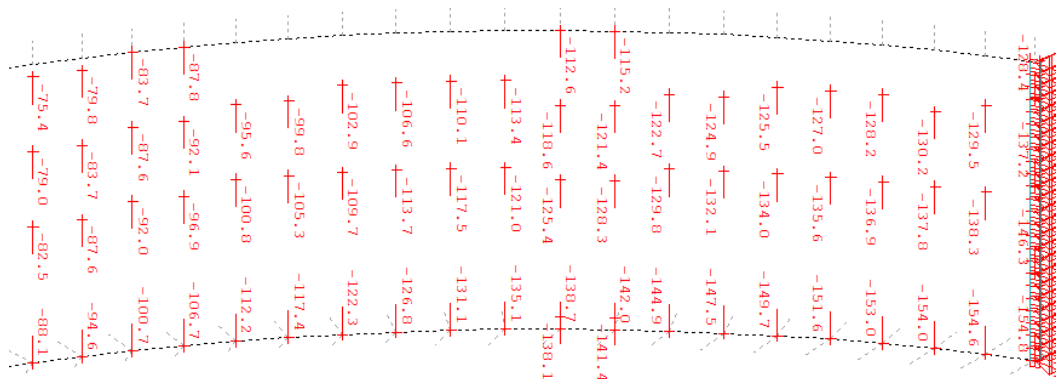
H_d is directly beared by the upper edge ring. No torsional moment needs to be considered.

11.4 Connection of the tank to the floor slab

According to EC 8-4, 3.5.2.3 the anchors are designed with an additional safety of 25%.



Picture 6: Derived lift off forces due to earthquake



Picture 7: Derived compressional forces due to earthquake

Load combination for maximum lift off forces (earthquake): LC1010

Maximum membrane force N_{yy} : $F_{max} = 1.25 * 131.1 = 164 \text{ kN/m}$

Minimum membrane force N_{yy} : $F_{max} = -155 * 2/3 = 103 \text{ kN/m}$



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The membrane forces are taken from a linear analysis. Hence the design spectrum may be used, which means that the derived value may be multiplied with 2/3. However, due to ductility reasons, for the anchor design the original spectrum is used to determine the dimensions.

c) resistances

weld joint as both-sided 3mm-fillet weld, strength S 235:

$$l_{aw} = 2 * 46 \text{ mm} = 92 \text{ mm} \quad A_w = 92 * 3 = 276 \text{ mm}^2$$

$$V_{w,Rd} = 276 * (360 / \sqrt{3} * 0.80 * 1.25) = 57365 \text{ N}$$

$$\text{Bearing capacity of the tank: } F_{y,Rd} = (46 + 3 * 2.5 * 2) * 3.0 * 355 / 1.10 = 59059 \text{ N}$$

$$\text{Capacity of the hoops } \varnothing 8: \quad N_{Rd} = 4 * 0.50 * 43.48 = 86.96 \text{ kN}$$

Anchoring of the lift-off forces in the floor slab via compound and compression of the lower flange:

$$\text{info:} \quad N_{Rd,comp} = 4 * 12 * 3.14 * 2.7 * 150 = 122145 \text{ N} \quad (\varnothing 12, l_{compound} \approx 150 \text{ mm})$$

$$\text{chosen:} \quad N_{Rd,comp} = 4 * 8 * 3.14 * 2.7 * 150 = 40694 \text{ N} (\varnothing 8, l_{compound} \approx 150 \text{ mm})$$

$$(\text{compression at lower flange:} \quad N_{Rd,comp} = 200 * (46 - 4) * 16.7 = 140200 \text{ N})$$

Forces from the tank can be beared by the anchors with max. 140 kN.

– capacity of the tank wall: $F_{y,Rd} = (b_{IPE} + a_w * 2.5 * 2) * t * f_{y,k} / 1.10$
with $t \geq 3.0 \text{ mm} \rightarrow F_{y,Rd} = 58.23 \text{ kN}$

$$\text{compression of concrete:} \quad f_{ck} = 25 / 1.50 = 16.67 \text{ N/mm}^2$$

$$\text{required area:} \quad A_{req} = 103300 / 16.67 = 6197 \text{ mm}^2 \quad b \geq 6.2 \text{ mm}$$

$$\text{chosen:} \quad \text{plates with dimensions } 300 \times 50 \times 10 - \text{S } 235$$

Connect the steel plates with tack weldings.

d) anchor type and distance

$$\text{max } a = 57.37 / 164 = 0.35 \text{ m} \quad (+1 \text{ additional anchor in the man hole area})$$

$$\text{Chosen anchor distance:} \quad a = 0.35 \text{ m} \rightarrow \text{Amount: } 84+1 \times \text{IPE } 80$$

Fit the steel plates between the anchors ($l_{Steelplate} \approx 300 \text{ mm}$).

12 Floor slab

12.1 Loads on the floor slab

12.1.1 Permanent load

LC 1: Dead weight

$$g_{\text{slab}} = 0.25 * 25 = 6.25 \text{ kN/m}^2$$

Bearing forces of LC1 are taken over from the tank (see annex B).

12.1.2 Variable Loads

LC 2: Hydrostatic load

$$q_u = 100.0 \text{ kN/m}^2 \quad (\text{at top of the floor slab})$$

Bearing forces of LC 2 are taken over from the tank (see annex B).

LC 3: Wind from +X

Bearing forces of LC 3 are taken over from the tank (see annex B).

Temperature loads

Bearing forces of LC 11...13 of are taken over from the tank (see annex B).

LC 11: Constant warming of 28 K

LC 12: Constant cooling of 28 K

LC 13: Circumferential distributed warming of 10 K

12.1.3 Earthquake loads

The load combination LComb 2900: 1.00 LC1 + 1.00 LC1010 + 0.20 LC4 is considered.

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12.2 Parameters

Exposition: XC 4

Chosen: concrete C 25/30 steel S 500 $c_v = 4.0 \text{ mm}$ $w_{k,max} = 0.2 \text{ mm}$

12.3 Required minimum reinforcement due to limitation of crack widths

$d_s = 9 \text{ mm}$ $k_c = 1.0$ $k = 0.8$ $f_{ct,eff} = 0.26 / 2 = 0.13 \text{ kN/cm}^2$

$\sigma_{sd} = (w_k * 3.48 * 10^6 / d_s)^{0.5} = 278 \text{ N/mm}^2$

req. $A_s = k_c * k * f_{ct,eff} * A_{ct} / \sigma_{sd} = 0.80 * 1.00 * 0.13 * 1250 / 27.8 = 4.68 \text{ cm}^2/\text{m}$

12.4 Minimum reinforcement due to ductility

$W = b h^2 / 6 = 100 * 25^2 / 6 = 10417 \text{ cm}^3$

$M_{cr} = 0.26 * 10417 = 2708 \text{ kNcm}$

$A_{s,min,dukt} = 2708 / (0.90 * 20.5 * 50) = 2.94 \text{ cm}^2/\text{m} < A_{s,min,Hydratation}$

Chosen: area reinforcement → Q 424A or eq. (top and bottom)
bar reinforcement (edge region) → Ø12/15

Seasoning of the concrete shall be carried out very carefully in order to prevent cracks. The use of foils on top of the base course is prescribed to prevent cracks in the young concrete age.

12.5 Reinforcement due to static loads

Design cases:

LCB1 → Statically required reinforcement

LCB2 → Crack width limiting reinforcement

LCB4 → Ductility reinforcement

LCB10 → Maximum required reinforcement of LCB 1, 2, 4

The reinforcement derived within point 12.3 is adequate.

Refer to annex F for further details.

12.6 Reinforcement due to Earthquake

The reinforcement derived within point 12.3 is adequate.