REPORT

Generic approach for pipe systems and pipe racks

General approach for pipe systems and pipe racks for the seismic verification of industrial facilities in Groningen

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Classification
Open

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DNV·GL
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Table of Contents

1 Introduction and Starting Points

1.1 Introduction

This document summarizes the approach for the seismic assessment for pipe systems and pipe racks in Groningen, The Netherlands. The document has been prepared in accordance with the GBoD documents [2],[3]. The goals of this document are 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 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 pipe systems on pipe racks is required.

1.2 Seismic Verification Model

As of December 2017, the seismic verification method for industrial facilities in Groningen can be either the semi-deterministic method ‘LoC-toets’ as described in the GBoD [2],[3], or alternatively the ‘risico gebaseerde rekenmethodiek’ developed by TNO/Deltares [5]. The approach for pipe systems and pipe racks described in this document can be used in the context of the ‘LoC-toets’.

1.3 Scope of Document

This document discusses the seismic verification of existing pipelines supported by steel structures resting on top of pile foundation, including the verification of the supporting structure. The purpose of this document is to provide a generic approach for these structural systems, which is complementary to the GBoD documents [2] and [3].

The seismic verification of the complex structural systems examined here, requires the involvement of expert engineers of various background. This document is composed such that it integrates the available knowledge of engineers of different background, i.e. structural engineers, pipe engineers and geotechnical engineers, so that the seismic verification procedure can be carried out in the most straightforward, yet accurate manner.

The generic approach described in this document is suitable for metallic pipe systems according to EN 13480-3.
Non-metallic pipe systems are not within the scope of this document, but possibly a similar approach can be adopted in the latter case for the seismic verification. It is up the consultant to determine if this is the case.

1.4 Flowchart

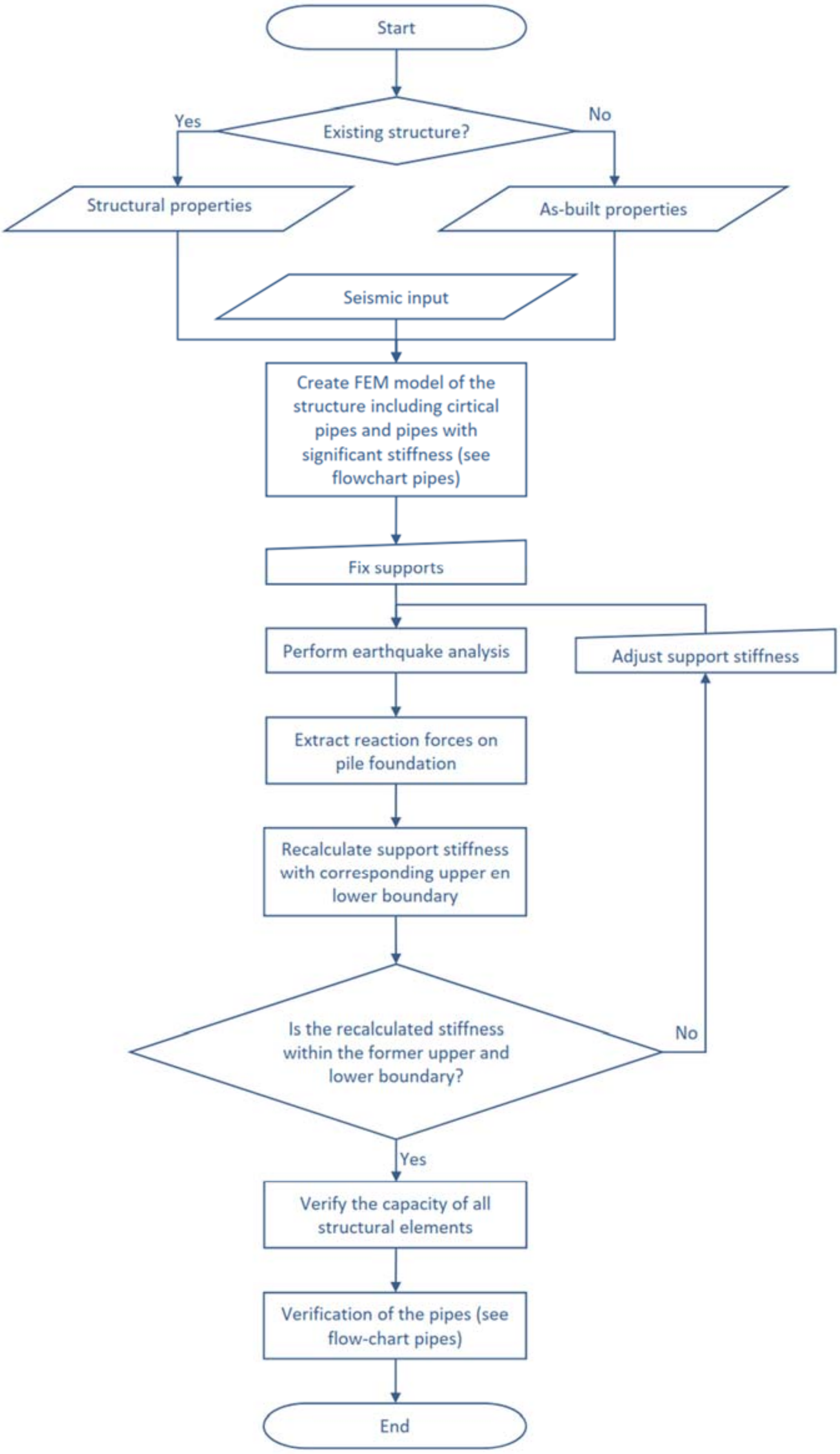

Figure 1-1: Work flow of the seismic verification of pipelines on steel structures with a pile foundation

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1.5 Standards, Guidelines and Other Documents

General documents

- [1] Rapportage werkgroep Maatgevende aardbevingsbelasting voor de industrie, 4 november 2016
- [2] Generic Basis of Design (GBoD) for the structural verification of industrial facilities in Groningen: a first screening of the seismic capacity, A. Tsouvalas, A.V. Metrikine en J.G. Rots, 24 oktober 2016, TUDelft
- [3] Explanatory notes for the “LoC Toets” in application to the industrial facilities in Groningen, A. Tsouvalas, A.V. Metrikine en J.G. Rots, 1 februari 2017, TU Delft
- [4] Lessons’ learned: “LoC Toets” in application to the industrial facilities in Groningen, A. Tsouvalas, 24 february 2018, TU Delft
- [5] Handreiking Fase 2, R.D.J.M. Steenbergen, P. Meijers, juni 2018, TNO

Dutch Standards and Eurocode

- [6] NEN-EN 1990:2011
- [7] NPR 9998:2015
- [8] NPR 9998:2018
- [9] NEN-EN 1991 serie
- [10] NEN-EN 1992 serie
- [11] NEN-EN 1993 serie
- [12] NEN-EN 1998-1
- [13] NEN-EN 1998-4:2007
- [14] NEN-EN 1998-5:2005
- [15] NEN-EN 9997-1:2018
- [16] EN 13480 -3

Other standards, guidelines and literature

- [17] Soil liquefaction during earthquakes, Idriss, I.M., R.W. Boulanger, EERI, MNO-12, 2008
- [18] Alpan, I. (1970). The geotechnical properties of soils. Earth-Science Reviews 6, pp. 5-49.

2 Description of the installation

This section shall provide information to have a general understanding of the purpose of the object and potential risks in case of damage related to an earthquake, as a result of the Qualitive Risk Analyses (phase 1). This information is added to show the context of the LoC-toets.

Information of objects and site conditions shall be included. With regard to the soil investigation, it is advised not only to collect available information from the specific object, but also from the surrounding environment.

3 Inventory of available information

The inventory on the available information shall distinguish between superstructure, foundation, site conditions and pipes

The required information for the verification consists of drawings and reports.

Superstructure:

- Drawings of the steel structure, main geometry and details
- Loads on the structure, due to different loadcases, for instance permanent loads, loads due to installations, pipelines and variable loads

Foundation

- Drawings of foundation beams and pile slaps, including reinforcement
- Drawings of piles (if applicable), including reinforcement

Site conditions:

- Site plan with the locations of the CPT's and/or boreholes. The site plan shall show key features recognizable in the field
- Results of CPT's – if available - presented on a CPT- diagram including the ground surface elevation relative to NAP, and the coordinates of the CPT, cone information and application class. Preferably also the CPT's results shall be made available in GEF-file format as well
- Results of boreholes including groundwater level readings.
- Laboratory test results

Pipes:

- Geometry, material
- Support conditions of the superstructure
- Loads, including loads of the fluid

With respect to the available data, this section shall include any comments regarding suitability and quality of information for verification purposes, like:

- Quality,
- format, legibility
- degree of verifiability
- Missing information

In case of any doubts or uncertainties detailed screening and further research, including one or more visual inspections may be necessary. For this phase of the LoC-toets, destructive investigations or inspection pits initially are not considered.

Due attention shall also be given to the quality and uncertainties related to available information regarding ground conditions and (additional) site investigation (fieldwork with associated laboratory testing and monitoring whatever is deemed applicable) fieldwork and since these may have significant impact on the analyses.

4 Seismic verification procedure

4.1 Limit State

The structure, including the pipes and the foundation, must be verified in the Ultimate Limit State (ULS) according to article 2.1.2. of the NEN-EN 1998-4:2007.

$E_d \leq R_d$

With:
E_d = design value of the load according to article 4.1 with q = 1.5
R_d = design value of the resistance conform article 5.6 of NEN-EN 1998-4:2007 and NEN-EN 1998-1 article 4.4.2.2.

Verification of the steel structure shall take place in accordance with NEN-EN 1998-1 chapter 6. For the steel structure a q-factor of 1.5 must be used, indicating verification based on concept a; low dissipative structural behaviour. Therefore, the steel structure can be verified using NEN-EN 1993-1 without further requirements. The concrete structure can be verified using NEN-EN 1992-1 without further requirements.

The metallic pipe systems need to be evaluated according to NEN-EN 13480-3: 2017 Chapter 12, the safety factor (k) used in the calculation is 1,2.

4.2 Safety Factors

The safety factors for structural elements are provided in Table 1. These values are based on the NPR-9998:2018 and are independent of the Consequence Class or the Reliability Class.

Table 1: Safety factors for structural elements

Material	Pre-/post seismic situation	Seismic situation
Concrete γ_m (γ_c)	1.5	1.5
Steel reinforcement γ_m (γ_s)	1.15	1.15
Structural steel γ_m	1.0	1.0
Masonry γ_m	1.5	1.5
Masonry γ_s	1.0	1.0

The characteristic soil parameters may be determined by probing or other types of investigation according to the national annex of Eurocode 7: NEN-EN 1997-1+C1+A1:2016/NB:2019. The characteristic values can be converted to design values using the partial factors provided in NEN-EN 1997-1+C1+A1:2016/NB:2019 for the pre-seismic and post-seismic situation and according to NPR 9998:2018 for the seismic situation, as shown in Table 2.

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Table 2: Safety factors for soil parameters

Material	Pre-/post seismic situation	Seismic situation
Foundation general		
Internal friction angle $\gamma_{\phi'}$	1.15	1.15
Effective cohesion γ_c	1.6	1.6
Undrained shear strength	1.35	1.35
Cyclic undrained shear strength	N/A	1.25
Weight	1.1	1.1
Stiffness	1.3	1.0
Pile foundation (based on CPT's)		
Partial material factor γ_M (for compressive loading)	1.20	1.20
Correlation factor ξ	1.39 (one CPT for non-stiff structures)	1.39 (one CPT for non-stiff structures)
Pile tip class factor γ_p	NEN-EN 1997-1+C1+A1:2016 / NB:2019 : table 7c/7d	NEN-EN 1997-1+C1+A1:2016 / NB:2019 : table 7c/7d
Pile shaft class factor γ_s	NEN-EN 1997-1+C1+A1:2016 / NB:2019 table 7c/7d	NEN-EN 1997-1+C1+A1:2016 / NB:2019 table 7c/7d

It is noted that the NEN does not provide a pile tip class factor for pile tips in clay. In this case the maximum tip resistance may, according to international applications (e.g. API, AASHTO), be determined based on the undrained shear strength multiplied with the capacity coefficient $N_C = 6 * (1 + 0.2 * (z / D)) \leq 9$, with z equal to the penetration of the pile in the clay and D equal to the pile diameter.

4.3 Failure Mechanisms

The following failure mechanisms are considered:

Structural failure mechanisms (STR):

- Failure of the steel structure (section, stability [local and global instability of the steel elements]), connections;
- Failure of the reinforced concrete foundation structure including reinforcement failure (when relevant);
- Failure of the connection between the concrete foundation structure and the steel structure;
- Failure of the pile foundation (when relevant);
- Failure of the pipes (this assessment to be accomplished together with the piping engineers).

Geotechnical failure mechanisms (GEO):

- Insufficient vertical capacity of the pile foundation (including liquefaction effects);
- Insufficient tensile capacity of the pile foundation (including liquefaction effects);
- Excessive deformations resulting from liquefaction settling under the pile tip.

The geotechnical failure mechanisms are only relevant in case failure of the foundation results in failure of the superstructure or in case deformation limits are exceeded.

5 Seismic input

5.1 Peak ground acceleration and acceleration response spectra

The horizontal and vertical response spectra for the acceleration are defined in the GBoD [2].
There is a regular update on the use of the new response spectrum ordinates from Shakemaps.

The importance factor is already included in the response spectra.

5.2 Behaviour factor (q)

The behaviour factor q for the piping bridge as well as the pipes has the same value of 1.5 in all directions. Reference is made to article 2.4(2) of NEN-EN 1998-4:2007 [[13]].

5.3 Seismic input for geotechnical analyses

Seismic loading is defined along three principal directions, one vertical and two horizontal. The two horizontal directions are assumed to be independent from each other. (par. 5.1 of [1]).

The workgroup "Maatgevende Aardbevingsbelasting" has indicated that for the envisaged verification purposes the liquefaction analyses shall be based on a moment magnitude M_w of 5.0 in combination with the relationship between the shear stress reduction coefficient r_d and the depth according to the NPR9998:2015-D.6. [7].

6 Soil

6.1 Site conditions

The description of the site conditions shall include the variations in the ground surface conditions and ground surface levels underneath, and in the close vicinity of structures as well as to greater distances. Any historical data on past activities (like ground improvement as part of site preparatory works), already terminated or on-going, or future planned activities shall be mentioned especially related to the changing of the ground surface profile.

Any uncertainties related to the site conditions that may have a positive or negative effect on the response of the site or the structure to seismic loading shall be – as a minimum qualitatively – be identified.

6.2 Ground conditions and pre-seismic ground parameters

This section shall describe and discuss the nature and quality of the available data regarding the local ground conditions. Representative CPT profiles shall be selected as basis for the envisaged geotechnical analyses.
One or more representative ground profiles shall be derived along with a set of soil parameters relevant to the various analyses. These are used as reference for the determination of the ground parameters to be used under seismic and post-seismic conditions (refer to section 6.3), and in order to validate the schematised ground conditions against the design and/or as-built data of the existing structure and its condition.

6.3 Ground parameters for seismic and post-seismic analyses

The geotechnical soil parameters given in the previous section 6.2 may need to be adjusted for seismic and post-seismic conditions depending on the sensitivity and response of the individual strata to the seismic loading.

6.3.1 Strength reduction of granular materials

In non-cohesive, granular materials, the effect of a seismic event is the generation of temporary excess pore pressure.

Sections 3.2.1, 3.2.2.3.2, 10.3.4.3.2 to 10.3.4.3.4 and 10.4.1 of the NPR 9998:2018 provide guidance on how to determine the values of the soil's strength and stiffness parameters before, during and immediately after the earthquake in relation to shallow foundations and axially loaded pile foundations.
The NPR 9998:2018 does not provide guidance for laterally loaded piles where generally the angle of internal friction φ is required as input for the soil strength. For these cases, the angle of internal friction φ may be reduced for the seismic and post-seismic situation using the equation below:

$$\varphi_{liq,rep} = MAX\left[atan\left((1 - r_u)tan(\varphi_k')\right); 3\right]$$

Hereby,
$\phi_{liq;rep}$ angle of internal friction during or immediate after the earthquake
ϕ_k' characteristic value of the angle of internal friction before the earthquake

r_u residual excess pore pressure (=the ratio of the pore pressure over the effective vertical stress prior to the earthquake, u/σ'_v) determined as function of the safety against liquefaction (according to the NPR.) It is assumed that during the earthquake 50% of the residual excess pore pressure is reached, to increase to 100% in a short period after the earthquake.

For completely liquefied soil the cyclic residual strength is taken as absolute lower boundary of strength. When the Factor of Safety against liquefaction is greater than γ_L = 2.0, the excess pore pressures are considered being negligible.

For the current verification purposes of the relationship between the safety factor against liquefaction and the residual excess pore pressure ratio r_u given in Annex D of the NPR9998-2018 shall be used.

6.3.2 Cyclic strength reduction cohesive materials

Cohesive soils may exhibit strength loss due to seismic loading referenced to as 'cyclic softening' (see for example: Idriss and Boulanger, 2008 [17]). In lieu of site-specific test results, the undrained shear strength of normally consolidated and over-consolidated saturated cohesive soils is assumed to be reduced by 20% and 30%, respectively, due to softening under seismic loading. Depending on the sensitivity of the cohesive soil to remoulding, other values may be considered more appropriate for a specific case.

This reduced undrained shear strength is applicable during the earthquake as well as immediately after the earthquake.
It is noted that especially during the earthquake this may be an over-conservative assumption. Depending on the sensitivity of the verification analyses to the effect of softening an additional site investigation may be considered to determine the strength loss.

In addition to the above, the soil resistance is reduced to zero at a depth of 5D below the pile head to account for the possible development of a gap between the pile and the soil that is filled with water.

The relevant reduced parameters are determined according to:

Cone resistance:

$$q_{c,red,rep} = q_{c,i} \cdot 0.8$$

Undrained shear strength:

$$c_{u,red,rep} = c_{u,i} \cdot 0.8$$

Cohesion:

$$c'_{red,rep} = c' \cdot 0.8$$

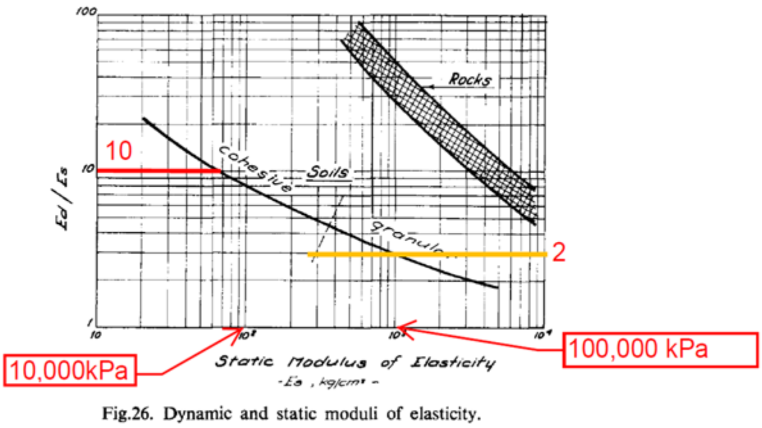
Angle of internal friction:

$$\varphi_{red,rep} = atan\big(0.8 \cdot tan(\varphi_k)\big)$$

6.3.3 Dynamic Soil Stiffness

The short term dynamic stiffness or small strain shear modulus (Go) preferably shall be estimated from in-situ measured shear wave velocities (using seismic CPT) or from correlations with the cone resistance and other soil properties. (Hardin and Black (1968); Hardin and Drnevich (1972); Kim & Novak (1981); Hardin and Blandford (1989)).

The codes do not provide clear guidance on the estimation of the dynamic soil stiffness to be considered. In lieu of further information, the dynamic stiffness may be estimated from the figure below. The shear modulus can be derived from the thus determined dynamic stiffness and the Poisson's Ratio ν (for non-cohesive soils 0.25, for cohesive soils 0.5).



Figuur 1: Relationship between static and dynamic stiffness of the soil (Alpan, [18])

6.4 Ground water levels

This section shall provide minimum, maximum and average groundwater heads for each aquifer. If applicable any potential effects of long-term ground water extraction and/or infiltration activities may be addressed.

6.5 Soil Liquefaction

6.5.1 Criteria for liquefaction susceptibility

Unless stated otherwise in this document, criteria for liquefaction susceptibility according to section 10.2 of the NPR 9998:2018 are applicable.

It is noted that for the envisaged verification purposes, the liquefaction potential shall also be evaluated for design values of the peak ground acceleration at ground surface below the threshold value of 0.125g given in section 10.2 of the NPR 9998:2018.

The liquefaction hazard may be neglected when the peak ground acceleration at ground surface is not greater than the threshold value and at least one of the following conditions for sandy layers is fulfilled:
- the sands have a clay content greater than 20% with plasticity index PI > 10;
- the sands have a silt content greater than 35% and, at the same time, the SPT blow count value normalised for overburden effects and for the energy ratio N1(60) > 20 or CPT cone resistance value normalised for overburden effects qc1 > 8 MPa;

- the sands are clean, with the SPT blow count value normalised for overburden effects and for the energy ratio N1(60) > 30 or CPT cone resistance value normalised for overburden effects qc1 > 12 MPa.

With respect to other main soil types, the following assumptions may be considered:
- Layers of gravel or gravelly material are not susceptible to liquefaction provided good drainage conditions are present.
- Layers of loam may require cyclic triaxial testing to assess the susceptibility to liquefaction or softening.
- Layers of clay or peat generally have low susceptibility to liquefaction.

In addition to the above criteria the liquefaction hazard may be neglected when designing or verifying piled foundations if:

a) only layers of clay and/or peat to the following depths are present (the greatest depth is applicable):
   1) till 15 m below ground surface, or,
   2) till 5 m below pile tip level, or,
   3) till 10 times the pile diameter below pile tip level;

b) sand layers are present with a thickness not greater than 0.5 m separated by clay- or peat layers with a thickness of 1m or more;

c) the safety against liquefaction γ_L is at least 2.0;

For shallow foundations, the liquefaction hazard may be neglected if the safety against liquefaction γ_L is at least 2.0. If the safety against liquefaction γ_L is less than 2.0, the excess pore pressure shall be accounted for in the design or verification calculations of the foundation. Reference is made to annex F of the NEN-EN 1998-5:2004 for guidance to calculate the seismic bearing capacity of shallow foundation. If the unity check is greater than 1, detailed calculations may be considered distinguishing between several moments during and after an earthquake.

The effect of the excess pore pressure due to the earthquake is to be taken into account according to:

a) Situation during the earthquake
   - Use the maximum value of the horizontal peak ground acceleration at ground surface in combination with a reduced excess pore pressure ratio $r_{u;d}$ to be determined as function of the safety factor against liquefaction γ_L:
     - If γ_L < 0.5 use $r_{u;d}$ = 1 (assuming full liquefaction);
     - If 0.5 < γ_L < 1.0 determine the value for $r_{u;d}$ through interpolation in the range 0.5 ≤ $r_{u;d}$ < 1 (partial liquefaction);
     - In all other case, (γ_L > 1.0), a value equal to 50% of the excess pore pressure ratio at the end of the earthquake shall be used.

b) Situation immediately after the earthquake
   - Use the maximum value of the excess pore pressure ratio. The ground acceleration can be assumed zero. The verification of the stability is to be done in accordance with the NEN 9997-1.

6.5.2 Verification safety against liquefaction

In accordance with the GBoD [2], the safety against liquefaction shall be verified using the method described by Boulanger and Idriss (2014)[17].

As indicated by the werkgroep Maatgevende Aardbevingsbelasting, the liquefaction analyses shall be based on a moment magnitude M_w of 5.0. The depth reduction factor r_d and magnitude scaling factor MSF shall be determined according to section D.5 and section D.6 of the NPR9998:2015, respectively.
It is noted that the use of version 2015 of NPR9998 is explicitly referred to by the werkgroep Maatgevende Aardbevingsbelasting.

7 Finite Element Model

This chapter discusses the starting points and design considerations for the finite element model (FE-models).

Piping Engineers and Structural Engineers are used to work in different software packages. This document is written such that it makes use of this common practice. The Structural Engineers does the verification of the structure in a FEM packages for example SCIA Engineer. The Piping Engineer does the verification of the pipes in a package suitable for pipe verification, for example CAESAR II.
To do the verification of the pipes, the pipes are part of the model of the Structural Engineers. In this way the stresses due to the earthquake can be found. Besides that, the Piping Engineer models the pipe system in the calculation program to determine the stresses due to other effects (for instance dead weight, temperature, internal pressure). By adding both stress components the total stresses are found.

In contrast to the above one can also decide to model both the structure and the pipes in one integrated model and do the verification based on this single model.

7.1 Geometry of the structure

The FE-model should be constructed conform the usual FE-model considerations. There are no special or addition requirements concerning the geometry of the structure for this seismic verification.

7.2 Modelling of the pipes

The pipes are modelled to determine the stresses due to the earthquake.
Regarding the pipes in the FEM model the following starting points should be used.

- A pipe should be physically modelled when:
  - The piping engineer defines the pipes as critical based on the stress level in existing pipe calculations and/or based on expert judgement and/or
  - the structural engineer defines the pipes as relevant in terms of dynamic contribution (when the stiffness and distributed mass of the pipe has a significant influence on the structural behaviour of the structure).
- It is important to correctly model the type and location of the pipe supports.
- Modelling the bending radius of the pipes is advised as it might result in a reduction of stresses in the pipe.
- Pipes which are not physically modelled should be incorporated in the model as point or line masses.

The critical pipes are modelled by the piping engineer. It’s recommended to check if the model of the piping engineer matches the model of the structural engineer. This can be done by giving the pipes in model of the piping engineer imposed deformation based on the results of a single mode from the FEM model. Then the stresses in the pipes from both models can be compared.

7.2.1 Fluid or Gas Content Volume

If possible, the governing situation (empty or full pipes) for earthquakes is determined. If it's not possible to determine the governing situation both situations should be assessed. Situation one, where the pipes are empty and situation two, where the pipes are full. In situation one only the weight of the pipes is modelled. In situation two also the weight of the fluid or gas content is incorporated as point of line masses.

7.2.2 Bending radius of pipes

In general, the largest stresses in the pipe systems occur in the branch connections. While modelling the pipe systems the actual bending radius of the elbows need to be considered. In order to evaluate the actual stresses, the Stress Intensification Factor (SIF) should be considered in the calculation.

7.2.3 Additional Masses

All significant masses present on the structure shall be incorporated in the FE-model. This can be done either by physically modelling the element or by incorporating the mass of the element as a point, line or surface mass.

7.3 Modelling of the pile

In the structural model, the piles may be i) modelled explicitly, as structural elements with given dimensions including both bending and axial stiffness; or ii) replaced by a set of vertical, horizontal and rotational springs. In the structural model due care must be given to the modelling of the connection between the foundation beam and the pile head to ensure that the relationship between horizontal displacements and moments in the pile head as well as the foundation beam are not un-conservative. This may be the case if the supports are modelled centrical in the foundation beam.

The verification of the pile capacity shall be based on the results of appropriate software or established by analytical methods.

7.4 Flexible base or fixed base

The type of supports for the structure shall be defined by the geotechnical expert and structural engineer together. By their definition the model can be either flexible base of fixed base.
In case flexible base is the most suitable an iterative process between structural engineer and geotechnical engineer is needed to determine spring values.

The soil-pile interaction under seismic loading condition may involve separate analyses for the lateral and the axial pile response. In either case, the pile dimensions should be modelled in a physically correct manner. The connection between the pile and the foundation element must be modelled by defining a suitable support.
The analyses for lateral pile-soil interaction shall be carried out by an analytical program that is suitable for the analyses of piles (for example DSingle Pile, LPile or comparable). A FEM based program to verify the soil-pile interaction can also be used but is usually required only in special cases.
In general, it can safely be considered that the pile has returned to its' original position after each event of horizontal loading provided no plastic hinge has developed in the pile and/or pile deformation in the soil has been negligible. In lieu of formal guidance, in practice a value of 5% of the pile's outer diameter during past loading often is used as threshold value.

In case of liquefaction, imposed deformations must be applied.

Differential deformations cause the highest stresses. The difference to be applied in the structural model is defined as 50% of the total settlement (section E.1 of NPR 9998:2018).

8 Response Spectrum Analysis

8.1 Natural Frequencies

A crucial step in the response spectrum method of analysis is the determination of the maximum number of modes to be included in the modal summations.
According to 4.3.3.3.1(3) of NEN-EN1998-1 a minimum of 90% participating mass should be incorporated.

In systems in which many local members contribute to the response, it is recommended to perform an additional check on the remaining 10% modal mass. Because local modes might cause significant stress concentrations to the local members [4].

In buildings where the foundation forms a substantial part of the building's mass, it is conceivable that no 90% particulate mass is found in the MRSA, as a result of the very high natural frequency of the foundation.
In these cases, the participating mass of the foundation can be added separately. The horizontal load component can then be determined based on the acceleration at t = 0.01.

8.2 Modal Superposition

For the model superposition either the SRSS combination rule or the Complete Quadratic Combination (CQC) method can be applied in accordance with NEN-EN 1998-1 4.3.3.2.

8.3 Combination Rules

Load combinations according to NEN-EN 1990 6.4.3.4 (6.12a).

For the pipes the load effect due to the earthquake and due to other effects are determined separately. The effect due to the earthquake by the structural engineer, the effect due to the other effects by the piping engineer. Therefore, in the FEM for the verification of the pipes separate load combinations containing only the loads due to the earthquake should be used.

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9 General and Structural Verification

9.1 Structural Elements

9.1.1 Steel

As q-factor the value of 1.5 is used (paragraph 5.2). Because of this according to NEN1998 -1 par. 6.1.2 design concept a) (Low dissipative structural behaviour) may be used. This means that the resistance of the members and of the connections should be evaluated in accordance with EN 1993 without any additional requirements.

9.1.2 Concrete

As q-factor the value of 1.5 is used (paragraph 5.2). Because of this according to NEN1998 -1 par. 5.3.1 Seismic design for low ductility (ductility class L), following EN 1992-1-1:2004 without any additional requirements other than those of NEN 1998-1 par. 5.3.2 may be used.

9.2 Pipes

Occasional stress should be checked according to NEN-EN 13480-3 paragraph 12.3.3. According (12.3.3-1) the allowable design stress is multiplied by factor k. In case of an earthquake k = 1,2 can be used. If higher values of the factor k are used (up to 1,8) then additional testing is needed before the system is taken into operation. This means the safety factor 1,8 is used for the safe shutdown of the plant, according to the Design Code EN 13480-3 Chapter 12. (only applicable for metallic pipe systems and not applicable for non-metallic pipe systems)
The “Werkgroep Maatgevende aardbevingsbelasting” has indicated that the most conservative value should be used. This means that k = 1.2.

If the method of two separate models is adopted for the analysis of the complete system, the stresses following from the MRSA are added to the pipe stresses as determined by the piping engineers. Note that the stresses from the MRSA should be the stresses only due to the earthquake. The stresses due to other effects are determined by the model of the Piping Engineer

The level of detail of the stress verifications depends on how high the stress level in the pipes is. The following approach can be followed (Figure 2), from low stress to higher stress:

- In case the governing stress from the MRSA is not exceeding 20 % of the allowable design stress f_f, no further verification is needed. This is because in normal occasional loads (e.g. normal snow, normal wind) factor k = 1. In case of an earthquake k = 1,2
- In case the governing stress from the MRSA is exceeding 20 % of the allowable design stress f_f, further verification is needed:
    - o First the governing stress from the MRSA (maximum stress from the total pipeline) and the governing stress from the model of the Piping Engineer (also maximum stress from the total pipeline) can be added. Total stress shouldn’t exceed k x f_f
    - o If the total stress exceeds k x f_f than at all locations for all critical pipes the resulting stresses from the MRSA and from the model of the Piping Engineer should be added.

The flange joints and the equipment nozzle connections are the weakest elements in the system. The mass of the equipment is higher than the mass of the pipe systems, resulting in different displacements and higher stresses during earthquake conditions.

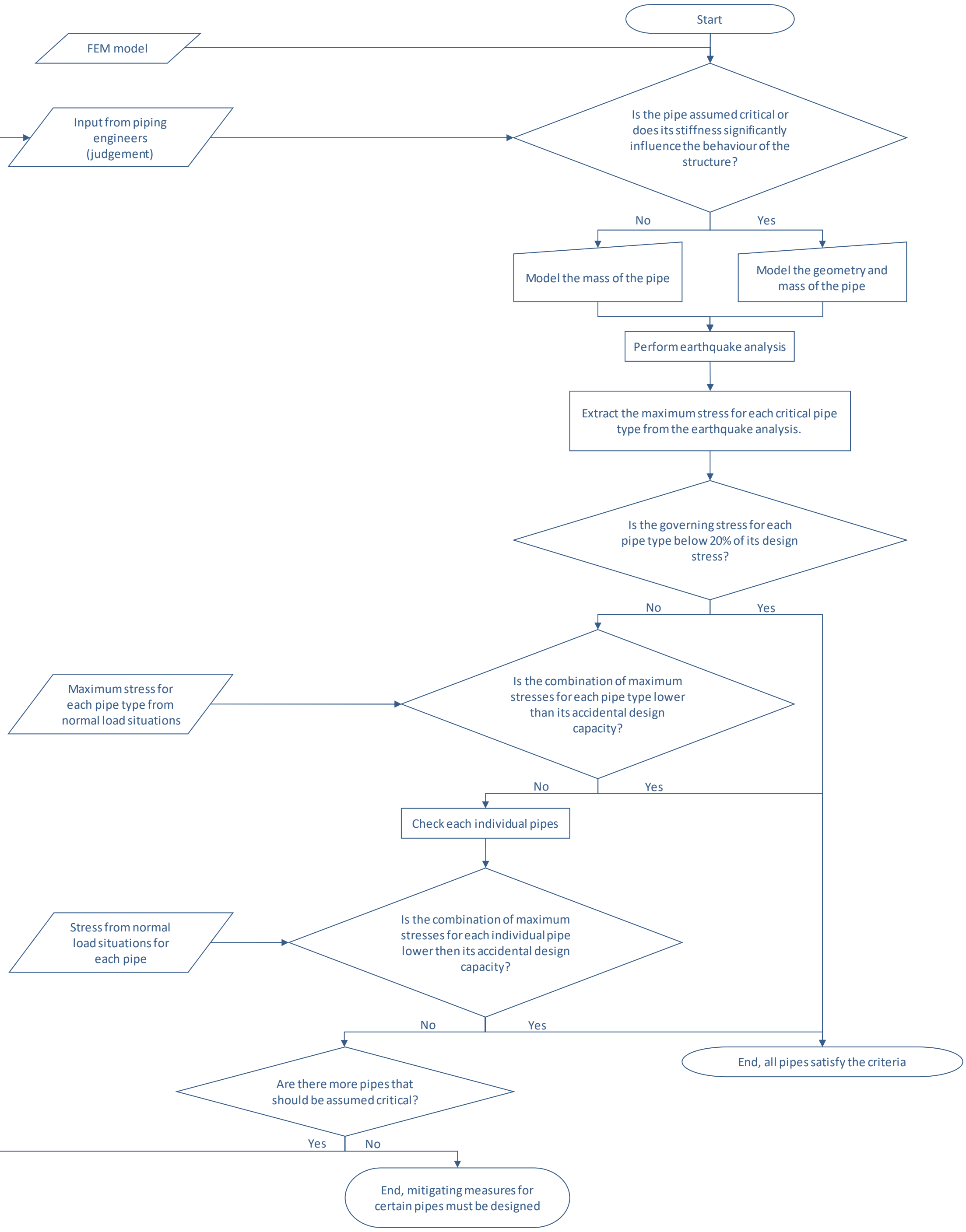

Figure 2: Work flow for pipe systems in case two separate models are used for the structural and pipe verifications

10 Pile Foundation Verification

This chapter describes the determination of the stiffness for a pile foundation.

10.1 General considerations

The geotechnical design shall take into account the possible degradation of the soil properties due to cyclic loading, thus performing the geotechnical strength calculations with the appropriate soil properties for the seismic level considered (see also section 6.3). A differentiation is made between the (A) seismic and (B) post-seismic situations:

A. In the seismic situation the load increase on the piled foundation due to dynamic soil-structure interaction shall be accounted for in combination with a 'reduced' level of soil degradation, as the pore pressure and cyclic degradation gradually build up during the earthquake.

   Local yielding or geotechnical failure may be permitted provided the exceedance is temporary and the damage levels of structure remain within the allowable criteria.

B. In the post seismic situation mass effects are not present (static type of soil-structure interaction); however the build-up of excess pore pressures and cyclic strain degradation is maximal which may result in a decrease of the bearing capacity.

   Re-consolidation may occur after dissipation of the excess pore pressure resulting in additional settlements (post seismic or post liquefaction settlements) and possibly also in an increase of negative shaft friction.

   Due consideration shall be given to the effect of possible re-distribution of the loads resulting from the seismic situation. The overall effect may be an increase of differential settlements of individual foundation elements. In this situation the stability of the structure needs to be guaranteed.

It is noted that the purpose of the LoC-verification analyses is to assess whether the superstructure is subject to failure or unacceptable displacements a result due to the considered earthquake level. A unity check for the foundation greater than 1.0 only means that the foundation does not meet the LoC-verification criteria for the foundation and not necessarily that the superstructure indeed is subject to failure or unacceptable displacements. This would require more complex analyses that allow for plastic behaviour and re-distribution of loads and stresses. Analyses of this nature are beyond the scope of the LoC-verification.

10.2 Pile Capacity

The axial pile capacity for the pre-seismic conditions shall be determined using the method described in the NEN-EN 1997-1 as reference and starting point for the pile capacity under seismic and post-seismic conditions. For the seismic and post-seismic conditions shaft resistance and base resistance may need correction as outlined in earlier section of this document.

10.3 Determination of the axial and lateral foundation springs

Upper- and lower bound approach

The response of the superstructure shall be based on a lower- and upper bound approach with respect to the values used for the vertical, horizontal and rotational spring stiffness. The lower- and upper boundary values may be determined by dividing the characteristic value of the small strain spring stiffness by $\sqrt{2}$ or

by multiplying the representative value of the small strain spring stiffness with √2, respectively. The verification analyses shall be based on the secant value of the spring stiffness using the appropriate load or stress level. See also Figure 10-1.
It is recommended the verification to include a check on the difference between the forces used in the geotechnical model and the deformations resulting therefrom, versus the reaction forces in the structural model loads and the deformations resulting therefrom. It is suggested to allow a difference of not greater than 10% to decide on most appropriate next step in the verification.

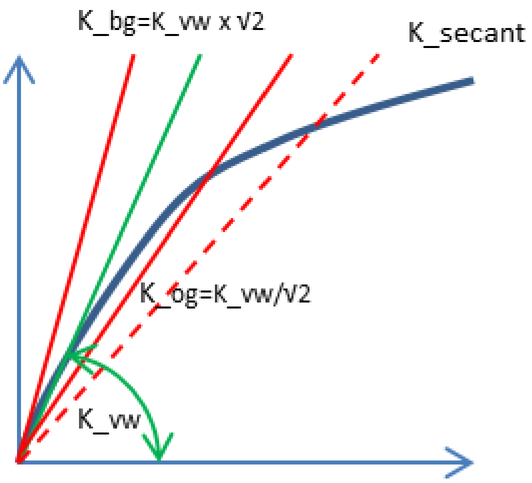

Figure 10-1 Definitions for the lower- and upper bound values for the spring stiffness

The vertical spring stiffness for the seismic and post-seismic conditions shall be determined relative to the pre-seismic spring stiffness which can be derived the appropriate load-displacement relationship given in the latest Dutch geotechnical design code NEN9997-1.
The lateral spring stiffness may be determined from a load-displacement relationship developed for project specific ground and foundation characteristics using suitable software. For the seismic and post-seismic situation different load-displacement relationships may be applicable depending on the effect of the earthquake on the soil properties (see also section 6.3).

10.4 Seismic soil – pile interaction

In the seismic soil-pile interaction the following can be distinguished:

1 Horizontal and vertical inertial loading of the superstructure caused by the seismic response of the structure

2 Kinematic loading related to
   a. temporary horizontal ground displacements during the earthquake
   b. permanent horizontal displacements due soil movement resulting from loss of subsurface stability or lateral spread (kinematic loading).

3 Permanent vertical displacement related to:
   a. Reduction of the pile's bearing capacity due to liquefaction. As a result, pile displacement is required to allow positive shaft friction to develop or further being mobilised.

b. Down drag forces as a result of liquefaction induced settlements in the post-seismic situation.

c. Liquefaction induced settlements in the post seismic situation of layers below the foundation but within its' depth of influence

As the structural and geotechnical models and analyses are decoupled (considered in separate models), these interaction mechanisms must be modelled by a discreet interface.

- In the structural model this is achieved by representing the soil as linear springs (vertical and horizontal) representing the vertical and horizontal pile-soil interaction, but which can also be used to apply kinematic loads due to lateral soil displacements and /or differential settlements.
- The possible effect of additional down-drag forces shall be directly accounted for in the pile bearing analysis.
- Hereby, it is verified that the reaction force is not exceeding the maximum soil resistance and whether permanent deformations may become a risk for the functionality or safety.

10.4.1 Kinematic loading due to horizontal soil displacements

For LoC verification purposes, kinematic forces on the pile arising from the deformation of pile surrounding soil due to the passage of seismic waves may be estimated from the peak ground acceleration at surface, the dominant natural period of the soil and the thickness of the soil layers between the head and the base of the pile. For general guidance reference is made to sections 5.4.2 and 6 of part 5 of Eurocode 8 (NEN-EN 1998-5:2005)

For a first appraisal of the effect of kinematic loading, the maximum displacement may assume to occur at ground surface where also the peak ground acceleration is maximum, and that the seismic induced displacements reduce to zero at pile base level following a parabola-like profile.
The effects of the kinematic and inertial loading of the structure will be assumed to act simultaneously. The pile response is analysed using a model with non-linear soil springs (appropriate p-y curves) with the pile being subjected to these kinematic forces along the shaft and a reaction force from the structure at the pile's head.
The horizontal subgrade modulus of the soil shall be based on dynamic Young's modulus values. In the model, the pile head is modelled fixed but with a horizontal movable end. An initial equivalent spring value $k_{h;i}$ can be calculated from the deformation of the whole foundation structure caused by a horizontal unit load at the base of the foundation cap or beam (or pile top):

$k_{h;i}$ = unit load / (number of piles x displacement foundation structure).

10.4.2 Seismically imposed permanent vertical displacements

The total permanent vertical deformation caused by earthquake loading comprises densification and liquefaction induced re-consolidation settlement.

Layers located above the ground water table may densify due to earthquake vibration causing additional surface settlement and hereby resulting in an increase in negative shaft friction. However, provided these layers have been assumed to fully contribute already to the negative shaft friction under non-seismic conditions, the potential effect of densification on the axial pile response is expected to be negligible and can be disregarded in the verification.

Densification of (saturated) layers below the ground water table is assumed to be already incorporated in the liquefaction induced re-consolidation settlements.

Post-seismic, re-consolidation settlement due to liquefaction shall be determined using the method described in the latest NPR 9998. This method is dependent on the safety against liquefaction (γ_L) and the initial (pre-seismic) relative density (R_e) of the sandy soils.

Displacements above and below the pile tip level have different effects and therefore need to be considered separately.

Reference is made to section 10.4 of the NPR 9998:2018 for further guidance.

Due consideration shall be given to differential settlement and potential effect therefrom (See NPR 9998:2018).