Seismic verification of foundations of industrial assets in Groningen

Performance criteria and assessment methods

Nationaal Coördinator Groningen

17 February 2021

Project	Seismic verification of foundations of industrial assets in Groningen
Client	Nationaal Coördinator Groningen

Document	Performance criteria and assessment methods
Status	Draft version 01
Date	17 February 2021
Reference	108064/21-002.668

Project code	108064
Project Leader	F. Besseling MSc
Project Director	R.A. de Heij

Author(s)	A.Bougioukos MSc, J. de Greef MSc
Checked by	F. Besseling MSc, M. Versluis MSc
Approved by	F. Besseling MSc

Initials

Digitally signed by
Floris Besseling
Date: 2021.02.17
15:50:48 +01'00'

Address

Witteveen+Bos Raadgevende ingenieurs B.V.
Leeuwenbrug 8
P.O. Box 233
7400 AE Deventer
The Netherlands
+31 570 69 79 11
www.witteveenbos.com
CoC 38020751

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INTRODUCTION, BACKGROUND AND SCOPE

1.1 Introduction

This document summarizes methods for performance-based foundation assessment, taken from NEN NPR 9998:2020, and evaluates their applicability to seismic verification of industrial assets in Groningen. The document is written as an explanatory report, aiming to help engineers to apply performance-based seismic evaluations for the industry seismic risk assessment program led by Nationaal Coördinator Groningen (NCG). Performance-based methods can be of added value to evaluate actual safety risks in case limit-equilibrium verifications are not satisfied.

1.2 Background

Over the past years the NCG has facilitated the development of methods for seismic risk assessment of industrial facilities in Groningen. The overall framework is explained in a document titled 'Handreiking Aardbevingsbestendigheid Industrie, Fase 2a/b (LoC-methode) en fase 2c' (Witteveen+Bos, 2019). For more details the reader is referred to this document.

Two methods have been developed for phase 2a/b assessments, being the so called LoC method and the so called risk-based method. The LoC method has been developed by the Werkgroep Maatgevende Aardbeving (WMA) and the risk-based method development has been led by Deltares and TNO. Both methods in essence prescribe limit-equilibrium verifications to assess seismic structural integrity in the ultimate limit state (ULS). Should a structure not satisfy the limit-equilibrium verifications of phase 2a/b, then permanent deformations or damage of an structural element are to be expected for the design seismic load. In this case, according to NCG's programme, an additional effort is required in phase 2c to better quantify consequences of local failure or effectively mitigate seismic risks.

One of the options for phase 2c assessment is to move from ULS verification on structural component level to performance-based assessment focussing on loss of containment or structural collapse risks. This effectively means that not just the strength capacity of individual structural members is verified, but instead the effect of capacity exceedance of a structural element is evaluated. Performance-based assessments help to better understand the actual consequences of local failure / damage.

The concept of performance-based assessment can be applied to foundations as well. Damage of foundations might be acceptable in some cases, provided that risks in terms of loss of containment of structural collapse remain within acceptable limits. In such cases a foundation may not satisfy the seismic ULS but still not compromise the ultimate seismic performance criteria of the structure. The overall seismic performance of an assets in this case may still prove to be sufficient. NEN NPR 9998:2020 rules for foundations of existing structures are based on this concept, aiming to only require foundation upgrading for those situations where seismic foundation failure does contribute to structural collapse risks. Therefore, NPR 9998 methods for foundation assessment are of added value for seismic assessments of industrial assets.

Over the recent years a decay of seismic activity has been observed for the Groningen field. As a result probabilistic seismic hazard has decreased and liquefaction risks have decreased as well. This trend is expected to continue in the future. Moreover, research results have become available which increased the understanding of the actual safety risks associated with seismic damage of foundations. As a result, NEN NPR 9998:2020 nowadays includes generalized exclusion criteria which exclude the need for further foundation assessments under certain conditions. Such criteria are also relevant for industrial assets and are therefore discussed in the present report.

1.3 Scope and limitations

This document is written for structural and geotechnical engineers working at seismic assessments of industrial assets in Groningen. The document summarizes NEN NPR 9998 (performance-based) foundation assessment methods and helps engineers to understand the added value of these methods for industrial asset verifications in Groningen. In addition this document reflects on NEN NPR 9998 generalized exclusion criteria for further seismic foundation assessment and evaluates applicability to industrial assets.

The present document is not a code or standard. It is the responsibility of the reader to comply with all the relevant standards and regulations and assess if the concepts discussed in this document are applicable to a specific facility or structure. 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 structures and foundations is required.

1.4 Document outline

Following after this introduction, chapter 2 describes the seismic verification framework of industrial assets in Groningen. Phases of the general process and evaluation methods are introduced, together with a discussion on performance criteria and their relation foundation damages is highlighted. Chapter 3 summarizes methods for shallow foundations. Chapter 4 presents an evaluation of NEN NPR 9998 developments for piled foundations. Chapter 5 concludes this report and lists recommendations.

2

VERIFICATION FRAMEWORK

2.1 Phases in the process and evaluation methods

Risk assessments for industrial assets in Groningen are organized in two phases: Phase 1 and Phase 2. A general overview of these phases and more is presented in the Handreiking Aardbevingsbestendigheid Industrie (Witteveen+Bos, (2019)).

In summary, Phase 1 comprises a qualitative risk assessment of an industrial site based on (Deltares & TNO , 2018b).

Phase 2 comprises quantitative assessments (calculations) in order to verify the capacity of industrial assets which were selected as high-risk in Phase 1. Two methods are developed for Phase 2 assessments, being the LoC-toets (WMA, 2016, 2017a/b and 2019) - summarized in the Handreiking Aardbevingsbestendigheid Industrie (Witteveen+Bos, (2019)) - and the risk based method (Deltares&TNO, (2018)). Both methods don't clearly describe how to foundation assessments should be coupled to targeted performance criteria of industrial assets. Instead a general reference is made to the Eurocodes, especially NEN-EN 9997-1 for geotechnical verifications.

Recently, frameworks have been developed to optimize the Phase 1 and Phase 2 approach. The 'Selectiemethodiek' (Arcadis, (2020)) qualitatively re-evaluates Phase 1 assessments based on a more uniform approach and to more recent insights. Ongoing follow-up studies of the Selectiemethodiek focus on the development of a generalized quantitative risk calculation tool for common industrial assets.

2.2 Seismic performance criteria of assets

In seismic design practice, typically differentiation is made between performance criteria and damage criteria. Performance criteria relate to the overall target performance of a structure. Damage criteria are a derivative of them specifying performance in terms of acceptable damage on a structural component level.

At the highest level, seismic performance criteria for industrial assets are set as follows (Commissie Meijdam (2015), RIVM (2016), Deltares&TNO (2018)):

1. Safety risks for the residents living in the vicinity of industrial sites shall not increase significantly due to the release of hazardous substances as a result of an earthquake.
2. Safety risks of employees of industrial companies shall not increase significantly due to earthquakes.
3. Environmental risks shall not increase significantly due to the release of hazardous substances as a result of an earthquake.

Both phase 2 assessment methods (the LoC-toets and the risk-based method) basically aim for these same performance criteria, but the seismic load to be used to verify if structural capacity is sufficient are defined differently. Here, the risk-based method differentiates among structures based on consequences classes, following international practice and Groningen specific evaluations. The LoC-toets method instead sets a performance requirement for a location specific fixed deterministic earthquake scenario.

Performance criteria listed above are verified in Phase 2a/b for preselected industrial assets by means of structural capacity verification of the ultimate limit state (ULS) following Eurocode. If the capacity/strength of an element is exceeded there is a risk for structural damage (due to earthquakes). The evaluation of implications of ULS exceedance could be part of the phase 2b or 2c assessment.

2.3 Seismic performance criteria of foundations

Global performance criteria in terms of LoC and loss of global structural integrity should be translated to structural element limit states. This applies to foundation elements as well. Seismic damage of foundations is acceptable, provided that the overall stability of a structure is maintained and provided that resulting deformations are within acceptable limits. In this case there would be no severe impact on the performance of the superstructure.

This is 1 to 1 consistent with NEN NPR approach for (residential) buildings. Therefore it is proposed to take benefit from NEN NPR 9998 developments for foundation assessment of existing buildings in order to optimize seismic foundation assessments of existing industrial structures in Groningen. In NEN NPR 9998 the relation between foundation damage and severe consequences for the building superstructure (near-collapse limit state) is established through a limit on differential seismic settlement. This limit is set to 20 mm/m by NEN NPR 9998, based on a literature review of international literature (NPR 9998:2018 background document, Deltares, (2018)).

Developments of NEN NPR 9998 for foundations can be roughly grouped into the following three categories:
1 Clarifications and additions to Eurocode 8 methods for limit equilibrium verifications of foundations.
2 Simplified methods to estimate seismic foundation settlements.
3 Exclusion criteria for which no further seismic foundation assessment is required.

The following chapters summarize these developments and evaluate applicability to industrial structures in Groningen.

For shallow foundations the following aspects are addressed:
- Bearing capacity evaluation according to NEN NPR 9998 for seismic and post-seismic conditions, which are basically additions to Eurocode 8 (chapter 3.2).
- Simplified procedure to estimate shallow foundation settlements, based on regression analysis of advanced finite element simulation studies (chapter 3.3).
- General exclusion criteria for seismic shallow foundation assessments (chapter 3.4).

For piled foundations the following aspects are addressed:
- Geotechnical bearing capacity and seismic settlement prediction according to NEN NPR 9998, which basically is an integration of liquefaction triggering assessment and floating pile calculation (chapter 4.2).
- Exclusion criteria for kinematic load assessments for verification of pile shaft capacity (chapter 4.3.1).
- Simplified methods to evaluate post-damage pile head residual bearing capacity (chapter 4.3.2).
- General exclusion criteria for seismic pile foundation assessments (chapter 4.3.3).

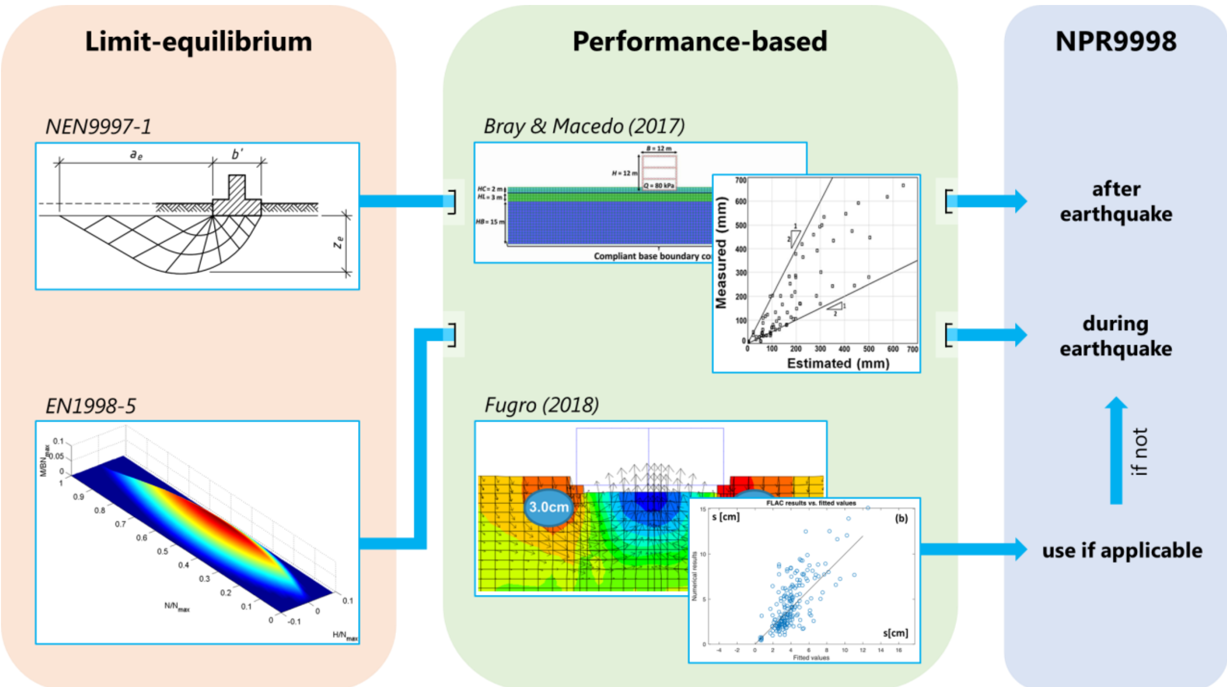
3

SHALLOW FOUNDATIONS

3.1 Approach and verifications

Initially the NEN NPR 9998 verification procedure for shallow foundations was based solely on limit-equilibrium checks, but for very common shallow founded strip footings it was extended with a performance based design approach based on a variety of finite element calculations using PM4Sand, in line with the procedure by Bray & Macedo (2017) for shallow foundations. In the latest version of the NEN NPR 9998 a flowchart is included to determine whether the settlement of shallow foundations could lead to near-collapse of the superstructure. Near-collapse here is related to a differential foundation settlement of 20 mm/m or larger.

In the subsequent flowchart the encoded analysis procedures and their relations are presented. The strength-based limit equilibrium methods will be discussed in paragraph 3.2, the performance based methods in which the settlements are estimated in paragraph 3.3. General exclusion criteria according to NEN NPR 9998 are discussed in paragraph 3.4.



3.2 Limit-equilibrium calculation- bearing capacity

Two design codes form the basis of the methods of the NEN NPR 9998 guideline:
- Eurocode 8 for the bearing capacity during the earthquake.
- NEN 9997-1 (the Dutch version of Eurocode 7) for the bearing capacity after the earthquake.

For the purpose of seismic verification of existing structures in Groningen these two codes are combined into optimized methods adopted by NEN NPR 9998. These methods can be applied for industrial structures as well.

3.2.1 Bearing capacity during the earthquake

For the bearing capacity check during the earthquake Eurocode 8 (in the informative annex F) provides a drained and an undrainded calculation procedure in which external forces, moment and soil inertia forces are accounted for. Specifically for strip footings in Groningen this method has two main disadvantages:

- Embedment depth cannot be accounted for in the simplified method.
- Only a single homogeneous soil layer can be used as an input and no fluctuations of the phreatic surface can be accounted for.

The method is derived for a strip footing, obeying a Tresca strength criterium of the subsoil in which numerical fitting parameters are derived by minimizing the maximum resisting work (Pecker, 1997). The strength of the soil, expressed as the ultimate bearing capacity N_{max} under a vertically centered load can be determined for either purely cohesive (F.2) or purely cohesionless (F.3) soils.

The expression for purely cohesionless soil may be used for dry soils or saturated soils without significant pore pressure build-up. Therefore this expression can be used in case of sandy soil where the liquefaction safety factor FS_{liq} is higher than 2.0.

For purely cohesive soils the ultimate bearing capacity depends on either the undrained shear strength c_u of the soil, or the cyclic undrained shear strength $\tau_{cy,u}$. This allows for the inclusion of temporal excess pore pressures that have developed during the earthquake (while the inertia forces are still acting), but moreover for strip footing the positive effect of the embedment depth can be included. Since over 50 % of the bearing capacity of shallow strip footings may come from the fact that the footing is embedded, the original formulation in Annex F can be very conservative. The expression included in the NPR 9998 is the following:

$$\tau_{cy;u} = \left(1 - r_{u;50\%}\right) \tan\varphi \cdot \left(0.5\frac{G}{B} + 0.7B\rho g + \sigma'_{v;z;d}\right) \qquad G \leq \frac{N_{max;dr}}{1.5}$$

Herein $0.5G/B$ represents the effective stress from the permanent self weight of the superstructure G. Its value is limited by the ultimate bearing capacity divided by a factor of 1.5 to avoid that the strength can increase indefinitely with increasing load. The factor 0.5 represents that the average stress underneath- and next to the foundation is used. The term $0.7B\rho g$ represents the vertical effective stress due to the soil weight at the influence depth, which is approximately located at a depth of $0.7B$ below the foundation level. The effective stress due to the embedding above the foundation level is represented by $\sigma'_{v;z;d}$.

The shortcoming that only a single soil layer can be entered remains, so conservatively a low-representative value of φ found below the foundations level should be used.

3.2.2 Bearing capacity after the earthquake

After an earthquake the inertia force is no longer present. This implies that the method as presented in NEN 9997-1 can be used. This method does allow for combining a layered subsoil by calculating a weighted average of the strength of the subsoil. NPR 9998 suggests reducing the bearing capacity factor that accounts for the self weight of the soil, N_γ, directly by a factor $\left(1 - r_{u;100\%}\right)$ and alternatively allows to reduce the layer strength parameters directly. From a analytical point of view the latter is preferred, since this allows for differentiation of liquefaction susceptibility of different soils layers, rather than applying a reduction to the cumulative bearing capacity.

It is recommended not to perform the punching and squeezing checks according to NEN 9997-1 because these relatively simple methods are derived for cohesive soil layers in static conditions. When assessing a cohesionless layer of which the strength gradually decreases, these methods could lead to erroneous results. It is therefore recommended to perform a simple finite element model if liquefaction is expected to occur below the initial influence depth. An upper bound of the depth in which this additional check is recommended is given by the depth at which the foundation load causes a vertical effective stress increase of 20% or more compared to the free-field condition (assuming the load is spread at an inclination of 2V:1H). Significant liquefaction underneath this level is expected not to affect the bearing capacity. The latter is a deviation from the limit-equilibrium methods and the outcome should therefore be compared to an allowable criterium.

3.3 Settlements

If a limit-equilibrium verification is not satisfied it means that the design load (temporarily) exceeds the capacity. In this case permanent deformation develop, which cannot be calculated based on a limit-equilibrium verification. For this reason performance based methods are developed that allow for an estimation of seismic displacements.

Such procedures should ideally be backed by case history data, but scale tests and variation studies with (advanced) calculation procedures are often used for method validation as well. Bray & Macedo (2017) developed a simplified analysis procedure in which the combined building settlement due to ratcheting and post-liquefaction consolidation can be estimated, based on some key input parameters. Underlying basis of this relation is an elaborate set of soil-structure interaction calculations in which the geometrical, strength and seismic loading parameters were varied. According to Bray & Macedo the shear-induced ratcheting settlement, expressed as D_s is given by the following equation:

$$\ln(D_s) = -7.48 + 4.59 \cdot \ln(Q) - 0.42 \cdot \ln(Q)^2 + 0.014 \cdot LBS + 0.58 \cdot \ln\left(\tanh\left(\frac{HL}{6}\right)\right) - 0.02 \cdot B +$$
$$0.84 \cdot \ln\left(CAV_{dp}\right) + 0.41 \cdot \ln(Sa1) + \varepsilon$$

Table 3.1 Parameters of Bray & Macedo method with validity ranges

Parameter	Meaning	Validity range
Q	foundation contact pressure	20 - 240 kPa
LBS	liquefaction-induced building settlement index	not known exactly
HL	cumulative thickness of soil where $FS_{liq} < 1$	1 - 18 m
B	foundation width	6 - 24 m
CAV_{dp}	cumulative absolute velocity	0.22 - 3.2 g-s
$Sa1$	spectral acceleration at $T = 1s$	not known exactly
ε	model error	normally distributed with mean 0.00 and sd 0.50

Note that two parameters have been selected to represent building/foundation characteristics (Q, B), two have been selected to represent motion characteristics (CAV_{dp}, $Sa1$) and two have been selected to represent the liquefaction susceptibility of the subsoil (LBS, HL).

The range of parameter LBS is not known exactly., but there appears to be no limit to its applicability given the reported range of HL. It is noted that in the soil-structure interaction calculations by Bray & Macedo a non-liquefiable layer thickness HC with a minimum value of 1 meter is adopted, but this parameter did not end up in the final regression as it is reported that this crust thickness is indirectly included in LBS. The range of spectral accelerations is at $T = 1s$ is not reported, but PGA values varied between 0.15 and 1.2g.

Apart from the minimum crust thickness, a limitation of the Bray & Macedo method is the fact that the minimum foundation width in their simulations was 6 meters. As a consequence the method has been concluded not to be calibrated for small foundation dimensions like the strip foundations typically encountered for residential buildings in Groningen. For industrial structures typically foundation geometry dimensions are more in the ranges covered by Bray & Macedo. Because of the limitations for residential building strip foundations Fugro (2018) performed a Groningen specific analysis using PM4Sand soil-structure interaction analyses of which the results is the following equation (note that in contrast to the equation by Bray & Macedo (2017) post-seismic reconsolidation settlements are included in this equation).

$$\ln(s) = 2.570 + 0.200 \cdot \ln(Q) + 0.742 \cdot B - 0.454 \cdot H_{crust} + 1.924 \cdot \ln(\tanh(H_{liq})) - 0.031 \cdot D_r + 0.588 \cdot \ln(D_{5-75}) + 1.900 \cdot \ln(Sa_{T=0.7s})$$

Table 3.2 Parameters of NEN NPR 9998 method with validity ranges

Parameter	Meaning	Validity range
Q	foundation pressure	20 - 120 kPa
B	foundation width	0.25 - 0.70 m
H_{crust}	non-liquefiable crust thickness	0.5 -1.0 m
H_{liq}	liquefiable layer thickness	0.5 - 10.0 m
D_r	liquefiable layer relative density	30 - 50%
D_{5-75}	ground motion significant duration	2.6 - 10.4 s
$Sa_{T=0.7s}$	spectral acceleration at $T = 1s$	0.27 - 0.55 g
ε	model error	normally distributed with mean 0.00 and sd 0.458

Note that, similar to the approach by Bray & Macedo (2017), two parameters have been selected to represent building/foundation characteristics (Q, B), two have been selected to represent motion characteristics (D_{5-75}, $Sa_{T=0.7s}$) and three have been selected to represent the liquefaction susceptibility of the subsoil (H_{crust}, H_{liq} and D_r). The reported range of spectral accelerations correspond to a PGA varying between 0.10 and 0.30g.

For industries and infrastructural projects the study by Fugro (2018) will in many cases not be directly applicable because the range of foundation dimensions will typically be exceeded. On the contrary the method by Bray & Macedo may be better applicable. In between the validity ranges of the Bray & Macedo and the Fugro studies there exists a gap. A possibility could be to use both relations and adopt envelop predicted foundation settlements for further evaluations.

Such evaluation whether or not seismic settlements are acceptable is done by comparison of the predicted seismic settlement to a settlement criterion which by NEN NPR 9998 is set to 20 mm/m. This level of differential settlement is deemed applicable to near-collapse limit states of buildings. For industrial structures the structural engineer could estimate if this limit value applies and if not select another limit value for allowable differential settlement.

3.4 General exclusion criteria for seismic shallow foundation assessment

Combined results of liquefaction hazard studies for Groningen (Green et al., 2018) and results of studies focussing on the effects of seismic shaking in terms of foundation settlements (Fugro (2018)) have substantiated a high level exclusion criterium for shallow foundations adopted in NEN NPR 9998:

For a design seismic load level below 0.125 g the risk of a foundation failure induced near-collapse limit state is sufficiently low an no further seismic assessment of the shallow foundation is required.

For sites with design peak ground acceleration below 0.125 g no further assessment of liquefaction triggering and stability or settlement of a shallow foundation of a building is required according to NEN NPR 9998. This exclusion criterion is not related to a specific type or geometry of shallow foundation and therefore applies to shallow foundations of industrial structures as well.

Table 3.3 lists deterministic and probabilistic peak ground acceleration (PGA) levels for assessments of the main industrial areas in Groningen. The LoC-toets shakemap values are derived from KNMI (2018) (organized by Sweco (2019)). The probabilistic values, associated with the mean return period (MRP) 475, 975 and 2475 years, are taken from seismischekrachten.nen.nl. According to the NEN NPR 9998 exclusion criterium these PGA levels imply that shallow foundation assessments including liquefaction triggering calculation for industrial assets are, given the decreasing seismic hazard, only still required for high risk structures in the industrial areas of Delfzijl, Hoogezand and Winschoten, which are assessed based on the 2475 years MRP spectrum. For such assessment the methods as discussed in paragraphs 3.2 and 3.3 could be used. For further development of the Selectiemethodiek (Arcadis, (2020)) or the Quick risk calculation tool (Witteveen+Bos, (2020) it could be argued that only for consequence class IV and V structures (table 4.1 of Deltares&TNO (2018)) in Delfzijl, Hoogezand and Winschoten a specific seismic assessment of the risk contribution of shallow foundation failure seems necessary. Here an area-based approach might be more efficient though, given the low liquefaction susceptibility indicated for these areas by the NEN NPR 9998 webtool.

Table 3.3 Peak ground acceleration values based KNMI (2018) / Sweco (2019) and seismischekrachten.nen.nl scenario T4, GMMv6

	LoC-toets KNMI 2018 shakemaps	PGA MRP 475 yr [g]	PGA MRP 975 yr [g]	PGA MRP 2475 yr [g]
Delfzijl (Chemiepark)	0.09	0.08	0.12	0.17
Eemshaven	0.05	0.05	0.07	0.09
Hoogezand	0.08	0.08	0.11	0.16
Veendam	0.04	0.05	0.07	0.10
Winschoten	0.05	0.06	0.09	0.13

4

PILE FOUNDATIONS

4.1 Approach and verifications

The NEN NPR 9998 approach to seismic pile foundation verification basically comes down to an evaluation of the following aspects:

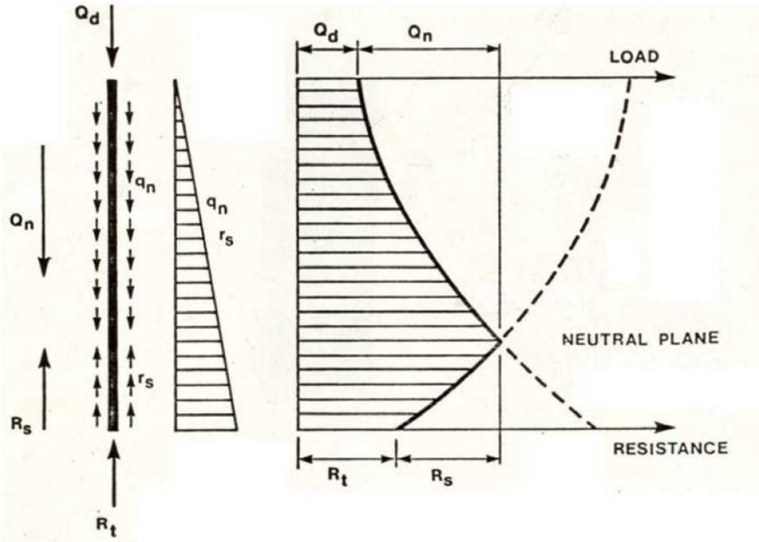
1 Is tension capacity of piles required for global overturning stability?
2 Are pile settlements due to liquefaction (GEO limit state) acceptable from a near-collapse perspective?
3 Is axial structural capacity of piles maintained for the design seismic load?

Exceedance of the lateral capacity of piles is not a criterion in itself, provided that the piles maintain their load bearing function. The first aspect can simply be addressed by structural engineer based on the results of the superstructure model. For the last two aspects NEN NPR 9998 provides simplified methods and criteria in order to allow for efficient verification. This chapter summarizes these methods of NEN NPR 9998, which may also be applied for the assessment of industrial structures. Paragraph 4.2 addresses pile settlements due to a loss of geotechnical bearing capacity by (partial) liquefaction. Paragraph 4.3 addresses the evaluation of residual axial structural capacity of piles in case of exceedance of lateral capacity.

4.2 Geotechnical bearing capacity

The geotechnical bearing capacity of piles is provided by the pile tip resistance and the pile shaft resistance, which make equilibrium with the load acting on the pile head and the negative skin friction that may act along part of the pile. Although using different notation compared to NEN 9997-1, this is presented in figure 4.1 as a resulting normal force diagram along the pile. The maximum normal force is found at the neutral plane: that location at which the relative movement between soil and pile is zero. It is possible to find the location of the neutral plane by performing an interaction calculation.

Figure 4.1 Concept of equilibrium of axially loaded piles



Earthquake shaking can affect the equilibrium condition of piles. Either due to transient higher vertical loads acting on the pile head, or due to liquefaction induced strength degradation of cohesionless soils. Settlements associated with liquefaction induced strength degradation can be calculated which allows to assess the potential consequences to the superstructure.

In the background document of NEN NPR 9998 (Deltares, (2018a)) a method for seismic settlement calculation of piles is presented that is aligned with the Dutch code of practice NEN 9997-1. This method is summarized in the next two paragraphs. For seismic assessments it is recommended, just as for shallow foundations, to perform two separate calculations to determine the seismically induced pile settlement during the earthquake s_{during} and the post-seismic pile settlement s_{after} and add these individual terms.

4.2.1 Pile settlement during the earthquake

During the earthquake, depending on the dynamic behaviour of the superstructure, there may be an increase of the pile load, while no seismic settlements have occurred. At limited downward displacement of the pile (several mm) the cumulative negative friction (Dutch: *negatieve kleef*) force F_{nk} will change direction and sign and result in positive friction. Hereby it is good to emphasize that in the Dutch code the formulation of positive- and negative friction differ; the former is a function of the measured cone tip resistance and the latter a function of the soil strength and lateral earth pressure coefficient at rest. From this perspective the following reasoning has been adopted in the NPR 9998:

- If the quasi-static temporal vertical load increment F_{dyn} is smaller than the initial cumulative negative friction force F_{nk} no analysis has to be performed and no additional settlements are expected to occur, compared to the current situation.
- If the quasi-static temporal vertical load increment F_{dyn} is larger than the initial cumulative negative friction force F_{nk}, the displacement can be determined by considering the pile settlement as a function of the combined SLS load F_{rep} and F_{dyn} accounting for positive skin friction along the entire shaft. Here also the reduction of the cone tip resistance (with a factor of $\sqrt{(1-r_{u;50\%})}$) should be used to account for the excess pore pressure that may have developed during the earthquake.
  - If the calculated settlement from this procedure is smaller than the initial (pre-seismic) situation, then the component F_{dyn} has no influence on the pile settlement and there will be no additional settlement compared to the current situation.
  - If the calculated settlement from this procedure is larger than the initial (pre-seismic) situation, then the different between the calculated settlement and the initial situation can be denoted as s_{during}.

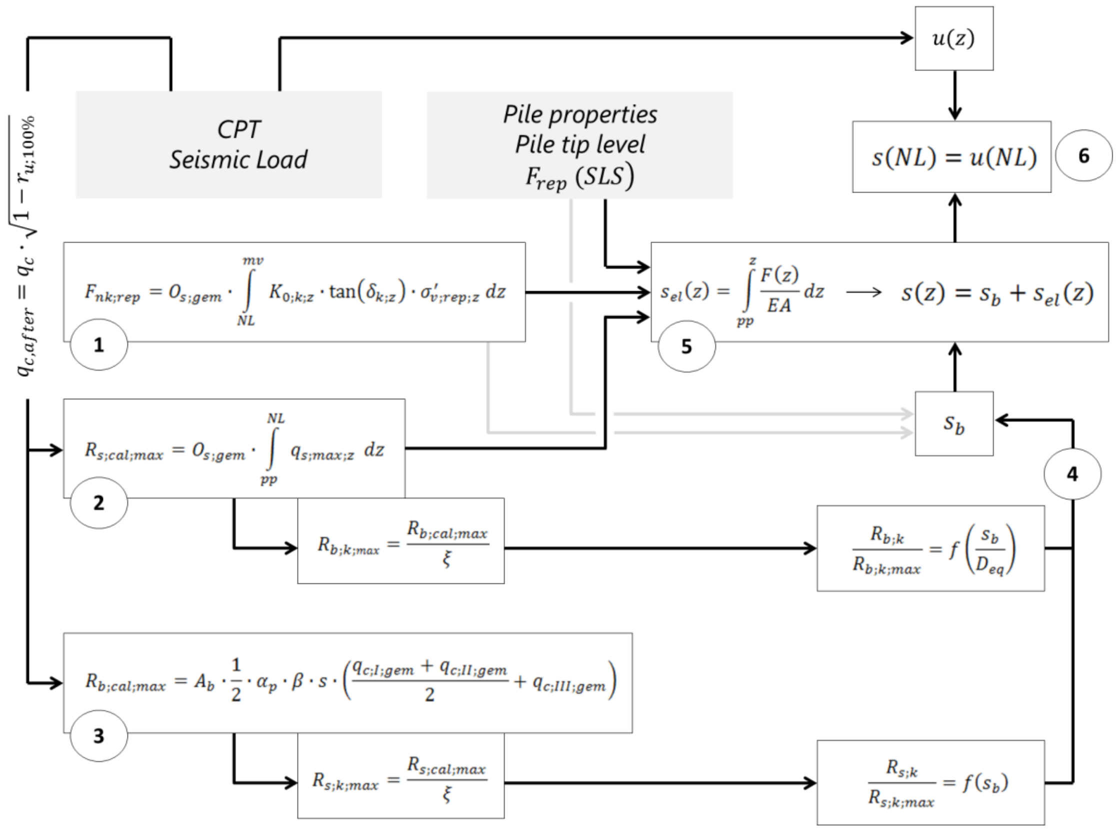
4.2.2 Pile settlement after the earthquake

After the earthquake two phenomena are expected which, slightly conservatively, can be expected to occur simultaneously:

- a strength reduction represented by a reduction of the cone tip resistances by a factor of $\sqrt{(1 - r_{u;100\%})}$
- post-seismic consolidation settlement as a consequence of dissipating excess pore pressures

These two effects may have an effect on the post-seismic displacement of the pile s_{after}. To quantify this displacement an interaction calculation is needed in which the pile behaviour including the two phenomena listed above, are compared to the pre-seismic pile behaviour. In the NPR 9998 background document a flowchart is presented (in Dutch) which in a slightly different format is presented below.

Figure 4.2 Flowchart for calculation of liquefaction induced post-seismic settlement of piles



The calculation comprises the following steps:

1. The cumulative negative skin friction force F_{nk} is calculating by integration of the negative skin friction from the neutral plane (NL) to the ground surface level (mv).
2. As mentioned above, the calculation of the positive skin friction depends on the cone tip resistance and the cumulative expected positive skin friction force $R_{s;cal;max}$ is calculated by integration from the pile tip level (pp) to the neutral plane (NL). Using the ξ factor to account for possible soil variability, the characteristic value of the positive skin friction force $R_{s;k;max}$ is calculated.
3. The expected value of the pile bearing capacity $R_{b;cal;max}$ is calculated using the Koppejan rule which takes a weighted and truncated average of different zones above and below the pile tip level. The characteristic value $R_{b;k;max}$ is obtained by using the ξ factor to account for soil variability.
4. The pile tip settlement s_b under a representative load F_{rep} and possibly a negative skin friction F_{nk} load, is determined by the combined stiffnesses of the shaft (function of the absolute displacement) and the pile tip (function of the displacement relative to the pile tip diameter D_{eq}). Here an important remark is made that there exists a difference between the 2012 and 2016 versions of NEN9997-1. In the 2012

version the displacement is calculated by comparing the acting load with expected values of pile shaft- and tip bearing capacities, $R_{s;cal;max}$ and $R_{b;cal;max}$, whereas in the 2016 version the displacement is calculated by comparing the acting load with characteristic values of pile shaft- and tip bearing capacities, $R_{s;k;max}$ and $R_{b;k;max}$. Note that the latter is presented in the flowchart of Figure X, although this does not mean that this is recommended, in particular for existing structures. In particular at high mobilization percentages in SLS and limited shaft resistances, the difference in expected displacements can be very significant.

5 The pile head displacement is composed of the displacement at the base s_b, the elastic deformation of the pile along the pile $s(z)$ and possibly settlements that occur below the pile tip level. The elastic deformation of the pile section between a depth z and the pile tip level (pp) is calculated by integration of the normal force divided by the axial stiffness over this section.

6 The neutral plane (NL) is found at that depth z at which there is no net displacement between the pile and the surrounding soil. To distinguish post-seismically induced pile settlement from pre-seismic settlement of the pile, an indicative settlement profile after installation of the pile $u_0(z)$ should be assumed with which the same interaction calculation can be performed to obtain the pre-seismic pile settlement. The interaction calculation can then be performed using reduced values of the cone tip resistances and total settlement profile $u(z) = u_0(z) + u_{eq}(z)$ in which $u_{eq}(z)$ can be determine using the volumetric settlement approach by Yoshimine et al. (2006). The difference in outcome between both calculation is the seismically induced pile settlement.

Regarding use and results of the method, the following should be stated:

- The more accurate the estimation of the initial settlement profile, the better the prediction of earthquake induced pile settlement.
- In cases of limited to medium liquefaction effects along the shaft of the pile, the pile head displacement is expected to be very small. Naturally this statement is qualitative as the behaviour depends on factors such as initial negative skin friction and the ratio between initial pile tip- and shaft mobilization.
- Liquefaction in the zone around the pile tip will yield a significant impact on the pile displacement behaviour. In general pile tips are positioned in densely packed and/or deep layers which are less prone to liquefaction, but exceptions can always occur. Based on recent efforts for NEN NPR 9998 2020 update it is concluded that the area around Overschild forms such an exception.

4.3 Structural capacity

If the elastic structural capacity of a pile is concluded to be exceeded due to earthquake load, this implies a risk of crack formation and or plastic deformation. The degree of damage in this case affects the residual load bearing capacity of a cross section. Possible effects of structural capacity exceedance of piles are (partial) loss of horizontal and vertical bearing capacity and/or vertical pile head settlement.

4.3.1 Structural capacity along the pile shaft

Kinematic loads can cause the exceedance of structural capacity along the pile shaft deeper below the ground surface. Kinematic loads on piles are caused by ground displacements due to earthquake waves. The effects of kinematic loads on piles are mainly concentrated at the interface of layers with large stiffness difference. The response of the pile to seismic waves in the ground results in internal forces in the pile.

In NEN-NPR 9998, in line with Eurocode 8 part 5 clause 5.4.2, exclusion criteria have already been included that indicate in which cases kinematic pile loads do not need to be considered. The same consideration applies for the pile foundations of industrial facilities.

4.3.2 Structural capacity of the pile head

During horizontal seismic shaking of a foundation relative to the ground, bending moments and shear forces occur in the pile, which concentrate at the pile head. Changes in axial load can also occur due to vertical shaking or rocking motion of a structure. In accordance with the NC criterion of NPR 9998, the effects of these loads may lead to local damage, but not to (partial) collapse of the structure. A similar starting point could be adopted for industrial structures: local pile damage is acceptable, provided that no LoC or global loss of structural stability results. This is therefore the limit state to be verified.

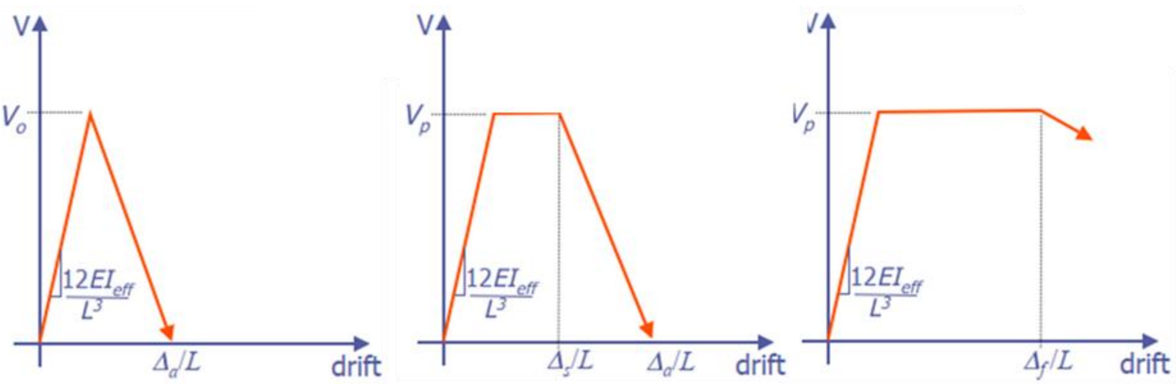
Insufficient axial residual load-bearing capacity of piles occurs if horizontal displacements become too large. NEN-NPR9998 does not set specific requirements for horizontal (permanent) displacements, other than the requirements for relative displacement of building storeys (inter-story drift limits). The requirements for relative displacements of building storeys do not apply to pile foundations. Instead, horizontal displacements of the foundation do only become critical if:
- This triggers P-delta effects.
- This causes loss of axial pile bearing capacity.

Elwood & Moehle (Elwood & Moehle, 2003) have developed models that describe the residual bearing capacity of concrete columns after exceeding the horizontal capacity as a function of seismic drift. The Elwood-Moehle models give limit values for horizontal displacements of columns above which horizontal or axial load-bearing capacity is lost. These limit values are significantly lower than the values for which P-delta effects become critical according to EN 1998-1. This is because the Elwood-Mohle values are based on loss of cross-section capacity while the P-delta limits according to EN 1998-1 are based on geometrically non-linear effects that increase loads on cross-sections. It can therefore be assumed for piles that if the Elwood-Moehle limit values are met, the criteria for P-delta effects are met as well.

The Elwood-Moehle models distinguish between:
- Shear columns (shear force capacity is critical when loaded horizontally);
- Flexural columns (columns in which a (plastic) hinge will form because shear capacity large compared to bending moment capacity);
- Shear-flexural columns (columns for which a combined shear force - bending moment failure mechanism develops).

The limit values of the displacement at which loss of shear force or axial capacity occurs depend on the type of column (refer to Figure 4.3). For more information reference is made to (Elwood & Moehle, 2003).

Figure 4.3 Failure models according to Elwood & Moehle (2003), left: shear columns, middle: shear-flexural columns, right: flexural columns



The Elwood-Moehle models have been used in studies for Groningen (Fugro 2018, Witteveen+Bos 2018, NEN Taakgroep 2, 2020). The study by Fugro (2018) showed that at very high seismic loads, residual bearing capacity of piles is still retained. The first study by Witteveen+Bos (2018) concludes that when a foundation is considered as a system of both piles and embedding in the ground, this results in a higher total horizontal capacity. This total horizontal capacity is also retained to some extent at horizontal displacements greater than the displacement where shear capacity of piles is exceeded according to Elwood-Moehle. The conclusion of previous two studies is also confirmed by observations in practice, which show that heavily damaged piles continue to provide bearing capacity after an earthquake.

In a in support of NEN NPR 9998 2020 update (NEN Taakgroep 2, (2020)), parametric analysis for the estimation of the developed horizontal displacements at the foundation of residential buildings were performed. The analysis were performed as Nonlinear Push-Over Response Spectrum (NLPO) simulations, of 2-degree-of-freedom systems, where one degree-of-freedom corresponds to the superstructure and the other degree-of-freedom corresponds to the foundation. The calculated horizontal displacements at foundation level can be compared with the displacement limits according to the Elwood-Moehle models. The calculation approach was validated against NLTH analyses of two residential buildings.

The parametric study was performed for a database of parameter variations which was set up based on advice reports for strengthening (Versterkingsadvies rapportage's) of residential buildings. As a result, the variation parameters that were used for the NLPO simulations of the NPR 9998 development do not cover the wider range of parameter variations that is encountered in industrial facilities (Table 4.1, and Appendix I).

Table 4.1 Variation parameters NLPO calculation in Witteveen+Bos (2020) and in the present study.

Parameter	**Variations for residential buildings [NEN Taakgroep 2, 2020]**	**Variations for industrial facilities buildings [present study]**
Pile dimensions	220, 250, 290 mm	220, 350, 500 mm
Equivalent clamping depth of pile	For sand: 3D, 6D, 9D For clay: 7D, 10D, 13D	For sand: 3D, 6D, 9D For clay: 7D, 10D, 13D
Fundamental period of the building with fixed base	0.15, 0.30, 0.45 s	0.2, 0.4, 0.8, 1.0, 1.5 s
Building mass	150, 300, 500 ton	250, 500, 1000, 3000 ton
Gap formation between beam and ground	with gap formation	with gap formation
Share of total mass in superstructure (upper degree of freedom of 2-DOF)	50, 75, 90 %	10, 50, 90 %
Share of total mass in foundation (lower degree of freedom of 2-DOF)	50, 25, 10 %	90, 50, 10 %
Axial capacity per pile	150, 250, 350 kN	100, 350, 500 kN, (750, 1000 kN for pile dimension 500 mm)
Ratio between compressive load applied on the pile versus axial capacity of the pile	1.0	0, 0.5, 1.0
Seismic load	Response spectra* for the following locations: Groningen city (peak ground acceleration 0.10g) Delfzijl (peak ground acceleration 0.15g) Ten Post (peak ground acceleration 0.20g) Loppersum (peak ground acceleration 0.25g)	Response spectrum* only for Delfzijl (peak ground acceleration 0.15g)**

* According to seismischekrachten.nen.nl, GMM v6, time path T4.
** Only the spectrum with pga 0.15g is examined in the current study since this is the limit criterium that is checked.

NEN NPR 9998 task force 2 concluded that the NLPO based method seems capable to reasonably predict lateral foundation displacements, and therefore can also be used to estimate seismic post-damage pile foundation residual bearing capacity. The method is suggested to be used for this purpose accordingly. For industrial assets this could be suggested as well. For more details and information regarding the calculations steps one is referred to the NEN NPR 9998 background document (NEN Taakgroep 2, (2020)).

Furthermore, NPR 9998 task force 2 concluded that no further assessment of structural capacity of pile foundations of existing CC1a/b structures needs to be carried out for locations where a design peak ground acceleration lass than or equal to 0.15 g applies for a return period that applies to near-collapse limit state verification. In order to identify whether the exclusion criterium (pga ≤ 0.15 g) for the assessment of pile foundations applies also for industrial facilities, the parametric study is extended for a wider range of parameter variations (refer to table 4.1). The results are discussed in 4.3.3.

4.3.3 General exclusion criteria for seismic structural pile capacity assessment

Combined results of different studies for Groningen (Arup (2017a), Arup (207b), Fugro (2018), Witteveen+Bos (2018) and NEN taakgroep 2 (2020) have substantiated a high level exclusion criterium for piled foundations in NEN NPR 9998. This criterion is included in the latest revision of NPR 9998 [NPR 9998:2020], par. 10.4.1:

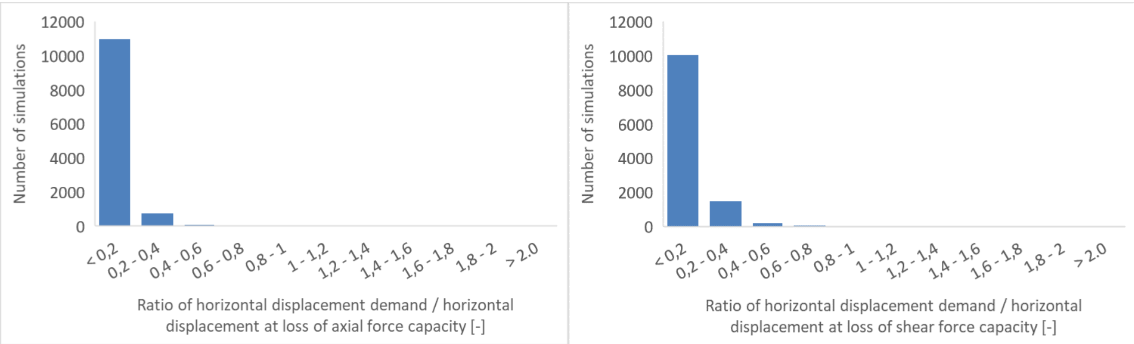
For a design seismic load level below 0.15 g for pile foundations of buildings in consequence classes CC1a and CC1b, only a GEO limit state verification is required. Verification of pile structural capacity is not required for under these conditions.

This criterion for NPR 9998 has been assigned to CC1a and CC1b buildings only, because the supporting analyses did cover the typical characteristics of residential buildings that fall in these categories. The concept behind the exclusion criterion however applies to any type of building, provided that no global overturning risks apply for the design seismic load level. Global overturning risks occur if successive tension failure of piles is calculated for the design seismic load. In this case structural capacity of piles needs to be sufficient.

As reported in paragraph 4.3.2, in the present project the simulations performed for CC1a/b buildings for NEN NPR 9998 development have been extended to a wider range of structure characteristics. Calculated displacement at the foundation level for a total of almost 12,000 simulations is compared with the corresponding limit of horizontal displacement (drift) at shear and axial failure according to the Elwood-Moehle model. The ratio between the calculated displacement and the limit of horizontal displacement at loss of shear and axial load capacity according to Elwood-Moehle is taken as a measure of the probability that the vertical load-bearing capacity of the pile foundation will be affected. The results of these simulations are summarized by the charts of figure 4.4.

The percentage of simulations that result in a drift corresponding to loss of vertical bearing capacity for the design spectrum of Delfzijl (pga = 0.15 g) is 0 %. Delfzijl has the highest seismic hazard level of all industrial areas in Groningen (Sweco, (2019)). Based on these results, it is proposed to adopt the exclusion criterium of NPR9998:2020 for pile foundations of industrial facilities at locations where a peak ground acceleration equal to or less than 0.15 g applies for a design seismic load that applies to the LoC or near-collapse limit state.

Figure 4.4 Ratio of calculated displacement at foundation over displacement at loss of shear capacity (left) and over displacement at loss of axial capacity, for design spectrum Delfzijl (2475 years, T4)



The exclusion criterion only applies if piles structural capacity is not critical to maintain global overturning stability. With reference to table 3.3 it is anticipated that structural capacity assessments of piles are, given the decreasing seismic hazard, only still required for high risk structures in the industrial areas of Delfzijl and Hoogezand, when assessed based on the 2,475 years mean return period spectrum. For such assessment the methods as discussed in paragraphs 4.3 could be used in order to assess with limited effort if pile axial bearing capacity is maintained, should lateral failure occur.

For piles, the exclusion criterion could not be generalized to also include GEO limit state verification because of highly exceptional sites known near Overschild. In this region residential buildings are present which are founded on short piles (5 m length) of which geotechnical bearing capacity is obtained from thin loosely packed sand layers. No further assessment has been performed yet to conclude if this criterion could be released to also include no need for GEO limit state assessment for specific industrial areas like Delfzijl and/or Eemshaven.

For further development of the Selectiemethodiek (Arcadis, (2020)) or the Quick risk calculation tool (Witteveen+Bos, (2020) it could be argued that only for consequence class IV and V structures (table 4.1 of Deltares&TNO (2018)) in Delfzijl and Hoogezand any seismic assessment of the risk contribution of pile structural failure seems necessary. An area-based evaluation for the specific industrial areas is considered an efficient approach to prove such criterion to apply to the GEO limit state as well, given the highly exceptional soil conditions that prohibited NPR 9998 from adopting a generalized exclusion criterion that also covers geotechnical bearing capacity.

5

CONCLUSIONS AND RECOMMENDATIONS

This report evaluates possible optimization of seismic foundation assessments of industrial structures in Groningen. Two methods have been previously developed for seismic integrity assessment of industrial structures, being the LoC-toets method by WMA and the risk-based method by Deltares&TNO. It is concluded that both methods more or less align regarding the performance criteria: LoC or global loss of structural stability shall be mitigated. Both methods differ when it comes to target reliability. Based on the performance criteria of structures, damage criteria of foundations can be derived. Given the performance criteria set for existing industrial assets in Groningen, damage to foundations in itself is acceptable provided that foundations maintain their function and foundation damage doesn’t trigger severe consequences for the superstructure. A clear analogy with the NEN NPR 9998 philosophy for (residential) buildings exist, for which the near-collapse limit state is evaluated.

For shallow foundations seismic ‘damage’ is typically expressed in terms of (differential) settlement due to liquefaction. Simplified methods exist to estimate seismic foundation settlements in literature. NEN NPR 9998 includes such method specifically developed for strip footings in Groningen. These methods can be useful for seismic verification of industrial assets in Groningen as well. Depending on the characteristics of a specific case a suitable settlement estimation method can be selected. Under certain conditions seismic settlement of severe magnitude can be excluded for shallow foundations. NEN NPR 9998 sets a threshold at 0.125 g peak ground acceleration, below which no seismic shallow foundation verification is necessary. This threshold is not related to any foundation characteristic and therefore applies to shallow foundations of industrial assets as well.

For piled foundations seismic damage materializes as settlement or damage of piles. The present document has outlined methods included in NPR 9998 for prediction of liquefaction induced seismic settlements of piles. Seismic damage of piles might occur in cases where the shear or bending moment capacity of piles is insufficient. Resulting pile damage is only critical when the load bearing function of the piles is compromised. Various studies have been performed focussing on this aspect, resulting NEN NPR 9998 to exclude any structural capacity assessment of piles for design peak ground acceleration levels below 0.15 g. Additional simulations reported in the present document have indicated that this threshold applies to industrial assets as well. Where this threshold applies to STR limit states only, it should be noted that according to NEN NPR 9998:2020 for GEO limit states of piles a lower threshold could apply under extremely unfavourable conditions. An area-based approach for industrial regions in Groningen could efficiently overcome this limitation of NEN NPR 9998:2020 and further optimize the approach for industrial assets.

A combination of the methods described above results a more performance-based approach to foundation assessments for industrial assets, in contrast to the more strength-based approach mostly used in practice so far. The present document summarizes these methods and helps engineers to understand the added value of these methods for industrial asset verifications in Groningen. Integrating these methods and concepts of NEN NPR 9998 into future developments of methods for seismic assessment of industry in Groningen is recommended. Further development into a ‘Blauwdruk’ document for seismic foundation assessments is possible. Alternatively, the concepts and methods discussed here could be integrated in the ongoing developments of the Selectiemethodiek (Arcadis, (2020) and the related quick seismic risk calculation tool for industry (Witteveen+Bos, (2020)).

6

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Appendices

APPENDIX: INVENTORY OF INDUSTRIAL STRUCTURES FROM A PERSPECTIVE OF EFFECTIVE LOAD ON PILE FOUNDATIONS

Component	Pile dimensions	No. piles	Vertical bearing capacity [kN]	Pile material	Concrete class	Rebars	Stirrups	Reinforcement class	Foundation stiffness [MN/m]	Foundation beam [mm x mm]	Plan dimensions [m x m]	Height [m]	Weight of superstructure [ton]	Weight of foundation [ton]	Stiffness / Eigenperiod of superstructure	Load per pile (avarage)
Calciner	Shallow foundation	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
HCL tank	Shallow foundation	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Shaft kiln	Shallow foundation	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Silo building	Shallow foundation	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Steel structure (Active Carbon Filters at the roof)	Ø220 - 320	231	-	concrete	-	-	-	-	-	450 x 500/950	54 x 49	7 - 13	1425	1654	-	131
Settling tank	Ø450	12	790-1100	concrete	C28/35	6 Ø16	Ø8-300	B500	~60	600 x 800/900	8 x 8	9.9	600	9	K~125 MN/m	498
Nitrogen tank	-	-	-	-	-	-	-	-	-	-	6 x 5,3	4	11	-	-	-
Hopper packing area	220x220 - 350x350	-	300-863	concrete	C30/37	-	-	-	20	-	8 x 7,5	~15	133	-	-	-
Granulate hopper	220x220	4	300	concrete	C30/37	-	-	-	20	-	4,8 x 4,8	~10,5	26	12	K~1-2 MN/m	93
feedhopper building	250x250 & Ø250	64	~ 800	concrete	C30/37	-	-	-	20	-	24 x 17,5	~22	633	391	T= 0,7-2 s	157
Hot water tank	220x220 - 350x350	-	300-863	concrete	C30/37	-	-	-	-	-	2,4 x 2,4	~5,5	50	-	T~ 1 s	-
Sodium silicate tank	220x220	6	-	concrete	C30/37	-	-	-	20	-	~ 4 x 4	~5,3	150	8	T~ 0,4 s	258
Production building	-	-	-	-	-	-	-	-	-	-	31 x 68	23	2200	-	T~ 3,16 s	
Fuel storage tank	Shallow foundation	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Glycol tanks T14A/B	290x290	74	200-340	prefab prestress concrete	-	-	-	-	-	450 x 600	42,5 x 50	~5,8	240	2390.625	T = 0,55 - 0,7 s	349
Control building	220x220	39	470-860	prefab concrete	-	-	-	-	10-15	400 x 500	15 x18	~3	55	200	T~ 0,3 s	64
Marine arms	Not relevant, in the sea	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Fuel storage tanks	Shallow foundation	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Chloorleidingen op leidingbrug, Leidingbaan3 (and Leidingbaan3)	Ø420 (Ø350)	10 (10)	-	concrete	C30/37	6 Ø16 (5 Ø14)	Ø10-200	B500	-	500 x 500	-	7 - 9	~95	~95	T = 0,2 - 0,5s	93
Chloorkoeling en -droging	Ø324	33	990	Vibropaal	C35/45	4 Ø8	Ø8--	B500A	-	800 x 500	16,2 x 17,9	27	249	286	T = 0,2s	159
Chloorcompressiegebouw	Ø324	18	843	Vibropaal	C28/35	-	-		14.5	1050 x 500	10,5 x 11,5	4 - 8	89	204	T = 0,16s	160
Silotank AT-2893	Ø323	37	-	Staal	-	-	-	-	-	-	-	-	-	-	-	-
Kolom AC-4402	Vibrexpalen Ø406	7	-	Vibrexpalen		4 Ø12	-	-	22-55	500 x 700	~7 x 7	~ 23	294	61	T = 0,15 - 0,7s	498
Leidingbrug LN-22010 met chloorleiding	Vibrexpalen Ø406	22	-	Vibrexpalen	-	minimum	-	-	26	600 x 600/800	~100 long	~13,5	201	101	T = 0,1 - 0,8s	135
Besturingsgebouwen	Vibrexpalen Ø406 (& avegaarpalen Ø450, CFA)	57 (17)	-	Vibrexpalen (avegaarpalen CFA)	C20/25	6 Ø10 (6 Ø12)	Ø8-300	-	18,8 (33,3)	-	33 x 41	2,8 - 6,8	2818	846	T = 0,3 - 0,45s	486
Sectie 1200 (& AC-1202) [incl. ractors AR-1201A&B and tank AT1201]	Vibrexpalen Ø406 (& Atlaspalen Ø 360)	53 (7)	-	Vibrexpalen (& Atlaspalen CFA)	C20/25	6 Ø10 (4 Ø 14)	-	-	-	-	12 x 54	~26	3039	-	T = 0,7s	497