Generic approach liquid storage tanks

General approach liquid storage tanks for the seismic verification of industrial facilities in Groningen

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INTRODUCTION AND STARTING POINTS

1.1 Introduction

This document summarizes the approach for the seismic assessment of liquid storage tanks in Groningen, The Netherlands. The document has been prepared in accordance with the GBoD documents ([1] and [2]). Its goal is to provide a correct (compliant to the most relevant and newest design codes and relevant literature) and complete (covering all relevant failure mechanisms) overview on this topic. It also aims to help engineers perform calculations in a straightforward manner.

It is the responsibility of the reader to comply with all the relevant regulations for which specialized knowledge and experience in the field of seismic design of liquid storage tanks is required.

1.2 Seismic verification method

As of December 2017[1] the seismic verification method for industrial facilities in Groningen can be either the semi-deterministic method ‘LoC-toets’ as described in the GBoD, or alternatively the ‘risicogebaseerde rekenmethodiek’ developed by TNO/Deltares. The verifications listed in this generic approach have at the moment only been reviewed and approved by TU Delft for use with the ‘LoC-toets’. These verification however can be applied with both methods, but the application of this document together with the ‘risicogebaseerde rekenmethodiek’ has not yet been validated by TNO/Deltares. Therefore, the application of this document in combination with the ‘risicogebaseerde rekenmethodiek’ should be done with prudence.

1.3 Scope of document

This document discusses the seismic verification of existing steel liquid storage tanks according to simplified provisions which can be found in European standards such as NEN-EN 1998-4 [10] and NEN-EN 14015 [13]. From previous studies on the subject it became apparent that relevant standards contain minor errors and are, at points, incomplete and unclear when it comes to the implementation of the theory in practical design cases. This leaves room for different interpretations by the end users which is not wanted, especially when the applied methods contradict some other, often applied, design standards. This document has been prepared to clarify these aspects and to set a generic framework for use of the different standards and guidelines to liquid storage tanks in Groningen. This generic approach is not a substitution of the aforementioned design standards and is complementary to the GBoD documents ([1] and [2]). The objective of this technical note is to make a first verification of liquid storage tanks in Groningen in a uniform, complete, effective and quick manner, preferably without the need for more complex finite element (FEM) calculations. The procedure involves:

1 Determination of the seismic loads on the tank according to the GBoD.
2 Verification of the tank structure with respect to failure due to exceedance of strength and/or stability according to the failure modes specified in this generic approach.

[1] More information on the two methods are related background documents can be found on the NCG website: https://www.nationaalcoordinatorgroningen.nl/themas/c/chemische-industrie

3 Verification of the foundation structure as explained in the generic approach.
4 Conclude on whether the tank fulfils the seismic verification outlined in this generic approach.

This generic approach has been specifically written for liquid storage tanks which are:
- Vertical and cylindrical.
- Made of steel.
- Located above ground.
- Under atmospheric pressure (with or without floating roof).
- Are containing liquids of ambient temperature.
- Situated at ground level by means of shallow or piled foundation.

Liquid storage tanks that do not fulfil the aforementioned characteristics[1] cannot be analysed according to the procedures described in this document. Although some content of this document can be applied to these other types of storage tanks, this document is not specifically written for them. Therefore the usage of this document for purposes other than the ones for which it is meant for, should be done with great cautiousness from the engineer.

Tanks which are exposed to only minor seismic action do not have to be verified. Article 3.2.1, clause 5(P) of NEN-EN 1998-1 suggests the following threshold values for both the horizontal and vertical[2] seismic excitations:
- $a_g \leq 0.04g$ ($0.39\ m/s^2$).

If a liquid storage tank complies to the above characteristics and fulfils the verifications elaborated in this document, it is safe to conclude that the tank can withstand the seismic load corresponding to the selected verification method and no further calculations are required. If not, then further steps are required by the tank's owner and/or consultant, e.g.:
- Perform a more detailed (FEM) analysis.
- Consider strengthening of the tanks (seismic retrofitting).
- Reduce the fluid content volume to the level that does fulfil the required seismic load according to this generic approach.

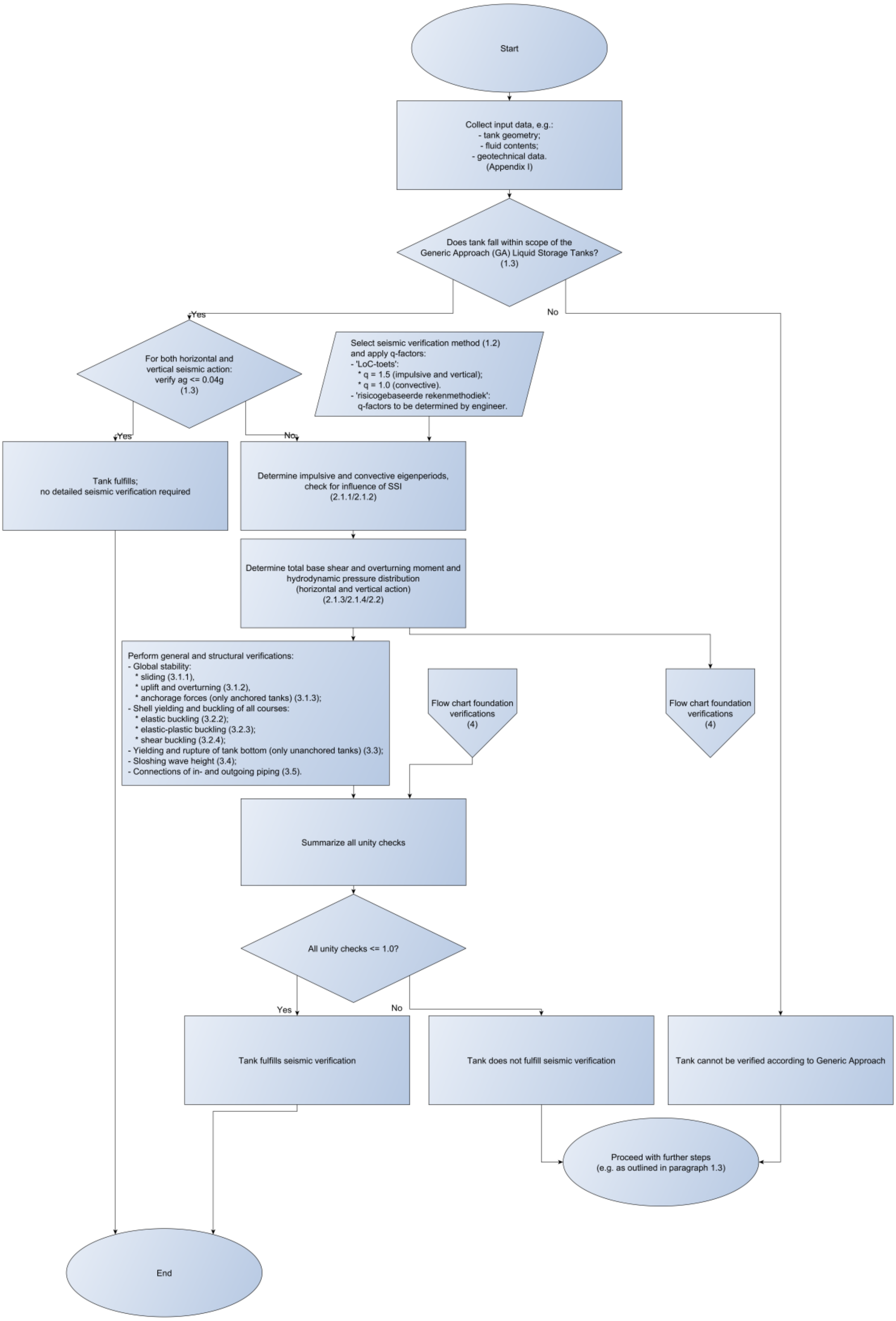
1.4 Flow chart

A flow chart depicting the steps for seismic verifications (from chapters 2 and 3) is shown in figure 1.1. A flow chart for the tanks foundation verifications is provided in chapter 4.

1 Tank structures like silos, horizontal tanks or storage containers made of fibre-reinforced plastics (FRP).

2 The aforementioned clause of NEN-EN 1998-1 specifically holds for the horizontal seismic action. Given the fact that the V/H ratio of the induced earthquakes in Groningen is (in general) quite high compared to tectonic earthquakes, it is recommended by TU Delft to apply the same threshold values for the vertical seismic action (and until more thorough studies show otherwise). Also refer to paragraph 2.2 which explains the importance of the vertical component of the earthquake load.

Figure 1.1 Flow chart for seismic analysis of liquid storage tanks in Groningen



1.5 Standards, guidelines and other documents

In accordance with the GBoD ([1],[2]) the verifications in this document are based on the Eurocodes, more specific NEN-EN 1998-4. A full list of applied standards, guidelines and other documents is presented below:

General documents

[1] Bijlage 4, Rapportage werkgroep Maatgevende aardbevingsbelasting voor de industrie: Naar een snelle, simpele, transparante en robuuste toets op de aardbevingsbestendigheid van de chemische industrie in Groningen (4-11-2016).

[2] TU Delft (2017), Explanatory notes for the 'LoC Toets' in application to the industrial facilities in Groningen, Doc. Nr. CM-2016-19D1, 1 February 2017.

Dutch standards and Eurocodes

[3] Nederlandse praktijkrichtlijn NPR 9998:2015 - Assessment of buildings of erection, reconstruction and disapproval - Basic rules for seismic actions: induced earthquakes.

[4] Nederlandse praktijkrichtlijn (Ontw.) NPR 9998:2017 - Assessment of structural safety of buildings in cases of erection, reconstruction and disapproval - Basic rules for seismic actions: induced earthquakes.

[5] NEN-EN 1992-1+C2:2011+NB:2016, Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings.

[6] NEN-EN 1993-1-1+C2+A1:2016+NB:2016, Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings.

[7] NEN-EN 1993-1-6:2007+C1:2009+NB:2011, Eurocode 3: Design of steel structures - Part 1-6: General - Strength and stability of shell structures.

[8] NEN-EN 1993-4-1:2007+C1:2009+NB:2012; Eurocode 3: Design of steel structures - Part 4-1: Silos.

[9] NEN-EN 1993-4-2:2007+NB:2012; Eurocode 3: Design of steel structures - Part 4-2: Tanks.

[10] NEN-EN 1998-4:2007, Eurocode 8 - Design of structures for earthquake resistance - Part 4: Silos, tanks and pipelines.

[11] NEN-EN 1998-5:2005, Eurocode 8 - Design of structures for earthquake resistance - Part 5: Foundations, retaining structures and geotechnical aspects.

[12] NEN 9997-1+C2:2017, Geotechnical design of structures - Part 1: General rules.

[13] NEN-EN 14015:2004, Specification for the design and manufacture of site built, vertical, cylindrical, flat-bottomed, above ground, welded, steel tanks for the storage of liquids at ambient temperature and above.

Other standards, guidelines and literature

[14] API standard 650 (2012), Welded Tanks for Oil Storage, Eleventh edition, effective date: February 1 2012.

[15] Design Recommendations for Storage Tanks and Their support with Emphasis on Seismic Design (2010 edition), Architectural Institute of Japan.

[16] NZSEE Seismic Design of Storage Tanks (2009), Recommendations of a Study Group of the New Zealand Society for Earthquake Engineering.

[17] Housner, G.W. (1954), Earthquake pressures on fluid containers, California Institute of Technology Pasadena.

[18] Haroun, M.A., Housner, G.W. (1981); Seismic Design of Liquid Storage Tanks; Journal of the Technical Counsils of ASCE.

[19] Scharf, K. (1990), Beiträge zur Erfassung des Verhaltens von erdbebenerregten, oberirdischen Tankbauwerken, Fortschritt-Berichte VDI, Reihe 4. Bauingenieurwesen, Nr. 97, VDI Verlag, Düsseldorf.

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[24] Bray, J.D., Macedo, J. (2017), 6th Ishihara lecture: Simplified procedure for estimating liquefaction-induced building settlement., Soil Dynamics and Earthquake Engineering, Volume 102, November 2017.

1.6 Failure mechanisms which are elaborated in this document

It should be noted that hand calculations from code provisions cannot comprehend all failure modes. The following list gives a summation of failure modes which are discussed in design codes, can be verified analytically and should be performed in the seismic verifications (also refer to [2] and NEN-EN 1998-4):
- Global stability of anchored and unanchored (uplifting) tanks.
- Yielding and (meridional and shear) buckling of the shell.
- Yielding and rupture of the tank bottom (annular and bottom plates).
- Sloshing wave height with respect to available freeboard.
- Connections of in- and outgoing piping.
- Tank foundation verification including soil liquefaction.

Global buckling of stiffening girders has been observed as a failure mechanism in the case of structures subjected to ground excitation caused by tectonic earthquakes. However is seems unlikely for tanks in Groningen given the relatively low seismic loads and other failure mechanisms which are expected to occur before global buckling of the stiffening girders becomes governing. For that reason, this failure mechanism is not included in this generic approach[1].

1.7 Fluid content volume

EN 1998-4, article 2.5.2, clause (4)P states that levels of filling should be considered: empty or full. The seismic verification for the full case should be performed with the conservative upper limit of the operational fluid content that is representative over time. If such data is unknown, the analysis should conservatively be performed at the maximum fill level. In general, the empty fill case is not governing for the specific liquid storage tanks within the scope of this document because of the much reduced fluid mass compared to the full case.

1.8 Example calculations

For example calculations on seismic verification of liquid storage tanks one is referred to:
- Annex E of API 650 [14].
- Annex B of NZSEE Seismic Design of Storage Tanks [16].

Although they do not follow the exact same procedure as described in this technical note, they provide insight in how to apply the provisions and equations given in the design standards and guidelines referred to in this generic approach.

[1] Nor it is treated in simplified recommendations/standards on seismic design such as ([14], [16]), as it is a failure mechanism that requires a FEM analysis to analyse properly.

2

MODAL RESPONSE SPECTRUM METHOD OF ANALYSIS (MRSA)

2.1 Simplified model for horizontal seismic action including soil-structure interaction (SSI)

2.1.1 Fundamental eigenperiod for tanks on rigid foundations

In annex A.3.2.2 of NEN-EN 1998-4 [10], the simplified analysis for liquid storage tanks is explained. In principle, the method follows Housner's ([17],[18]) approach in which a tank-liquid system is modelled as two uncoupled single-degree-of-freedom (SDOF) systems with an impulsive mass and a convective mass. The method includes the tank wall flexibility in the calculation of the fundamental eigenperiod of the system. In NEN-EN 1998-4 two expressions for the fundamental eigenperiod for a rigid foundation are presented:

- Equation A.24, based on Scharf [19] and DIN 4119; a predecessor to (NEN-)EN 14015. The given expression is, however, **wrongfully copied from Scharf and should include a factor 2 in the denominator. The correct expression (expressed in terms of period) should read**:
  $T_f = 2 \cdot R \cdot (0.157 \cdot \gamma^2 + \gamma + 1.49) \cdot ((E \cdot s(\varsigma)) / (\rho \cdot H))^{-1/2}$ (for $\varsigma = 1/3$).
- Equation A.36, similar to API 650 [14], annex E. This expression is also used by NEN-EN 14015, annex G and the New Zealand recommendations on seismic design of storage tanks [16].

In general, equation A.24 gives a lower fundamental period and thus a lower spectral acceleration (in most cases the fundamental period is smaller than the peak period of the response spectrum). Combined with the fact that the equation A.36 is more widely adopted internationally, the usage of equation A.36 has a clear preference and will be adopted hereafter.

2.1.2 Fundamental eigenperiod for tanks on non-rigid foundations (including SSI)

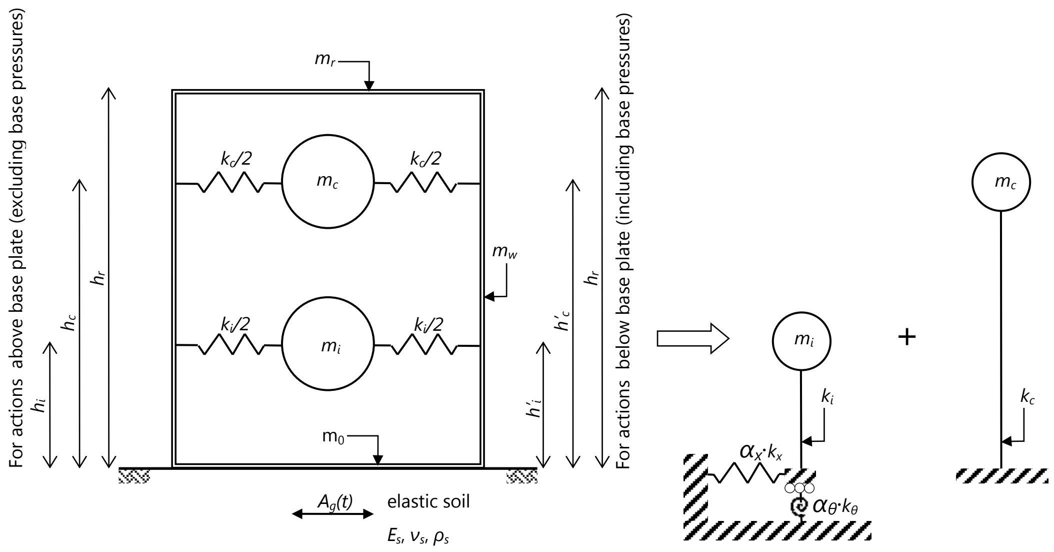
The simplified model as described in annex A.3.2.2 assumes a rigid foundation and therefore does not include the period elongation caused by soil-structure interaction (SSI). For Groningen, the soil conditions cannot be described as rigid and SSI period elongation shall be included to better define the effective eigenperiod of the tank. This only applies to the impulsive mode. The convective period(s) (for ground-supported tanks) are taken as independent of tank or soil flexibility. SSI will also result in increased damping, but this effect shall not be included with the 'LoC toets'.

Soil-structure interaction in given in NEN-EN 1998-4, annex A.7, equation A.53 where a simplified procedure is given. This procedure can also be applied on the simplified two SDOF system in annex A.3.2.2 by applying $m_f = m_i + m_w + m_r$ and $h_f = h_i$. The frequency dependent factors α_x and α_θ can be obtained by an iterative process. This simplified procedure uses a homogenous elastic halfspace to describe the underlying soil. The selection of the elastic soil parameters should therefore be chosen such that the homogenous soil predicts the actual (multi-layered non-linear) soil behaviour underneath the tank. More details can be found in Gazetas [20] and the New Zealand recommendations [16]. Annex B of the latter [16] includes example calculations on how to perform this iterative SSI procedure.

The selection of the elastic soil parameters should be performed by an geotechnical engineer. It is recommended to perform an sensitivity analysis to discover the influence of the elastic soil parameters on the fundamental period and thus the spectral acceleration of the impulsive mode. From this the most conservative parameters should be selected[1].

A visual presentation of the mechanical model of the simplified procedure with SSI is provided in figure 2.1.

Figure 2.1 Simplified uncoupled two SDOF model for flexible tanks with SSI (period elongation, no damping) based on elastic soil for horizontal seismic action (with NEN-EN 1998-4 nomenclature)



2.1.3 Total base shear and overturning moment and combination rule

The overall overturning moment and base shear force can be calculated by NEN-EN 1998-4, annex A.3.2.2.2. The contribution of the impulsive and convective modes in the physical quantity of interest, i.e. shear force or overturning moment, shall be based on the absolute summation rule (as presented in the Eurocode).

2.1.4 Hydrodynamic masses and pressure distributions

The impulsive and convective hydrodynamic mass and acting heights shall be obtained from linear interpolation of table A.2 from NEN-EN 1998-4[2], or identical, from annex G.2 of NEN-EN 14015.

2.2 Vertical seismic action and combination rule

Although the vertical action (NEN-EN 1998-4, annex A.2.2 and A.3.3) of the earthquake will not cause a base shear or overturning moment, it will increase or decrease the internal pressure exerted on the walls of the tank. For this reason it should be included in the buckling verification as discussed in chapter 3. The spectral acceleration should include the effect of soil-structure interaction obtained from annex A.7 of NEN-EN 1998-

[1] Alternatively, one can simply select the fundamental period of the tank-fluid-soil system as the peak period of the response spectrum and perform the verifications outlined in the generic approach. If the tank fulfils all the verifications with this conservative upper limit approach, one can conclude that the tank passes the seismic verification method ('LoC toets' or the 'risicogebaseerde rekenmethodiek') without the need of performing the SSI analysis.

[2] Alternatively, they can be obtained in more detail from NEN-EN 1998-4, annexes A2.1.2 and A2.1.3.

4. The procedure is similar to the one as described in section 2.1.2 for horizontal seismic actions. The vertical and horizontal seismic action should be combined with the absolute summation rule[1].

2.3 Behaviour factors

For the 'LoC toets' the behaviour factors q on linear analyses are set ([2],[10]) to:
- q = 1.5 for the horizontal impulsive mode and vertical modes.
- q = 1.0 for the horizontal convective mode.

This is in accordance with article 4.4 of NEN-EN 1998-4, in particular clauses (1)P, (2)P and (3)P as defined therein for liquid storage tanks.

1 Because of the relatively short duration of induced earthquakes, the SRSS rule or the other rules mentioned in article 4.3.3.5 may be unconservative for liquid storage tanks. Therefore, for the screening described in this generic approach the absolute summation rule is adopted.

3

GENERAL AND STRUCTURAL VERIFICATIONS

3.1 Global stability of anchored and unanchored (uplifting) tanks

3.1.1 Sliding

Unless special measures against sliding have been taken, the sliding resistance should be calculated as the lowest coefficient of friction in the system times the acting vertical load. The vertical acceleration component shall be included in this verification. One is referred to equation 6.2 of NEN 9997-1 [12]:

$H_d \leq R_d + R_{p;d}$

in which:

H_d: the design value of the base shear force calculated from section 2.1.3.
$R_{p;d}$: the design value of the resisting force acting on the sides of the foundation (if available).
R_d: $\mu \cdot (V'_d - F_{v;d})$.
μ: the lowest coefficient of friction in the foundation plane (e.g. soil-concrete of steel-concrete).
V'_d: the effective vertical (static) force acting perpendicular to the foundation plane.
$F_{v;d}$: the force (absolute value) resulting from the vertical seismic excitation from paragraph 2.2.

3.1.2 Uplift and overturning

In case of unanchored tanks, the overturning moment can cause uplift of the base. The main effect of uplift is an increase in the axial compressive stress in the shell and possible rupture in the annular/bottom plates. For anchored tanks and the axial stress due to overturning follows from the overturning moment divided by the elastic section modulus of thin walled ring ($\pi \cdot R^2 \cdot t$). For unanchored tanks (or anchored tanks with ductile anchor behaviour), the axial compressive stress will increase as a result of uplift and therefore this effect needs to be taken into account when verifying shell buckling. The amount of uplift and increase of normal forces in the shell can be calculated[1] by linear interpolation with NEN-EN 1998-4, annexes A.9.1 through A.9.3.

3.1.3 Anchorage forces in case of anchored tanks

Depending on the type of anchors, the resistance of anchored tanks should be calculated accordingly to NEN-EN 1990 series. The occurring maximum force in the anchors depends on the overturning moment and assumed (linear or plastic) distributed of anchor forces. Details can be found in annex G.5 of NEN-EN 14015 and [16]. Note that equation G.11 of NEN-EN 14015 is valid for a linear distribution and **should read D^2 (diameter squared) instead of D_2**.

[1] The compressive force at the bottom shell (including the effect of uplift) can alternatively be calculated with NEN-EN 14015, annexes G.4.1 and G.4.2, with the convenience of direct calculation of the load increment factor without the need of linear interpolation.

3.2 Yielding and buckling of shell courses

3.2.1 Introduction

Types of shell buckling

Three types of shell buckling can be identified when evaluating liquid storage tanks:
- Elastic buckling.
- Elastic-plastic buckling ('elephant foot' buckling).
- Shear buckling.

They are discussed in more detail in sections 3.2.2, 3.2.3 and 3.2.4, respectively. Note that shear buckling is not included in ([13], [14], [16]), but needs to be verified according to the Eurocode[1].

Buckling verifications in higher shell courses

It is important to realise that the hydrodynamic pressure exerted on the inner walls and plate of the tank structure during seismic excitation (caused by both horizontal and vertical seismic excitation), defines not only the loading that the structure experiences but also its resistance to deformation of a certain type. Whereas the former statement is obvious, i.e. definition of loading, the latter one needs further explanation. It should become clear that an increase in the hydrodynamic pressure can significantly alter the hoop stress that the shell membrane experiences and, in turn, its own resistance. Thus, in case of tanks of varying wall thickness, all shell courses need to be verified against the aforementioned buckling failure modes.

To illustrate this point, figure 3.1 shows a typical shear force and overturning moment distribution over the height of the wall measured from the base plate. Anticipating the fact that the distributions of both the shear force and the overturning moment are non-linear with height, one cannot exclude the possibility that an upper shell course (of reduced thickness) buckles while the bottom one remains intact.

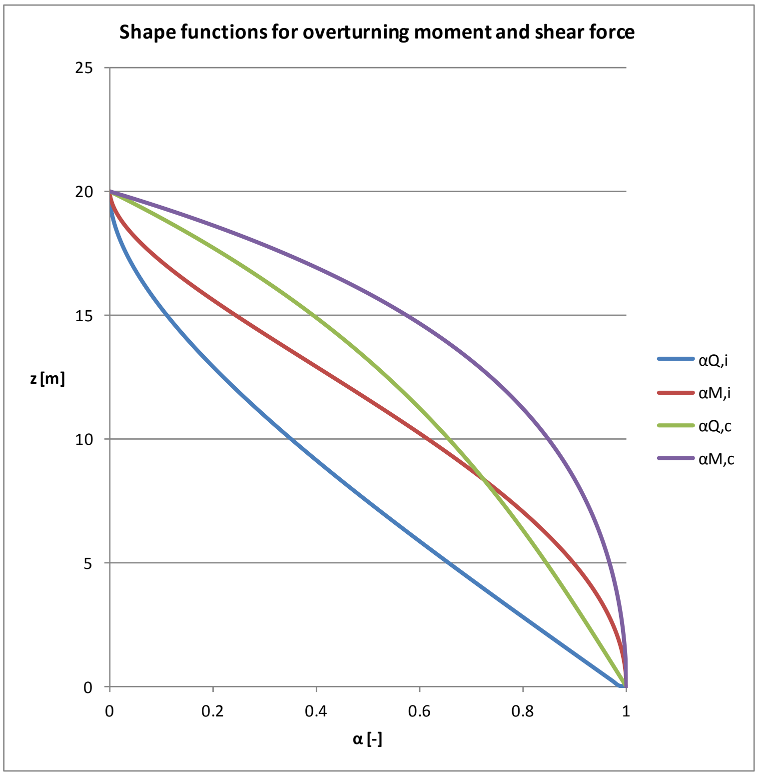
This phenomenon can be explained as follows. The resulting net shear force and moment decrease with increasing height and are maximum at the base of the structure. However, this does not mean that the resulting local stress reduces equally with height as the latter is also based on the shell course thickness. Additionally, one cannot a priori know the buckling resistance at a given height as the latter depends on both the thickness of the shell course and the exerted hydrodynamic pressure. **To exclude unexpected buckling of the upper shell courses when the latter are of reduced thickness, a buckling verification is required for all shell courses.**

This also holds in the case in which the design of the wall thickness is based on the linear hydrostatic pressure (decrease linearly with height) since the latter is non-compliant with the highly non-linear hydrodynamic pressure distribution. Clearly, for the case of shell of constant wall thickness buckling verification at the base of the structure suffices. As the impulsive component is dominant in the seismic response one can also estimate the shear force or bending moment at any level in the tank by integration of the stress distribution from NEN-EN 1998-4, annex A.2.1.2.

From figure 3.1 also follows that linear approximations (such as suggested by NEN-EN 14015, annex G.4.4) should be applied with caution, as they can underestimate the overturning moment in the bottom shell courses.

[1] NEN-EN 1998-4, article 3.5.2.2, clause (1)P, note (a)

Figure 3.1 Example of distribution of shear force (Q) and overturning moment (M) due to impulsive (i) and convective (c) action over the height of a H = R = 20 m tank



Shear and moment distributions along the height:

$M_i(z) = \alpha_{Mi}(z) \cdot M_{i,z=0}$ with $\alpha_{Mi}(z) = 1 - \frac{\int_0^z p_i(z) \cdot z \cdot dz}{\int_0^H p_i(z) \cdot z \cdot dz}$

$M_c(z) = \alpha_{Mc}(z) \cdot M_{c,z=0}$ with $\alpha_{Mc}(z) = 1 - \frac{\int_0^z p_c(z) \cdot z \cdot dz}{\int_0^H p_c(z) \cdot z \cdot dz}$

$Q_i(z) = \alpha_{Qi}(z) \cdot Q_{i,z=0}$ with $\alpha_{Qi}(z) = 1 - \frac{\int_0^z p_i(z) \cdot dz}{\int_0^H p_i(z) \cdot dz}$

$Q_c(z) = \alpha_{Qc}(z) \cdot Q_{c,z=0}$ with $\alpha_{Qc}(z) = 1 - \frac{\int_0^z p_c(z) \cdot dz}{\int_0^H p_c(z) \cdot dz}$

In which:

$M_{i,z=0}$: the overturning moment due to impulsive action at base level (z = 0).
$M_{c,z=0}$: the overturning moment due to convective action at base level.
$Q_{i,z=0}$: the base shear (z = 0) due to impulsive action.
$Q_{i,c=0}$: the base shear due to convective action.

For more details, one is referred to NEN-EN 1998-4, annex A.2.1.

In the verification of a tank with multiple shell courses, each shell course can be considered as an equivalent cylinder with a constant wall thickness equal to that of the shell course and cylinder length equal to the total length between the boundaries of the tank[1].

Partial factor on buckling γ_{M1}

Annex A.10 of NEN-EN 1998-4 provides simplified meridional buckling verification similar to NEN-EN 1993-4-1 and/or NEN-EN 1993-1-6. The last two codes do include the Eurocode's partial factor on stability γ_{M1}, while NEN-EN 1998-4 does not. For shell buckling, this partial factor is equal to 1.1 according to the Dutch

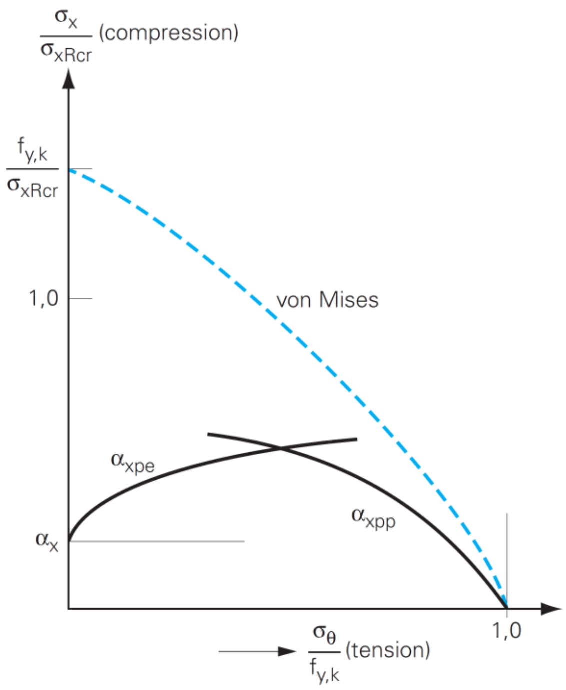
[1] As mentioned in annex D.2.2 of NEN-EN 1993-1-6

National Annex. In accordance with the aforementioned Eurocodes, a partial factor γ_{M1} = 1.1 shall be included when applying the simplified expressions as given in NEN-EN 1998-4, annex A.10.

Effect of internal pressure and vertical seismic action on buckling verifications

Compared to the case of an empty tank, the internal pressure initially stabilizes the tank against buckling. In case of increasing internal pressures, the circumferential hoop stress approaches the yield stress; close to this point elasto-plastic buckling can occur. Because of this, the hydrodynamic pressure due to vertical action needs to be combined with the horizontally induced hydrodynamic pressures. This phenomenon is visualized in figure 3.2. On the horizontal axis the ratio hoop stress / yield stress is presented. The lines α_{xpe} and α_{xpp} define the elastic and elastic-plastic imperfection factors from NEN-EN 1993-1-6. The actual meridional buckling stress is the lowest of the two meridional buckling types.

Figure 3.2 Schematic influence of tensile hoop stress on the meridional buckling stress



With elastic (meridional) buckling and shear buckling the internal pressure stabilizes the tank buckling and has therefore a positive effect. The pressure taken into account, in the verification of the capacity of the shell, shall therefore be the lowest possible. For elastic-plastic buckling (meridional) increasing internal pressure decreases the allowable (buckling) stress. Therefore the highest possible internal pressure needs to be considered.

In summary:

- **Elastic buckling: minimum internal pressure with absolute summation = $p_H + (p_i + p_c - p_v)$.**
- **Elastic-plastic buckling: maximum internal pressure with absolute summation = $p_H + (p_i + p_c + p_v)$.**
- **Shear buckling: minimum internal pressure = $p_H - p_v$.**

In which:

p_H	=	hydrostatic pressure.
p_i	=	rigid impulsive pressure caused by horizontal ground excitation (with SSI).
p_c	=	convective pressure caused by horizontal ground excitation (with SSI).
p_v	=	combined hydrodynamic pressure due to vertical seismic excitation (with SSI)
	=	$(p_{vr} + p_{vf})^{1/2}$.
p_{vr}	=	rigid hydrodynamic pressure due to vertical seismic excitation (with SSI).
p_{vf}	=	flexible hydrodynamic pressure due to vertical seismic excitation (with SSI).

All hydrodynamic pressures are calculated with annex A of NEN-EN 1998-4 [10]. As explained in paragraph 2.2, the absolute summation rule is used to combine horizontally and vertically excited modes.

3.2.2 Verification of elastic buckling

A simplified approach for verifying elastic buckling is provided in NEN-EN 1998-4, annex A.10.2 [10]. This verification includes the effect of internal pressure p as explained in the previous section. Equation A.62 of [10] can be expressed in terms of a unity check as follows:
Unity check = σ_m / {(0.19 · σ_{c1} + 0.81 · σ_p)} / γ_{M1}

Alternatively, the elastic buckling verification can be performed in more detail (if so required) with NEN-EN 1993-4-1 or NEN-EN 1993-1-6. If the unity check exceeds 1, then the seismic verification is not satisfied.

3.2.3 Verification of elastic-plastic buckling

A simplified approach for verifying elastic-plastic buckling is provided in NEN-EN 1998-4, annex A.10.3 [10]. This verification includes the effect of internal pressure p as explained in section3.2.1. Equation A.63 of [10] can be expressed in terms of a unity check as follows:
Unity check = σ_m / {σ_{c1} · [1 - $(p \cdot R)^2$ / $(s \cdot f_y)^2$] · (1 - 1 / (1.12 + $r^{1.5}$)) · [(r + f_y / 250) / (r + 1)]} / γ_{M1}

Note that equation A.69 of NEN-EN 1998-4 should read $r^{1.5}$ (as shown in the above equation) and not $r^{1.15}$ ([7],[8],[16],[19]).

Alternatively, the elastic buckling verification can be performed in more detail with NEN-EN 1993-4-1 or NEN-EN 1993-1-6. If the unity check exceeds 1, then the seismic verification is not satisfied.

3.2.4 Verification of shear buckling

Eurocode provisions

According to article 3.5.2.2 of NEN-EN 1998-4, the shell of steel tanks have to be verified against shear buckling. Other design codes, such as NEN-EN 14015 or API 650 do not consider shear buckling in detail.

Contrary to meridional buckling, the Eurocode does not make a distinction between empty and filled tanks for shear buckling verification. This means that the formulae included in the Eurocode, deal with the case of empty tanks alone. The result of this is that an unrealistically low shear buckling resistance is calculated when applying the shear buckling verifications of NEN-EN 1993-1-6, annex D.1.4 to tanks filled with liquid. To avoid a far too conservative calculation of the shear buckling resistance of the shell, and until a revised version of this Eurocode becomes available considering the positive effect of internal fluid pressure, it is advised [2] to use the Japanese 'Design recommendations for storage tanks and their support with emphasis on seismic design (2010 edition)' [14] to verify the shear buckling capacity of the shell courses[1]. For the shell courses that are dry (not in contact with the liquid), the verification of NEN-EN 1993-1-6 still applies.

Japanese code provisions and shear buckling capacity

In summary, the Japanese recommendations state that if the hoop stress (caused by the internal pressure) is higher than 30 % of the yield stress, the plastic shear capacity ($f_y/\sqrt{3}$) will be governing and no shear buckling failure mechanism will develop. Figure 3.3 shows this effect: constant allowable shear stress $\tau_{x\theta,Rd}$ in case of NEN-EN 1993-1-6 verification, but increasing shear stress capacity with the Japanese design

[1] The Japanese code provisions account for the presence of the internal pressure.

recommendations. In practice, this implies that filled tanks are generally not susceptible to shear buckling, apart from the upper shell courses in which the effect of internal pressure is minimal.

Figure 3.3 Schematic influence of tensile hoop stress on the shear buckling stress

The Japanese design recommendations however do not apply partial factors on the buckling capacity, nor do they include imperfections such as the classes A,B,C of the Eurocode. It is therefore recommended to linear interpolate between the value $\tau_{x\theta,Rd}$ calculated with NEN-EN 1993-1-6 for p = 0 and f_y / √3 for σ_θ < 0.3 · f_y, as shown in figure 3.3. If the unity check exceeds 1, then the seismic verification is not satisfied.

3.3 Yielding and rupture of the tank bottom (unanchored tanks)

For unanchored tanks, in case of uplift, the tank bottom - especially the annular ring plate - has to withstand the tensile force from the shell resulting from the overturning moment. Without FEM calculations, the tank bottom can be evaluated by NEN-EN 1998-4 in terms of radial membrane stress (annex A.9.4) and plastic rotation (annex A.9.5). As NEN-EN 14015, annex G.3 specifically calculates the required width and thickness of the annular ring, it is required to also verify the tank bottom for these provisions in the seismic verification of liquid storage tanks.

3.4 Sloshing wave height with respect to available freeboard

The tank should have sufficient available freeboard f to prevent overflow and/or damage to the roof structure. The freeboard freeboard f can be estimated by equation A.15 of NEN-EN 1998-4. Please note that this expression calculates the wave amplitude d and not the wave height H (which is twice the amplitude). Contributions from higher order modes can be calculated as follows:
$d_1 = 0.837 \cdot R \cdot S_e(T_{c1}) / g$
$d_2 = 0.073 \cdot R \cdot S_e(T_{c2}) / g$
$d_3 = 0.028 \cdot R \cdot S_e(T_{c3}) / g$

The unity check follows from:
Unity check = d_{max} / f

If the freeboard proves to be insufficient, the seismic verification is not satisfied. In case of fixed roof tanks with insufficient freeboard, sloshing waves will hit the roof. Additional calculations are required to prove the resistance of the roof is sufficient in those cased to withstand earthquake loads.

3.5 Connections of in- and outgoing piping

Unlike the verifications mentioned previously, the verification of the nozzle-shell connection cannot be easily performed with basic calculations alone and therefore FEM analyses are required in most cases. The reasons for this are:

- Piping systems have multiple supports and are therefore statically indeterminate. The occurring forces or differential deformations between the tank and piping are dependent on the stiffness of adjacent piping. This complicates a combined analytical MRSA of the tank-piping system.
- The capacity of the connection may be approximated by the analytical expression from e.g. annex P of API 650 or the WRC 107/297/537 bulletins, but not all connections meet the condition to apply these analytical methods (e.g. small D/t ratios, oblique connections, connections with reinforcement pads, etc.).

NEN-EN 1998-4 (article 4.5.2.3) does provide however a simple verification of the deformation requirement of the shell-nozzle connection. **This verification shall be minimally performed in the seismic verification.** As a reference, table E-8 of API 650 gives design seismic displacement of piping attachments to tanks. The engineer should be able to judge whether this verification suffices or that a more detailed model is required.

If the shell-nozzle connection does not meet the requirement of NEN-EN 1998-4 (article 4.5.2.3), a more detailed analysis can be performed using FEM software that has been specially designed to verify these connections. Two cases should be verified:

- The forces or imposed deformations and rotations (due to horizontal/rocking motion, uplift and/or sliding) from the tank on the shell-nozzle connection.
- A MRSA of the piping system with the tank modelled as e.g. a clamped support in order to evaluate inertia forces from the piping itself.

4

TANK FOUNDATION VERIFICATIONS

4.1 2-step calculation procedure

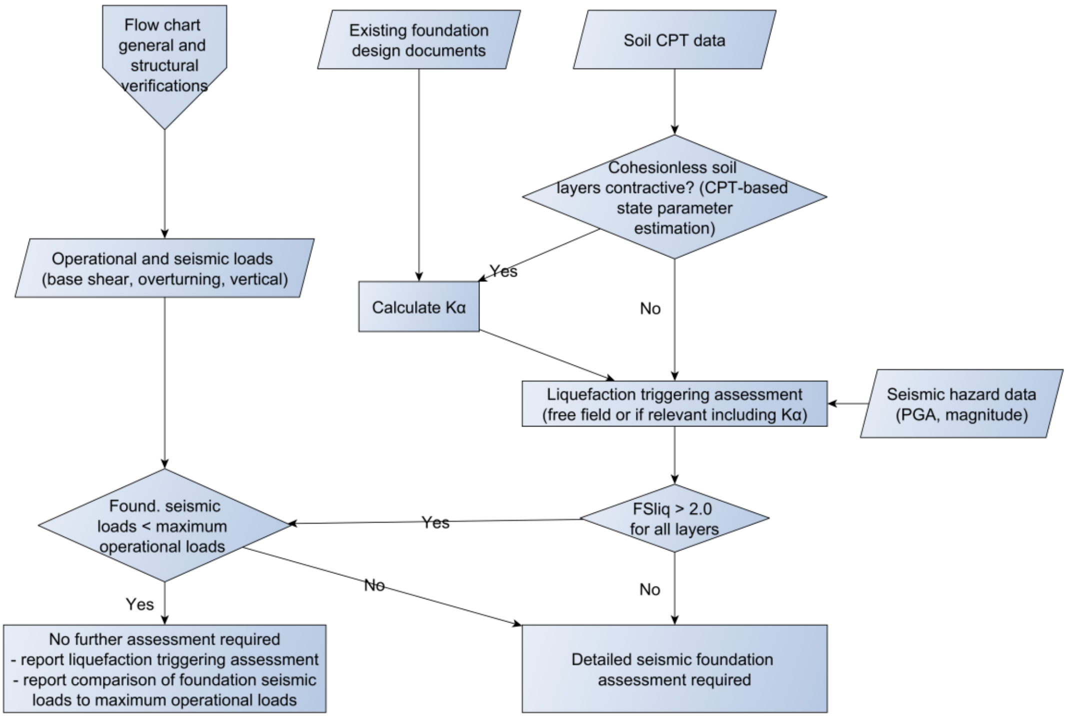
The assessment framework for liquid storage tank foundations in Groningen follows a 2-step calculation procedure. Both steps comprise quantitative calculations (not only screening) and are therefore sufficient to conclude on the expected LoC of a tank. The aim of a 2-step procedure is to limit the effort required for detailed calculations only to specific cases for which this is deemed necessary. The 2 steps comprise:

Step 1: This calculation step is performed for all tanks. Calculations being part of this step are quick and give solid and complete conclusions on foundation capacity. If step 1 does satisfy foundation capacity requirements then step 2 is not required. The framework for step 1 is described in detail in this document.
Step 2: This calculation step is to be considered only when the tank, after following step 1, does not satisfy foundation capacity requirements. Step 2 comprises detailed (finite element method) calculations of seismic foundation performance. The exact scope of this step depends on the outcome of step 1, the foundation typology and will be case-to-case specific. The framework for this step is briefly described in this document and references to relevant codes and guidelines are given[1].

The 2-step procedure that is followed is illustrated by figure 4.1.

[1] Step 2 requires expertise, knowledge and an experienced end user, who is able to carry out detailed FE computations including SSI in the nonlinear regime.

Figure 4.1 Flow chart for tank foundation assessment



4.2 Step 1: basic tank foundation calculations

Step 1 consists of two main components:
- Calculation of foundation loads from (global) seismic foundation loads and comparison of these seismic foundation loads to design foundation loads by operational load cases.
- Soil classification, identification of contractive cohesionless soils and liquefaction triggering assessment.

Based on combined conclusions from these two components an overall conclusion on potential criticality of the tank foundation subject to seismic action can be made and the possible requirement for further, more detailed calculations can be substantiated.

4.2.1 Evaluation of foundation loads level

Seismic calculations for the tank superstructure (as described in chapters 2 and 3 of this document) result in design seismic foundation loads (global base shear and global overturning moment). These seismic foundation loads are transformed into foundation pressures or loads on foundation elements. Subsequently these loads are compared to design foundation pressures, or design loads on foundation elements in operational conditions, in order to conclude on the relevance and potential criticality of the seismic load case.

4.2.2 Liquefaction triggering assessment

Following the GBoD ([1], [2]), CPT-based liquefaction triggering assessments shall follow Boulanger and Idriss (2014) [21].

The Boulanger and Idriss (2014) method follows a similar framework as the older Idriss and Boulanger (2008) method [22] with adjustments on some calculation parameters. The latest published versions of NPR 9998 ([3], [4]) prescribe a slightly different procedure, based on Idriss and Boulanger (2008) with specific adjustments. All these different procedures that have been published in the past for liquefaction triggering assessment follow the same CSR -CRR framework but deviate in terms of calculation parameters (magnitude scaling, CRR-curves) and correction factors for specific conditions (layered deposits, initial static shear stress, effective stress levels).

In the context of liquid storage tanks and the specific situation in Groningen, two factors are of special importance:

- **Static shear stress**: For liquid storage tanks with shallow foundations, static shear stress levels in the soil are typically high. Static shear is known to potentially severely increase liquefaction potential for contractive soils. The method by Boulanger and Idriss (2014), as prescribed by GBoD, does not address the impact of static shear stress. A modification of the Boulanger and Idriss (2014) method for static shear therefore shall be adopted for contractive soils if further substantiation by either numerical studies or laboratory experiments is not available. Corrections as outlined in [22] can be followed.
- **Layered deposits**: a correction factor on CPT based liquefaction resistance (CRR) may be applied in accordance with NPR 9998 (factor K_H).

4.3 Step 2: detailed tank foundation calculations

The framework for detailed tank foundation calculations depends on the foundation type and case-specific conditions. Therefore being exact and complete for any combination of tank geometry, foundation type and soil conditions is not possible within the scope of this document. Procedures are therefore only roughly described in this document and references to relevant codes and guidelines are provided.

Above ground, vertical cylindrical welded steel storage tanks typically have either tank pad, plate or piled raft foundations. Seismic foundation calculation methods differ for these foundation typologies. Seismic foundation assessments can be performed either strength-based (limit equilibrium methods) or performance-based (calculation of expected deformation levels). If limit equilibrium method calculations indicate that foundation capacity is insufficient then performance based calculations will be performed prior to disapproval of the foundation and engineering of strengthening measures. Table 4.1 sets out per foundation type which methods can be adopted.

Subsequent paragraphs elaborate further on specific important aspects related to assessment methods listed in table 4.1.

Table 4.1 Summary of foundation assessment methods

Foundation typology	Limit equilibrium calculation	Reference documents	Deformation based evaluation	Reference documents
rigid plate foundation	shallow foundation calculation (co-seismic and post-seismic)	NEN 9997-1+C2:2017	estimation of volumetric compaction settlements and ratcheting settlements	NPR 9998 or [23] for volumetric compaction, [24] for ratcheting *
			NLTHA, accounting for excess pore water pressure build up and dissipation for liquefiable soils	various *
tank pad foundation	finite element calculation (static, co-seismic and post-seismic))	NEN 9997-1+C2:2017, NEN-EN 1998-5:2005	estimation of volumetric compaction settlements and ratcheting settlements	NPR 9998 or [23] for volumetric compaction *
			NLTHA, accounting for excess pore water pressure build up and dissipation for liquefiable soils	various *
pilled raft foundation	check if kinematic pile loads can be neglected	NEN-EN 1998-5: 2005	calculation of post seismic settlements	NPR 9998 and NEN 9997-1+C2:2017
	co-seismic calculation of piles as function of base shear, global overturning and vertical action	NEN 9997-1+C2:2017	NLTHA, accounting for excess pore water pressure build up and dissipation for liquefiable soils	various *
	static post seismic pile bearing capacity calculation	NEN 9997-1+C2:2017		

* Codes/standards that substantiate a complete framework that can be applied for performance based evaluations of liquid storage tank are not available. Expertise, knowledge and experienced end user are required to proceed with such assessments.

4.3.1 Rigid plate foundations

A basic (limit equilibrium method) screening on foundation bearing capacity for liquid storage tanks on a rigid plate foundations comprises conventional shallow foundation bearing capacity calculation procedures. NEN 9997-1+C2:2017 prescribes this procedure and can be used for both co-seismic and post-seismic scenarios.

Following the NEN 9997-1+C2:2017 method for shallow foundation bearing capacity the dimensions of the slip surface are calculated based on weighted averaging over the various soil layers. It should be assessed beforehand, based on the soil layering and liquefaction potential of the different soil layers, whether the method results at a realistic failure surface.

Soil strength degradation due to liquefaction is estimated based on the liquefaction triggering procedure. When using the NEN 9997-1+C2:2017 effective stress procedure, degradation effects can be incorporated by a direct reduction on internal friction angle φ. Minimum residual bearing capacity can be determined based on undrained (total stress) calculation according to article 6.5.2.2.(g). The minimum residual strength S_r should be defined based on CPT or lab test data.

Tank settlement should be evaluated for tanks on subsoil including layers with liquefaction potential. Methods prescribed by the reference documents listed in table 4.1 can be followed. The accuracy of simplified approaches like Yoshimine et al. (2006) [23] and simplified approaches to estimate ratcheting settlements [24] should be verified when applied to liquid storage tanks.

4.3.2 Tank pad foundations

The analytical bearing capacity verification described in NEN 9997-1+C2:2017 cannot be applied for tanks on pad foundation, because local (edge) failure mechanisms are typically decisive over global failure mechanisms. Assessment of the dominant failure modes should be based on finite element model calculations. Finite element calculations for co-seismic and post-seismic stability should be performed for a complete verification.

Soil degradation due to liquefaction is incorporated in these models similarly as outlined for plate foundations in section 4.3.1.

Seismic settlement verification for tanks on pad foundations are performed following similar methods as reported for tanks on plate foundations.

4.3.3 Piled raft foundations

Co-seismic foundation capacity evaluation should include both geotechnical (GEO) and structural (STR) limit states. Verifications are prescribed in NEN 9997-1+C2:2017 [12]. Both vertical, horizontal and overturning load components shall be verified.

Limit state GEO can be assessed using suitable software like e.g. D-Pile Group or suitable finite element software. Liquefaction effects can incorporated by modifying the pile-soil interaction springs in e.g. D-Pile Group or by modification of material properties of liquefiable layers in finite element analysis in accordance with paragraph 4.2.

Structural limit state verifications should include pile loads from both inertial and kinematic seismic actions according to the relevant codes and guidelines ([4], [11]). Envelop pile internal forces are in this case defined as the sum of inertial and kinematic load components.

For piled raft foundations significant liquefaction induced settlements are neglected in absence of liquefiable layers below the pile neutral plane. For other cases post-seismic settlements for tanks on piled rafts are evaluated based limit equilibrium method calculations in line with the relevant codes ([4], [11], [12]). Co-seismic settlements for piled raft foundations cannot be evaluated with a limit equilibrium method and need integrated finite element models to be calculated.

Appendices

I

APPENDIX: MINIMUM REQUIRED INPUT DATA FOR SEISMIC VERIFICATION AS DESCRIBED IN THIS GENERIC APPROACH

Required information for verification of steel liquid storage tanks according to "Generic approach storage tanks"

General

The table below gives a list of the minimum required information to do the seismic verification acccording to the Generic approach storage tanks. The companies can enter the required data per tank if they wish, but the company should nevertheless provide all source data so that the consultant can verify this information and apply in the calculations.
This means that the list should be consulted on which data needs to be provided. In general the following source data is required:

- technical information about the steel tank, foundation and in-/outgoing piping: design drawings, design calculations, inspection reports, etc.
- a selection of photographs on the tank and details (in-/outgoing piping), these can be taken from the outside from ground level.
- geotechnical information (foundation drawings, foundation design reports with calculations, CPT-data (preferably in a digital .gef format, alternatively .pdf format) or soil survey report.

Input		Description	Unit / tank ID	1	2	3	4	5
Company		*Name of company*	-					
Location		*Geographic location of tank (e.g. Delfshaven)*	-					
Contact person		*Enter name and contact info in case of additional questions*	-					
Type of liquid		*e.g. petrol or natural gas condensate*	-					
Liquid density		*Density in kg/m^3 , e.g. 1000 kg/m^3 for water*	kg/m^3					
Tank material		*Select type of steel or other material (please mentioned this under "additional comments"*	-	(carbon) steel	(carbon) steel	(carbon) steel	(carbon) steel	(carbon) steel
Type of anchorage		*Select unachored or anchored. If anchored, please enter details under "additional comments"*	-	unanchored	unanchored	anchored	unanchored	anchored
(Inside) diameter of tank		*The (inside) diameter of the tank in m*	m					
Tank shell height		*The height of shell, exluding roof in m*	m					
Representative fluid level		*Enter the highest fluid level that is representative for most of the operational time (e.g. nominal fill level)*	m					
Type of roof		*e.g. dome, or open top tank with floating roof, etc.*	-					
Mass of roof		*Enter the mass of the fixed roof in kg (excluding the floating roof)*	kg					
Mass of wind girders		*Enter the total mass of the stiffening girders*	kg					
Height of main stiffening girder		*Height between bottom of tank and the main (wind) girder*	m					
Thickness shell courses (from bottom to top)	t_1	*Wall thicknesses in mm from bottom shell course to top course. Leave others blank.*	mm					
	t_2		mm					
	t_3		mm					
	t_4		mm					
	t_5		mm					
	t_6		mm					
	t_7		mm					
	t_8		mm					
	t_9		mm					
	t_{10}		mm					
Height of shell courses (from bottom to top)	h_1	*Height of each shell course from bottom shell course to top course. Leave others blank.*	mm					
	h_2		mm					
	h_3		mm					
	h_4		mm					
	h_5		mm					
	h_6		mm					
	h_7		mm					
	h_8		mm					
	h_9		mm					
	h_{10}		mm					
Steel grade or yield stress shell courses (from bottom to top)	f_{y1}	*Steel grade of each shell course from bottom shell course to top course. Leave others blank. Examples are S355J2, or St. 37, or FE 360 B FN, or Grade B.*	-					
	f_{y2}		-					
	f_{y3}		-					
	f_{y4}		-					
	f_{y5}		-					
	f_{y6}		-					
	f_{y7}		-					
	f_{y8}		-					
	f_{y9}		-					
	f_{y10}		-					
Thickness of annular ring plate		*Plate thickness of the annular ring*	mm					
Width of annular ring plate		*Width of the annular ring measured inwards from the shell (neglect width outside shell)*	mm					
Steel grade annular ring plate		*Steel grade, Examples are S355J2, or St. 37, or FE 360 B FN or Grade B*	-					
Thickness of bottom plate		*Plate thickness of the bottom plates*	mm					
Welds bottom plate		*Select type of welds: full penetration butt welds (stompe lassen) or lapped fillet welds (hoeklassen)*	-	butt welds	lapped welds	lapped welds	lapped welds	lapped welds
Outside diameter of foundation		*The total diameter of the steel bottom foundation plate*	m					
Type of foundation and dimensions		*e.g. concrete slab on piles, or shallow foundation on concrete slab or compacted mound (terp)*	-					
CPT-data		*Please enter if CPT (sonderingen) data is available, and if so, in which format (digital or analog)*	-	none	none	none	none	none
Additional comments		*Please enter any other relevant information for tank analysis and add photos, drawings and other technical documents if available.*	-					