Investigation of Stresses in Cylindrical Flat Bottom Tanks with Ring Shaped Foundation due to Uneven Settlement

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DIPLOMA THESIS

for

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"Investigation of stresses in cylindrical flat bottom tanks with ring shaped foundation due to uneven settlement"

It is common praxis today to measure the settlement of a flat bottom tank during testing at the tank shell or eventually at the surface of the foundation. The displacement measured in this way may not be the settlement of the soil. For unanchored tanks due to the large stiffness of the tank shell in the longitudinal direction the variation of the deformations of the tank shell along the circumference will be much smaller than that of the settlement of the soil. By a ring shaped foundation the deformation at the lower boundary of the tank shell will be reduced compared to the unevenness of the settlement. High membrane stress resultants will occur due to settlement if the tank is anchored. This anchorage for operation conditions including wind requires heavy foundation rings or thick concrete bottom plates. The stiffness of these types of foundation will reduce the variation of the deformation that has to be used when calculating the stresses in the tank shell.

Scope:

1.) The influence of a ring shaped foundation on the stresses in flat bottom tanks shall be investigated by a finite element calculation. The finite element model shall include tank (shell, bottom and boundary conditions at the roof), ring foundation and soil. In addition to the stress calculation the deflections of the soil, the upper surface of the foundation ring and of the tank shell shall be compared.

2.) The stresses calculated for tanks with foundation calculated according to 1.) shall be compared with those for the same tanks without foundation ring.

3.) Models for the hand calculation and results from former investigations shall be compared with the results obtained under 1.) and 2.) and if possible extended to include the influence of the foundation ring.

4.) The relevant parameters shall be defined and the results shall be clearly presented.

Period of performance: 8 weeks.

Start time:

Date of delivery:

April 03rd 1998 shall: May 29th 1998 is:

Supervisor: U. Hornung

Karlsruhe, March 25th 1998

(Univ.-Prof. Dr.-Ing. H. Saal)

Preface

I would like to state that I wrote this thesis on my own, and I did not use any other sources except those listed in the references.

Sydney, 19/06/1998

Acknowledgements

I am grateful to Prof. H. Saal and Prof. P. Ansourian for allowing me to write my Diploma thesis at the University of Sydney. I would like to thank also my supervisor Mr. U. Hornung, in Karlsruhe, for his valuable and helpful advice despite the long distance.
For their assistance in preparing my thesis and their help during my stay at the University of Sydney, I especially thank Cynthia Caballes, Tim Wilkinson, Danny Kim, Hadi Helbawi and Mohammad Jonaidi.

0 CONTENTS

1 INTRODUCTION

In building a steel tank as economically as possible, one attempts to use the minimum number of components for the desired application. The simplest scenario consists only of a tankshell, tankbase and sometimes a roof. It is important to have a soil with uniform consistency and stiffness so that a uniform settlement of the entire building when the tank is filled e.g. with water. This, however, is not always possible in regions with subterranean mining or poor ground preparation. Also, tankshell because of the weight of shell and roof there is a concentrated pressure under the tank shell and therefore a larger settlement there.

The best protection against uneven settlement is a continuous concrete foundation plate. However, this requires a lot of extra material and added costs. Therefore, only a foundation ring is used under the tankshell. Schneider showed in [15] that the unfounded part of the tankbase is quite insensitive against axisymmetric settlement and that the failure results more in the tensile strength than in the stability of the tank.

While handling an open top tank there are almost no significant stresses in the tank shell, however, large displacements of the tank top result. To avoid them –e.g. for tanks with a swimming roof-, the top is fixed to a circular girder. This constraint results in high axial stresses especially at the lower bound of the tank shell. Palmer [14] developed an analytical model for getting the axial stresses in empty, cylindrical tank shells with a constant thickness in dependence to the girder stiffness and the settlement shape. Hofmann [5] expanded the model for several thicknesses. He considered also the tank base and water filled tanks. The investigations in the following thesis are made with fixed roof tanks, which can be compared with an open top tank and a primary wind girder having an infinite moment of inertia.

Palmer's efforts are confined to anchored tanks, which means that the tank follows exactly the applied settlement. Hofmann [5] compared the anchored and unanchored tanks. Würfel [16] produced a parameter study about the behavior of unanchored tanks. This thesis is to broaden the knowledge about this behavior and the difference between anchored and unanchored tanks.

In addition, the thesis will investigate the effect of a foundation ring. In the previous research, the foundation ring is supposed to exist for the tanks – whether anchored or not. However, the displacements applied in these treatises describe the shape of the deformed foundation under the tank shell. In this thesis, the settlement is applied to the underground and the special behavior of foundation ring and the tank shell is to be investigated.

Because of the complexity of the FE-model and the use of new features for modeling more attention was paid to the parts relating the handling of the FE-calculation.

2 SUMMARY OF RESULTS FROM PREVIOUS RESEARCH

Kamyab [7] and Palmer [14] used a form of localized settlement for anchored tanks, which is shaped as shown in Fig. 0-1.

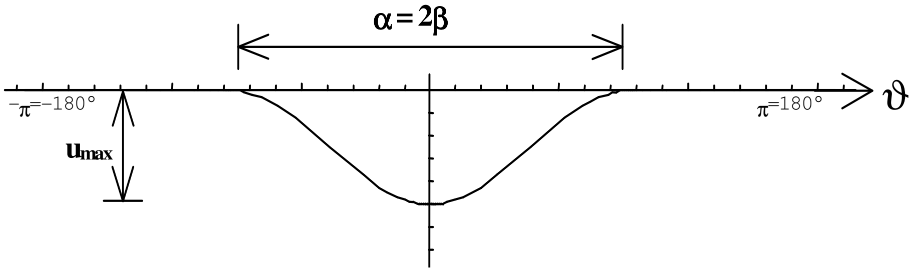


Fig. 0-1: Exact shape of settlement

This is the settlement shape also used in this thesis. However, because of the symmetry only one half of the settlement is formed by function (1). Another difference to the one Palmer used is the substitution of β for α/2. All angles are in radians.

$$\left.\begin{aligned} u &= -\frac{u_{\max}}{2}\left(1+\cos\frac{\pi\vartheta}{\beta}\right) && 0 < \vartheta < \beta \\ u &= 0 && \beta < \vartheta < \pi \end{aligned}\right\} \qquad (1)$$

This function can be expressed as a Fourier series described by

$$u = \sum_{n=0}^{\infty} u_n \cos(n\vartheta + \varphi_n) \quad ,0 < \vartheta < \pi \qquad (2)$$

The Fourier analysis gives the results:

a) phase angle $\varphi_{n=0}$

b)

$$\left.\begin{aligned} u_n &= -\frac{u_{\max}}{n\pi}\sin n\beta \frac{1}{\left[1-\left(\frac{n\beta}{\pi}\right)^2\right]} && ,n \neq \frac{\pi}{\beta} \\ u_n &= -\frac{u_{\max}}{2n} && ,n = \frac{\pi}{\beta} \end{aligned}\right\} \qquad (3)$$

Previous research found that the reaction of the tank due to uneven settlement depends on the wave number of the settlement around the circumference. For n=0 a uniform settlement is produced and n=1 causes a rigid tilt of the tank which does produce significant local stresses or deformations. For all n>1, the applied settlement has more than one minimum or maximum value. The result of an open top tank without a top girder is a large radial displacement of the top. If the top edge is constrained this radial displacement changes into vertical stresses within the tank shell. The stresses produced by every single Fourier component can be superimposed for getting the total stress amount. Palmer investigated the stresses regarding the single wave numbers [14].

- For $n<n_1$ only small values of the vertical stress/force in the tank shell occur and the assumption of the inextensional analysis is correct. The wave shapes formed by these components produce long waves and the significant values of the Fourier composition are in this area.
- Between n_1 and $n_2>n_1$, the above assumption is no longer valid, but the shell still behaves as a membrane.
- For $n>n_2$ the circumferential bending stiffness of the shell becomes more and more relevant and the shell does not react only as a membrane anymore.

Palmer investigated the stresses only for cylindrical tanks with a constant wall thickness. Hoffmann expanded this model for a tank with three courses [5]. A three-course tank is used in the German standard DIN 18800-4 as an imaginary substitute for any desired tank [2]. By that the effect of a more course tank is taken into account but the number of the necessary equations can be reduced.

The thicknesses of these courses are t_u, t_m, and t_o and the height of them are h_u, h_m, and h_o. t stands for the thickness of the lowest course of the original tank and E, ν are mechanical properties of the steel the tank is made by. The tank radius is r and its height is H. I_G stands for the inertia moment of the top girder. In this case the fixed roof tank is simulated by $I_G=\infty$. Hofmann showed the convergence of the stresses in dependence to I_G. Their values do not change for an I_G between 3.5e11 mm^4 and 1.0e12 mm^4 [5]. Therefore I_G=4e11 mm^4 is assumed for a 'rigid' primary wind girder.

The axial forces at the shell bottom for each n are:

$$n_{z(z=0)} = -3\frac{Et_u}{H}u_{\max}\frac{m^4\left(m^2-m_1^2\right)^2}{1+\bar{\lambda}m^2\left(m^2-m_1^2\right)^2\Omega_I\frac{t_u}{t_o}+m^4\left(m^2-m_1^2\right)^2\frac{1}{2}\left(3\Omega_{II}-\Omega_{III}\right)\frac{t_u}{t_o}}\cos\vartheta \quad (4)$$

with

$$\bar{\lambda} = \frac{\lambda}{\sqrt[4]{1-\nu^2}}; \qquad m = \frac{n}{n_1} \qquad m_1 = \frac{1}{n};$$

$$n_1 = \left(36\left(1-\nu^2\right)\right)^{1/8}\left(\frac{r}{H}\right)^{1/2}\left(\frac{r}{t}\right)^{1/4}\left(\frac{1}{\varsigma}\right)^{1/8}; \quad \lambda = \sqrt{\frac{3}{2}}\,2(1+\nu)\frac{r}{H}\sqrt{\frac{t}{r}}\,\sqrt[4]{\varsigma}; \quad \varsigma = \frac{I_G}{Ht^3/12(1-\mu)^2}$$

and

$$\Omega_I = \xi_u\frac{t_o}{t_u}+\xi_u\frac{t_o}{t_m}+\xi_o$$

$$\Omega_{II} = \varsigma_u^2\frac{t_o}{t_u}+\varsigma_m^2\frac{t_o}{t_m}+\varsigma_o^2+2\xi_u\xi_m\frac{t_o}{t_u}+2\xi_u\xi_o\frac{t_o}{t_u}+2\xi_m\xi_o\frac{t_o}{t_m}$$

$$\Omega_{II} = \varsigma_u^3\frac{t_o}{t_u}+\varsigma_m^3\frac{t_o}{t_m}+\varsigma_o^3+3\xi_u^2\left(\xi_m+\xi_o\right)\frac{t_o}{t_u}+3\xi_m^2\left(\xi_u+\xi_o\right)\frac{t_o}{t_m}+3\xi_o^2\left(\xi_u+\xi_m\right)\frac{t_o}{t_u}+6\xi_u\xi_m\xi_o\frac{t_o}{t_m}$$

$$\xi_u = \frac{h_u}{H}; \qquad \xi_m = \frac{h_m}{H}; \qquad \xi_o = \frac{h_o}{H}$$

The axial force declines linearly from the shell bottom to its top. The equivalent stresses σ_x can be obtained by dividing n_x by the thickness of the single courses. Depending on the different thicknesses the maximum value of the stress is not necessarily located at the shell bottom [5].

Above calculation is only valid for an anchored tank without, tank base and/or foundation. The difference between of those model and the models including all tank components are investigated in *chapter 5*.

3 GEOMETRY OF TANKS AND APPLIED LOADS

This chapter will describe the used tanks and the applied loads as well as the shape of the settlement. The used units are [N] for forces and [mm] for length declarations. If the angle α of the settlement area is used one must be aware that this angle means the opening of the whole settled area. In this thesis is only one half of the tank modeled. Therefore the angle β which appears in the diagrams is half of α.

3.1 Model geometry

Three tanks are selected, each distinguished by the ratio between radius and height. This is supposed to show the difference in the investigated behavior in dependence to the axial bending stiffness of the tank shell.

The tank base is composed of two parts. The inner part with no intended stability is 6mm thick. The outer ring is supposed to allow the shell to submit bending moment. For this its thickness is about 70% of the lowest course thickness. The size of the outer part is chosen to lie completely over the foundation ring.

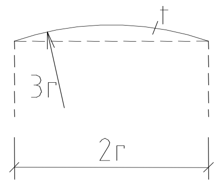
The foundation depth is set to 0.8m, which is the frost relating depth for foundations in Germany. Its width is 1.50m. The foundation is not used for all models since a comparison between tanks with and without foundation is required. For other investigations the outer part of the tank base is connected with the top nodes of the foundation ring.

Both foundation ring and outer base ring overhang the tank shell about 0.5m
For a detailed overview of the model geometry see the tab 0-1.

3.2 Static loads

3.2.1 Roof load

API Standard 620 gives the lowest limit for the roof skin thickness at 4.7mm and thicker values have to be chosen for tanks with different radius [1]. The radius of the dome is chosen to be triple the tank radius.



By using steel as material with γ=77.0*10^{-6} there results a roof load of 7.96e-5*r*t*φ $^{N}/_{mm^3}$ along the circumference of the tank top. φ is used as a factor regarding the roof structure. Its value is φ=1.1 for tank1*x* and tank2. Because tank3*x* has a bigger span additional girders, bracings, etc. are necessary and therefore φ=2.0. Distributed on the top tank nodes of the 60 elements around the half of circumference the load per node is 2293 N for tank1*x*, tank2*x* and 26988 N for tank3*x*

Half of these values are applied on the nodes at the symmetry plane.

3.2.2 Dead weight

The self weight is calculated in ABAQUS by the gravity constant g=9.81 $^{m}/_{s^2}$ and the density of steel (ρ=7.85*10^{-6} $^{kg}/_{mm^3}$) and concrete (ρ=2.5*10^{-6} $^{kg}/_{mm^3}$)

3.2.3 Water pressure

The pressure on to the shell elements mounts linear from the top with p=0 $^{N}/_{mm^2}$ to the bottom where the maximum pressure is p=γ_w*h with γ_w =9.81*10^{-6} $^{N}/_{mm^3}$.

3.2.4 Overview of values

	tank1x			**tank2x**			**tank3x**		
radius	10000			10000			21500		
height	11100			21400			15426		
number of courses	**4**	*2775*	*15.0*	**6**	*2600*	*11.0*	**6**	*2571*	*15.5*
details of courses		*2775*	*11.0*		*2500*	*9.5*		*2571*	*14.7*
(number,thickness,height)		*2775*	*9.0*		*2300*	*8.5*		*2571*	*14.2*
		2775	*7.1*		*4600*	*7.5*		*2571*	*13.7*
					9200	*7.0*		*2571*	*13.0*
					2300	*6.0*		*2571*	*13.0*
thickness of roof plate	5.0			5.0			7.0		
base thickness *(middle part/outerring)*	6.5	10.0		6.0	8.0		6.0	9.0	
roof load [N/mm]	4.4			4.4			24.0		
water pressure at bottom [N/mm²]	0.109			0.237			0.151		

Tab 0-1: Details of used tanks

3.3 Settlement

Purpose of this thesis is to investigate the stresses due to a cosine shaped settlement. Analytical calculations show the dependence of these results on the Fourier coefficients of the cosine function (see *chapter 0*). The settlement is generated in a FORTRAN subroutine, which is included in the ABAQUS input file. At first this subroutine reduces the node number to the coordinates of the nodes and then determines the respective settlement for this node. This can be either the exact cosine function or the composition of the single Fourier components. An overview about the applied settlement for different n_{max} and the Fourier components for different angles is given in Appendix 4. The components for different maximum settlements u_{max} distinguish only between a constant factor, which is u_{max} and therefore the coefficients must vary only for different angles α.

The maximum settlement for tanks with foundation is related to the settlement applied on the spring nodes of the outer edge of the foundation. The difference between the settlement under the tank shell and under the outer edge is r/ (r+500mm) with r=tank radius of the tanks described above. This gives a settlement of $u=0.952u_{max}$ for tank 1*x* and tank 2*x* as well as a settlement of $u=0.977u_{max}$ for tank3*x*.

4 FINITE ELEMENT MODELING

The following chapter will show the constellation, loading and features of the Finite Element Model. It will also discuss the plausibility of the results are.

The finite element calculation was done by the program ABAQUS v5.6.

4.1 Model

The model consists of tankshell, tankbase, foundation ring and soil. Because of the symmetric shape and load, only one half has been generated. Hence, the number of elements could be halved. Fig.4-1 shows a shaped model with all components.

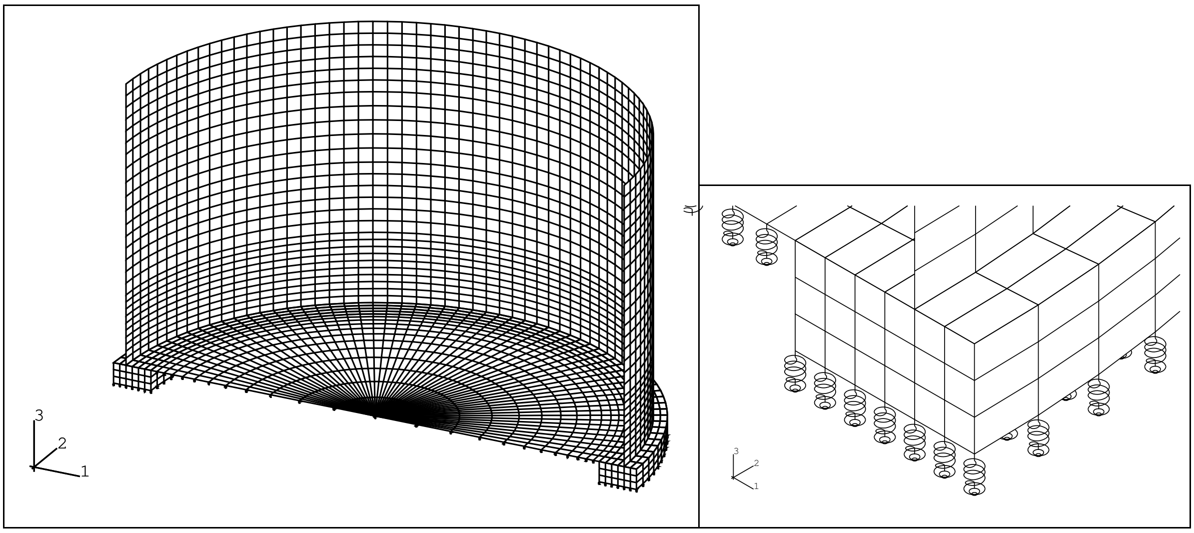


Fig. 4-1: tank13 with detail of springs and foundation

All nodes and elements are transformed into a cylindrical coordinate system. By using ABAQUS POST for postprocessing, all stresses/strains could be evaluated due to this system. The node displacements are not displayed in cylindrical coordinates by ABAQUS POST. As such, it was necessary to create a special output file as an interface to another post processor, specifically MSC/PATRAN. The values of the settlement due to the cylindrical system were verified to be correct in the created output file, however, PATRAN does not recognize the cylindrical system and draws a wrong deformed model. Therefore, PATRAN could only be used for getting the right values for diagrams with radial/tangential displacements.

The ratio between the dimensions of the elements is kept below 3.0.

4.1.1 Tank

The element used for the tankshell is a shell element for finite strain applications. The element S4R is a 4-node, double curved element for thin and thick shells. The average characteristic element length in the models lays about 450mm. By a shell thickness between 6 and 16mm, there results a ratio of 1/28. The criterion in ABAQUS for thin or thick shells is a ratio of 1/15, so a thin shell can be assumed.

This element uses a reduced integration and offers hourglass control. At every node, there are 6 degrees of freedom. The membrane strains are finite and suitable for large strain analysis. The shell thickness can change according to the Poisson ratio. Concerning the reduced integration (i.e. one Gauss integration point in each direction less than necessary for exact integration), a softer behavior of the structure in compare to the exact integration is expected.

The tankshell consists of several courses with different thicknesses. The difference of the radius because of the thickness change between two courses is neglected. The element size within one course is constant whereby the size changed between the single courses. The lowest course got the smallest elements because of the expected biggest change of stresses. The other courses are adapted intending a mesh refinement towards the lowest course. The highest course got more elements because of the roof load and the top boundary conditions.

The same shell element is used for the tankbase. In the centre, it is necessary to use a triangle element (S3R) instead of a 4-node element, which is full compatible to the element S4R. The base is composed of a thin plate in its inner part and a thicker ring outside over the foundation. The elements over the foundation ring have a more or less constant size. The size of those in the middle part decreased from the tank centre towards the inner foundation edge by a constant factor. Hence, a continuous refinement towards the expected higher displacements close to the foundation is achieved.

4.1.2 Foundation

The foundation ring is modeled by the element C3D8, which is a 8-node linear brick element. Its nodes are connected with 3 degrees of freedom each (u_r, u_ϑ, u_z). The difference to a 20-node solid elements appears only for very high stresses where the quadratic brick element is more accurate regarding the Poisson ratio. As you can see later, the stresses in the foundation and its displacement stay on a level, which does not affect big changes in thickness of the elements. So this effect due to the Poisson ratio can be neglected.

4.1.3 Soil

There are a few possibilities of generating a soil in ABAQUS. Creating a soil by using solid elements is only meaningful, if one is interested in the results of the underground but for any other purpose too many variables are wasted for unneeded results. There is an element option for defining an elastic foundation onto an element face. This is recommended, if one is interested in the different settlement due to an uneven E-modulus of the soil.

Concerning a proper defining of the uneven settlement, the ground is built with the spring element SPRING1. This element simulates a spring acting between one node and the ground in a fixed direction. Its stiffness is calculated by k=EA/l. E stands for the E-modulus of the ground and l is an imaginary length. For the supported area A of one spring, it is assumed that half the distance to the next spring in each direction counts/belongs to it. The stiffness varies with the radius. Without a foundation it would have been possible to use an average stiffness. Schneider showed in [15] that the axisymmetric difference of settlement and therefore the different stiffnesses are not very significant for the stability of the tank. Besides, the stiffness directly under the shell would be constant. However, by the use of a foundation and a constant average stiffness there would be –depending on radius of the tank and the dimensions of the foundation- a deviation of about 5.. 9%. Springs with the same stiffness are arranged in circles around the centre of the tankbase. Because of the same radius, all springs in one row support the same area. The springs at the symmetry axis got the half stiffness.

4.2 Contact Modeling

The purpose of this work is to examine the different settlement of ground, foundation and tank. Therefore, it is necessary to manage the contact in that way that only pressure can be submitted and the single components of the model are able to separate from each other. For this, six surfaces have been generated. Two at the top and at the bottom of the foundation ring as well as two for the middle part and the outer ring of the tankbase. The nodes of the springs underneath the base and the foundation form the last two surfaces. It is not necessary that the nodes of the master- and the slave surface fit exactly together, but ABAQUS performs better if it is so because ABAQUS handles the contact relations by creating internal contact elements between a pair of corresponding nodes of the two contact surfaces.

For the sake of completeness, I would like to mention the disadvantages over the use of GAP-elements. The opening of the gaps must be calculated by the user from the coordinates/ displacements of the nodes of the two surfaces while postprocessing with ABAQUS POST. It is possible to get the opening e.g. with MSC PATRAN, if a special output file is generated during the analysis. The presentation of the pressure/forces on the surfaces is not very suitable for evaluating and creating diagrams. For generating an anchored tank it is not possible to connect only the tank shell with the corresponding nodes of the foundation ring. The reason herefore is an overlaying of constraints and the internal contact elements at the same nodes. This would cause zero-pivots during the FE-calculation instead of creating two separated surfaces beside the tank shell the whole outer base ring is connected to the top nodes of the foundation.

4.3 Fixed Boundary Conditions

The tanks used in this thesis are fixed roof tanks. A fixed roof can be simulated by a top girder with an infinite moment of inertia or fixed boundary conditions at the top of the tank. The latter method was chosen. Hence, all top nodes are constrained in the radial and tangential direction. The vertical movement of the tank is controlled by the springs.

The foundation ring has to be constrained against free moving into any direction and rotating around any axis. The vertical direction is also supported by the springs. The remaining degrees of freedom could have been fixed by constraining two points in radial and tangential direction. The analysis showed that this was not sufficient. There appeared negative eigenvalues and numerical singularities. It was necessary to constrain the foundation ring as followed: It is left unconstrained between ϑ=0°.. 90° and has radial constraints for ϑ=90°.. 180° at every edge node of the bottom. For applying a larger settlement area or increasing the maximum settlement, it was necessary to constrain above nodes even in the tangential direction. These constraints are not too far from the reality. Those radials can be compared with the resistant of the soil around the ring. The change of length due to normal force is negligible because of low tangential forces and the influence of the tensional stiffness is small in comparison with the flexural rigidity. Because only two nodes on the bottom of the foundation ring are constrained, the tank is still able to bend around the r-axis.

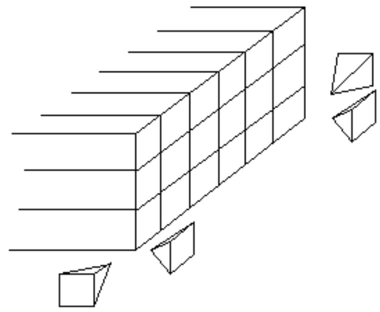


Fig 4-2: Boundary conditions of foundation ring for ϑ=90°..180°

All nodes at the symmetry plane are constrained against tangential displacement. Additionally, the nodes of the tank, which have also all rotational degrees of freedom, are constrained against tangential and vertical rotations.
For an anchored tank, the outer base ring is connected to the correlative fundament nodes in r-, ϑ- and z-direction.

4.4 Materials

Two material types have been used. Tankshell and -base are made by steel with an E-Modulus of 2.1e5 $^{N}/_{mm^2}$. The foundation is made by concrete with an assumed E-Modulus of 2.6e4. Both steel and concrete are assumed to have an elastic behavior and a Poisson ratio of 0.3. The elastic behavior is correct, as long as there are not very high stresses and/or deflections.

4.5 Loading

The load is applied in two steps. In the first, a linear step, the static loads are generated. The tank is completely filled with water. The water is modeled with a -from the top to the bottom linear increasing- pressure onto the faces of the shell and base elements. Concentrated forces at the top nodes of the tankshell simulate the roof weight. The self-weight of tank and foundation is calculated automatically by the density, gravity constant and the volume of every single element.

In the second step, the settlement has been applied by using a FORTRAN-subroutine. It could be chosen between a settlement composed of a single cosine function or several Fourier-coefficients of this cosine function. The settlement increases from 0 mm at the centre to its maximum value at the outer edge of the foundation. All nodes covered by the user subroutine, even those with no settlement, are constrained by ABAQUS in the vertical direction. Therefore, the springs cannot act anymore as springs. Therefore, two different areas are used. For the settlement composed of the single cosine function the area the user subroutine is applied on is limited. The limit is the maximum angle of the settlement. For the Fourier variant, the subroutine constrains all spring nodes of the model.

The procedure for the second step is a nonlinear static analysis. The iteration is performed by the Newton method. Depending on the convergence of the single increments at one step ABAQUS changes automatically between Newton-Raphson (for every increment there is one stiffness matrix) and the modified Newton-Raphson method (one global stiffness matrix for all increments).

For finding the equilibrium of forces and moments, ABAQUS calculates as much iteration as necessary or as maximal defined. Especially for the first increment, ABAQUS needs many iteration because of severe discontinuities. Those can be avoided by changing the default values of the size of the first increment and the maximum allowed number of iterations. The smallest first increment used was 1e-7 out of 1.0. All following increments converged after 1or 2 iterations. For a faster calculation, the factor for the automatic increasing of the increments was enlarged from its default value 1.5 to 5. Obviously, the more elements have been used the more iterations were necessary and the smaller was the first increment. The same effect appeared for a larger settlement area, maximum value of the settlement or only a few components of the Fourier analysis.

4.6 Mesh fineness

The precision depends a lot on the grade of the mesh fineness. However, more nodes/elements require a longer time for calculation and heavy demand on the system resources and capability. For getting some results about convergence, I ran jobs with different fineness by doubling the number of variables. The difference between tank10 and tank13 is the number of elements within a cross section. Tank14 has twice the number of elements as tank13 along the circumference. In tank13 and tank10 every element covers an angle of ϑ=3°. Hence, the accordant angle of tank14 was ϑ=1.5°.For details of the partitioning of the cross section of these three tanks see [Appendix 1].

The problems in ABAQUS had following sizes:

	Number of nodes	Number of elements	Total number of variables
tank10	4333	4612	14646
tank13	8725	9457	29835
tank14	17308	18877	ca. 60000

In the number of nodes and elements are also those included, which are created internal by ABAQUS e.g. for modeling the contact problem. The total number of variables includes all nodes, elements and LaGrange multipliers. The applied settlement is a single cosine function with u_{max}=50 mm and α=100° (for the description of the settlement shape see *Chapter 2*)

4.6.1 Assessment of results

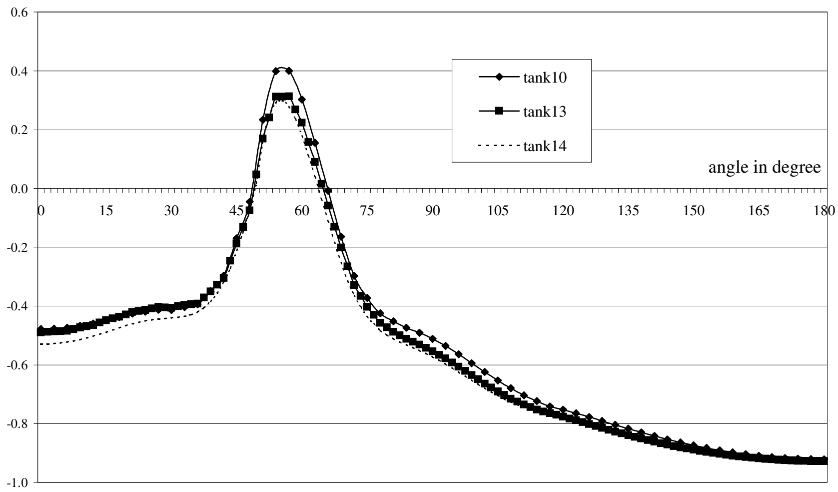


Fig. 4–3: Vertical displacement of connection between tank shell and bottom in mm

Fig. 4–3 shows the vertical displacement of the tank shell bottom. There is a significant difference between tank10 and tank13, but almost none between tank13 and tank14 in the area, where the shell rises into positive values. Whereas there must be a distinction between tank10/13 and tank14 at the uplifted end. The only point where all curves are almost identical at is around ϑ=50° where the whole load of the uplifted part weighs on the foundation within a narrow area. Therefore, the forces in this area are high enough that the relative number of elements in that area did not count anymore. In the area outside from ϑ≈50° tank10 and tank13 indicate always a smaller displacement than tank14 does. Those models behave more stiffly than tank14. Their maximum possible change of gradient (curvature=$\delta^2 u_z/(\delta\vartheta)^2$) of the displacement shape is always less because of fewer elements along the circumference,.

Due to the vertical settlement, there results a radial displacement of the tankshell. For open top tanks without any top girder, it is on the top edge that the maximum value is reached. In my models the top is constrained, so the radial displacement is demonstrated in Fig 4–4 in the middle of the tank height.

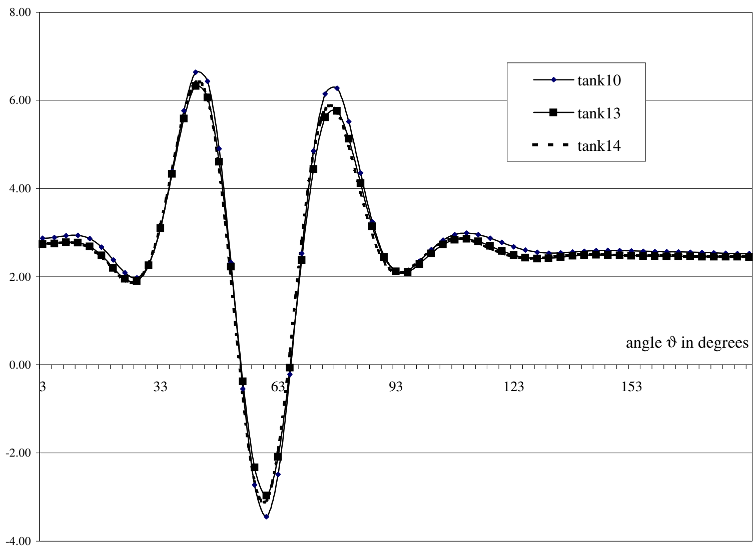


Fig 4–4: radial displacement in the middle of the tank height in mm

There is a constant positive, i.e. outwards directed, displacement due to the water pressure. The radial displacement because of the vertical displacement is superimposed. Tank10's positive and negative maximum values are bigger than those of tank13 and tank14 are. This is because less elements per cross section react in more extreme and bigger values than in a cross section with more elements, whereas there is almost no difference by using a different number of elements around the circumference.

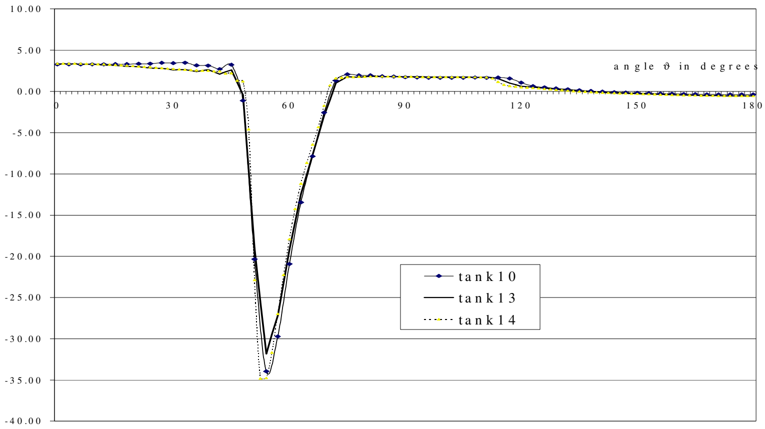


Fig. 4–5: axial stress at the bottom of the tank shell in $^{N}/_{mm^2}$.

Regarding the axial stress in the tank shell in Fig. 4-5, one can see the advance of more elements around the circumference. Tank14 does not have the peaks at about ϑ=48° as tank 10 and 13 have. Because of the little area, where the tank can submit the load of the uplifted part there is a big change of stresses with a little change of angle. The appearance of those little peaks and the exactness of the maximum value of the stress depend a lot on the position of the point, where the applied settlement is finished. In addition, because of the maximum negative value, it can be seen again that the fewer shell elements in tank10 behave more stiff than the more numbers in tank13. Tank10 can submit more pressure within a certain area.

Finally, examined the deformed foundation ring in dependence to the mesh fineness, Fig 4–7 shows the vertical displacement of the foundation top surface. The selected node line was the one under the tank shell. Again, it can be seen that tank13 behaves more stiffly than tank14 because it has fewer elements. However, tank10 is very soft in the uplifted area. This phenomenon has following explanation: The foundation ring of tank10 is modeled by 2x5 elements whereas tank13/14 exists of 6x3 elements in the cross section. This causes following force distribution at the element nodes in tangential direction:

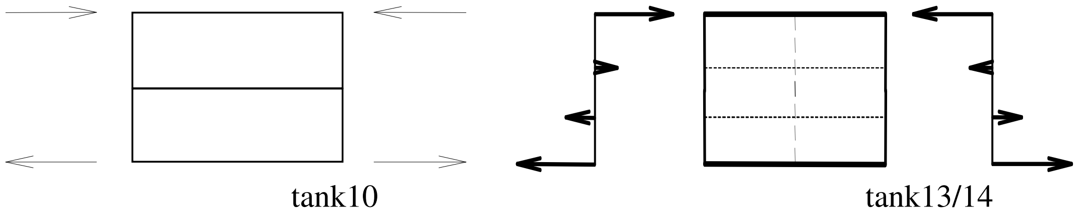


Fig 4–6: Distribution of bending stresses in foundation onto the element nodes

The main load results from bending. Therefore, at the middle nodes of tank10 there is the neutral axis. The pressure on the top and bottom nodes is the reason for larger stresses and strains in the solid elements of tank10 than on those of tank13/14. This results in a larger displacement of the foundation in tank10.

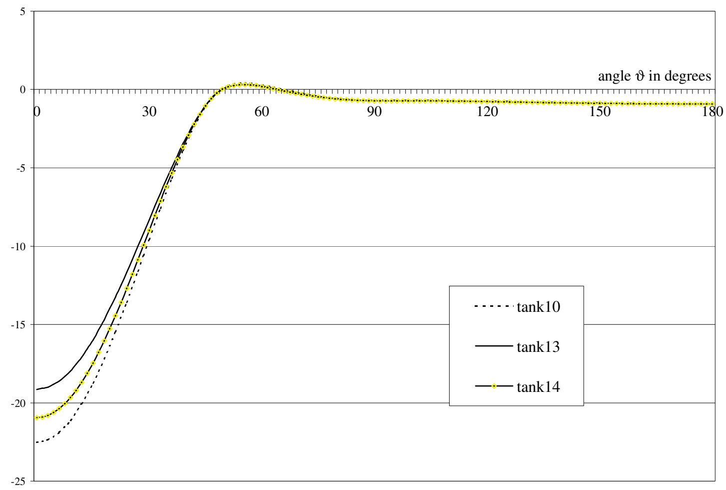


Fig 4–7: vertical displacement of foundation in mm

4.6.2 Summary

Above itemization shows that tank10 distinguishes too much from the two others. Tank14 would be the best to use. ABAQUS is able to handle this high number but the needed time amount is much more than the one of tank13. Additionally, ABAQUS POST in the momentary configuration can not process the whole model. The memory management does not allow models with this size and not all results could be retrieved. It is possible to model a mesh refinement in the area from the uplifted end up to an angle where there are hardly any additional stresses produced by the settlement. This would limit the number of elements. Concerning the different angles of the applied settlement, this requires different models and their results can not be compared with each other.

Therefore, tank13 was used for the investigations. Considering above diagrams, one must be aware that approximate values can be received which are assimilable to results of tank14. Following corrections are necessary:

- Multiply the settlement of foundation and tankshell of tank13 in the uplifted area with x=1.1
- The maximum axial stress in the tankshell of tank13 can be multiplied with x=1.1

5 FE-CALCULATION RESULTS

The following will show the effects of single components on the tank, in particular its stresses and deformations. Despite the fact that some of those are already investigated in the literature they will be raised as a general overview.

In most cases only the axial force instead of the axial stress is shown. The difference is the constant factor 1/t with t=thickness of the lowest course. In comparisons where different thicknesses are used, the axial stress is used instead. Most of the following diagrams are created by using tank1*x*, which is described in *Chapter 3*. All given axial forces/stresses will be always shown at the lower end of the tank shell.

5.1 Overview about single components of the tank

5.1.1 Effect of fixed roof

The significant vertical stresses are caused by constraints of the tank top. Those are due to a fixed roof or a primary wind girder used for swimming roof tanks. This constraint prevents ovalization of the top and e.g. a getting stuck of the swimming roof. To visualize the deformations see Fig. 5-1. This diagram shows the radial displacement in the middle of the tank height. It can be assumed that the displacement increases linearly from $\Delta r=0$ at the bottom to its maximum value at the top. The radial deformation of the fixed roof tank is $\Delta r=0$ at the top and at the bottom. This radial displacement consists of two parts: The constant part $\Delta r\approx 2.5$mm is attributed to the deformation due to the water pressure. Superimposed, there is the deflection caused by the vertical displacement of the lower bound.

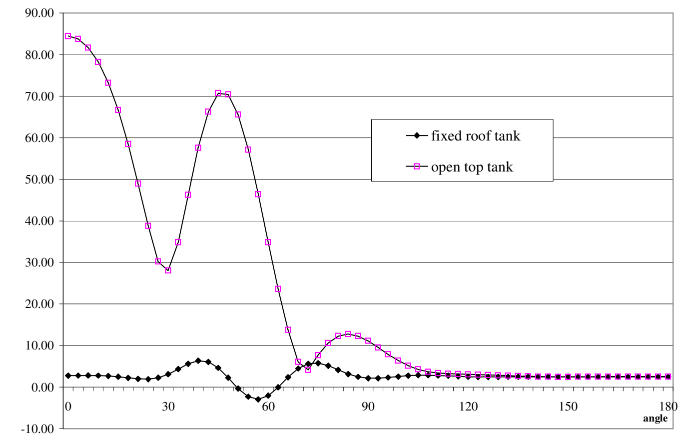


Fig. 5-1: radial displacement of an open top and a fixed roof tank

5.1.2 Consideration of the tank base

Palmer did his first investigations with an empty tank without a base. As it can be seen in Fig.5-2, the effect of filling the tank with water and adding a tank base results in higher shell stresses in the uplifted area of the tank. The water pressure on the base is partly taken by the shell in this area. This additional loads prevents the shell from uplifting for small values of the shell displacement u. One can see this effect in the area of ϑ=90..180°. The tank without the base (this can be compared with an empty tank) lifts up in the area of ϑ=90..180° whereas the shell bottom of the tank with the base is still subjected to compressive stresses.

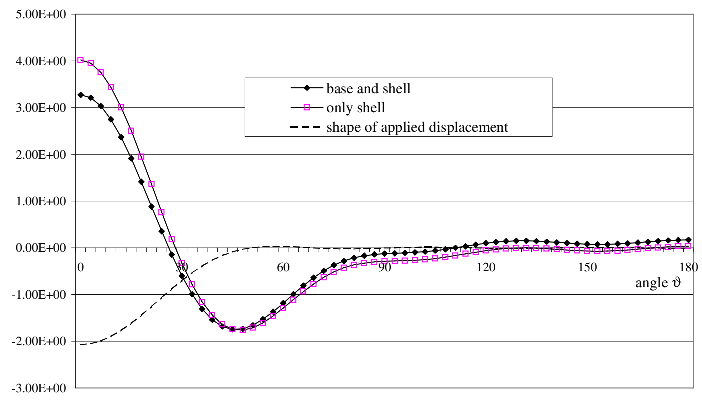


Fig. 5-2: Effect of filled tank and tankbase on the axial force in the tank shell

5.1.3 Anchorage of the tankshell

The effect of the filling as described above can be compared to an anchored tank. The purpose of the anchorage is to prevent an uplifting of the shell. This results in higher vertical stresses.

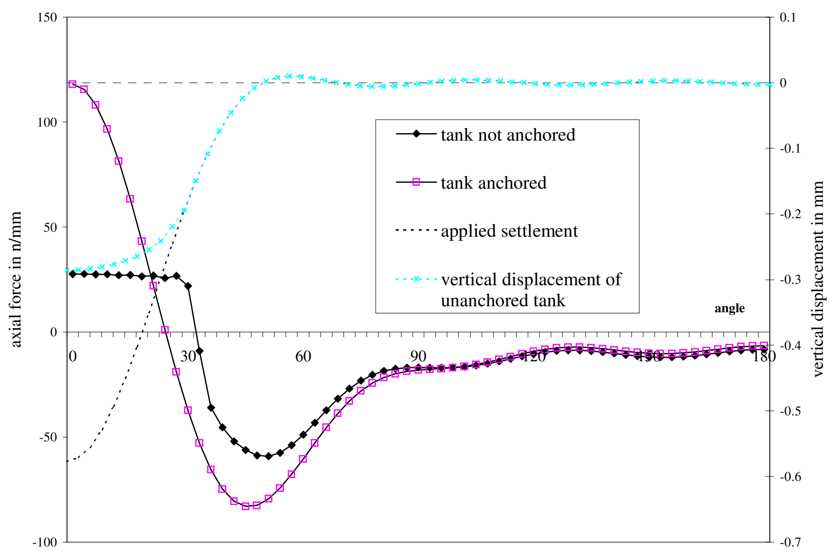


Fig. 5-3: Axial force and vertical displacements for anchored and unanchored tank

Fig. 5-3 shows the difference in the axial force between an anchored and unanchored tank. It also shows the shape of the applied settlement, which is u_{max}=0.6mm, α=100° and n=6, and the displacement of the unanchored tank. As it can be seen this tank uplifts already for small values of u_{max}. The stresses in the uplifted tank reach a maximum value and stay constant as long as the uplifted area per unit does not change.

5.2 Effect of foundation

Like the tanks without a foundation there is a difference whether a tank is anchored or not. The applied settlement in Fig.5-4 has its maximum value at u_{max}=5mm and is extended to α=100° with the wavy line for n=4. For clarity, the applied soil settlement and the foundation displacement of the unanchored tank is not shown. The shape of the applied settlement is exactly the same as the foundation of the unanchored tank. It follows the soil completely to its maximum value. Because of the small u_{max}=5mm there is no uplifting of the unanchored foundation. The behavior for larger settlements is shown further down. In Fig.5-4, uplifting of the unanchored tank occurs between ϑ=0..54° and in the second and third 'valley' of the applied settlement. The tank is to stiff too follow this settlement in these areas.

More evidence is given by the axial forces in Fig.5-5. The constant axial force in the uplifted areas can be observed clearly at the unanchored tank. This effect is described above. Because the first uplifted area of the unanchored tankbase, the force for ϑ=0..54° is bigger than in the two other uplifted areas.

There is also an uplifted part at the anchored tank, but this is outside of the foundation and is supported by the foundation. The foundation, however, is not able to support the base in the whole uplifted area and passes the load on to the tank shell. This is the reason for the almost horizontal profile of the stress curve of the anchored tank for $\vartheta \approx 0.27°$.

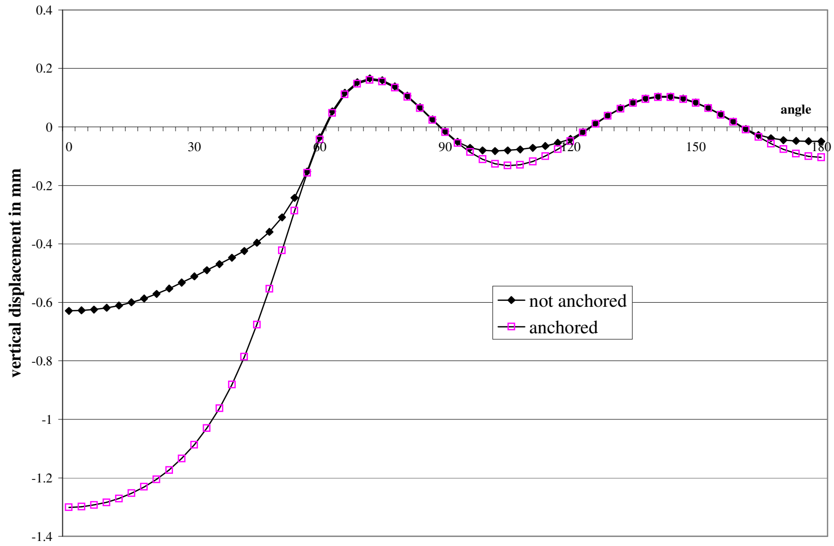


Fig. 5-4: Axial force for unanchored and anchored tanks with foundation

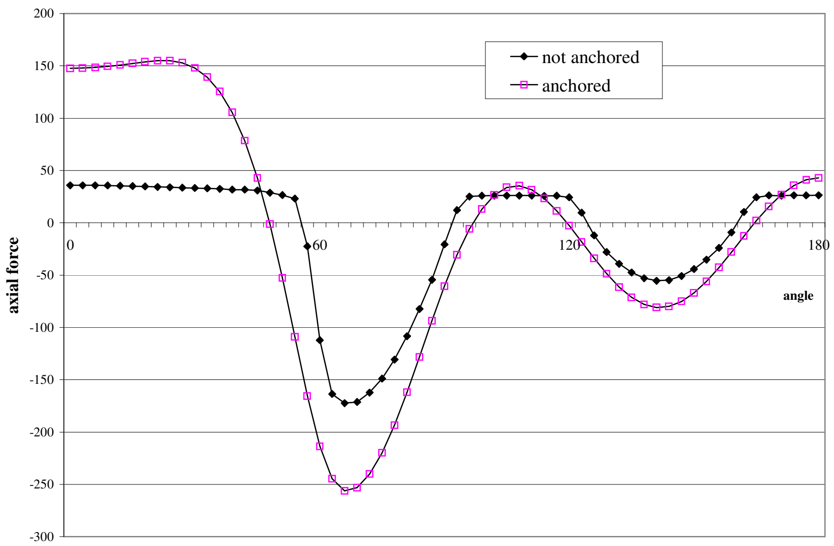


Fig.5-5: Vertical displacement for unanchored and anchored tanks with foundation

5.2.1 Behavior of the uplifted tank base

Fig.5-6 shows the cross section of foundation and parts of tank shell and base as well as the uplifted area for an unanchored model loaded with the exact shape of the settlement and u_{max}=50mm and α=100°. These diagrams can also be found in Appendix 5. The hatched parts show the uplifted area. This area can only be approximated according to the reached accuracy by the used mesh fineness of the model. Inside the tank shell the water pressure is depicted.

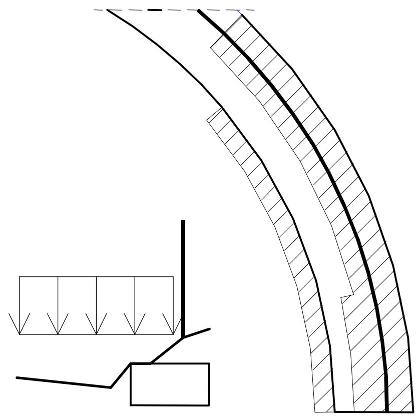


Fig. 5-6: Demonstration of uplifted parts

The uplifted area over the foundation inside the tankshell is A=355.2e4mm². Therefore a force F_w=p*A=0.109*355.2e4=387.2kN must be carried off by the tank shell and the foundation. The sum of the axial shell force in the uplifted area is F_s=363.8kN. The sum of roof load and self-weight of the tank shell is f_D=4.38+9.99=13.37 N/mm. Along the uplifted area this results in a total value of F_D=f_D 46.5° r *3π/180=108.6kN. The self-weight of the uplifted tank base is neglected.

The a rate of the uplifted load taken by the tank shell is therefore (363.8-108.6)/387.2=0.659. This rate of 65.9% is close to the splitting force of a beam with linear distributed load pinned at the one end and rigidly fixed at the other. The vertical reaction force at the latter end is 62.5%. Hence, the uplifted part over the foundation can be regarded as such a beam and the connection shell-base can be assumed to be rigidly fixed. The difference of 3.4% lies within accuracy of the model and the FE-calculation

The uplifted part inside of the foundation ring can be compared with a pinned beam. The total water pressure force is F_w=264.2kN. The foundation reaction force F=185.5kN can be calculated as an average pressure from the pressure distribution on the foundation surface (Appendix 6), and subtracting the water pressure on the foundation in this area.

The reaction force for the pinned beam is therefore F=274.2-(1-0.659)*387.2=141.5kN. This equivalent to 53.6% of the water pressure F_w=264.2kN, and again within 3.6% error range for the splitting reaction force of a pinned beam.

5.2.2 Description of foundation ring behavior

In Fig. 5-7 the exact settlement shaped is applied for an unanchored tank with several values of the angle α, and a maximum settlement u_{max}. The settlement of these models is applied only for ϑ=0..50°. The spring nodes for ϑ=50..180° are left unconstrained to model the soil in this area. This results not only in an applied settlement for the tank, but also a rise. The difference is shown in Appendix 5.2. This does not really conform to the reality but the results are still useful. The water pressure presses onto the springs and the max. deformation of about 0.9mm is equivalent to the expected uniform settlement if there would be a tank filled only with water and without a predefined settlement.

For a better comparison, the displacement for u_{max}=50mm is shown in Fig. 5-7 together with the axial force for several u_{max}. In all cases uplifting of both the foundation from the soil and the tank from the foundation occurs.

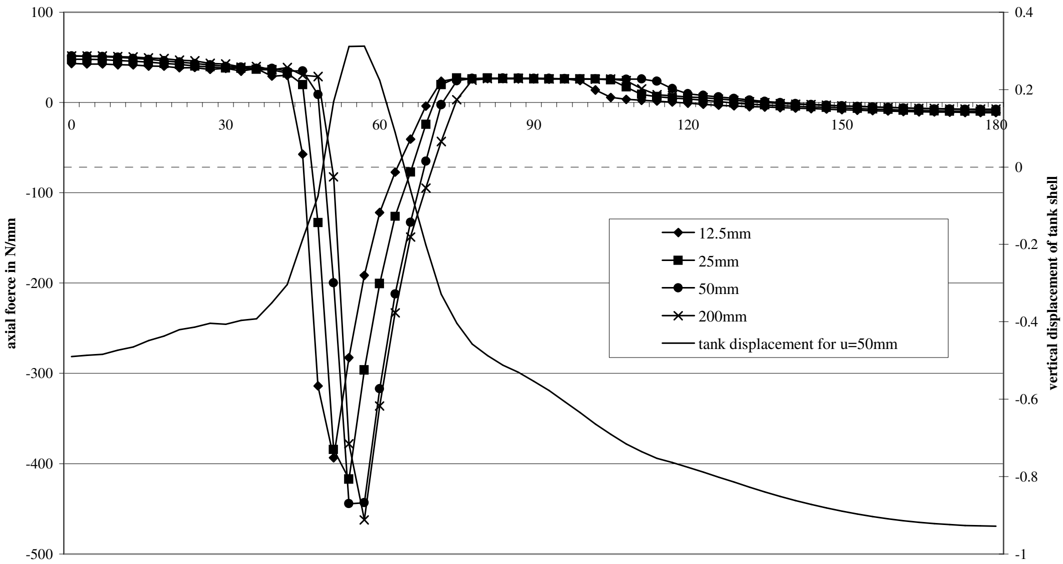


Fig. 5-7: axial force for exact settlement, α=100° and different u_{max}
vertical displacement of lower tank shell bound

For ϑ=180°, the axial force is f≈13N/mm, which is close to the load from the combined weight of the tankshell and the roof (f_D= 4.4+9 N/mm). The almost horizontal parts show again the effect of uplifting on the axial stresses. The second uplifted area around ϑ=90° is caused by the beam behavior of the foundation ring. The ring behaves similarly to a beam with linear distributed load as shown in Fig. 5-8 and its upwardly bent part causes a second uplifting of the tank shell.

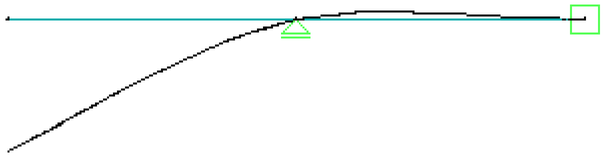


Fig. 5-8: deformation of beam with linear distributed load

Appendix 7 shows the deformed foundation ring for both an unanchored and an anchored model. The one for the anchored tank is bent downwards and there is also torsion because of the water load in the tank inside part of the foundation area. The same water load also presses on the unanchored foundation ring. This ring, however, is twisted outwards. The reason is that the anchorage constrains the anchored foundation in the tangential, radial and, to a certain degree, even in the vertical direction. As such, the torsion is caused by the type of loading and the cross section shape, whereas the torsion of the unanchored ring is the result of the displacement behavior of a curved cantilever beam loaded with a distributed vertical load.

5.2.3 Effect of different parameters of the exact settlement on the tank shell

Fig. 5-7 already showed that the maximum amount of axial force/stress of unanchored tanks does not vary very much in size for different settlement u_{max}. The most noticeable effect of increasing soil displacement is a very steep rise in shell stresses within a short distance. The steeper the applied soil settlement is the steeper is this rise.

More significant are the stresses for different angles on which settlement is applied. On the next page the following results are shown:

- Fig. 5-9a: Axial force at lower bound of tank shell (u_{max}=100mm)
- Fig. 5-9b: Vertical displacement of lower bound of tank shell (u_{max}=25mm)
- Fig. 5-9c: Vertical displacement of tank, foundation and soil (α=150° and u_{max}=100mm)

The shape of the axial stresses is already described above. Obviously, because of the neg. axial force at ϑ=0 for α=25° there is no uplifting of the tankshell. The applied settlement angle is too small and so the foundation is stiff enough to support the tank shell in this area. The foundation ring, however, bends enough to cause an uplifting in the area, where the settlement is not applied anymore.

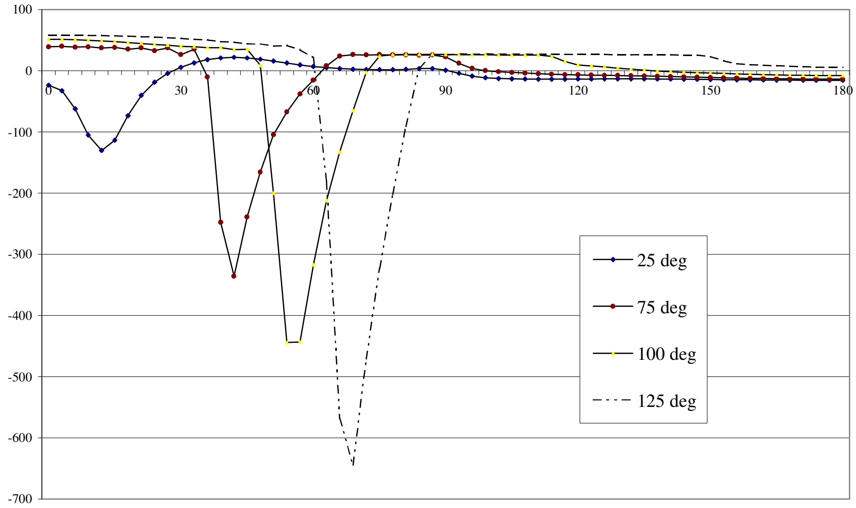


Fig. 5-9a: Axial force at lower bound of tank shell (u_{max}=100mm)

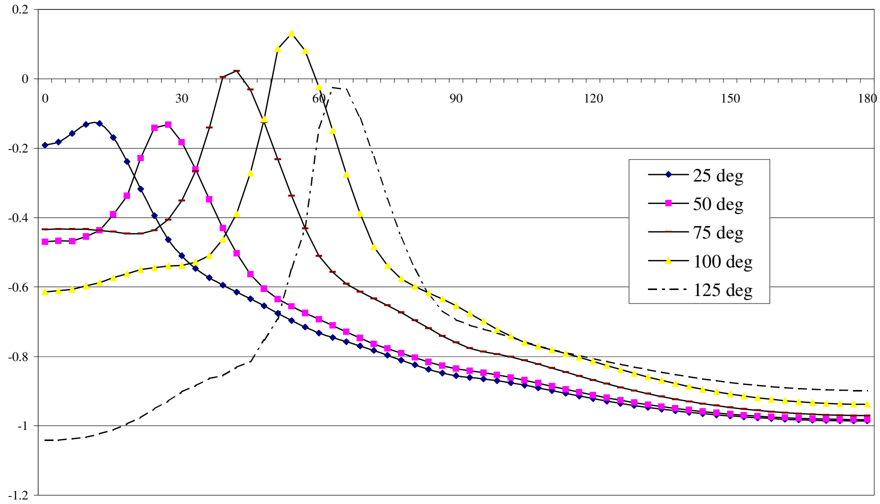


Fig. 5-9b: Vertical displacement of lower bound of tank shell (u_{max}=25mm)

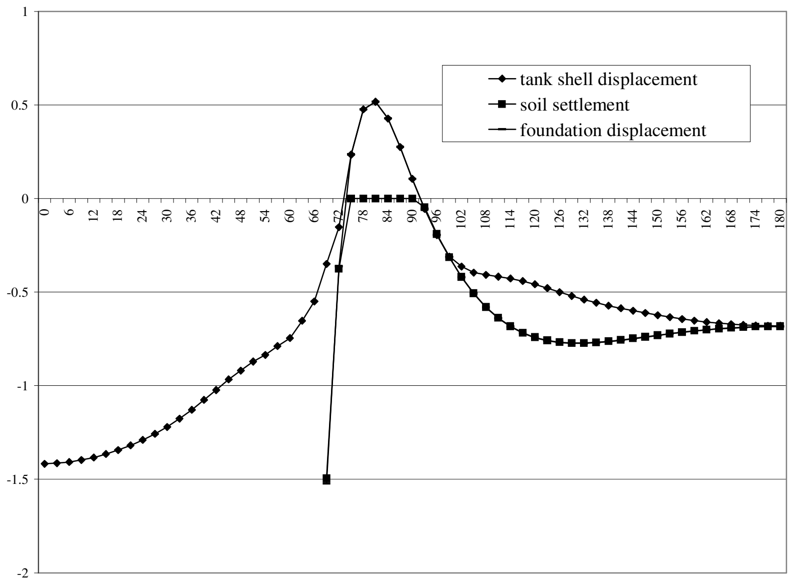


Fig. 5-9c: Vertical displacement of tank, foundation and soil (α=150° andu_{max}=100mm)

Another peculiarity appears for α=125°. The foundation is bent downwards by the water pressure on the tank base in the uplifted area. The tankshell is sitting on the foundation like a seesaw. The tensional axial force caused by the uplifted base shows also an uplifting up to ϑ=180°. The shell is therefore fully supported for $\vartheta\approx$60.. 87°.

This is also confirmed by the displacements in Fig. 5-9b. The curve for α=25° reaches the maximum displacement relating to the chosen soil stiffness. However, although there is no shell uplifting for α=125° and u_{max}=25°, the shell does not reach the full settlement of $u_s\approx$0.9mm. With u_{max}=25mm, the applied settlement is very small, and as such, so is the settlement of the foundation. This means that the foundation ring supports almost all of the uplifted base, and so, the tensional force in the shell stays at a low level. Additionally, the shell is not bent as it is for u_{max}=100mm (Fig. 5-9c) where also an uplifting of the foundation from the soil occurs at ϑ=102..174°.

Fig. 5-9c shows the different displacement of foundation and tankshell together with the applied exact settlement. For the sake of clarity the displacement of soil and foundation is not shown for ϑ=0..60°. The foundation is separated from the soil for ϑ=0..60° (uplifting).

5.2.4 Comprehension of Fourier composed settlement

In the following the difference in the results due to different foundation conditions is examined. The cases investigated by previous research assumed the existence of a foundation but the displacement of the shell, as described above, is given by a deformed foundation. This deformation, however, is not equivalent to the deformations of the underground. The following diagrams show the axial forces of a tank without a foundation and with two different foundation rings. The stiffness of the foundation ring depends on its E*I, where E is the E-modulus and I the moment of inertia. The E-Modulus of a concrete foundation is generally constant because only the minimum required reinforcement is used and this amount is normally not changed. Another E*I is usually achieved by a change of the cross section size of the foundation, however, this would require a new FE-Model. To circumvent this, the E-modulus was changed from original E=2.6e4N/mm to E=50e4. This corresponds to an enlargement of the foundation height from h=0.8m to h=2.14m.

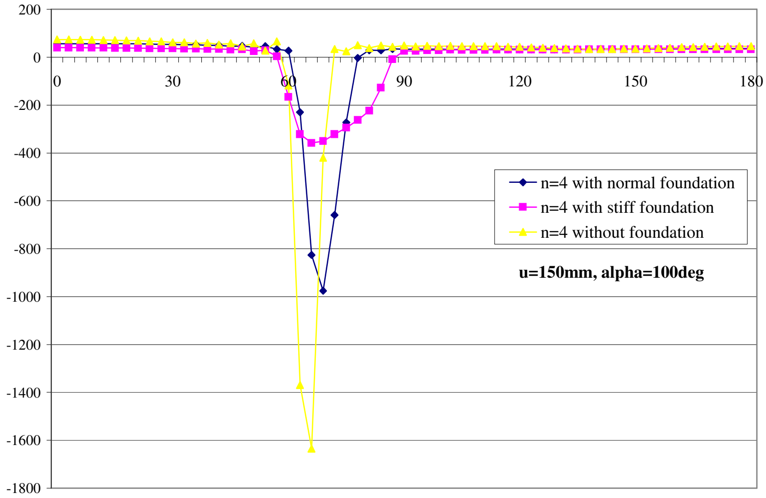


Fig. 5-10a: Different foundation conditions for n=4

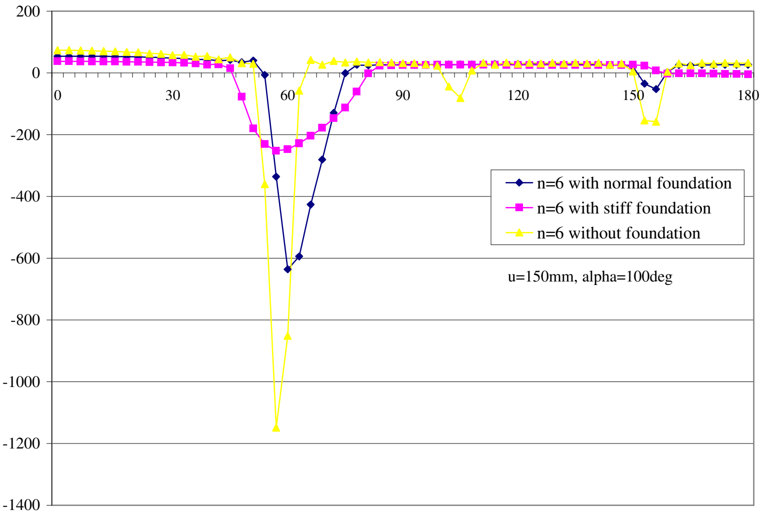


Fig. 5-10b: Different foundation conditions for n=6

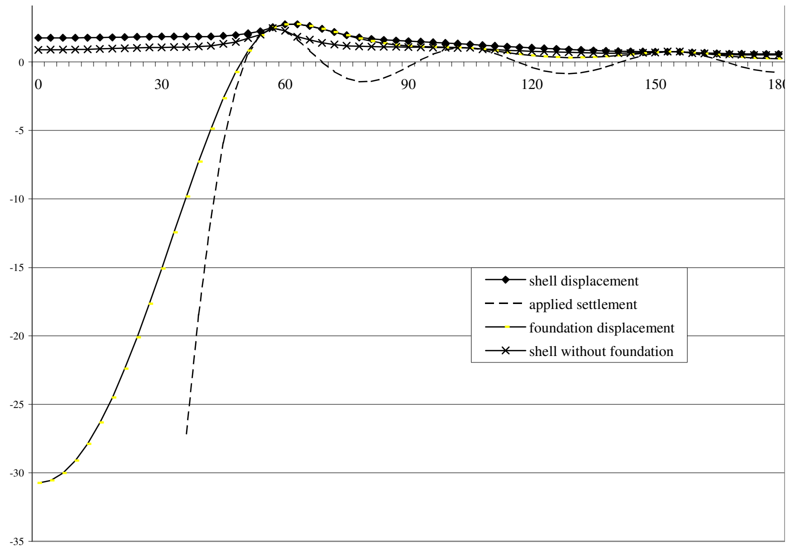


Fig. 5-10c: Different displacements of model components for n=6

The settlement in Fig. 5-10a-c is applied with α=100° and u_{max}=150mm. The effect can be clearly observed. The foundation lowers the maximum stresses and distributes the pressure onto a bigger area. With the foundation the area, where the pressure is submitted, is enlarged. Normally the edge of the applied settlement is too steep such that the tank shell can follow it. The settlement is also too steep for the foundation, however, because the foundation moment of inertia is less then the axial bending stiffness of the tank shell, the foundation compensates for the stress peaks. In the calculated models the maximum axial stress amounts to 60% of the stresses without the foundation, and 21%, if the calculation was done with the stiff foundation. As already shown, the foundation ring does not support the shell in the area between the settlement peaks because it is too soft in comparison to the tank shell. The base, however, is still supported by the foundation ring.

5.2.5 Influence of tank geometry

A characteristic for the behavior of the tank shell is the geometry of the tank. It can vary in the radius [r], height [h], number of courses, course thicknesses [t] etc. The main effect is shown by the different bending stiffness and the main criteria is the ratio r/h. The following diagrams will show the effect of this relation between the stiffness of the tank and the foundation. The foundation cross section stays constant.

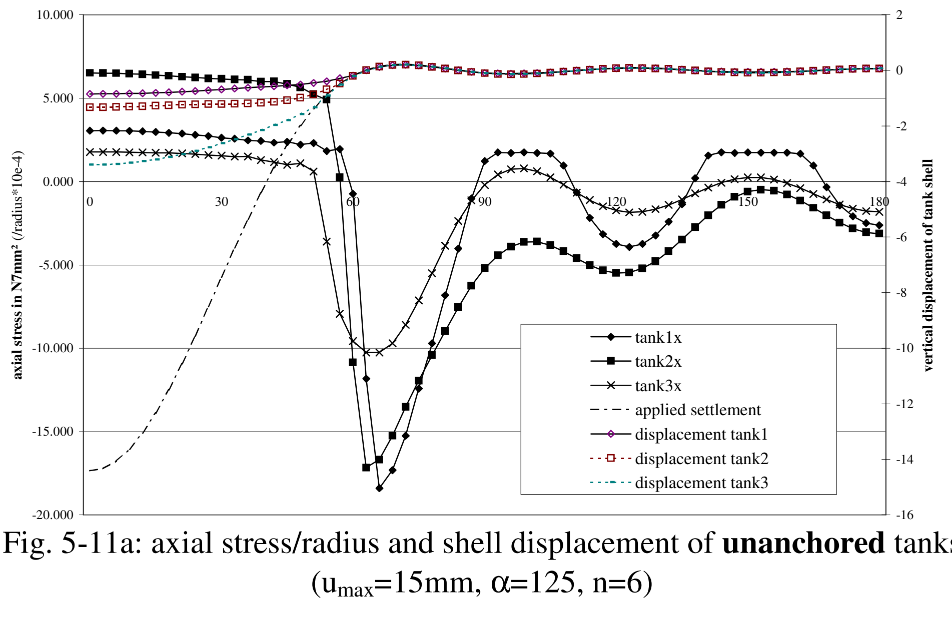


Fig. 5-11a: axial stress/radius and shell displacement of **unanchored** tanks (u_{max}=15mm, α=125, n=6)

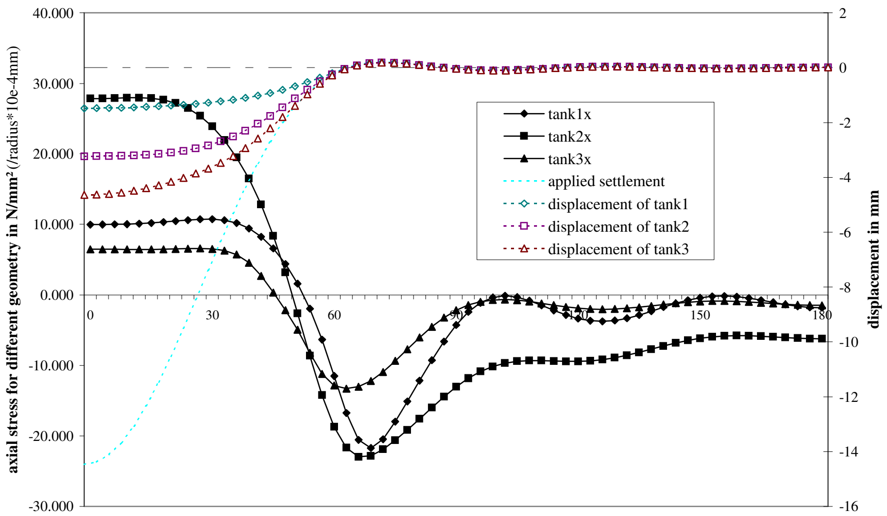


Fig. 5-11b: axial stress/radius and shell displacement of **anchored** tanks (u_{max}=15mm, α=125, n=6)

Because of the bigger radius of tank3x the load per unit angle carried by the tank shell is larger than for tank1x and tank2x. For a better comparison in Fig. 5-11a/b, the stress curve of tank3x is divided by r_{tank3x}/r_{tank1x}=2.15. The profile of the stress curve is not considered here. The different behavior due to uplifting in the area ϑ=90..180° of tank1x and tank3x in comparison to tank2x, and the different values of stresses and displacements result from the dependence of the stresses on the wave number n and the tank geometry as shown in *chapter 2*.

6 SKETCH OF AN ANALYTICAL MODEL

Previous research has already found analytical solutions for the stresses in the tank shell. It would be good if those models can be extended for tanks with a foundation ring. For this, it is necessary to have information about the ability of the foundation to distribute the soil deformation more evenly to the tank shell. Additionally, one must know how much the foundation is relieving the tank shell from the load of the uplifted tank base.

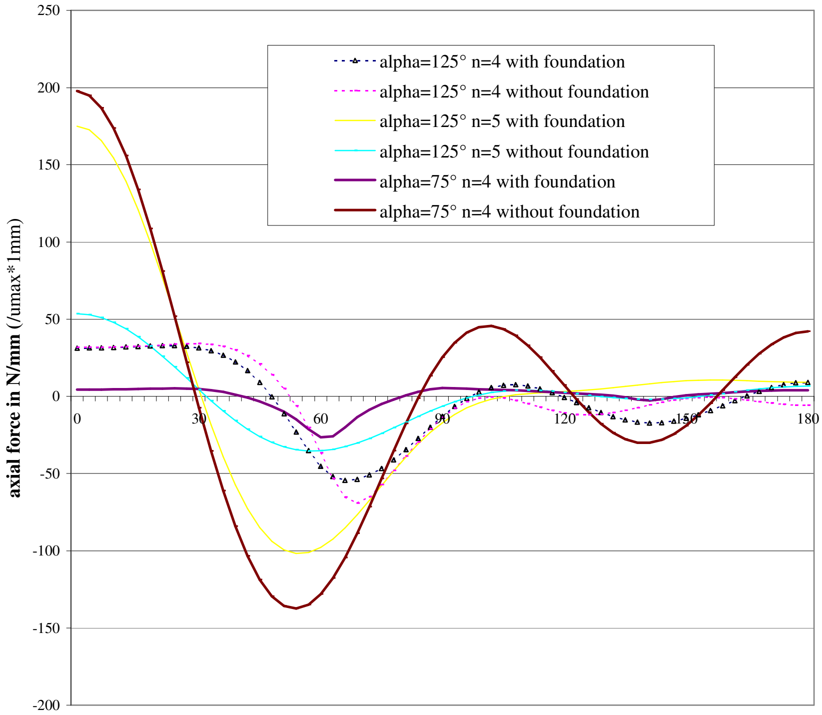


Fig 6-1: difference of tanks with and without foundation

In this chapter a first attempt is shown for having a procedure to derive the maximum compressive axial stress of a anchored tank with foundation from the stresses of an anchored tank without a foundation ring. The main idea is to approximate the shape of the deformed foundation and to define a new settlement for the same tank but without a foundation. The maximum stress for this tank can be get according to *Chapter 2* and its maximum compressive stress should be similar to the one of the model with foundation.

For this first draft following assumptions are made:

- The tank is anchored, i.e. the tank shell is connected to the foundation.
- The contact area between soil and foundation ring is concentrated at the high points of the applied settlement. The foundation is assumed to be too stiff to follow the applied settlement. The
- The foundation ring is able to give the shell the necessary support in the uplifted area.
- The curvature of the foundation is small enough, that the curvature radius of the foundation middle is approximate the curvature radius of the foundation top surface.

The settlement is given as $u = \sum_{n=0}^{n} u_n \cos(n\vartheta)$ $,0 < \vartheta < \pi$ (equation (2) in *chapter 2* with n coefficients).

Its maxima can be found by equating the first derivation to zero:

$$\frac{\partial u}{\partial \vartheta} = -\sum_{n=0}^{n} n u_n \sin(n\vartheta) \quad = 0 \tag{5}$$

The assumed positions of the maxima are $\vartheta=\vartheta_k$.

The foundation is assumed to behave as a beam with linear distributed load q_f. This load is composed of the roof load and the weight of the tank shell as well as the water pressure on the foundation. For a first try, it is assumed that q_f is a constant pressure. The beam length is $l=r*\pi$ with r=middle radius of foundation ring.

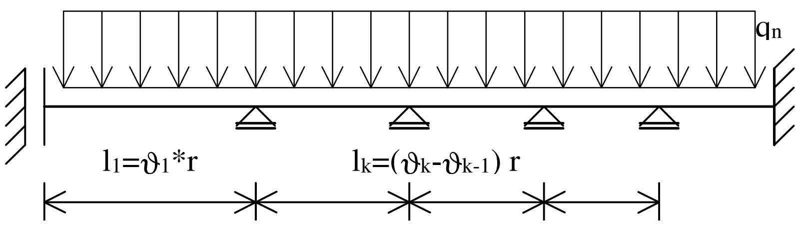


Fig.6-2: model for foundation ring

Relating to this model, the bending moment diagram function is $M=M_{(\vartheta)}$.Now the deflection curve w_f of the foundation ring can be obtained from the equation

$$EI\frac{\partial^2 w_f}{\partial \vartheta^2} = -M_{(\vartheta)} \tag{6}$$

and the correlative boundary conditions. E stands for the E-modulus of the concrete and the moment of inertia is $I=b*h^3/12$ with h and b as height and width of the foundation.

The deflection curve w_f describes the vertical displacement of the foundation centre and, as assumed above, the displacement of the foundation top. This curve can be seen as a composition of Fourier components. The appropriate coefficients can be obtained by a Fourier analysis. For getting the maximum compressive stress in the shell, a settlement composed of the new Fourier components must be taken. The tank is calculated due to this new settlement as described in c*hapter 2*.

Fig.6-3 shows an approximation for the derivation described above. At first a calculation was done for an anchored tank with foundation and u_{max}=15mm, α=125° and n=5. The first maximum of the applied settlement is found to be located at ϑ=72°. The maximum foundation deflection is $w_{f\ (\vartheta=0°)}$=1.478mm. This value is not calculated due to above static foundation model but it is taken from the FE-calculation of the founded tank. As it can be seen, uplifting occurs in the calculation of the tank with foundation and, therefore, q_f is not a constant pressure anymore. For a better comparison, the FE-calculated value is taken to show the general working of above described idea.

With the displacement $w_{f\ (\vartheta=0°)}$=1.478mm and α=144° two calculations were done. One with a tank including the water filling and the other only consisting of the tank shell. The applied settlement for the foundationless tanks is shaped as shown in equation (1). All models are anchored.

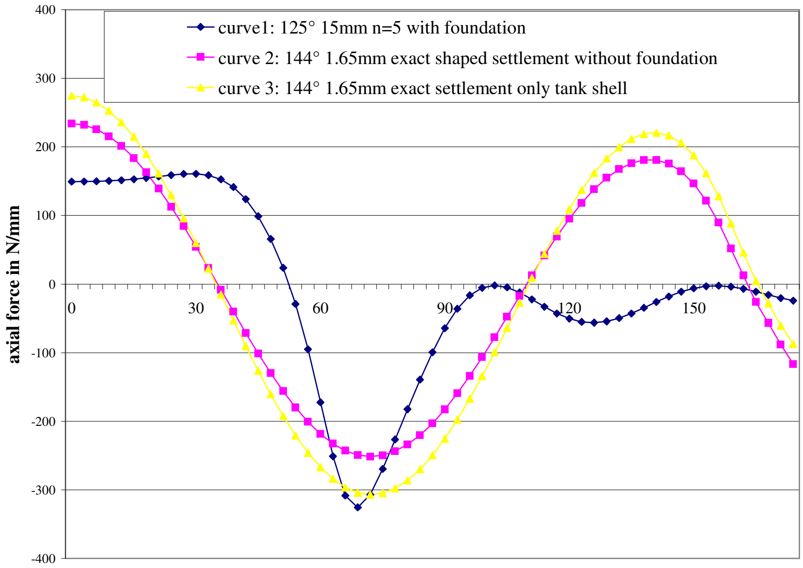


Fig. 6-3: Comparison of anchored tanks with and without foundation

As it can be seen the maximal axial compressive stress can be compared quite well. The maximum stress of the model without a base is closer to the stress of the founded tank. This effect is described in *chapter 5*. Additionally, the analytical solution described in *Chapter 2* is valid for a tank shell without a base.

7 CONCLUSION

This thesis dealt with the effect of a foundation ring on a cylindrical flat bottom tank due to uneven circumferential settlement. By the use of a foundation the tank behaves in general similar to a tank without a foundation. The foundation ring reduces the maximum strains and stresses and takes parts of the uplifted tank base.

Previous FE-models were extended to a tank model closer to the reality. It includes soil, tank shell, tank base and a foundation ring. The tank is assumed to be a fixed roof tank. For further investigations the boundary condition for this roof can be replaced by a circular primary wind girder.

A first idea was shown for getting the maximum compressive stress of the founded tank from the unfounded tank. Main problem for future investigations might be finding the point where the foundation is not able to support all loads from base and shell anymore. In the area where foundation and base are hanging on the shell the pressure q_d on the foundation must be replaced by a tensional linear distributed load.

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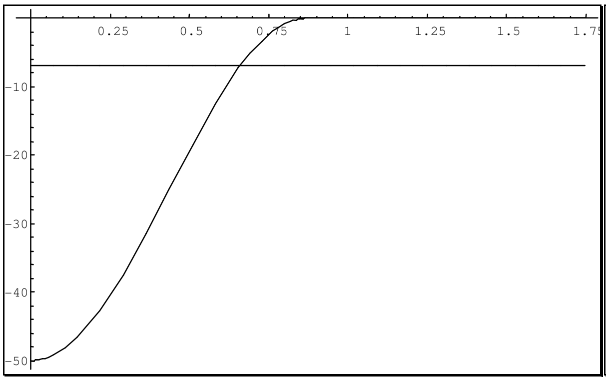
9 APPENDIX



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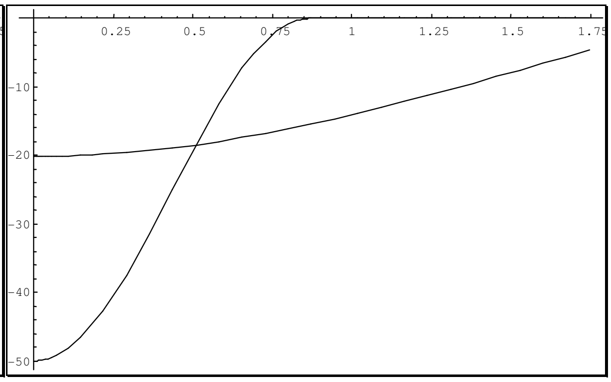

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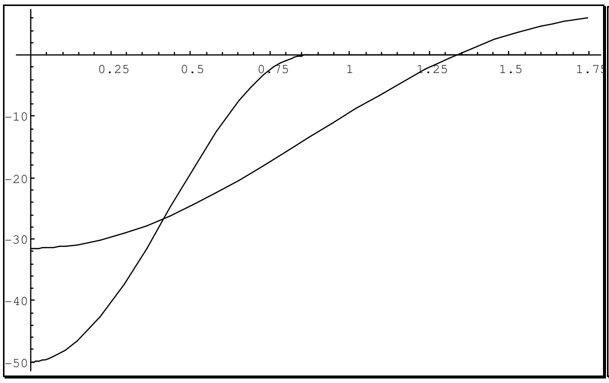

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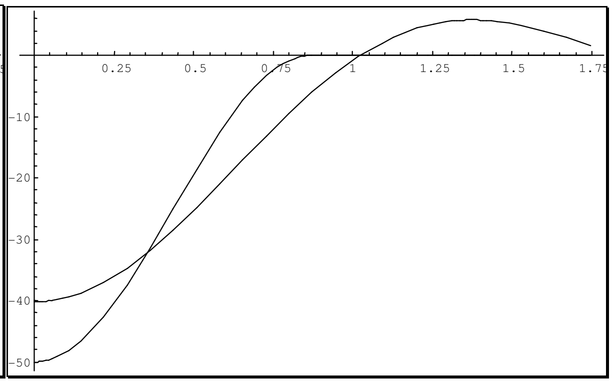

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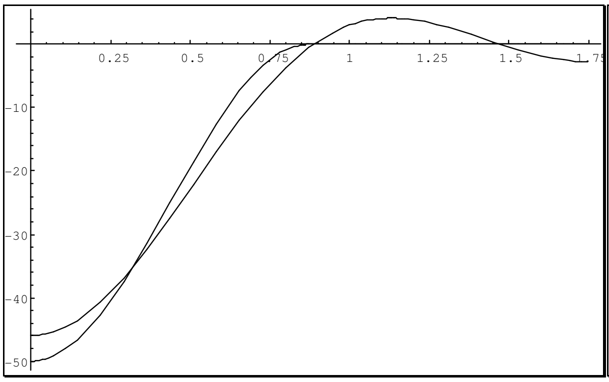

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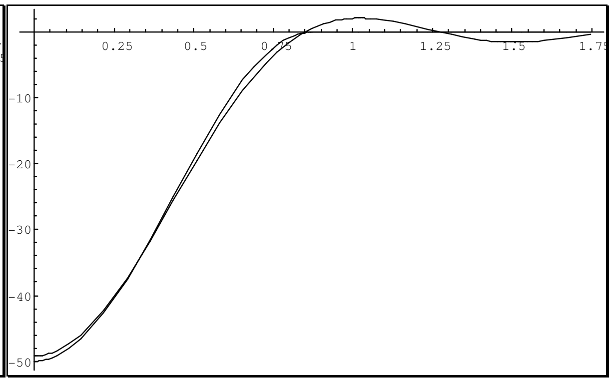

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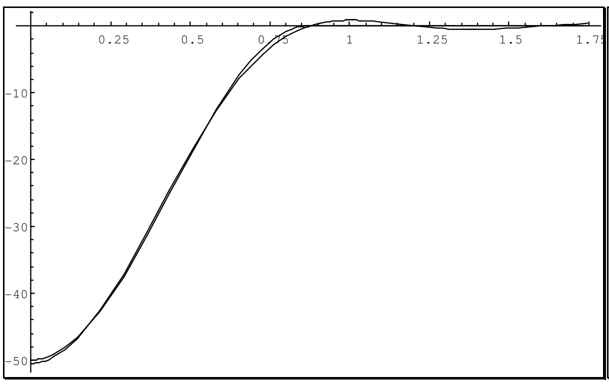

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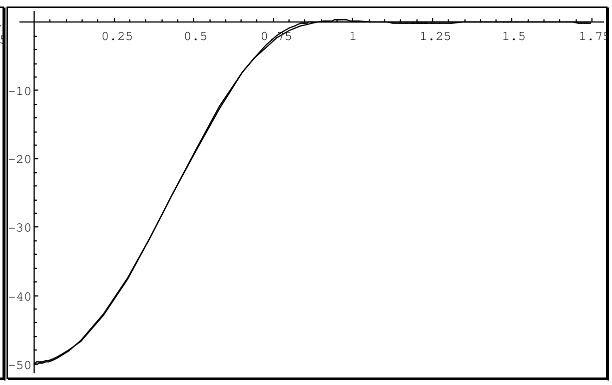

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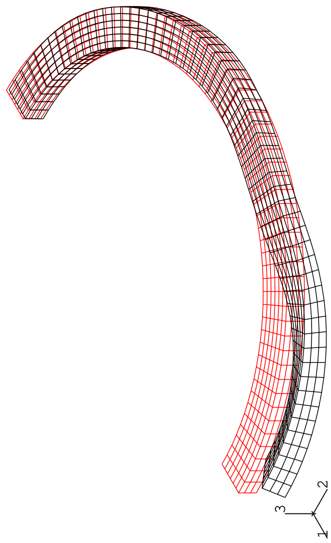
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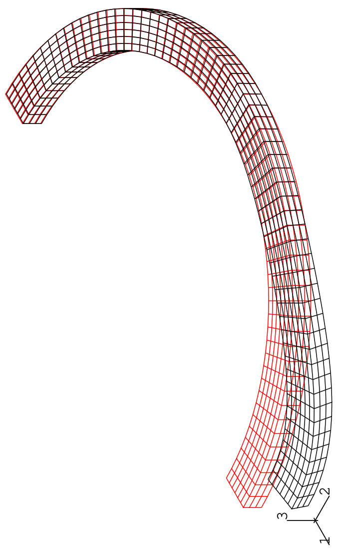
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