EHS

EHS beratende Ingenieure für Bauwesen GmbH

job.-no.: 13033N1C

page: I

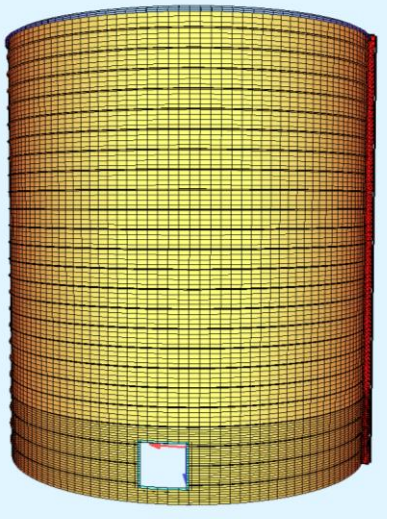
Am Alten Rathaus 5 · 34253 Lohfelden
Telefon: +49 561 95088-0 ·
Telefax: +49 561 95088-99
kontakt@ehs-ingenieure.de ·
www.ehs-ingenieure.de

Amtsgericht Kassel: HRB 3800

Geschäftsführer:
Dr.-Ing. Hans-Heinrich Osteroth
Dipl.-Ing. Guido Göttlicher
Dr.-Ing. Björn Schmidt-Hurtienne
Dr.-Ing. Dirk Sperling

Prokuristen:
Dr.-Ing. Katja Abel
Dipl.-Ing. Heiko Rausch

STATICAL CALCULATION



Subject: Construction of a spiral folded steel tank with a membrane roof and a floor slab

Tank designation: Hydrolyse tank

Site: Vermont Technical College
124 Main Street in 05061 Randolph, Vermont, USA

Client: Lipp GmbH
Industriestraße 27 in 73497 Tannhausen

Engineer: E H S beratende Ingenieure für Bauwesen GmbH
Am Alten Rathaus 5, 34253 Lohfelden

Document revisions			
Date of dispatch	Pages	Lipp GmbH	Checking engineer
06.08.2013	I.1, II.1, III.1 – 3, 1 – 20, V.1	PDF, 1x	-

I Table of contents

II List of annexes (program input and results)

III List of annexes (structural fundamentals)

All annexes are provided on demand. Plans of the tank are offered by Lipp.

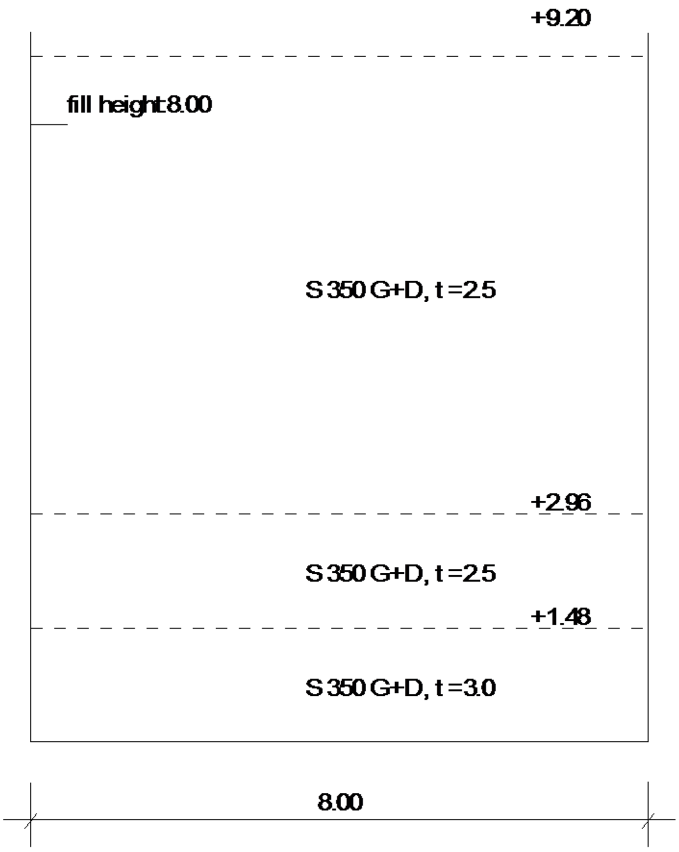
IV List of pictures

V List of tables

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)

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
Static material	
1000	1.20 LC1 + 1.60 LC4 + 0.50 LC3
1100	1.20 LC1 + 1.00 LC3 + 1.00 LC10 + 0.50 LC4
Earthquake	
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	1.38	1051	3.76
1100	1106	2.75	1171	12.64

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	1.922
2100	2907	0.0110	2106	1.828
2200	2011	0.0110	2205	1.828
2300	100	0.0110	2305	1.500
2400	101	0.0110	2406	1.375
2500	102	0.0110	2505	2.312
2600	103	0.0110	2608	1.400

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 confirmed before the commencement of the construction.

Tank geometry:	diameter D:	8.00 m
	tank height H:	9.20 m (max fill height H_f = 8.00 m)
	stored medium:	slurry, γ = 10 kN/m³
Loads:	snow	→ s = 50 psf = 2.40 kN/m²
	wind	→ v = 115 mph, w = 1.63 kN/m²
	earthquake	→ PGA: a_g = 0.147 g = 1.44 m/s²
	risk category	→II (importance factors are all 1.00)
Material properties:	Steel S 235	yield strength = 235 N/mm²
	(t ≤ 40 mm)	tensile strength = 360 N/mm²
	Steel S 355	yield strength = 355 N/mm²
	(t ≤ 40 mm)	tensile strength = 490 N/mm²
	Steel S 500SB	yield strength = 500 N/mm²
		max. strain = 50 ‰
	Concrete C 25/30	max. cylindrical compression = 25 N/mm²
		max. cubic compression = 30 N/mm²

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 Kreiszylinderschalen 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 spiralgefalzten Flüssigkeits-behältern 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 kN/m² (assumed additional load for insulation and gravel)

P = 1.00 kN/m (pre-stress in X and Y direction)

prescribed deflection in middle of membrane: f = 250 mm

8.1.2 Varying loads

LC 2: Snow

ground snow load: p_g = 50 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 psf

Sloped roof snow load: $p_s = C_s * p_f$ = 31.5 psf = 1.51 kN/m²

LC 3: Wind pressure

Basic wind speed: V_0 = 115 mph = 51 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: $K_z = 2.01 * (9.2 / 274.32)^{2/9.5} = 0.98$ (top of roof,Exposure class C)

Velocity pressure: $q_z = 0.613 * K_z * K_{zt} * K_d * V_0^2$ = 2167 N/m² = 2.17 kN/m²

Pressure factor: max $q_w = 1.45 * q_z$

LC 5: Maximum pressure

$q_{k,max}$ = 0.30 kN/m²

LC 6: Minimum pressure

$q_{k,min}$ = -0.30 kN/m²

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

<u>LC 1: Dead weight</u>

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

<u>LC 2: Hydrostatic load</u>

$h_{f,max}$ = 10.00 m γ = 10 kN/m³

q_{max} = 10.0 kN/m² (at top of floor slab)

<u>LC 3: Wind from +X, tanks in seriation</u>

Height level factor: $K_z = 2.01 * (9.2 / 274.32)^{2/9.5} = 0.98$ (top of tank, Exposure class C)

Velocity pressure: $q_z = 0.613 * K_z * K_{zt} * K_d * V_0^2 = 2167$ N/m² = 2.17 kN/m²

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.

<u>LC 4: Snow load</u>

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

<u>LC 9: Minimum pressure</u>

$q_{k,min}$ = -0.30 kN/m²

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

<u>LC 10: Maximum pressure</u>

$q_{k,max}$ = 0.30 kN/m²

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

<u>Temperature loads</u>

LC 11: Constant warming of 28 K

LC 12: Constant cooling of 28 K

LC 13: Circumferential distributed warming of 10 K

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:

site class: C $\rightarrow$ PGA = 1.20 * 0.122 = 0.147 g = 1.44 m/s²

$S_s = 0.246$ g $F_a = 1.20$ $S_{MS} = 0.246$ g * 1.20 = 0.296 g = 2.94 m/s²

$S_1 = 0.087$ g $F_v = 1.70$ $S_{M1} = 0.087$ g * 1.70 = 0.147 g = 1.45 m/s²

$T_0 = 0.10$ $S_{M1} / S_{MS} = 0.500$ $T_S = S_{M1} / S_{MS} = 0.497$ s $T_L = 6.000$ s

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	930 kN	1038 kN

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

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

$C_1 = 6.21$ $k_i = M_i / M = 0.76$ $C_{con} = 1.48$ $k_c = M_c / M = 1 - (M_i / M)$

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

H/R = 10.00 / 4.00 = 2.00

$M = \pi * R^2 * H * \rho = 402$ t

$T_{imp} = C_1 * \rho^{0.5} * H / ((t_{mean} / R)^{0.5} * E^{0.5}) = 0.14$ s

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

$Q_{imp,d} = S_{e,imp} * (k_i * M) = 898$ kN

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

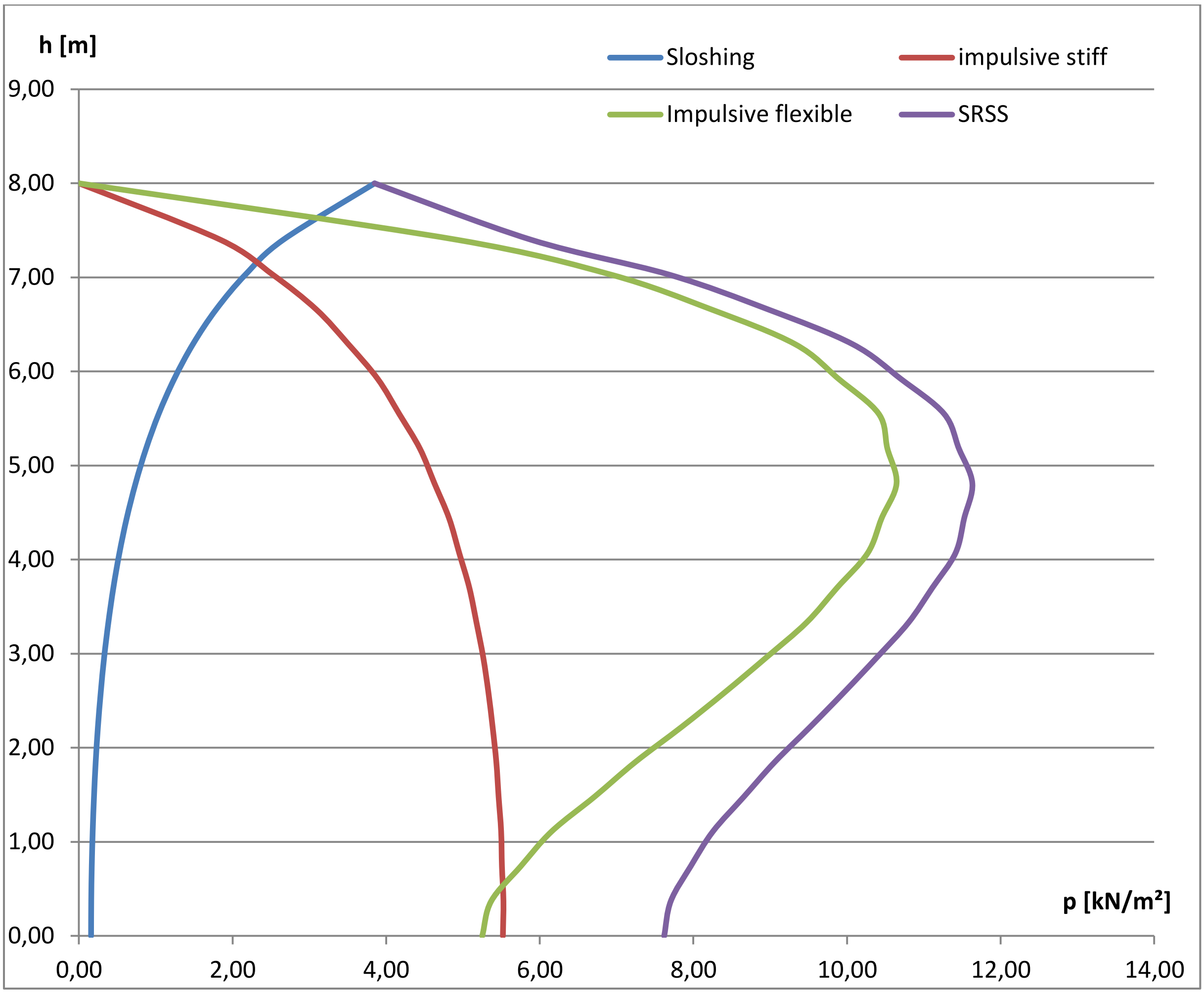
$S_{e,con} = S_{M1} / T = 0.49$ m/s²

$Q_{con,d} = S_{e,con} * (k_c * M) = 140$ kN

$Q_d = Q_{con,d} + Q_{imp,d} = 1038$ kN

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)

fill height	sloshing	impulsive rigid	impulsive flexible	SRSS
0.000	0.158	5.520	5.251	7.620
0.370	0.160	5.527	5.368	7.706
0.740	0.167	5.507	5.752	7.965
1.110	0.179	5.497	6.154	8.254
1.480	0.196	5.464	6.714	8.658
1.850	0.218	5.434	7.247	9.061
2.220	0.247	5.386	7.845	9.519
2.590	0.284	5.331	8.411	9.962
2.960	0.328	5.264	8.950	10.389
3.330	0.383	5.176	9.472	10.800
3.700	0.448	5.083	9.870	11.111
4.070	0.527	4.951	10.270	11.413
4.440	0.622	4.819	10.451	11.526
4.810	0.737	4.629	10.645	11.632
5.180	0.874	4.439	10.526	11.457
5.550	1.039	4.169	10.413	11.264
5.920	1.240	3.891	9.892	10.702
6.290	1.484	3.508	9.325	10.073
6.660	1.787	3.089	8.232	8.972
7.030	2.168	2.522	6.924	7.681
7.400	2.660	1.842	4.919	5.887
8.000	3.850	0.000	0.000	3.850

Table 2: Derived pressures due to earthquake

The circumferential distribution of these loads is assumed to be $p = p_0 * \cos\theta$. 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

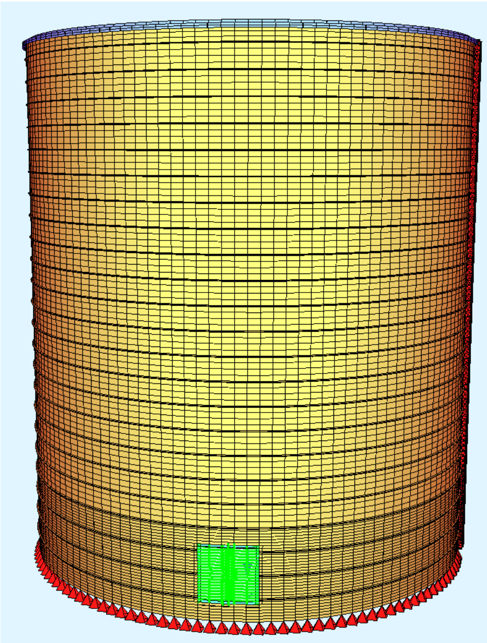
LC 1000	redistributed earthquake pressure (according to picture 2)
LC 1001	earthquake load resulting from the dead weight of the tank
LC 1004	reference earthquake pressure distribution (according to picture 2)
LC 1010	redistributed earthquake pressure + earthquake load from tank dead weight

Wind loads do not have to be considered alongside 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

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 ≤ 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 = $2.1 * 10^8 * 0.012 = 2.52 * 10^6$ 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.

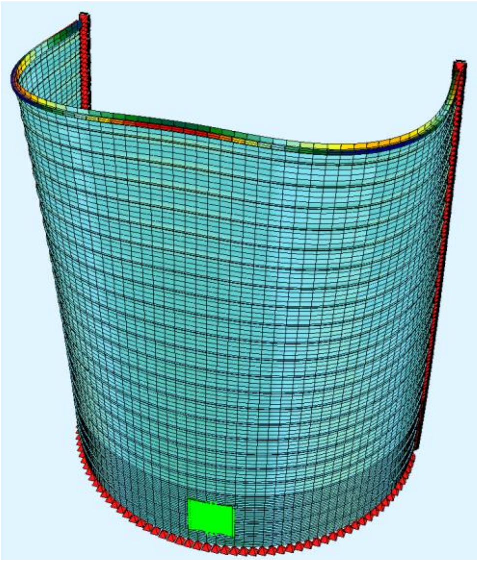
For this tank the limit states LS2 (cyclic plastification) and LS4 (fatigue) are not decisive and therefore not further investigated.

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.

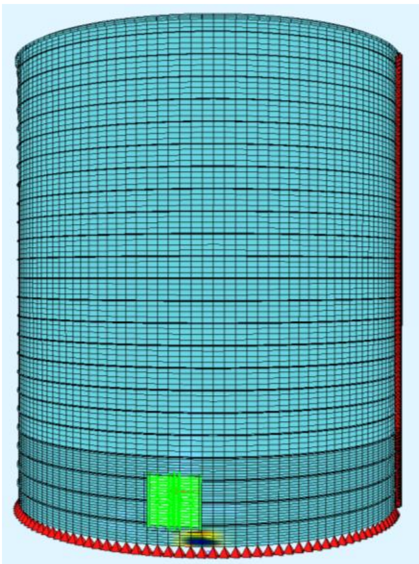


Picture 4: 1st Eigenform for LComb 1000 (LC1051), r_{Rcr} = 3.76

$Parameters$: $\overline{\lambda_0} = 0.40$ $\beta = 0.60$ $\eta = 1.00$ $Q = 16$ $\alpha = 0.50$

LK1000: $r_{Rpl,d} = 1.38$ $\lambda_\theta = (r_{Rpl,d} / r_{Rcr,d})^{0.5} = 0.61$ < 1.12 → partly plastical buckling

$\chi_p = 1 – \beta ((\lambda_\theta - \lambda_{\theta,0}) / (\lambda_p - \lambda_{\theta,0}))^\eta = 0.83$ $r_{Rd} = r_{Rpl,d} * \chi = 1.14 > 1$



Picture 5: 1st Eigenform for LComb 1100 (LC1151), r_{Rcr} = 2.70

b) Buckling of the cylindrical shell

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

Decisive measuring length $l_{g,x} = 4 \sqrt{rt} = 4 * \sqrt{(4000 * 3.0)} = 438$ mm

Decisive measuring length $l_{g,\theta} = 2.3\ (l^2 * r * t)^{0.25} = 2.3 * (9200^2 * 4000 * 3.0)^{0.25} = 2309$ mm

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

$\Delta w_{eq,x} = \max (l_{g,x} * 0.025 ; 4 * 25 * 0.025) = 10.95$ mm

$\Delta w_{eq,\theta} = \max (l_{g,\theta} * 0.025 ; 4 * 25 * 0.025) = 57.73$ 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): $r_{Rd} = 1.14 > 1.0$ (LComb 1000)

Decisive proof of buckling (earthquake): $r_{Rd} = 1.40 > 1.0$ (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

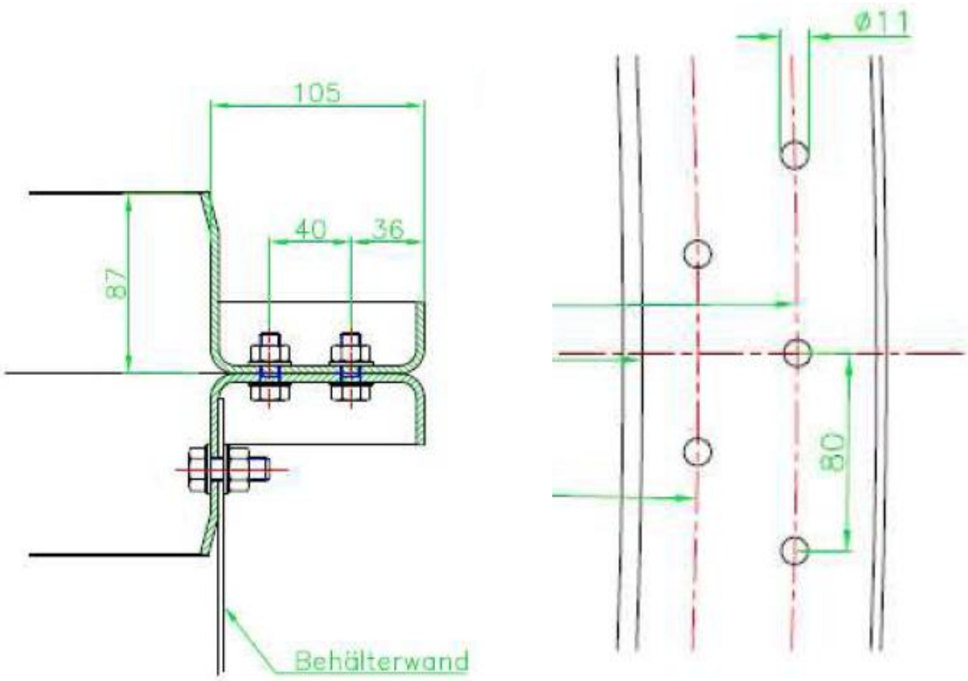


Bild 1: Sketch of the connection

decisive: LC12 of the mebrane

membrane forces: $H_d = 65$ kN/m

chosen bolts: M 12-8.8 (tensile strength $f_u = 800$ N/mm²), distance less than 100 mm

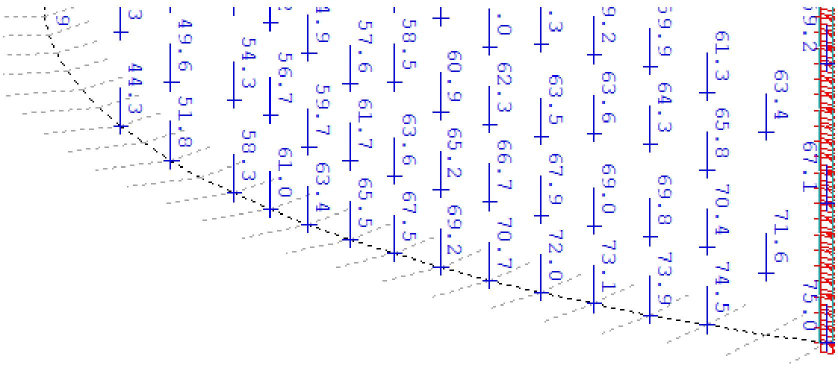
Pre-tension of bolt: 47.2 kN → $F_{s,R,d} = 1.0 * 0.2 * 47.2 \ / 1.25 = 7.55$ kN

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

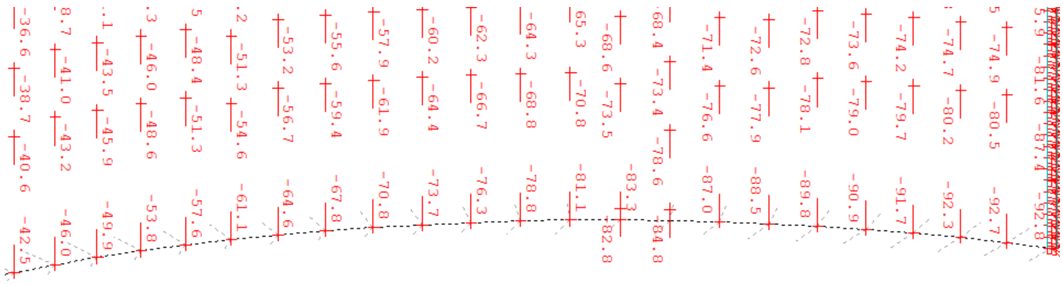
H_d is directly beared by the upper edge ring. No torsional moment needs to be considered.

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}: $F_{max} = 1.25 * 75.0 = 94$ kN/m

Minimum membrane force N_{yy}: $F_{max} = -93 * 2/3 = -62.0$ kN/m

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 mm = 92 mm A_w = 92 * 3 = 276 mm²

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

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

Capacity of the hoops Ø8: N_{Rd} = 4 * 0.50 * 43.48 = 86.96 kN

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

info: $N_{Rd,comp}$ = 4 * 12 * 3.14 * 2.7 * 150 = 122145 N (Ø12, $l_{compound}$ ≈ 150 mm)

chosen: $N_{Rd,comp}$ = 4 * 8 * 3.14 * 2.7 * 150 = 40694 N (Ø8, $l_{compound}$ ≈ 150 mm)

(compression at lower flange: $N_{Rd,comp}$ = 200 * (46 – 4) * 16.7 = 140200 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 ≥ 3.0 mm → $F_{y,Rd}$ = 58.23 kN

compression of concrete: f_{ck} = 25 / 1.50 = 16.67 N/mm²

required area: A_{req} = 62000 / 16.67 = 3719 mm² b ≥ 3.7 mm

Taking into account the compound of the tank wall with the mortar in the rill and the distance between the anchors no additional plates are needed.

d) anchor type and distance

max a = 57.37 / 94 = 0.61 m (+1 additional anchor in the man hole area)

Chosen anchor distance: a = 0.35 m → Amount: 72+1 x IPE 80

12 Floor slab

12.1 Loads on the floor slab

12.1.1 Permanent load

LC 1: Dead weight

g_{slab} = 0.25 * 25 = 6.25 kN/m²

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 kN/m² (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.

12.2 Parameters

Exposition: XC 4

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

12.3 Required minimum reinforcement due to limitation of crack widths

d_s = 9 mm k_c = 1.0 k =0.8 $f_{ct,eff}$ = 0.26 / 2 = 0.13 kN/cm²

$\sigma_{sd} = (w_k * 3.48 * 10^6 / d_s)^{0.5} = 278$ N/mm²

req. $A_s = k_c * k * f_{ct,eff} * A_{ct} / \sigma_{sd}$ = 0.80 * 1.00 * 0.13 * 1250 / 27.8 = 4.68 cm²/m

12.4 Minimum reinforcement due to ductility

W = b h² / 6 = 100 * 25² / 6 = 10417 cm³

M_{cr} = 0.26 * 10417 = 2708 kNcm

$A_{s,min,dukt}$ = 2708 / (0.90 * 20.5 * 50) = 2.94 cm²/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.