Handreiking Aardbevingsbestendigheid Industrie

Deel 2: toepassing van de LoC-methode

Nationaal Coördinator Groningen

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INHOUDSOPGAVE

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INTRODUCTIE

1.1 Revisiebeheer

In 2019 is de 'Handreiking Aardbevingsbestendigheid Industrie - Fase 2a/b (LoC-methode) en Fase 2c' opgesteld [ref. 1]. Het document beschrijft zowel het gebruik van de LoC-methode, als het proces van de diverse aardbevingsonderzoeken. Nieuwe ontwikkelingen, zoals de toepassen van de Selectiemethodieken en de invoering van de 'Beleidsregel vergoeding kosten aardbevingsbestendige industrie Groningen' [ref. 3] (hierna genoemd: Beleidsregel), vergen een bijgewerkte versie. De Handreiking Aardbevingsbestendigheid Industrie uit 2019 is hierop aangepast. Voor de 2022 revisie is het document tevens opgedeeld in 2 losse delen:

1. 'Handreiking Aardbevingsbestendigheid Industrie - Deel 1: procesbeschrijving uitgebreide beoordeling Fase 1, Fase 2 en Fase 3' [ref. 2]: dit deel beschrijft het proces van de verschillende fases van de onderzoeken;
2. 'Handreiking Aardbevingsbestendigheid Industrie - Deel 2: toepassing van de LoC-methode' (voorliggende document); dit deel beschrijft de inhoudelijke toepassing van de LoC-methode.

In het vervolg zijn de beide documenten verkort aangeduid als 'Handreiking deel 1 procesbeschrijving' en 'Handreiking deel 2 LoC-methode'.

1.2 Scope en doelstelling van deze Handreiking deel 2 LoC-methode

De voorliggende Handreiking deel 1 procesbeschrijving omvat een handreiking voor overheden, de chemiebedrijven en hun consultants voor de Fase 2 (kwantitatieve) onderzoeken naar de aardbevingsbestendigheid van industriële installaties met gevaarlijke stoffen in Groningen. Voor deze Fase 2 zijn twee methoden ontwikkeld die beide kunnen worden toegepast [ref. 3] door de chemiebedrijven voor bestaande bouw, verbouw en nieuwbouw:

- de LoC-methode (of LoC-toets), ontwikkeld door de Werkgroep Maatgevende Aardbevingsbelasting (WMA) ([ref. 7] tot en met [ref. 10]);
- de Risico-gebaseerde rekenmethodiek voor Fase 2 (hierna genoemd: Risico-gebaseerde rekenmethodiek), ontwikkeld door Deltares/TNO [ref. 16].

Voor de Risico-gebaseerde rekenmethodiek van Deltares/TNO is sinds juni 2018 een uitgebreide handreiking [ref. 16] beschikbaar, voor de LoC-methode zijn losse adviesdocumenten beschikbaar. In de voorliggende Handreiking zijn de adviezen voor de LoC-methode verzameld, is hun rangorde aangegeven en wordt een algemene toelichting inclusief *lessons*' *learned* gegeven geeft voor het toepassen van deze methode. In dit document wordt één overkoepelende handreiking voor de LoC-methode gedocumenteerd.

De doelstelling van dit document is als volgt:

- het presenteren van één overkoepelend document voor toepassing van de LoC-methode in Fase 2;
- het bevorderen van de uniformiteit van de Fase 2 berekeningsrapporten bij toepassing van de LoC-methode;
- het bijdragen aan het verduidelijken, versnellen en optimaliseren van het proces bij het toepassen van de LoC-methode.

English translation of the above listed scope and purpose of this *'Handreiking Aardbevingsbestendigheid Industrie - Deel 2: toepassing van de LoC-methode' [ref. 2];* ('Guideline Earthquake resistant Industry - Part 2: application of the LoC-method')

The presented document describes a guideline for governments, chemical companies and their consultants for assessments into the seismic capacity of industrial installations containing hazardous contents in Groningen, The Netherlands. This part 2 describes the application of the Phase 2 LoC-method.

For Phase 2, two methods have been developed which both can be applied by the chemical companies for assessing existing installations, renovations or the seismic design of new installations:
- The LoC-method (or 'LoC toets': LoC assessment), developed by the 'Werkgroep Maatgevende Aardbevingsbelasting' (WMA) ([ref. 7] up to and including [ref. 10]).
- The Risk-based calculation method for phase 2, developed by Deltares/TNO [ref. 16].

For the Risk based calculation method for phase 2 by Deltares/TNO, a single guideline is available since June 2018 [ref. 16]. For the LoC-method several separate documents are available. These documents have been compiled and ranked in this Guideline. Next to that, is a general explanation is provided of the application of the LoC-method including lessons learned. This document forms a complete guideline for the LoC-method with respect to the technical aspects.

The objectives of this document are as follows:
- to form one complete document for the application of the LoC-method in Phase 2.
- to enhance uniformity of the Phase 2 calculation reports when applying the LoC-method.
- to contribute to the elaboration, acceleration and optimization the process within the application of the LoC-method.

The introduction of the 'Beleidsregel' and other recent developments necessitate an update of the 2019 version of this document [ref. 1]. Compared to this previous version, the 2022 update has now has been split up into two separate documents:
- Guideline Earthquake resistant Industry - Part 1: process description elaborate assessment Phase 1, Phase 2 and Phase 3'; this part describes the process of the several phases of the assessments;
- Guideline Earthquake resistant Industry - Part 2: application of the LoC-method [ref. 2]; this part describes in the technical application of the LoC-method.

1.3 Lijst met afkortingen en terminologieën

BoD	Uitgangspuntennota (Basis of Design)
CPT	Sondering (Cone Penetration Test)
CQC	Volledige kwadratische combinatieregel (Complete Quadratic Combination)
DCL	Lage ductiliteitsklasse uit Eurocode 8 (Ductility Class Low)
DCM	Medium ductiliteitsklasse uit Eurocode 8 (Ductility Class Medium)
DCH	Hoge ductiliteitsklasse uit Eurocode 8 (Ductility Class High)
DL	Damage Limitation, grenstoestand voor aardbevingsbestendigheid NPR 9998
DS	Damage State: mate van schade bij een aardbeving
EEM	Eindige Elementen Methode, is gelijk aan FEM
FEM	Finite Element Method, is gelijk aan EEM
GBoD	Algemene uitgangspunten LoC-methode (Generic Basis of Design), zie [ref. 13]
GMM	Ground Motion Model
LoC	Loss of Containment
MRSA	Spectrale modale responsberekening (Modal Response Spectrum method of Analysis)
NC	Near Collapse, grenstoestand voor aardbevingsbestendigheid NPR 9998
NCG	Nationaal Coördinator Groningen
NEN	Nederlandse Norm, Koninklijk Nederlands Normalisatie-instituut

NPR (9998)	Nederlandse Praktijkrichtlijn (9998)
PGA	Maximale grondversnelling op maaiveld (Peak Ground Acceleration)
q-factor	Gedragsfactor Eurocode 8
SD	Significant Damage, grenstoestand voor aardbevingsbestendigheid NPR 9998
SRSS	Kwadratische combinatieregel (Square Root of the Sum of Squares)
SSI	Grond-constructie interactie (Soil Structure Interaction)
T1, T2, etc.	Tijdvak waarvoor aardbevingsbelastingen zijn berekend (bijvoorbeeld T5: 1-10-2021 t/m 30-9-2023)
UGT	Uiterste grenstoestand NEN-EN 1990, is gelijk aan ULS
ULS	Ultimate limit state NEN-EN 1990, is gelijk aan UGT
WMA	Werkgroep Maatgevende Aardbevingsbelasting

2

REFERENTIES EN DOCUMENTATIE

2.1 Introductie

In de navolgende tabellen is (per paragraaf gesorteerd naar categorie) een zo compleet mogelijk overzicht gegeven van alle relevante documenten die van toepassing zijn op de LoC-methode voor het kwantiteit beoordelen van aardbevingsrisico's voor industriebedrijven. Voor elk document is een korte toelichting gegeven en aangegeven waar het document kan worden gevonden[1]. Per paragraaf is aangegeven wat de hiërarchie van de documenten is (indien van toepassing).

Voor aanvullende, algemene documentatie wordt naar de Handreiking deel 1 procesbeschrijving [ref. 2] verwezen.

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1 Het streven is om nieuwe versies en nieuwe documenten te plaatsen op de website van de NCG, thema chemische industrie: https://www.nationaalcoordinatorgroningen.nl/onderwerpen/chemische-industrie

2.2 Algemene documenten

Tabel 2.1 geeft een overzicht van algemene documenten waaronder de Handreiking deel 1 Procesbeschrijving [ref. 2] en de Beleidsregel [ref. 3].

Tabel 2.1 Overzicht algemene documenten voor het beoordelen van industriële installaties in Groningen op aardbevingsbestendigheid

Referentie	Auteur	Document	Kenmerk	Versie en datum	Bron	Toelichting
[ref. 1]	Witteveen+Bos	Handreiking Aardbevingsbestendigheid Industrie - Fase 2a/b (LoC-methode) en fase 2c	110756/19-012.389	Definitief d.d. 25 juli 2019	-	Handreiking voor het gebruik van de LoC-methode en algemene procesbeschrijving. Opgevolgd door de combinatie van het voorliggende document Handreiking deel 2 LoC-methode en de Handreiking deel 1 procesbeschrijving [ref. 2].
[ref. 2]	Witteveen+Bos	Handreiking Aardbevingsbestendigheid Industrie - Deel 1: procesbeschrijving uitgebreide beoordeling Fase 1, Fase 2 en Fase 3	127427/22-008.261	Definitief d.d. 7 juni 2022	https://www.nationaalcoordinatorgroningen.nl/onderwerpen/chemische-industrie	Handreiking met daarin de procesbeschrijving van de aanpak voor de aardbevingsbestendigheid voor de industrie.
[ref. 3]	Ministerie van Economische zaken en Klimaat (EZK)	Beleidsregel vergoeding kosten aardbevingsbestendige industrie Groningen	nr. WJZ / 20220974	30 januari 2021	https://wetten.overheid.nl/jci1.3:c:BWBR0044799&z=2021-02-10&g=2021-02-10	Beleidsregel met voorwaarden vergoeding kosten aardbevingsonderzoeken (inclusief toelichting). Gepubliceerd op 9 februari 2021 in de Staatscourant Nr. 6830.
[ref. 4]	Nationaal Coördinator Groningen (NCG)	Beleidsregel vergoeding aardbevingsbestendige industrie Groningen - Overzicht vergoedingen, methodes en aan te leveren documenten	-	20 mei 2021	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/publicaties/2021/5/20/overzicht-vergoedingen-methodes-en-aan-te-leveren-documenten-aardbevingsbestendige-industrie-groningen/Overzicht+vergoedingen%2C+methodes+en+aan+te+leveren+documenten+aardbevingsbestendige+industrie+Groningen.pdf	Beknopt overzicht van de Beleidsregel [ref. 3] met overzicht vergoedingen, methodes en aan te leveren documenten.
[ref. 5]	Nationaal Coördinator Groningen (NCG)	Meerjarenprogramma Aardbevingsbestendig en Kansrijk Groningen 2017-2021	16140933	Definitief	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/beleidsnotas/2016/december/23/meerjarenprogramma-2017-2021/Meerjarenprogramma+Aardbevingsbestendig+en+Kansrijk+Groningen+2017-2021.pdf	Paragraaf 3.8 is relevant voor de industrie.
[ref. 6]	Nationaal Coördinator Groningen (NCG)	Addendum bij Meerjarenprogramma Aardbevingsbestendig en Kansrijk Groningen 2017-2021	-	Definitief d.d. 30 juni 2017	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/rapporten/2017/juli/7/addendum-meerjarenprogramma-2017-2021/Addendum+Meerjarenprogramma+Aardbevingsbestendig+en+Kansrijk+Groningen+2017-2021.pdf	Hoofdstuk 4 is relevant voor de industrie.

2.3 Fase 2 documenten voor toepassing van de LoC-methode (ontwikkeld door WMA)

2.3.1 Algemeen en hiërarchie van documenten

In deze paragraaf staan de relevante documenten beschreven wanneer voor Fase 2 de LoC-methode wordt toegepast. Er is een onderscheid gemaakt tussen achtergronddocumenten (opgenomen in tabel 2.2), de inhoudelijke GBoD en aanverwante notities van TU Delft in tabel 2.3 en overige inhoudelijke documenten in tabel 2.5. De achtergronddocumenten (WMA adviezen) kunnen als informatief worden beschouwd wanneer de documenten uit tabel 2.3 en tabel 2.5 worden gehanteerd; er wordt dan ook voldaan aan de WMA adviezen.

Voor de volledigheid zijn de inhoudelijke documenten uit tabel 2.3 en tabel 2.5 opgenomen in bijlage I tot en met VII. Op deze manier ontstaat met de voorliggende Handreiking deel 2 LoC-methode één gecombineerd document waar voor de beoogde doelgroep van dit document alle inhoudelijke documenten zijn verzameld. Voor het consultants en overige gebruikers van de Handreiking deel 2 LoC-methode volstaat het in het vervolg om naar deze Handreiking deel 2 LoC-methode en bijlagen te verwijzen; het is dan niet meer nodig om naar de individuele documenten te verwijzen.

Voor een aantal specifieke veel voorkomende installaties is een Blauwdruk opgesteld (zie paragraaf 2.5), wat een generieke aanpak is voor één specifiek type installatie.

Qua prioriteit zijn de GBoD [ref. 13], Explanatory notes [ref. 14] en de ‘Lessons’ learned’ [ref. 15] van TU Delft leidend voor te volgen berekeningen, ongeacht het type installatie. Daarna volgen de aanvullende bepaling uit de voorliggende Handreiking deel 2 LoC-methode, de Blauwdrukken en overige toelichtende documenten (zie tabel 2.5). De hiërarchie van de te volgen documenten is weergegeven in afbeelding 2.1. Conform [ref. 15] zijn Eurocodes en andere Nederlandse normen altijd maatgevend boven internationale normen en richtlijnen.

Afbeelding 2.1 Hiërarchie van documenten bij toepassing van de LoC-methode

Niveau 1	• WMA adviezen[1)]. • GBoD, Explanatory notes en Lessons' learned van TU Delft. • Meest recente en geaccordeerde versie (door Stuurgroep Industrie) van het KNMI Shakemaps rapport[1)] en afgeleide respons spectra.	*[ref. 7], [ref. 8], [ref. 10]* *[ref. 13], [ref. 14], [ref. 15]* *[ref. 12]*
Niveau 2	• Aanvullende bepalingen uit de voorliggende Handreiking Aardbevingsbestendigheid Industrie - Deel 2: toepassing van de LoC-methode. • Blauwdrukken en overige toelichtende documenten.	*Zie tabel 2.5*
Niveau 3	• Normen (en richtlijnen) waar de documenten op niveau 1 en 2 naar verwijzen, waarbij 1) Nederlandse en 2) Europese normen (bijvoorbeeld Eurocodes) leidend zijn ten opzichte van 3) internationale normen en richtlijnen.	*Zie tabel 2.6*
Niveau 4	• Overige normen en richtlijnen die mogelijk van toepassing zijn voor de beoordelen van de installatie, waarbij waarbij 1) Nederlandse en 2) Europese normen (bijvoorbeeld Eurocodes) leidend zijn ten opzichte van 3) internationale normen en richtlijnen.	*Zie tabel 2.6*

Opmerking bij afbeelding:

1) Informatieve documenten.

Opgemerkt wordt dat de Blauwdrukken primair zijn ontwikkeld om de aanpak van veel voorkomende typen installaties te uniformiseren en het proces te versnellen. Ondanks dat de Blauwdrukken in afbeelding 2.1 boven nationale normen staan is het toegestaan om gemotiveerd van de Blauwdrukken af te wijken (bijvoorbeeld in aanpak of in normgeving) indien daar een gegronde aanleiding voor is. De redenen dat de in de hiërarchie de Blauwdrukken boven (inter)nationale normgeving zijn geplaatst, zijn als volgt:

- de Blauwdrukken zijn opgesteld in lijn met de GBoD [ref. 13], relevante Eurocodes en overige (inter)nationale normering, maar met specifieke aanvullingen voor gebruik van de normen voor het beoordelen van (chemische) installaties in Groningen. Dit is nodig omdat de meeste (inter)nationale normen zijn opgesteld voor tektonische aardbevingen en daarom niet altijd geschikt zijn voor de specifieke situatie in Groningen met geïnduceerde aardbevingen. In de Blauwdrukken worden daarom specifieke aanvullingen op normen gegeven voor toepassing in Groningen;
- het is een reeds geaccordeerde aanpak voor toepassing binnen het industriedossier Groningen [ref. 3].

2.3.2 Achtergronddocumenten LoC- methode (informatief)

De WMA-adviezen geven de aanleiding en onderbouwing van de ontwikkelde LoC-methode. In hoofdstuk 3 van de voorliggende Handreiking worden de belangrijkste uitgangspunten herhaald, waarbij de focus ligt op de toepassing van de methode voor verschillende situaties (bijvoorbeeld bestaande bouw en nieuwbouw).

Tabel 2.2 Overzicht achtergronddocumenten Fase 2 - LoC-methode

Referentie	Auteur	Document	Kenmerk	Versie en datum	Bron	Toelichting
[ref. 7]	Werkgroep Maatgevende Aardbevingsbelasting (WMA)	Rapportage werkgroep Maatgevende aardbevingsbelasting voor de industrie - naar een snelle, simpele, transparante en robuuste toets op de aardbevingsbestendigheid van de chemische industrie in Groningen	-	4 november 2016	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/rapporten/2016/november/04/rapportage-werkgroep-maatgevende-aardbevingsbelasting-voor-de-industrie/Rapportage+werkgroep+MA.pdf	Achtergronddocument van de Fase 2 LoC-methode.
[ref. 8]	Werkgroep Maatgevende Aardbevingsbelasting (WMA)	Aanvullende rapportage werkgroep Maatgevende aardbevingsbelasting voor de industrie - Aanvullende inzichten over de LoC-toets op de		28 juni 2017	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/publicaties/2017/6/28/aanvullend-advies-werkgroep-maatgevende-	Achtergronddocument van de Fase 2 LoC-methode.

Referentie	Auteur	Document	Kenmerk	Versie en datum	Bron	Toelichting
		aardbevingsbestendigheid van de chemische industrie in Groningen			aardbevingsbelasting/20170628+Aanvullend+advies+werkgroep+Maatgevende+aardbevingsbelasting+def.pdf	
[ref. 9]	Werkgroep Maatgevende Aardbevingsbelasting (WMA)	Notitie: beantwoording van nadere vragen over toepassing van de LoC-methode bij de industrie		december 2017	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/richtlijnen/2018/03/05/notitie-loc-methode-chemische-industrie/Notitie+beantwoording+vragen+WMA+toepassing+LoC-methode+december+2017.pdf	Achtergronddocument van de Fase 2 LoC-methode.
[ref. 10]	Werkgroep Maatgevende Aardbevingsbelasting (WMA)	Advies WMA over gevolgen afnemende aardgaswinning		7 april 2019	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/rapporten/2018/april/1/advies-wma-over-gevolgen-afnemende-aardgaswinning/Aanvullend+advies+WMA+april2018+gevolgen+gaswinningsbesluit.pdf	Update van WMA adviezen ([ref. 7] t/m [ref. 9]) voor 2019 en verder.
[ref. 11]	KNMI	Update 2018 Shakemaps for 'Maximum Considered Earthquake' Scenario in Groningen	-	12 december 2018	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/rapporten/2018/december/12/rapport-knmi-shakemaps-2018/Rapport+KNMI+shakemaps+2018.pdf	Informatief document wat de Shakemaps methode beschrijft en middels databestanden de respons spectra per bedrijfslocatie geeft die gebruikt worden bij de Fase 2 LoC toets methode. N.B. Gecontroleerd dient te worden bij aanvang van Fase 2 wat de meest recente geaccordeerde versie van het Shakemaps rapport is (zie voetnoot 1 op pagina 8). Op het moment van schrijven is de hier beschreven 2018 versie de meest actuele versie voor nieuwe onderzoeken.
[ref. 12]	Sweco	Mogelijke consequenties Shakemaps v5 op LoC-toetsen	Referentie-nummer: SWNL 0240418	25 maart 2019	-	Beschouwing van de 2018 versie van de seismische belastingen uit het KNMI Shakemaps.

De meest recente 2018 databestanden van het KNMI (respons spectra voor de LoC-methode) zijn gedeeld met de betrokken consultants of kunnen worden verkregen via industrie@nationaalcoordinatorgroningen.nl.

2.3.3 Technisch inhoudelijke documenten LoC-methode

De documenten van TU Delft (GBoD [ref. 13] en aanvullende notities [ref. 14] en [ref. 15], zie tabel 2.3) beschrijven inhoudelijk de berekeningsmethode van de LoC-toets. De GBoD is een algemeen uitgangspunten voor de berekeningen en is daarom leidend voor de berekeningen die de consultants uitvoeren in het kader van de LoC-methode. De GBoD beschrijft onder meer:
- de te hanteren aardbevingsbelasting (respons spectra) in combinatie met de LoC-methode, dit zijn de respons spectra volgend uit KNMI databestanden zoals beschreven in paragraaf 3.4.;
- definitie van de limit state waaraan wordt getoetst (uiterste grenstoestand volgens NEN-EN 1998-4);
- de te hanteren berekeningswijze van de seismische krachten, zijnde in principe de spectrale modale responsberekening (MRSA) of de zijdelingse belastingsmethode ('lateral force methode of analysis').

Tabel 2.3 Overzicht inhoudelijke documenten Fase 2 - LoC-methode

Referentie	Auteur	Document	Kenmerk	Versie en datum	Bron	Toelichting
[ref. 13]	TU Delft	Bijlage 4 van [ref. 7]: Generic Basis of Design (GBoD) for the structural verification of industrial facilities in Groningen: a first screening of the seismic capacity	CM-2016-19	Definitive d.d. 24 oktober 2016	zie bijlage 4 van [ref. 7]	Beschrijft de inhoudelijke toepassing van de Fase 2 LoC toets voor consultants. N.B. tevens opgenomen in bijlage I.
[ref. 14]	TU Delft	Explanatory notes for the 'LoC Toets' in application to the industrial facilities in Groningen	CM-2016-19-D1	1 februari 2017	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/publicaties/2017/februari/1/loc-toets-toelichting/022017+LoC+toets+TUDelft.pdf	Beschrijft de inhoudelijke toepassing van de Fase 2 LoC toets voor consultants. N.B. tevens opgenomen in bijlage II.
[ref. 15]	TU Delft	Lessons's Learned: 'LoC Toets' in application to the industrial facilities in Groningen	ES-DoS-2018-01	Draft d.d. 8 november 2018	-	Beschrijft de inhoudelijke toepassing van de Fase 2 LoC toets voor consultants plus een evaluatie van de ca. 40 gedane onderzoeken in de periode 2017-2018. N.B. tevens opgenomen in bijlage III.

2.4 Fase 2 documenten voor de toepassing van de Risico-gebaseerde rekenmethodiek (ontwikkeld door Deltares/TNO)

Voor de Risico-gebaseerde rekenmethodiek van Deltares/TNO is één handreiking beschikbaar die zowel het proces als de inhoudelijke aspecten behandeld. De referentie is in de navolgende tabel opgenomen.

Tabel 2.4 Overzicht document Fase 2 - Risico-gebaseerde rekenmethodiek

Referentie	Auteur	Document	Kenmerk	Versie en datum	Bron	Toelichting
[ref. 16]	Deltares/TNO	Handreiking Fase 2 - voor het uitvoeren van studies naar het effect van aardbevingen voor bedrijven in de industriegebieden in Groningen	1209036-000-GEO-0285-ga	Versie 7 d.d. juni 2018	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/richtlijnen/2018/06/08/rekenmethodiek-onderzoek-industrie-fase-2/Handreiking+Fase+2+rekenmethodiek+Deltares+TNO.pdf	Handreiking voor de Risico-gebaseerde rekenmethodiek voor Fase 2.

2.5 Blauwdrukken (generieke aanpakken)

Tabel 2.5 geeft een overzicht van de beschikbare Blauwdrukken.

Tabel 2.5 Overzicht Blauwdrukken en overige toelichtende documenten

Referentie	Auteur	Document	Kenmerk	Versie en datum	Bron	Toelichting
[ref. 17]	TU Delft	General procedure for checking the above-ground pipelines supported by a structural frame based on the provisions of EN1998-4:2007	-	-	-	Beknopte memo met beschrijving hoe leidingen en hun ondersteuningsconstructie te berekenen. N.B. tevens opgenomen in bijlage IV.
[ref. 18]	Witteveen+Bos	Generic approach liquid storage tanks - General approach liquid storage tanks for the seismic verification of industrial facilities in Groningen	103022/18-006.248	Final version 03 d.d. 28 april 2018	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/richtlijnen/2018/04/23/standaardmethode-opslagtanks/Standaardmethode++opslagtanks++-+Generic+approach+liquid+storage+tanks.pdf	Blauwdruk (generieke aanpak) voor het toetsen van stalen, cilindrische, verticale stalen opslagtanks. N.B. tevens opgenomen in bijlage V.
[ref. 19]	Royal HaskoningDHV	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	BC7415I&BRP 1809141342	0.3/Final d.d. 23 september 2019	https://www.nationaalcoordinatorgroningen.nl/binaries/nationaal-coordinator-groningen/documenten/richtlijnen/2020/01/28/standaardmethode-leidingen-en-leidingbruggen/Standdaardmethode+leidingen+op+leidingbruggen.pdf	Blauwdruk (generieke aanpak) voor het toetsen van leidingen op leidingbruggen. N.B. tevens opgenomen in bijlage VI.
[ref. 20]	Witteveen+Bos	Seismic verification of foundations of industrial assets in Groningen - Performance criteria and assessment methods	108064/21-002.668	Draft version 01 d.d. 17 februari 2021	n.t.b.	Blauwdruk (generieke aanpak) voor toetsen van fundaties. N.B. tevens opgenomen in bijlage VII.

Opgemerkt wordt dat de Blauwdrukken in tabel 2.5 primair zijn ontwikkeld en gereviewd om te gebruiken in combinatie met de LoC-methode, maar dat delen van de aanpak ook toepasbaar zijn bij de Risco-gebaseerde-rekenmethodiek. Het is aan de gebruiker om te beoordelen welke onderdelen van, en in welke mate, de Blauwdrukken kunnen worden gebruikt bij de Risico-gebaseerde rekenmethodiek. Zie ook het stroomschema in hoofdstuk 1 van de Handreiking deel 1 Procesbeschrijving [ref. 2].

2.6 Normen en richtlijnen

Tabel 2.6 geeft een beknopt en niet-limitatief overzicht van normen voor het beoordelen van constructies in Nederland, inclusief aardbevingsbestendigheid. Welke (intern)nationale normen er gehanteerd worden bij de LoC-methode en in welke hiërarchie is afhankelijk van het type installatie. Om deze reden wordt voor de specifieke toepassing van de normen naar de GBoD met aanvullingen van de LoC-methode (zie tabel 2.3). Hetzelfde geldt voor normen waar de Blauwdrukken (zie tabel 2.5) naar verwijzen.

Tabel 2.6 Beknopt overzicht normen en richtlijnen (niet-limitatief)

Referentie	Auteur	Document	Kenmerk	Versie en datum	Bron	Toelichting
[ref. 21]	NEN	NPR 9998:2020 - Beoordeling van de constructieve veiligheid van een gebouw bij nieuwbouw, verbouw en afkeuren - Geïnduceerde aardbevingen - Grondslagen, belastingen en weerstanden	NPR 9998:2020	december 2020	www.nen.nl	Praktijkrichtlijn voor het beoordelen van gebouwen op aardbevingsbestendigheid bij afwezigheid van een Nederlandse Nationale Bijlage bij EN 1998.
[ref. 22]	NEN	Eurocodes (NEN-EN 1990 t/m NEN-EN 1998)	-	-	www.nen.nl	Normen voor het constructief ontwerp van constructies in Nederland.
[ref. 23]	NEN	NEN 8700 serie (NEN 8700 t/m NEN 8707)	-	-	www.nen.nl	Normen voor het beoordelen van bestaande constructies in Nederland.

3

SCOPE, DOELSTELLING EN TOEPASSING VAN DE LOC-METHODE

3.1 Toelichting

De in dit hoofdstuk geschreven tekst is een beknopte samenvatting van de LoC-methode zoals ontwikkeld door WMA ([ref. 7] tot en met [ref. 10]).

3.2 Achtergrond, karakteristeken en beperkingen LoC-methode

De LoC-methode is ontwikkeld door de WMA als een eenvoudige en transparante toets op aardbevingsbestendigheid van industriële installaties in Groningen. Deze methode is ontwikkeld als sneller alternatief (opgeleverd in november 2016) voor de Risico-gebaseerde rekenmethodiek voor Fase 2 van Deltares/TNO (definitief opgeleverd in juni 2018). Kernpunten van de LoC-methode zijn:
- een maximaal denkbaar geïnduceerd aardbevingsscenario in het kerngebied van magnitude 5 op de schaal van Richter (M_{max}=5);
- er wordt proportioneel getoetst door bij de maximale aardbeving verwachte (meest waarschijnlijke) waarde van het versnellingsspectrum te gebruiken als input voor een simpele toets op aardbevingsbestendigheid in de uiterste grenstoestand (UGT) bij aardbevingsontwerpsituaties volgens de Eurocodes.

De technische uitwerking en te volgen aanpak van de methode staat beschreven in de GBoD met aanvullingen ([ref. 13] tot en met [ref. 15]), opgesteld door TU Delft. De te hanteren aardbevingsbelasting volgt uit de respons spectra behorende bij de door de Stuurgroep Industrie laatst geaccordeerde versie van het KNMI Shakemaps rapport. De meest recente spectra te gebruiken binnen de LoC-methode zijn uit 2018. (zie sectie 2.3.2).

De karakteristieken van de LoC-methode zijn:
- een eenvoudige, uniforme, lineair-elastische toets met een vaste gedragsfactor (q-factor) van 1,5: een ondergrenswaarde voor de oversterkte/ductiliteit van willekeurige constructies en hun detailleringen volgens NEN-EN 1998. Elke bestaande of nieuwe constructie kan daarom worden getoetst met de LoC-methode;
- de methode gebruikt een relatief lage q-factor van 1,5 wat leidt tot conservatieve resultaten voor constructies die over voldoende en aantoonbare herverdelingscapaciteit bezitten. De LoC-toets geeft voornamelijk een binair antwoord in de vorm van ‘voldoet’ of ‘voldoet niet’ op het gebied van aardbevingsbestendigheid (zie ook paragraaf 3.1.4 van [ref. 15]). Het toepassen van de LoC toets bij nieuwbouw kan leiden tot conflicten met de ontwerpprincipes van Eurocode waar de q-factor afhankelijk is van ductiliteitsklassen en waar detailleringseisen worden gesteld afhankelijk van de ductiliteitsklassen (laag, medium of hoog). In paragraaf hoofdstuk 3 van de Handreiking deel 1 procesbeschrijving wordt hier nader op ingegaan;
- er wordt gewerkt met één (locatieafhankelijk) semi-deterministisch bepaalde aardbevingsbelasting (respons spectrum), gebaseerd op het eerder genoemde scenario (M_{max}=5). Differentiatie op importantiefactor of herhalingstijd is niet mogelijk zoals gebruikelijk in onder meer Eurocode 8. Met andere woorden: alle te toetsen installaties worden aan dezelfde belasting onderworpen, ongeacht het type of de hoeveelheid gevaarlijke stof en aanverwante zaken als effectafstanden. De WMA heeft ervoor

gekozen om niet het gebruikelijke proportionaliteitsbeginsel aan te houden, maar in plaats daarvan uit te gaan van een voorzichtige interpretatie van het voorzorgsbeginsel;
- de toets is puur een constructieve verificatie van de UGT van de kritische **elementen** van een installatie / constructie en neemt niet mee het daadwerkelijk vrijkomen van (de hoeveelheid) gevaarlijke stoffen (LoC) door het voortschrijdend falen van het gehele **systeem** of (reeds aanwezige) beheersmaatregelen zoals LoD's;
- de methode geeft ook inzicht op het gebied van arbeidsveiligheid, maar dan alleen als toets op de scenario's waar constructief falen aan ten grondslag ligt (bijvoorbeeld het omvallen van een stelling of het scheuren van een leiding).

Te concluderen is dat de LoC-methode daarom primair een screeningmethode is voor het risico op onvoldoende aardbevingsbestendigheid en minder geschikt is als ontwerpmethode voor een geoptimaliseerd ontwerp. Opgemerkt wordt dat de LoC-methode met bijbehorende seismische input wel kan worden toegepast voor nieuwbouw, waarbij ook het gebruik van eventueel hogere q-factoren mogelijk is. Het toepassen van hogere q-factoren dient afdoende te zijn onderbouwd. Tevens is het aan te bevelen om bij onzekerheden dit uitgangspunt te laten reviewen door een onafhankelijke partij (bijvoorbeeld TU Delft). Specifieke aspecten van (het toepassen van de LoC-methode) bij nieuwbouw zijn gegeven in hoofdstuk 3 van de Handreiking deel 1 procesbeschrijving [ref. 2].

De alternatieve Fase 2 rekenmethode, de semi-probabilistische Risico-gebaseerde rekenmethodiek voor Fase 2 van Deltares/TNO [ref. 16], biedt wel de mogelijkheid om te differentiëren op gevolgklasse, bestaande bouw en nieuwbouw, q-factoren en gebruikt de respons spectra uit de NEN NPR 9998 webtool.

3.3 Definitie van uiterste grenstoestand en falen

Binnen de LoC-methode wordt de constructieve veiligheid van een installatie getoetst aan de uiterste grenstoestand (UGT) bij aardbevingsontwerpsituaties volgens de Eurocodes. De definitie van de UGT is als volgt [ref. 13].

Definitie uiterste grenstoestand NEN-EN 1998-4, artikel 2.1.1
(2)P For particular elements of the network, as well as for independent structures whose complete collapse would entail severe consequences, the ultimate limit state is defined as that of a state prior to structural collapse that, although possibly severe, would exclude brittle failures and would allow for a controlled release of the contents. When the failure of the aforementioned elements does not entail severe consequences, the ultimate limit state may be defined as corresponding to total structural collapse.

Niet-lineaire effecten worden niet expliciet in de analyse beschouwd maar zijn verdisconteerd in een gedragsfactor (of q-factor). Dit is een reductiefactor op de 5 % gedempte elastische spectra om tot de ontwerpspectra te komen. Tussen 0,01 s en de piekperiode kan de reductiefactor lineair worden geïnterpoleerd tussen q = 1.0 en q = 1,5. Voor de LoC-methode wordt q = 1,5 gehanteerd voor de horizontale en verticale spectra.

3.4 Respons spectra

3.4.1 Algemeen

De te hanteren respons spectra volgen uit de databestanden behorende bij het Shakemaps rapport [ref. 12] en volgen direct uit Shakemaps-berekeningen met het GMM voor de door WMA vaststelde maatgevende scenario's. In de databestanden zijn de respons spectra per locatie (in RD-coördinaten) bepaald. Indien de locatie van het betreffende bedrijf niet direct staat aangegeven, dient de locatie te worden geselecteerd welke het dichtst in de buurt ligt van de werkelijke locatie van het bedrijf. De Shakemaps en afgeleide respons spectra worden indien noodzakelijk herzien. Door de Stuurgroep Industrie zal worden vastgesteld of deze nieuwe versie gebruikt dient te worden bij nieuwe onderzoeken waarbij de LoC-methode wordt gehanteerd. Voor lopende onderzoeken (dit zijn onderzoeken waarbij het BoD reeds is geaccordeerd door TU Delft) hoeft het respons spectrum niet aangepast te worden naar de meest recente versie, tenzij de maximale grondversnellingen voor de betreffende locatie in absolute zin meer dan 10 % zijn gestegen. De meest recente versie van het Shakemaps rapport en de bijbehorende databestanden is uit 2018 (zie tabel 2.2).

De 2018 databestanden zijn gedeeld met de betrokken consultants of kunnen worden verkregen via industrie@nationaalcoordinatorgroningen.nl.

Opgemerkt wordt dat deze respons spectra fundamenteel anders zijn dan de response spectra van de NEN NPR 9998 webtool (te gebruiken voor Fase 1 en de Risico-gebaseerde rekenmethodiek voor Fase 2 van Deltares/TNO), zie ook het stroomschema in hoofstuk 1 van de Handreiking deel 1 procesbeschrijving [ref. 2]. De spectra behorende bij de LoC-methode representeren de belasting bij een deterministisch gekozen maatgevende aardbeving. De NEN NPR 9998 spectra zijn probabilistische locatie-specifieke respons spectra gekoppeld aan jaarlijkse overschrijdingskansen, in lijn met internationale richtlijnen.

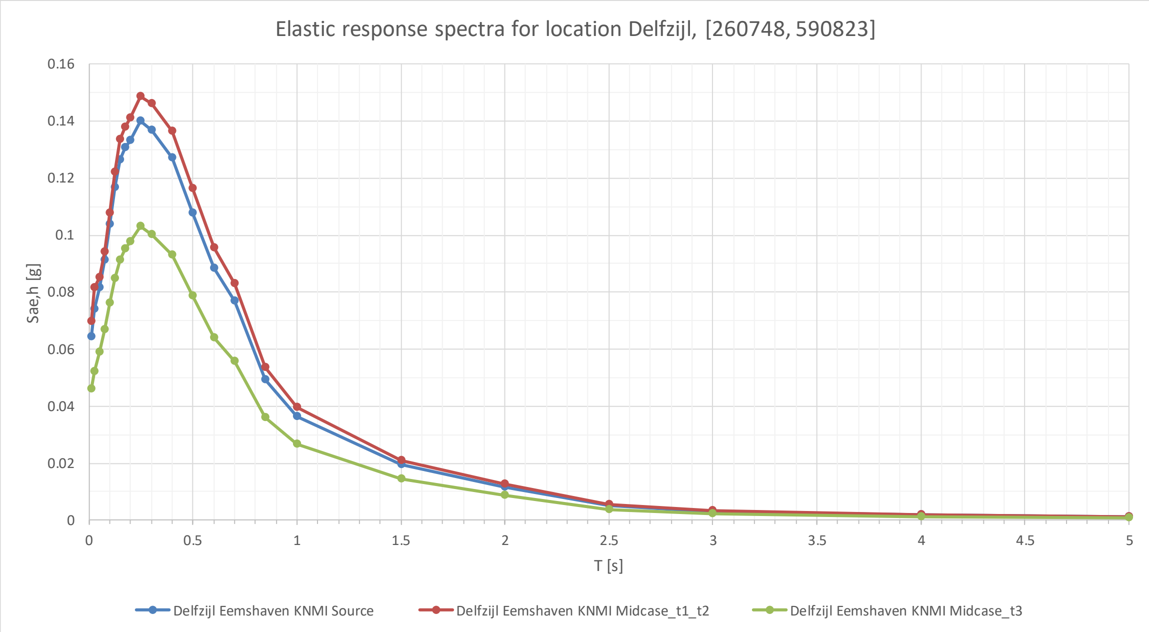
3.4.2 Toepassing 2018 spectra

De elastische spectra voor de LoC-methode zijn te vinden in de databestanden in de mappen 'ShakeMaps_v5_spectra_singleMax_scenario_...', deze komen overeen met het M_{max}=5 scenario ten grondslag ligt van de LoC-methode. De spectra zijn gegeven voor 22 industriële locaties ('...industry' datasets) en 45 site zones ('...sitesZonation' datasets). Hierbij zijn verschillende bronmodelocaties per industriële zones gedefinieerd:
- voor de industriële zone Delfzijl - Eemshaven gelden 3 bronmodelocaties:
  - KNMI source;
  - Midcase-t1-t2 (gecombineerd);
  - Midcase-t3;
- voor de industriële zone Hoogezand - Veendam is 1 gecombineerde bronmodellocatie gedefinieerd:
  - KNMI source + midcase-t1-t2-t3 (gecombineerd);
- voor de industriële zone Winschoten is ook 1 gecombineerde bronmodellocatie gedefinieerd:
  - KNMI source + midcase-t1-t2-t3 (gecombineerd).

Voor meer achtergrondinformatie wordt verwezen naar de beschouwing door Sweco [ref. 12].

Voor de scenario's / bronmodellocatie geldt dat het meest ongunstige gehanteerd dient te worden. In afbeelding 3.1 is een voorbeeld gegeven voor een willekeurige locatie in Delfzijl, waarbij geldt dat (voor deze locatie) het midcase-t1-t2 scenario maatgevend is.

Afbeelding 3.1 Voorbeeld horizontaal elastisch respons spectrum uit de 2018 datasets



Het verticale elastische respons spectrum kan worden berekend op basis van de V/H ratio geldend voor Groningen uit bijvoorbeeld afbeelding 3.2:

Afbeelding 3.2 V/H ratio geïnduceerde aardbevingen Groningen[2]

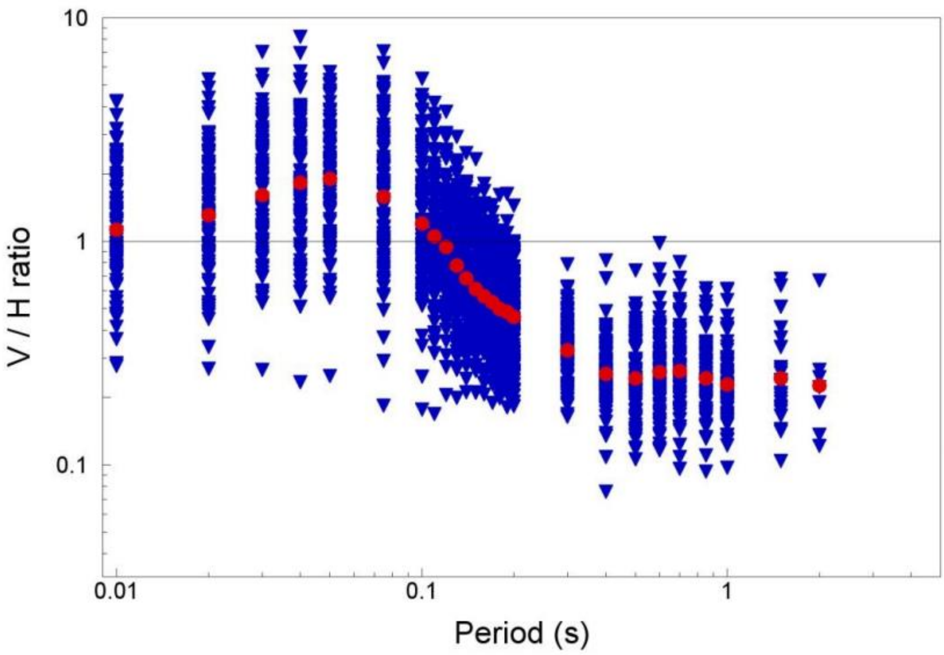


Figure 11.140. V/H rations of spectral accelerations from the Groningen recordings; the red dots show the average ratio at each oscillator period

[2] Ontleend aan pagina 345 van Bommer, J.J. et al (2015), Development of Version 2 GMPEs for Response Spectral Accelerations and Significant Durations from Induced Earthquakes in the Groningen Field, Version 2, 29 October 2015.

3.4.3 Wijzigingen in seismische dreiging

Zoals toegelicht in sectie 3.4.1 worden de Shakemaps en afgeleide respons spectra (indien noodzakelijk) periodiek herzien in opdracht van de NCG. De Stuurgroep Industrie stelt vast wanneer deze worden vrijgegeven voor het gebruik in combinatie met de LoC-methode. De bedrijven en consultant kunnen vervolgens beoordelen wat de wijzigingen in seismische dreiging zijn ten opzichte van eerdere jaren. Voor meer informatie hoe in het algemeen om te gaan met wijzigingen in de seismische dreiging wordt verwezen naar hoofdstuk 4 van de Handreiking deel 1 procesbeschrijving.

3.5 Resultaten LoC-methode in Fase 2b

Uit de Fase 2b LoC-toets volgt één van de twee volgende resultaten:
- ‘voldoet’, of;
- ‘voldoet niet’.

In het MJP [ref. 5] en diens addendum [ref. 6] is de volgende duiding gedefinieerd voor installaties die voldoen volgens de LoC-methode.

Duiding resultaten LoC-methode: installatie voldoet (conform paragraaf 4.2 van [ref. 6])

De in het MJP beschreven aanpak onder artikel 1c wordt nu zo opgevat: Als uit de LoC-toets blijkt dat naar verwachting geen gevaarlijke stoffen vrijkomen in het maatgevende aardbevingsscenario, dan kan worden geconcludeerd dat er bij de meest waarschijnlijke maximale grondversnellingen geen achteruitgang zal ontstaan in de omgevingsveiligheid en ook geen hogere blootstelling voor werknemers zal optreden als gevolg van het vrijkomen van gevaarlijke stoffen. Dan is er dus geen aanleiding voor (versterkings)maatregelen.

Voor installaties die niet voldoen wordt de volgende duiding gegeven:

Duiding resultaten LoC-methode: installatie voldoet niet (conform paragraaf 4.2 van [ref. 6])

Zoals bepaald in het MJP (art. 2c): als uit de LoC-toets blijkt dat bij bepaalde installaties kan worden verwacht dat in het maatgevende aardbevingsscenario gevaarlijke stoffen vrijkomen, vinden nadere berekeningen plaats, zodat deze uitslag snel en eenduidig geverifieerd kan worden. Vervolgens wordt bepaald welke versterkingsmaatregelen en/of aanpassingen in het bedrijfsproces nodig zijn om te voldoen aan een versterkingsniveau waarin geen LoC verwacht kan worden.

Wanneer een constructie niet voldoet aan de LoC-toets, is nog niet aangetoond dat er daadwerkelijk LoC optreedt. Aangetoond is dat er een verhoogd risico is op het constructief falen van één of meerdere onderdelen die een ongewenst vervolgscenario van LoC mogelijk maken. De installaties die worden beoordeeld met de LoC-toets zijn immers al geprioriteerd op basis van de Fase 1 onderzoeken. De benodigde maatregelen worden zo snel mogelijk geïnventariseerd in een aanvullende Fase 2c om een adequaat en tijdig actie te kunnen ondernemen. In hoofdstuk 5 van de Handreiking deel 1 procesbeschrijving [ref. 2] wordt nader ingegaan op Fase 2c.

4

LESSONS LEARNED LOC-METHODE

4.1 Toelichting

Als tussentijdse evaluatie van de LoC-methode heeft TU Delft (als externe reviewer) al een apart document opgesteld met leerpunten en veelvoorkomend reviewcommentaar, zie [ref. 15]. Deze zijn samengevat in paragraaf 4.2. Voor een uitgebreide omschrijving wordt verwezen naar bijlage III van deze Handreiking deel 2 LoC-methode waarin [ref. 15] is opgenomen.

In aanvulling daarop zijn in paragraaf 4.3 en verder diverse tips en concrete leerpunten opgenomen gebaseerd op de ervaring die de afgelopen jaren zijn opgedaan, op zowel inhoudelijk als procedureel vlak.

4.2 Belangrijkste leerpunten TU Delft

De leerpunten worden hieronder beknopt genoemd, samen met het paragraafnummer overeenkomstig [ref. 15].

- algemeen (3.1):
  - *3.1.1: Afknotten modale analyse*. Voor het bepalen voor het minimaal benodigde aantal trillingsvormen (modes) is het aan te bevelen niet alleen naar het percentage geactiveerde massa te kijken (artikel 4.3.3.3.1(3) van NEN-EN 1998-1), maar ook aanvullend te kijken of bij een toenemend aantal modes het resultaat niet meer significant wijzigt;
  - *3.1.2: Niet-constructieve onderdelen op hogere verdiepingen*. Deze onderdelen kunnen worden onderschat wanneer men alleen het 5 % gedempte respons spectrum hanteert. De volgende methoden kunnen worden gebruikt om dit te controleren:
    1. analytisch op basis van vergelijkingen 4.24 en 4.25 van NEN-EN 1998-1;
    2. numeriek door middel van een integraal EEM model met daarin zowel de constructieve (hoofddraagconstructie) als de niet-constructieve onderdelen;
  - *3.1.3: Aanpendelende gebouwen*. De te onderzoeken installatie of het te onderzoeken gebouw is constructief verbonden met een ander gebouw. Dit kan het dynamische gedrag van de te onderzoeken installatie of gebouw significant beïnvloeden en dit aspect dient daarom gemotiveerd te worden beschouwd in het BoD en berekeningsrapport (bijvoorbeeld met dynamische randvoorwaarden die het aanpendelende gebouw representeren);
  - *3.1.4: Gedragsfactor (q-factor) en herverdeling*. Het herverdelen van krachten wanneer een onderdeel faalt is niet verenigbaar binnen het toetskader van de LoC-methode omdat dit al ondervangen is in de gedragsfactor q = 1.5. Zie ook paragrafen 3.2 en 3.5 van deze Handreiking over dit ontwerp;
- gebouwen (3.2):
  - *3.2.1: Metselwerk wanden belast uit het vlak*. Zonder uitzondering voldoen alle getoetste metselwerk wanden niet wanneer deze met de LoC-methode worden berekend. Een simpele aanvullende toets, de niet-lineaire kinematische analyse berekeningsmethode (NLKA) kan gebruikt worden wanneer niet wordt voldaan met een lineair elastische berekening volgens de LoC-methode. Zie aanvullend ook paragraaf 4.3 van deze Handreiking over het toepassen van de LoC-methode voor metselwerk (controle)gebouwen;
  - *3.2.2 Relatieve verplaatsing tussen bouwlagen (inter-storey drift)*. De juiste methode om de relatieve verplaatsing van bouwlagen te berekenen is door de verplaatsing per vloer en per trillingsvorm te

combineren met een combinatieregel zoals de SRSS of de CQC. Het is incorrect om simpelweg de relatieve verplaatsing te berekenen uit de uiteindelijke berekende verplaatsingen per vloer;
- opslagtanks (3.3):
  - *3.3.1: Generieke aanpak (Blauwdruk)*. Voor opslagtanks dient de Blauwdruk 'Generieke aanpak opslagtanks' ([ref. 18], tevens opgenomen als bijlage V) te worden gevolgd als primaire richtlijn. Dit document is in lijn met de GBoD van de LoC-methode;
  - *3.3.2: 0,5 % demping van de convectieve massa*. Vergelijking 3.6 van NEN-EN 1998-1 dient te worden gevolgd om tot het 0,5 % (in plaats van 5 %) gedempte spectrum te komen voor het beoordelen van sloshing (klotsen van de vloeistof). Dit resulteert in een vergrotingsfactor van het 5 % gedempte spectrum van 1,36 voor de voor sloshing relevante perioden (eigenperioden groter dan de piekperiode van het respons spectrum);
  - *3.3.3: Combinatieregel*. Voor het combineren van de impulsieve en convectieve krachten (bijvoorbeeld het kantelmoment) dient de absolute sommatieregel worden gebruikt. Zie ook paragraaf 2.1.3 van bijlage V [ref. 18];
  - *3.3.4: Verificatie van de tankschalen en aansluitingen van leidingen*. Aandachtspunten met betrekking tot het toetsen van plooi van de verschillende (verjongde) schalen van een gesegmenteerde tank en het toetsen van aansluiting van leidingwerk (nozzles) zijn gegeven in het evaluatiedocument van TU Delft [ref. 15] en in bijlage V [ref. 18];
- stellingen (3.4):
  - *3.4.1: Massaverdeling*. Verschillende opslagconfiguraties in verticale en horizontale richtingen dienen te worden onderzocht om de maatgevende situatie te vinden (bijvoorbeeld asymmetrische configuraties omdat dit tot torsie-trillingsvormen kan leiden);
  - *3.4.2: Falen van containment*. Individuele objecten die uit de stelling kunnen vallen dienen ook getoetst te worden door bijvoorbeeld een realistische coëfficiënt van wrijving aan te nemen. Zie ook sectie 3.1.2 van [ref. 15];
  - *3.4.3: Pounding*. Er dient altijd beschouwd te worden of de stelling een naastgelegen gebouwonderdeel kan raken (pounding). Wanneer pounding optreedt, dient dit als LoC-scenario te worden beschouwd omdat het werkelijke gedrag te complex is om te berekenen. Verplaatsingen dienen te worden berekend op basis van het elastische spectrum (q = 1,0);
- leidingen (3.5):
  Aandachtspunten met betrekking tot het toetsen van leidingen op leidingbruggen (in aanvulling op de GBoD ([ref. 13], [ref. 14]) zijn opgenomen in:
  - de Blauwdruk (Generieke aanpak) over leidingen op leidingbruggen [ref. 19];
  - *'General procedure for checking the above-ground pipes supported by a structural frame based on the provisions of EN1998-4:2007'* [ref. 17];
  - paragraaf 3.5 van het evaluatiedocument van TU Delft [ref. 15].

4.3 Metselwerk (controle)gebouwen

Bij de Fase 1 onderzoeken zijn verschillende controlegebouwen geprioriteerd voor Fase 2 onderzoeken op basis van:
- arbeidsveiligheid van operators aanwezig in en/of nabij het gebouw;
- potentiële risico voor controle-, proces- en safeguardingsystemen gelokaliseerd in de controlegebouwen.

In veel gevallen gaat het hier om gebouwen met één of twee verdiepingen met wanden van ongewapend metselwerk. Uit de gedane LoC toetsen volgt zonder uitzondering dat deze gebouwen niet de (lineair-elastische) LoC toets doorstaan [ref. 15] bij de toetsing uit het vlak. Hierdoor ontstaat het risico dat het omvallen van de wand zorgt voor letsel aan personen en schade aan kritische apparatuur.

Mede ten grondslag hieraan ligt dat de LoC-toets is ontwikkeld voor constructies primair gemaakt van staal en/of gewapend beton. De effectiviteit van de toepassing van de LoC-toets op metselwerk (controle)gebouwen is daarom twijfelachtig. Derhalve wordt de volgende aanpak geadviseerd:

- bij gebouwen met primair een verblijfsfunctie voor personen: berekening voor bestaande/ver-/nieuwbouw volgens NPR 9998:2018 [ref. 21] in plaats van de LoC-methode;
- in overige gevallen: 1 van de volgende 2 opties:
  - berekening volgens de LoC-methode (bepaling van seismische belastingen, etc.) waarbij de sterktetoets uitgevoerd dient te worden als volgt:
    1. lineair-elastische sterkte toets volgens hoofdstuk 6 van NEN-EN 1996-1-1. Wanneer wordt voldaan zijn geen aanvullende stappen noodzakelijk;
    2. indien niet wordt voldaan bij stap 1: de niet-lineaire kinematische berekening (NKLA) volgens bijlage H van de NPR 9998:2018 dient te worden beschouwd. Voor meer informatie wordt naar [ref. 15] verwezen;
  - berekening uitvoeren volgens de NPR 9998:2018.

In de gevallen dat de NPR 9998 wordt gehanteerd kan er in hoofdstuk 2 van de NPR 9998 qua belasting onderscheid worden gemaakt naar de situatie die van toepassing is, op basis van onder meer de volgende aspecten:
- grenstoestand (NC, SD of DL), in de meeste gevallen zal NC van toepassing zijn;
- gevolgklasse, voor controlegebouwen zal in de meeste gevallen CC2 of CC1b van toepassing zijn;
- relevantie van constructieonderdeel:
  - primaire en secundaire seismische elementen (hoofdraagconstructie van het gebouw);
  - niet-seismische, constructieve elementen; dit zijn bijvoorbeeld de metselwerkwanden wanneer de hoofddraagconstructie uit stalen kolommen en liggers bestaat;
- bestaande bouw, verbouw of nieuwbouw.

4.4 Wijze van modellering fundering (grond-constructie interactie, SSI)

De stijfheid van de fundering en ondergrond kan het dynamische gedrag van de te toetsen installatie significant beïnvloeden, door het verlengen van de eigenperioden en toename in demping. Het laatste wordt binnen de LoC-methode niet beschouwd, maar periodeverlenging wel. In het BoD dient beschouwd te worden hoe grondconstructie interactie (SSI) wordt meegenomen in de analyse door:
- gemotiveerd te beschrijven of het voor de betreffende installatie kan worden verwaarloosd;
- het toepassen van flexibele randvoorwaarden (verende in plaats van vaste knoopopleggingen) in het EEM-model die de dynamische stijfheid van de fundering en ondergrond modelleren;
- het mee-modelleren van de gehele (paal)fundering in het EEM-model.

Het is aan te bevelen een gevoeligheidsanalyse te doen naar de invloed van de gekozen elastische grondparameters op de eigenperioden. Voor meer informatie over SSI wordt onder meer naar paragraaf 4.2 van [ref. 14] en paragraaf 2.1.2 van [ref. 18] verwezen.

Bijlage(n)

BIJLAGE: GENERIC BASIS OF DESIGN (GBOD) FOR THE STRUCTURAL VERIFICATION OF INDUSTRIAL FACILITIES IN GRONINGEN: A FIRST SCREENING OF THE SEISMIC CAPACITY [REF. 13]

Bijlage 4. Generic Basis of Design (GBoD) for the structural verification of industrial facilities in Groningen: a *first screening* of the seismic capacity

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1. Introduction

This Generic Basis of Design (GBoD) document provides an overview of the methodology to be followed for a *first screening* of the seismic capacity of the industrial facilities in Groningen. It gives a brief description of the seismic input (as provided in the report by KNMI) and the methods to be used for the verification of the structural capacity based on the availability of structural models. This is not a stand-alone document; the technical note at the end of this document as well as the KNMI report prepared under the auspices of the workgroup form an integral part of it.

The technical note composed by TU Delft includes a description of the method to be followed for the verification of the seismic capacity of the industrial facilities. In addition, for the seismic input the reference document is the report by KNMI which additionally includes a set of files with the ordinates of the 5 percent damped acceleration response spectra to be applied in the design.

2. Definition of the seismic action

The basic representation of the seismic action is provided in terms of ground acceleration response spectra in the two relevant directions, i.e. horizontal and vertical. The spectra are derived on the basis of the maximum considered earthquake to be expected as described in the KNMI report. Two sets of spectra are provided at each location:

i) The first set corresponds to the expected values of the ordinates of the 5 percent damped acceleration response spectrum obtained at the surface of ground and will be referred to hereafter as the "*acceleration response spectrum"* (defined either in the horizontal or the vertical direction).

ii) The second set corresponds to the increased - by a standard deviation - ordinates of the 5 percent damped acceleration response spectrum and will be referred to hereafter as the *"extreme*[1] *acceleration response spectrum"* (either in the horizontal or the vertical direction). The formula to be used to obtain the increased ordinates of the 5 percent damped acceleration response spectrum is given in the KNMI report.

2.1 Horizontal acceleration response spectrum

The representation of the seismic action in the horizontal direction is a 5 percent damped horizontal acceleration response spectrum at the surface of the ground derived at each location of interest as explained in detail in the KNMI report. The exact values of the spectral ordinates are explicitly provided at four structural periods namely, T=0.01s (effectively treated hereafter as the peak ground acceleration, PGA); 0.3s; 1.0s; 3s. The exact values of the ordinates of the spectrum are given in the KNMI report (files containing the spectral ordinates at each specific location). A continuous (as a function of the structural period T of the system) response spectrum can be derived on the basis of suitable interpolation of these four spectral ordinates to a characteristic acceleration response spectrum in the region. The resulting graphs per location are reproduced in the KNMI report.

The 5 percent damped horizontal acceleration response spectrum constitutes the representation of the seismic action in the horizontal direction with no further modification, i.e. use of importance factors.

[1] The word "extreme" here is chosen for convenience and it should not be related to any other definition of extreme events used in the literature.

2.2 Vertical acceleration response spectrum

The ordinates of the 5 percent damped vertical acceleration response spectrum are derived directly from the ones of the 5 percent damped horizontal acceleration response spectrum by multiplying the latter with a constant -per structural period T of the oscillator- factor. The multiplier can be obtained from the average values shown in the graph on p.345 in *Bommer et al.*[2]. The resulting 5 percent damped vertical acceleration response spectrum defines the seismic action in the vertical direction with no further modification, i.e. use of importance factors.

2.3 Displacement response spectrum

The displacement spectrum in the relevant direction, i.e. either horizontal or vertical, can be derived by the following formula:

$$S_d(T,\xi=5\%)=S_a(T,\xi=5\%)\times\left(\frac{T}{2\pi}\right)^2$$

In the formula above, $S_d(T,\xi=5\%)$ denotes the ordinate of the 5 percent damped displacement response spectrum, $S_a(T,\xi=5\%)$ is the ordinate of the 5 percent damped acceleration response spectrum, and T is the structural period of interest.

3. Calculations and structural verifications

The analysis of the structural integrity will be based on a two-step approach as described below.

Step 1:
An analysis will be carried out using the simplified method (effectively based on hand-calculations, i.e. a lateral force method as specified in the Eurocode 8) in which a detailed FE model of the structure is not required and the verification should be carried out in accordance with the ULS as defined in the relevant parts of the Eurocodes (see also the TUD technical note at the end of this document). An outline of a suitable model of this group is Model III of a liquid storage tank structure as described in the TUD technical note. Once the loads are defined and the resulting stresses and member forces are determined, the safety checks (failure modes of the structure and of the soil-foundation system) will be carried out in accordance with the relevant parts of the Eurocodes. Specific procedures for checking the capacity of the foundations and the liquefaction potential of the soil are included in the NPR 9998:2015 in accordance with the relevant parts of the Eurocodes[3]. Similar procedures should be followed for other type of structures[4] (other than liquid storage tanks).

[2] Bommer, J.J., Dost, B., Edwards, B., Kruiver, P.P., Meijers, P., Ntinalexix, M., Polidoro, B., Rodrigues-Marek, A., and Stafford, P.J. Development of Version 2 GMPEs for Response Spectral Accelerations and Significant Durations from Induced Earthquakes in the Groningen Field, Version 2, 29 October 2015.

[3] In the absence of a calibrated, region specific methodology for induced earthquakes, it is recommended to apply the Boulanger and Idriss (2014) method [Boulanger R. W. and Idriss I. M. (2014), 'CPT and SPT Based Liquefaction Triggering Procedures', UCD/CGM-14/01], using the Peak Ground Acceleration (PGA) values provided by KNMI at the surface of the soil profile as input (effectively the ordinates of 5 percent damped response acceleration spectrum at T=0.01s as defined in Appendices A and C). It is noted that applying the Boulanger and Idriss (2014) method without region-specific modifications will, most likely, result in a conservative assessment.

Step 2:
An analysis will be carried out using the modal response spectrum method of analysis with the inclusion of higher-order modes and the explicit consideration of a FE model of the structure for the determination of the stresses and strains (equivalent to either Model I or Model II of the TUD technical note). The verification should be carried out in accordance with the ULS as defined in the relevant parts of the Eurocodes. For example, such a model should provide an estimation of the spatial distribution of the internal pressures at the inner side of the liquid storage tank, and subsequently the stress resultants and member forces can be determined. Consequently, the safety checks (failure modes of the structure and of the soil-foundation system) need to be carried out in accordance with the relevant parts of the Eurocodes. Similar procedures should be followed for other type of structures (other than liquid storage tanks).

The analysis will be carried out separately for seismic input corresponding to two cases:

I. The expected values of the ordinates of the 5 percent damped acceleration response spectra, i.e. the *"acceleration response spectra"* as defined in section 2 above;
II. The increased (by a standard deviation) ordinates of the 5 percent damped acceleration response spectra as obtained by the use of the equation provided in the KNMI report, i.e. the *"extreme acceleration response spectra"* as defined in section 2 above.

Thus, for each structure, the steps above will lead to four calculations; two different methods [5] for the determination of the resulting actions (steps 1 and 2 above) and two sets of seismic input (*normal* and *extreme* ordinates of the 5 percent damped elastic response spectra).

4. *First screening* of the seismic capacity

The completion of the analysis proposed in section 3 (by performing the Modal Response Spectrum Method of Analysis or by performing the simple hand calculations when applicable) can result in one of the following three outcomes:

- **Group 1**: A structure satisfies the ULS-norm for both sets of seismic input, i.e. seismic set (i) and seismic set (ii).
- **Group 2**: A structure satisfies the ULS-norm for seismic input set (i) but not for seismic input set (ii);
- **Group 3**: A structure does not satisfy the ULS-norm for any of the two sets of ground input motions, i.e. neither seismic set (i) nor seismic set (ii).

Structures falling into the different groups are characterized by varying seismic capacity and each structure shall fall into only one of the three groups mentioned above. Structures in Group 1 have higher seismic capacity than the ones in Group 2, and structures in Group 2 have higher seismic capacity than the ones in Group 3. For the structures that fall into Group 3 strengthening measures should be proposed such that they are upgraded to one group higher.

[4] Once the verification of the structural integrity is completed, a verification of the individual components of the industrial facility (pipelines, connections, storage facilities, process equipment, etc.) needs to carried out in accordance with the existing international (and/or national) codes and practices.
[5] The application of the GBoD for the investigation of the seismic capacity in the four pilot case studies has provided insight into the cases in which a hand calculation was sufficient for the determination of the resulting seismic action in the structures. In a follow-up analysis it is thus not necessary to perform both steps 1 and 2 as described above. When the structures do not satisfy the structural regularity criteria in plan and/or elevation (section 4.2.3 of NEN-EN 1998-1), one can proceed directly with step 2 to verify the seismic capacity of the system by developing a suitable FE model and applying seismic actions set (i) and set (ii) as specified above.

5. Conclusions of the four pilot case studies

The application of the analysis proposed in this document to the four pilot case studies in the region of Groningen provided insight into a number of elements that need to be considered additionally to the ones mentioned under sections 1 to 4 above. These items are discussed further in this section and should be considered in the future application of this document.

5.1 Lateral force method versus Modal Response Spectrum method of analysis (MRA)

Structures within the pilot study were analysed by two methods:

i) hand calculations, which are effectively based on the lateral force method as described in section 4.3.3.2 of NEN-EN 1998-1 and the relevant parts of NEN-EN 1998-4 and
ii) Modal Response Spectrum method of Analysis (MRA) as specified in 4.3.3.3 of NEN-EN 1998-1 and the relevant parts of NEN-EN 1998-4.

The choice between these two methods is based essentially on the structural regularity conditions in plan and elevation as specified in section 4.2.3 of NEN-EN 1998-1. In addition, the lateral force method should essentially yield a conservative estimation of the resulting stresses and deformations in the structures which implies that higher order modes do not contribute to the structural response (or their contribution is negligibly small). For the structures analysed in this pilot investigation either the conditions of structural regularity as specified in NEN-EN 1998-1 were not met or the correct distribution of stresses and strains was not captured with the application of the lateral force method. Thus, for future studies, and in order to be able to capture the normative failure mechanisms in similar structural systems, a MRA is the minimum requirement (equivalent to either Model I or Model II of the TUD technical note). The lateral force method (effectively based on hand calculations) can only be applied to get a first rough estimation of the seismic capacity but not for the final determination of the seismic capacity in order to group the structures as mentioned in section 4 above.

5.2 FE models including fluid-structure interaction

When the analysis of the seismic capacity requires the modelling of coupled systems which contain structure-fluid interaction, i.e. FE models of liquid storage tanks, a careful choice of the software able to perform such an analysis is required. Results of this pilot study has revealed that regular FE software used for steel structures is not always suitable for modal analysis of liquid storage tanks when one considers models of the Type II of the TUD technical note in which the fluid is substituted by an added mass at the inner side of the wall of the steel tank. More elaborated software may be capable of dealing with models of Type I of the TUD technical note (fluid-structure interaction models in which the fluid is modelled explicitly). Careful choice of the appropriate FE package is thus required when performing these type of analyses.

5.3 Consideration of vertical component of the seismic action

For the verification of the seismic capacity of industrial facilities in Groningen the vertical component of the seismic action should always be taken into account. For the induced earthquakes in Groningen the ratio of vertical-to-horizontal accelerations can be considerably higher compared to ground motion recordings from tectonic earthquakes.

5.4 Correction formulae (A.24) of NEN-EN 1998-4 (p.52)

The expression (A.24) of NEN-EN 1998-4 on page 52 is retrieved from Scharf et al.[6] In the original paper by Scharf et al. a factor 2 is included in the denominator. Without this correction, equation (A.24) gives an incorrect estimation of the impulsive eigenperiod when compared to (A.35). Since the original reference document by Scharf et al. contains the factor 2 in the denominator it can be concluded that equation (A.24) is most likely wrongly copied from the original reference document. Thus, for future application of the GBoD, we advise the use of equation (A.24) of NEN-EN 1998-4 with the additional factor 2 in the denominator (resulting thus in the doubling of the eigenperiod).

6. Technical Note

The TU Delft technical note on liquid storage tanks starts at the following page.

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

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Contents

1. Introduction

This document provides a brief explanation regarding the calculation procedure that can be followed for the verification of the structural integrity of industrial facilities in the north of the Netherlands (Groningen) under seismic loading. Although the text is written with the design of liquid storage tanks in mind, the procedures described herein can be followed for other structures, i.e. silos, pipelines etc., provided that the relevant seismic input are available in each case. This document forms an integral part of the Generic Basis of Design (GBoD) which is prepared under the auspices of the workgroup "maatgevende aardbevingsblasting" chaired by Prof. dr. Ira Helsloot.

This document is not meant to substitute any standard procedure that is normally applied for the checking of the structural integrity of the industrial facilities. The focus is solely placed on the clarification of the adjustments needed to the standard methods because of the different approach followed in the derivation of the seismic input. The document contains some explanatory notes for the specialised structural engineer as well as some suggestions for modification of the standard methods in order to comply with the current form of the seismic input provided by KNMI. It is assumed that the engineer is already familiar with the design of structures of similar type as well as with the relevant design codes. Keeping that in mind, the document introduces only those changes that the engineer should consider when performing the calculations without discussing the calculation procedures in detail. The details of the calculation methods for assessing the structural integrity for different types of structures are given in the individual Basis of Design (BoD) documents as prepared by the engineering firms under the auspices of this workgroup.

The structure of the document is as follows. First, the definition of the seismic input is given with the emphasis placed on the differences with respect to standard methods used in earthquake engineering and the need to modify the soil damping during the structural analysis. Second, the definition of the design limit state is given which corresponds to the maximum credible earthquake to be expected in the region. Third, the *simple method* is described to analyse the seismic response of structures depending on the availability of various models and modelling approaches. Finally, an advanced method is presented should further investigation be required in the future.

2. Definition of the seismic input

2.1 Basic representation of the seismic action according to NEN-EN 1998-1:2004

The basic representation of the seismic action according to NEN-EN 1998-1:2004 **[1]** is an elastic ground acceleration response spectrum (called hereafter the elastic response spectrum) obtained at the surface of the soil. The standard practice in the derivation of an elastic response spectrum does not account for the non-linear soil response. In section 3 of NEN-EN 1998-1:2004, the elastic response spectra are derived on the basis of a linear response of the soil; for different soil types and intensity of the seismic events two sets of spectra are proposed (Type 1 and Type 2 elastic response spectra). Any corrections for additional dissipation provided by the soil, in the form of physically non-linear behaviour (or radiation damping), are incorporated at a later stage during the seismic analysis of the structural system. In particular, NEN-EN 1998-5:2004 provides a guideline for the increase in the overall critical damping ratio of the system to account for the inelastic response of the foundation soil at varying strain level; the latter related to the expected intensity of the ground motion.

2.2 Representation of the seismic action by KNMI

The seismic input provided by KNMI are based on site-specific analysis which takes into account the (physically) non-linear soil behaviour in the upper soil layers, i.e. Version 2 Ground Motion Prediction Equations (v2 GMPEs) for response spectral accelerations **[2]**. Thus, the representation of the seismic action, although similar to NEN-EN 1998-1:2004, is a response spectrum which accounts additionally for the inelastic response of the soil from the depth of the so-called "engineering bedrock" (-350m from the ground surface) until the soil surface. This response spectrum will be used as the main seismic input for the verification of the capacity of the industrial facilities without any modification (use of importance factors etc.), and will be called hereafter the Non-linear Response Spectrum (NLRS) with the understanding that the term *"non-linear"* refers here only to the (physically) non-linear behaviour of the soil material and not to that of the structure.

2.2.1 General background on the derivation of the NLRS

The derivation of the NLRS is based on vertical wave propagation through a soil column of an induced motion at a certain depth that takes into account the physical non-linearity in the upper soil medium. The wave propagation from the earthquake focus up to a predefined "bedrock" level (assumed at -350 m below the surface of the soil) is assumed linear. The response of the upper 350 m of the soil column includes the physical non-linearity of the soil material, albeit in an equivalent linear formulation **[3]**. In addition, during the derivation procedure, it is assumed that the surface of the ground is stress-free. This means that the NLRS corresponds to a free-field non-linear ground motion which does not consider the presence of the structure which rests on top of the soil (in the case of shallow foundations) or the presence of piles embedded into the soil (in the case of structures resting on pile foundations). One must note the presence of the structure, i.e. stiffness, mass, and damping, can alter the motion of the surface of the soil significantly, especially when large heavy liquid storage tanks are resting on relatively soft soils that can deform considerably. In other words, soil-structure interaction and its effect on the response spectrum are not considered in the derivation of the NLRS.

2.2.2 High-pass filtering of the derived spectral ordinates

The high-pass filtering (cut-off of the long structural periods) applied in the derivation of the NLRS should, ideally, not be less than T=6 seconds in view of the need to include response spectrum quantities at long structural periods associated with the convective motion of the liquid (sloshing of the liquid). In those cases in which the high-pass filtering has been applied at periods shorter than the one mentioned above, this should explicitly be mentioned to the structural engineer. When no spectral ordinates are provided at the long periods mentioned above - due to the limited amount of strong motion recordings available for the derivation of the NLRS as described in **[2]**- the corresponding ordinates of the NLRS can effectively be assumed equal to zero.

2.2.3 Fitting of the spectral ordinates to obtain the continuous NLRS

When the values of the NLRS to be applied in the design are not provided as a continuous function of the structural period T of the oscillator [Sa(T)], i.e. are given at a subset of structural periods and in the form of a discrete spectrum instead (T=0.01; 0.3; 1.0; 2.0 seconds), then the continuous NLRS can be obtained by suitable fitting (interpolation) of the spectral ordinates provided at the discrete periods to a characteristic acceleration spectrum for the region provided by KNMI. By suitable fitting the following is understood; the characteristic spectrum should be scaled accordingly to minimise the difference between the final NLRS (continuous spectrum) and the discrete values (misfit) for the structural periods of interest.

2.2.4 Basic information needed by the structural engineer

For the seismic analysis of the structural elements the following input need to be provided at each location: (i) contour maps with the peak ground acceleration ("shake maps" at T=0.01s); (ii) non-linear response spectra (NLRS) at the several locations (derived as described above); and (iii) the soil material properties, i.e. dynamic modulus of elasticity, Poisson's ratio and density, of the upper (few) soil layer(s) at the relevant strain level expected during the considered earthquake event as used for the derivation of the NLRS by KNMI. Any alternative representation of the soil material constants that can lead to the aforementioned basic quantities as, for example, the *Lamé* coefficients for the soil description, is also relevant. The information of the soil properties for this study should be gathered by appropriate SPT and/or SCPT performed for the site under consideration in accordance with the relevant international and/or national codes.

2.2.5 Correction of the KNMI response spectra for different values of the critical damping ratio ξ

In order to be applied by the engineer for the design of new structures (or the verification of the capacity of existing structures) according to the Eurocode provisions (NEN-EN 1998-4:2007 **[4]**), the NLRS should be provided for at least two values of the critical damping ratio of the structure, namely, ξ=5% and ξ=0.5%. The use of the correction factor "*η*" as suggested in NEN-EN 1998-1:2004 under 3.2.2.2(3) may not be applied directly in the provided NLRS since the latter are derived on the basis of non-linear soil behaviour. Thus, the derivation of response spectra for different values of the critical damping ratio of the structure may not be scaled linearly for values of ξ≠5% as it is customary in earthquake engineering. Since the scaling of the various branches of the NLRS for different values of *η* is generally unknown, it is recommended to provide directly the NLRS for (at least) the requested values of ξ mentioned above.

2.2.6 Recommendation regarding soil damping for the seismic analysis

In view of assumptions mentioned above, one may not make use of the additional material damping described under section 2.3.3 of NEN-EN 1998-4:2006 for the soil, since the non-linear soil behaviour is already exploited in the derivation of the NLRS. Due to the inherent uncertainties in the process of the derivation of site-specific spectra (and the deviation from the usually adopted procedure given in NEN-EN 1998-1:2004), it is advisable:

i) not to consider any additional damping for the soil (physical) non-linearity (as described in NEN-EN 1998-5:2004 **[5]**, section 4.2.3); and
ii) to consider only limited additional damping for the radiation of elastic waves into the soil in case that significant exploitation of the non-linear soil response is to be expected. This second component may need further investigation at a later stage in order to justify the amount of radiation damping to be applied in the design together with the use of the NLRS.

In those cases in which there is not sufficient data to justify the choice of additional soil radiation damping to be used in the analysis, the ordinates of the NLRS should be applied as are without further modification.

2.3 Additional notes

It is important to mention here that the derivation of a NLRS which considers the (physically) non-linear behaviour of the soil, does not necessarily lead to reduced seismic loads when it comes to structural design. In other words, and although the provided spectral quantities may be significantly reduced with respect to their equivalent ones obtained by considering a fully linear soil behaviour, the resulting seismic action to be considered in the design for the Ultimate Limit State (ULS defined in section 3 below) may not (necessarily) be reduced. Although seemingly paradoxical, this can happen due to several reasons as explained below.

2.3.1 Consideration of additional damping for the soil according to NEN-EN 1998-5:2004

First, the provisions in the NEN-EN1998-5:2004 allow for significant amounts of damping ratios to be used (once justified) due to the physically non-linear behaviour of the soil beneath the foundation and the radiation of energy away from the vibrating structures (damping ratios up to ξ_{max}=25% are allowed). Thus, for seismic events (yielding considerable strain levels in the soil), the combination of elastic spectra together with behaviour factors that incorporate such high percentages of damping may (very well) lead to design spectra ordinates less conservative that the ones obtained in the NLRS.

2.3.2 Significance of the constant acceleration plateau of the response spectrum

Second, the exploitation of the non-linear soil response together with the vertical wave propagation over a soil column may result in NLRS that concentrate the energy in a very narrow band of structural periods, i.e. a single peak (or limited constant acceleration plateau) in the acceleration response spectrum in contrast to a wider constant acceleration plateau usually adopted in the design (NEN-EN 1998-1:2004). The presence of such sharp peaks depends very much on the choice of the soil column characteristics, for example, its material properties and depth. A larger depth may lead to a wider constant acceleration plateau in the NLRS and smaller values of the response quantities. Thus, the choice of the depth of the soil column becomes critical, especially when non-linear behaviour is exploited. A sensitivity analysis with respect to this point is recommended as a further research item should NLRS be adopted as are in future designs.

For the design engineer, the presence of a single sharp peak in the NLRS may turn to be crucial for the determination of the seismic loads. Since the majority of the design methods rely on the use of the response spectrum method of analysis, it is important to look beyond the fundamental modes of vibration, since higher modes of vibration may turn out to be critical for the correct determination of the seismic load (especially when the fundamental mode is positioned away from the peak of the NLRS). In other words, the fundamental mode with high participation mass may not be sufficient to determine the design loads as higher modes (with reduced participation mass) may yield higher seismic loads if their natural periods are positioned closer to the peak (or *narrow* constant acceleration plateau) of the spectrum.

2.3.3 Significance of the soil-structure interaction effects

Third, the presence of a structure on top of the soil may alter the position of the peak of the NLRS (and in fact the shape of the spectrum itself) because the fundamental period of the structure-soil system differs from the one of the soil column alone. In traditional seismic design, which makes use of the classical elastic response spectra, this fact may not be that crucial because the shifting of the natural period of the structure-soil system may still lie within the relatively wide constant acceleration plateau. However, when this constant acceleration plateau vanishes altogether, a small modification of the natural period of the coupled system may yield a completely different seismic load. This issue needs further investigation and could be a point of further research should NLRS become standard practice in the seismic design.

2.3.4 Recommendation for future research

For all the reasons mentioned above, it is advisable to perform as an additional research item at least one analysis (at a later stage) in which the traditional design method based on the linear response spectrum is applied together with the necessary reduction to account for soil damping instead of the use of NLRS as seismic input.

3. Definition of the design limit state

3.1 Design limit state for controlled release of contents

The limit state to be checked is defined according to the Ultimate Limit State (ULS) given in section 2.1.2 of NEN-EN 1998-4:2007 as follows:

For particular elements of the network, as well as for independent units whose complete collapse would entail severe consequences, the ULS is defined as that of a state prior to structural collapse that, although possibly severe, would exclude brittle failures and would allow for a controlled release of the contents. When the failure of the aforementioned elements does not entail severe consequences, the ULS may be defined as corresponding to total structural collapse.

3.2 Damage limitation state (DLS) and required input

No Damage Limitation State (DLS) can be checked at the current stage with the input provided by KNMI as this would entail the need to perform a separate analysis to obtain a new set of acceleration response spectra corresponding to a different seismic scenario of a lower return period for the new design seismic event (DLS). The method proposed in NEN-EN 1998-4:2006 (section 2.2 under point (3)) for the reduction of the design seismic action with a factor of v=0.4~0.5 cannot be applied in this case since the derivation of the input acceleration response spectra is based on a non-linear soil response which cannot be scaled linearly with a constant reduction factor as proposed in NEN-EN 1998-4:2007.

To be able to check the DLS, KNMI should provide additional seismic input relevant to the correspondent seismic event which is not the maximum credible earthquake expected in the region but a seismic event of a higher probability of exceedance. The satisfaction of the DLS, should this be required for controlling the damage and costs, could be checked in the future. This is not part of the verification proposed in this GBoD.

4. Seismic analysis of structures (simple method)

4.1 Basic method of analysis

Based on the seismic input provided by KNMI as described above, the basic method of analysis, when hand calculations are not applicable according to the provisions of EN 1998-1 (2004) **[1]**, should be the **Modal Response Spectrum method of Analysis (MRSA)**.

4.1.1 Structural models for the verification of the design criteria for the Ultimate Limit State

The basic model to be used (when hand calculations are not applicable according to the provisions of EN 1998-1 (2004) **[1]**), should be a linear Finite Element (FE) model of the structure. In particular, for the case of liquid storage tanks, one of the three approaches described below should be followed based on the availability and type of FE models developed.

Model I

When the FE model of the structure **includes** the dynamic motion of the liquid, i.e. a dynamic FE model of the coupled tank-liquid system is available, then the modal response spectrum method of analysis can be carried out in accordance with:

a. NEN-EN 1998-1:2004 (specifically section 4.3.3.3);
b. NEN-EN 1998-4:2007 (section 4, considering additionally the specifications defined above regarding the modification of the soil damping);

For this analysis an eigenvalue problem of the tank-liquid system needs to be solved first followed by the modal response spectrum method of analysis which should be carried out in the usual way for obtaining the final stresses acting in the various parts of the shell structure.

The verification of the seismic capacity for the corresponding ULS should then be carried out in accordance with NEN-EN 1998-4:2007 taking into account the relevant parts of NEN-EN 1998-5:2004 and NEN-EN 1993-4-2:2007 **[6]**. In particular, the following should be checked:

a. Global stability of the liquid storage tank;
b. Yielding of the steel shell taking into account the special provisions given in NEN-EN 1998-4: Annex A.9 (for uplifting when required) as well as the provisions in NEN-EN 1998-4: Annex A.10;
c. Buckling in shear;
d. "Elephant foot" buckling and elastic buckling (Annex A.10);
e. Failure of the anchors (only for anchored tanks) and ductility checks;
f. Failure of foundation beneath the tanks;
g. Connections with adjusted piping and equipment based on the relative displacements to be expected;
h. Checking of wave sloshing height.

<u>Effects of soil liquefaction in the above verifications should be considered accordingly when relevant.</u>

<u>Model II</u>

When the FE model of the structure **does not** account explicitly for the dynamic motion of the liquid, i.e. a FE dynamic model of the coupled tank-liquid system is not available, then the modal response spectrum method of analysis can be carried out in accordance with:

a. NEN-EN 1998-1:2004;
b. NEN-EN 1998-4:2007. In particular, the specifications of section 4 hold for the liquid storage tanks. The dynamic pressures of the liquid at the inner side of the shell structure can be calculated according to NEN-EN 1998-4: Annex A. The calculation of the dynamic pressures to be applied at the inner side of the tank should account for the deformability of the shell structure for tanks composed of thin steel walls. For tanks made of thick concrete walls the deformability of the outer shell may be neglected.

For this analysis, the dynamic loads as a result of the liquid motion are applied as equivalent static loads at the inner side of the shell with predefined spatial distribution and an amplitude defined by the corresponding NLRS provided by KNMI. Subsequently, the seismic load cases are combined with the rest of the load cases relevant to the static situation. From that point on, the verification of the tank is done in accordance with standard practices commonly used in static design cases and the verification criteria are the same as described above.

<u>Model III</u>

When no suitable FE model of the structure is available, then the analysis can be carried out on the basis of the simplified methods as described in NEN-EN 1998-4: Annex A.3.2.2 **[7]**. Alternatively, the verification procedure can follow the provisions of the *API Standard 650: Eleventh Edition, June 2007* (Appendix E – Seismic design of storage tanks) taking into account the NLRS as provided by KNMI. However, it is recommended to investigate at least one case per facility in which a detailed FE model of the most critical elements is developed.

4.1.2 Inclusion of higher order modes and modal truncation

Due to the specific shape of the seismic input provided by KNMI in which the constant acceleration plateau of the design spectrum is largely missing, it is advisable to account for more than one rigid impulsive and flexible modes in the seismic analysis, especially in those cases in which the first rigid and flexible impulsive modes are positioned at the right-side (at longer periods) of the peak of the acceleration response spectrum. In such cases, it can happen that the higher order modes, although of much smaller participation mass, can contribute significantly to the total response due to larger spectral accelerations assigned to them. Thus, it is highly recommended to always check the contribution of the higher modes prior to modal truncation. In general, since the constant acceleration branch is largely missing from the new seismic input, it is to be expected that the results will be highly sensitive to the period of the fundamental impulsive mode of the liquid. Given the uncertainty in the estimation of the latter, especially when considering the relatively soft soils in the region, it is highly advisable to always check the results with respect to this element to verify their validity.

4.2 General remarks on structural analysis of liquid storage tanks

In all cases described previously the points described below need special attention.

4.2.1 Load cases

The seismic loads should be combined with the remaining load cases relevant to the accidental action effect (seismic action) in accordance with the relevant provisions of the Eurocodes.

4.2.2 Components of seismic input motions

When axisymmetric structures are analysed, i.e. cylindrical tanks, the earthquake load can be applied in one horizontal direction acting together with the vertical one. When two different horizontal input acceleration response spectra are provided at the same location, it is advisable to choose the most conservative one for the design, i.e. the one that results in the higher stresses at the structure. When non-axisymmetric structures are analysed all three components of the ground motion (two horizontal and one vertical) need to be applied simultaneously. In contrast to other cases, it is advisable to consider always the vertical component of the seismic action in the design due to the shallow depth of the foci of the Groningen earthquakes. KNMI provides a method to obtain the NLRS in the vertical direction.

4.2.3 Soil-structure interaction effects

The presence of a soft soil beneath the foundation of the structures can shift the natural periods of the tank-fluid system to longer periods compared to the one supported by rigid ground. This should be considered accordingly in the design by:

a. using flexible connection to the ground, i.e. springs in both radial and tangential direction applied at the bottom of the tank, in the dynamic models of **Type I** described previously. The spring constants should be derived based on the dynamic soil properties of the upper soil layer(s) as provided by KNMI and for the correspondent strain level expected during the seismic event (should be also clarified by KNMI if requested); or
b. modifying the natural periods of the oscillations of the impulsive modes as described in NEN-EN 1998-4: Annex A.7 in the quasi-static models of **Type II** mentioned previously.

When models of **Type III** are used, the provisions of NEN-EN 1998-4: Annex A.7 apply as well.

4.2.4 Behaviour factor for the non-linear response of the structure

A behaviour factor of q=1.5 can be applied to obtain the pressures exerted by the impulsive modes of the liquid in accordance with NEN-EN 1998-4:2007 when **Type I** and **II** models are used. Higher values of the behaviour factor can also be applied under certain conditions, but when not sufficient data are available to justify this choice it is advisable **not to use values of the behaviour factor higher than q=1.5** [1]. When the full dynamic models of **Type I** are used, the corresponding

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[1] According to section 2.4(2) of NEN-EN 1998-4:2007:
Use of q-factors greater than 1.5 in Ultimate Limit State (ULS) verifications is only allowed, provided that the sources of energy dissipation are explicitly identified and quantified and the capability of the structure to exploit them through appropriate detailing is demonstrated.

behaviour factor should be applied only to the impulsive tank-fluid modes of the system prior to modal superposition and not to the ones corresponding to predominantly convective motions of the liquid.

4.2.5 Reduction of NLRS provided by KNMI for q≠1

The reduction of the ordinates of the 5-percent damped NLRS with the behaviour factor q (to account for the nonlinear capacity of the structure) can be achieved as is customary for the various branches of the acceleration response spectrum. For the exact application of the q-factor, the provisions given in paragraph 3.2.2.5 of NEN EN1998-1:2004 (p.41-42) **[1]** can be used. More specifically, at the zero structural period there is no reduction of the elastic spectral ordinates. Then at the left and right corner of the constant acceleration plateau (if such a plateau exists) the elastic spectral ordinates can be divided by the q-factor. This procedure can then be repeated for longer structural periods as well. When a single peak exists in the NLRS (instead of a *constant acceleration plateau*), this peak can be reduced by the application of the q-factor in a similar manner.

Further investigation is recommended as an additional research item to determine the way of applying the behaviour factor q in cases where the inelastic response of the structural system needs to be considered together with the use of NLRS as derived by KNMI [2].

5. Seismic analysis of structures (advanced method)

For further research purposes an advanced analysis is suggested at this section. This is not part of the analysis covered in this GBoD. For such an analysis different input are required to represent the seismic action. In this section, we provide a general outline of the steps needed should such an analysis of this type be required in the future.

5.1 Seismic input required for advanced analysis methods

For an advanced method of analysis of the structures, the seismic input need to be modified in order to account for the soil-structure interaction effects. It is to be expected that the behaviour at the foundation level of the structure will be different when compared to the free-field motion. This is especially true when large structures (liquid storage tanks) are founded on relatively soft soil deposits as explained previously. In addition, the non-linear soil behaviour will be different as the stress distribution below the foundation level will change compared to the one obtained in the free-field. An advanced analysis of the structural systems should ideally account for:

i) Soil-structure interaction effects;
ii) Non-linear soil response (for seismic events that are expected to induce non-linear soil behaviour);
iii) Non-linear structural response (for seismic events that are expected to induce non-linear structural behaviour).

5.2 Modelling of the structure required for the advanced analysis methods

To be able to perform an advanced analysis the required seismic input should be provided in the form of appropriate time-histories of the seismic events to be expected under the correspondent limit state. For the ULS , this state corresponds to an earthquake for a predefined seismic hazard level in the area under investigation. The derived time histories should be provided at a reasonable depth below the structure, i.e. at such a depth that the effects of the structural response on the surrounding soil can be disregarded. The time histories should match the situation corresponding to a predefined seismic hazard at the chosen depth below the structure. The types of waves, wave propagation characteristics, and soil behaviour should match the conditions to be expected under the relevant circumstances. A suitable number of generated time histories should be provided to account for the inherent stochastic nature of earthquakes as described in NEN-EN 1998-1:2004. To properly account for the effect of soil-structure interaction the tank-liquid system has to be modelled together with the soil (or at least part of it), i.e. a tank-soil-liquid model is necessary. In this case, the classical modal response method of analysis cannot be applied and therefore suitable time histories (in terms of frequency content and amplitude characteristics) need to be provided at some level below the structure as described previously. The basic method of analysis in this case should be a non-linear time history (NLTH) analysis in accordance with NEN-EN 1998-1:2004 **[1]**. The verification criteria for global stability, yielding mechanisms and buckling failure modes remain the same as described previously.

6. References

[1] EN 1998-1 (2004): *Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*

[2] Bommer, J.J., Dost, B., Edwards, B., Kruiver, P.P., Meijers, P., Ntinalexix, M., Polidoro, B., Rodrigues-Marek, A., and Stafford, P.J. *Development of Version 2 GMPEs for Response Spectral Accelerations and Significant Durations from Induced Earthquakes in the Groningen Field*, Version 2, 29 October 2015.

[3] Darendeli, M. *Development of a new family of normalized modulus reduction and material damping curves*. Ph.D. Thesis, Department of Civil Engineering, University of Texas, Austin TX, 2001.

[4] NEN-EN 1998-4 (2007): *Eurocode 8: Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines*

[5] NEN-EN 1998-5 (2004): *Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects*

[6] NEN-EN 1993-4-2 (2007): *Eurocode 3: Design of steel structures - Part 4-2: Tanks*

[7] Malhotra, P.K., Wenk, T., Wieland, M. *Simple procedure for seismic analysis of liquid-storage tanks*. Structural Engineering International, 3/2000, pp. 197-201. Bijlage 5. Conclusies na proefberekeningen in vier pilot bedrijven

II

BIJLAGE: EXPLANATORY NOTES FOR THE 'LOC TOETS' IN APPLICATION TO THE INDUSTRIAL FACILITIES IN GRONINGEN [REF. 14]

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Title:	Explanatory notes for the "LoC Toets" in application to the industrial facilities in Groningen
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Disclaimer: This document has been prepared on request of the Nationaal Coördinator Groningen (NCG). The content of the present document is related to the "*Generic Basis of Design (GBoD) for the structural verification of industrial facilities in Groningen: a first screening of the seismic capacity*" which forms an integral part of the advice of the workgroup of Ira Helsloot "*Rapportage werkgroep Maatgevende aardbevingsbelasting voor de industrie: Naar een snelle, simpele, transparante en robuuste toets op de aardbevingsbestendigheid van de chemische industrie in Groningen* (4-11-2016)". The present document is not meant to substitute the latter; this document is a shorter version of the GBoD modified to meet the requirements of the "LoC toets" as requested by the NCG.

1. Introduction

This document provides an overview of the methodology to be followed for the so-called "LoC toets" to check the seismic capacity of the industrial facilities in Groningen. It gives a brief description of the seismic input (as provided in the report by KNMI) and the method to be used for the verification of the structural capacity based on the availability of structural models. This is not a stand-alone document; the technical note at the end of this document as well as the KNMI report prepared under the auspices of the workgroup of Ira Helsloot[1] form an integral part of it.

The technical note composed by TU Delft (see end of this document) includes a description of the method to be followed for the verification of the seismic capacity of the industrial facilities. In addition, for the seismic input the reference document is the report by KNMI (Bijlage 3 of the report of Ira Helsloot[1]) which additionally includes a set of files with the ordinates of the 5 percent damped acceleration response spectra to be applied in the design.

2. Definition of the seismic action

The basic representation of the seismic action is provided in terms of ground acceleration response spectra in the two relevant directions, i.e. horizontal and vertical. The spectra are derived on the basis of the maximum considered earthquake to be expected as described in the KNMI report. For the "LoC toets" the seismic action will be based on the <u>expected values of the ordinates of the 5 percent damped acceleration response spectrum</u>[2] obtained at the surface of ground which will be referred to hereafter as the "*acceleration response spectrum"* (defined either in the horizontal or the vertical direction).

2.1 Horizontal acceleration response spectrum

The representation of the seismic action in the horizontal direction is a 5 percent damped horizontal acceleration response spectrum at the surface of the ground derived at each location of interest as explained in detail in the KNMI report. The exact values of the spectral ordinates are explicitly provided at four structural periods namely, T=0.01s (effectively treated hereafter as the peak ground acceleration, PGA); 0.3s; 1.0s; 3s. The exact values of the ordinates of the spectrum are given in the KNMI report (files containing the spectral ordinates at each specific location). A continuous (as a function of the structural period T of the system) response spectrum can be derived on the basis of suitable interpolation of these four spectral ordinates to a characteristic acceleration response spectrum in the region. The resulting graphs for some typical locations are reproduced in the KNMI report. A similar curve fitting procedure should be followed for locations in which graphs are not explicitly provided based on the spectral ordinates provided at the four periods mentioned above. The 5 percent damped horizontal acceleration response spectrum constitutes the representation of the seismic action in the horizontal direction for the "LoC toets" with no further modification, i.e. use of importance factors.

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[1] Rapportage werkgroep Maatgevende aardbevingsbelasting voor de industrie: Naar een snelle, simpele, transparante en robuuste toets op de aardbevingsbestendigheid van de chemische industrie in Groningen (4-11-2016). https://www.rijksoverheid.nl/documenten/rapporten/2016/11/04/rapportage-werkgroep-maatgevende-aardbevingsbelasting-voor-de-industrie

[2] The expected values of the ordinates of the 5 percent damped acceleration response spectrum are equivalent to the set (i) ground motions defined in the GBoD (see Bijlage 4 of the report of Ira Helsloot - footnote 1 above).

2.2 Vertical acceleration response spectrum

The ordinates of the 5 percent damped vertical acceleration response spectrum are derived directly from the ones of the 5 percent damped horizontal acceleration response spectrum by multiplying the latter with a constant -per structural period T of the oscillator- factor. The multiplier can be obtained from the average values shown in the graph on p.345 in *Bommer et al.*[3]. The resulting 5 percent damped vertical acceleration response spectrum defines the seismic action in the vertical direction for the "LoC toets" with no further modification, i.e. use of importance factors.

2.3 Displacement response spectrum

The displacement spectrum in the relevant direction, i.e. either horizontal or vertical, can be derived by the following formula:

$$S_d\left(T,\xi=5\%\right)=S_a\left(T,\xi=5\%\right)\times\left(\frac{T}{2\pi}\right)^2$$

In the formula above, $S_d\left(T,\xi=5\%\right)$ denotes the ordinate of the 5 percent damped displacement response spectrum, $S_a\left(T,\xi=5\%\right)$ is the ordinate of the 5 percent damped acceleration response spectrum, and T is the structural period of interest.

3. Calculations and structural verifications

The seismic analysis shall be carried out in accordance with the provisions of the relevant parts of the Eurocodes and the seismic actions described in section 2 above.

3.1 Method of analysis

Two main methods can be used for the structural analysis, i.e.

a) Lateral force method as described in section 4.3.3.2 of NEN-EN 1998-1 and the relevant parts of NEN-EN 1998-4; or
b) Modal Response Spectrum method of Analysis (MRSA) as specified in 4.3.3.3 of NEN-EN 1998-1 and the relevant parts of NEN-EN 1998-4.

The choice between the two methods is based on the structural regularity conditions in plan and elevation as specified in section 4.2.3 of NEN-EN 1998-1. In addition, the lateral force method should essentially yield a conservative estimation of the resulting stresses and deformations in the structure which implies that higher order modes are not expected to contribute to the structural response. When the conditions of structural regularity as specified in NEN-EN 1998-1 are not met (or the correct distribution of stresses and strains is not expected to be captured with the application of the lateral force method), a MRSA is the minimum requirement (equivalent to either Model I or Model II of the TUD technical note - see Bijlage 4 of the report of Ira Helsloot[1]).

3.2 Ultimate Limit State (ULS) verification checks

When the analysis is based on the lateral force method as specified in the Eurocode 8, the verification shall be carried out in accordance with the ULS as defined in the relevant parts of the Eurocodes. An outline of a suitable model of this group is Model III of a liquid storage tank structure as described in the TUD technical note. Once the loads are defined and the resulting

[3] Bommer, J.J., Dost, B., Edwards, B., Kruiver, P.P., Meijers, P., Ntinalexix, M., Polidoro, B., Rodrigues-Marek, A., and Stafford, P.J. *Development of Version 2 GMPEs for Response Spectral Accelerations and Significant Durations from Induced Earthquakes in the Groningen Field*, Version 2, 29 October 2015.

stresses and member forces are determined, the safety checks (failure modes of the structure and of the soil-foundation system) shall be carried out in accordance with the relevant parts of the Eurocodes. Specific procedures for checking the capacity of the foundations and the liquefaction potential of the soil are included in the NPR 9998:2015 in accordance with the relevant parts of the Eurocodes[4]. Similar procedures shall be followed for other type of structures (other than liquid storage tanks).

When the analysis is based on the MRSA[5] with the inclusion of higher-order vibration modes, an explicit consideration of a FE model of the structure for the determination of the stresses and strains is required (equivalent to either Model I or Model II of the TUD technical note). The verifications shall be carried out in accordance with the ULS as defined in the relevant parts of the Eurocodes. For example, such a model should provide an estimation of the spatial distribution of the internal pressures at the inner side of the liquid storage tank, and subsequently the stress resultants and member forces can be determined. Consequently, the safety checks (failure modes of the structure and of the soil-foundation system) need to be carried out in accordance with the relevant parts of the Eurocodes. Similar procedures shall be followed for other type of structures (other than liquid storage tanks).

Once the verification of the structural integrity is completed with either of the two methods described above, a verification of the individual components of the industrial facility (pipelines, connections, process equipment, etc.) needs to be carried out in accordance with the existing international (and/or national) codes and practices.

4. Additional comments ("lessons learned" from the pilot studies)

In this section a few technical comments are included as a result of the application of the method in the pilot studies. These need to be considered for the "LoC toets".

4.1 FE models including fluid-structure interaction

When the analysis of the seismic capacity requires the modelling of coupled systems which contain structure-fluid interaction, i.e. FE models of liquid storage tanks, a careful choice of the software able to perform such an analysis is required. Results of the pilot studies have revealed that regular FE software used for steel structures is not always suitable for modal analysis of liquid storage tanks when one considers models of the Type II of the TUD technical note in which the fluid is substituted by an added mass at the inner side of the wall of the steel tank for the estimation of the fluid-tank vibration modes. More elaborated software may be capable of dealing with models of Type I of the TUD technical note (fluid-structure interaction models in which the fluid is modelled explicitly). Careful choice of the appropriate FE package is thus required when performing these types of analyses.

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[4] In the absence of a calibrated, region specific methodology for induced earthquakes, it is recommended to apply the Boulanger and Idriss (2014) method [Boulanger R. W. and Idriss I. M. (2014), 'CPT and SPT Based Liquefaction Triggering Procedures', UCD/CGM-14/01], using the Peak Ground Acceleration (PGA) values provided by KNMI at the surface of the soil profile as input (effectively the ordinates of 5 percent damped response acceleration spectrum at T=0.01s as defined in the KNMI report). It is noted that applying the Boulanger and Idriss (2014) method without region-specific modifications will, most likely, result in a conservative assessment.

[5] The application of the GBoD for the investigation of the seismic capacity in the four pilot case studies during the period July-October 2016 has provided insight into the cases in which a simple hand calculation based on the lateral force method was sufficient for the determination of the resulting seismic action in the structures. In the "LoC toets", when the structures do not satisfy the structural regularity criteria in plan and/or elevation (section 4.2.3 of NEN-EN 1998-1), one can proceed directly with the MRSA to verify the seismic capacity of the system by developing a suitable FE model and applying seismic actions specified in section 2 above.

4.2 Consideration of vertical component of the seismic action

For the verification of the seismic capacity of industrial facilities with the “LoC toets” in Groningen the vertical component of the seismic action shall always be taken into account. For the induced earthquakes in Groningen the ratio of vertical-to-horizontal accelerations can be considerably higher compared to ground motion recordings from tectonic earthquakes.

4.3 Correction formulae (A.24) of NEN-EN 1998-4 (p.52)

The expression (A.24) of NEN-EN 1998-4 on page 52 is retrieved from Scharf et al.[6] In the original paper by Scharf et al. a factor 2 is included in the denominator. Without this correction, equation (A.24) gives an incorrect estimation of the impulsive eigenperiod when compared to (A.35). Since the original reference document by Scharf et al. contains the factor 2 in the denominator it can be concluded that equation (A.24) is most likely wrongly copied from the original reference document. Thus, for the LoC toets we advise the use of equation (A.24) of NEN-EN 1998-4 with the additional factor 2 in the denominator (resulting thus in the doubling of the eigenperiod).

4.4 Shear buckling instability verification of liquid storage vertical steel tanks

In the pilot studies the shear buckling instability verification was based on the provisions of NEN-EN1998-4 (section 4) and NEN-EN1993-1-6 using a linear model of the structure and a strength-based verification method, i.e. calculation of elastic stresses with a FE model and verification of the critical shear buckling capacity of the shell membrane by the relevant formulae given in NEN-EN1993-1-6. According to NEN-EN1998-4 (section 4), shear buckling failure needs always to be verified for steel tanks subjected to seismic excitation in accordance with NEN-EN1993-1-6. However, the formulae provided in Annex D of NEN-EN1993-1-6 for the critical shear buckling stresses in the membrane of the shell structure do not account for the presence of the stored liquid and the effect of the existing hoop stress on the shear buckling instability. This effect is considered, for example, in the provisions of the Japanese code[7] and is documented in the literature by previous experimental work[8].

The inconsideration of the influence of the hoop membrane stresses -caused by the hydrostatic liquid pressure exerted on the inner surface of the shell- on the shear buckling stability, results at a critical shear buckling resistance which is very low in the case of filled tanks. Results of the pilot studies have clearly revealed this issue and highlighted the extremely low shear buckling capacity predicted by applying Annex D of NEN-EN1993-1-6. Until a solid conclusion is reached on the exact influence of the hoop stress on the instability of the shear type[9], it is advised to apply the Japanese code provisions for the checking of this particular failure mode[10].

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

[7] *Design recommendation for storage tanks and their supports with emphasis on seismic design, Architectural Institute of Japan, 2010.* Formulae (3.58-3.62) refer to allowable shear stress against seismic lateral force and take into account a positive correction term (increase of shear bucking capacity) in the presence of hoop stress as a result of internal pressure.

[8] *The effect of internal pressure on the buckling stress of thin walled circular cylinders under torsion*, NASA report, May 1944.

[9] The subject matter is still under discussion and no solid conclusion is yet reached on the exact effect of the internal pressure on the shear buckling capacity of thin-walled cylinders. Since the formulae included in the Eurocodes for this specific case do not consider the presence of internal pressure, while the Japanese code provisions do consider this special case, it is advised to apply the latter for the case in which the tanks are filled with liquid.

[10] More specifically, formulae [3.58-3.62] can be applied in deviation to the formulae given in Annex D of NEN-EN1993-1-6 for the shear buckling instability verification of liquid steel storage tanks subjected to seismic excitation.

For the case in which the tank is not filled with liquid the provisions of NEN-EN1993-1-6 for the shear buckling verification still hold and shall be used. All other shell buckling verifications for steel tanks (meridional buckling and circumferential buckling with co-existent internal pressure and any combination of those) shall be carried out in accordance with the formulae given in NEN-EN1993-1-6.

5. Technical Note

The TU Delft technical note on liquid storage tanks is reproduced in the following page for convenience.

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Contents

1. Introduction

This document provides a brief explanation regarding the calculation procedure that can be followed for the verification of the structural integrity of industrial facilities in the north of the Netherlands (Groningen) under seismic loading. Although the text is written with the design of liquid storage tanks in mind, the procedures described herein can be followed for other structures, i.e. silos, pipelines etc., provided that the relevant seismic input are available in each case. This document forms an integral part of the Generic Basis of Design (GBoD) which is prepared under the auspices of the workgroup "maatgevende aardbevingsblasting" chaired by Prof. dr. Ira Helsloot.

This document is not meant to substitute any standard procedure that is normally applied for the checking of the structural integrity of the industrial facilities. The focus is solely placed on the clarification of the adjustments needed to the standard methods because of the different approach followed in the derivation of the seismic input. The document contains some explanatory notes for the specialised structural engineer as well as some suggestions for modification of the standard methods in order to comply with the current form of the seismic input provided by KNMI. It is assumed that the engineer is already familiar with the design of structures of similar type as well as with the relevant design codes. Keeping that in mind, the document introduces only those changes that the engineer should consider when performing the calculations without discussing the calculation procedures in detail. The details of the calculation methods for assessing the structural integrity for different types of structures are given in the individual Basis of Design (BoD) documents as prepared by the engineering firms under the auspices of this workgroup.

The structure of the document is as follows. First, the definition of the seismic input is given with the emphasis placed on the differences with respect to standard methods used in earthquake engineering and the need to modify the soil damping during the structural analysis. Second, the definition of the design limit state is given which corresponds to the maximum credible earthquake to be expected in the region. Third, the *simple method* is described to analyse the seismic response of structures depending on the availability of various models and modelling approaches. Finally, an advanced method is presented should further investigation be required in the future.

2. Definition of the seismic input

2.1 Basic representation of the seismic action according to NEN-EN 1998-1:2004

The basic representation of the seismic action according to NEN-EN 1998-1:2004 **[1]** is an elastic ground acceleration response spectrum (called hereafter the elastic response spectrum) obtained at the surface of the soil. The standard practice in the derivation of an elastic response spectrum does not account for the non-linear soil response. In section 3 of NEN-EN 1998-1:2004, the elastic response spectra are derived on the basis of a linear response of the soil; for different soil types and intensity of the seismic events two sets of spectra are proposed (Type 1 and Type 2 elastic response spectra). Any corrections for additional dissipation provided by the soil, in the form of physically non-linear behaviour (or radiation damping), are incorporated at a later stage during the seismic analysis of the structural system. In particular, NEN-EN 1998-5:2004 provides a guideline for the increase in the overall critical damping ratio of the system to account for the inelastic response of the foundation soil at varying strain level; the latter related to the expected intensity of the ground motion.

2.2 Representation of the seismic action by KNMI

The seismic input provided by KNMI are based on site-specific analysis which takes into account the (physically) non-linear soil behaviour in the upper soil layers, i.e. Version 2 Ground Motion Prediction Equations (v2 GMPEs) for response spectral accelerations **[2]**. Thus, the representation of the seismic action, although similar to NEN-EN 1998-1:2004, is a response spectrum which accounts additionally for the inelastic response of the soil from the depth of the so-called "engineering bedrock" (-350m from the ground surface) until the soil surface. This response spectrum will be used as the main seismic input for the verification of the capacity of the industrial facilities without any modification (use of importance factors etc.), and will be called hereafter the Non-linear Response Spectrum (NLRS) with the understanding that the term *"non-linear"* refers here only to the (physically) non-linear behaviour of the soil material and not to that of the structure.

General background on the derivation of the NLRS

The derivation of the NLRS is based on vertical wave propagation through a soil column of an induced motion at a certain depth that takes into account the physical non-linearity in the upper soil medium. The wave propagation from the earthquake focus up to a predefined "bedrock" level (assumed at -350 m below the surface of the soil) is assumed linear. The response of the upper 350 m of the soil column includes the physical non-linearity of the soil material, albeit in an equivalent linear formulation **[3]**. In addition, during the derivation procedure, it is assumed that the surface of the ground is stress-free. This means that the NLRS corresponds to a free-field non-linear ground motion which does not consider the presence of the structure which rests on top of the soil (in the case of shallow foundations) or the presence of piles embedded into the soil (in the case of structures resting on pile foundations). One must note the presence of the structure, i.e. stiffness, mass, and damping, can alter the motion of the surface of the soil significantly, especially when large heavy liquid storage tanks are resting on relatively soft soils that can deform considerably. In other words, soil-structure interaction and its effect on the response spectrum are not considered in the derivation of the NLRS.

High-pass filtering of the derived spectral ordinates

The high-pass filtering (cut-off of the long structural periods) applied in the derivation of the NLRS should, ideally, not be less than T=6 seconds in view of the need to include response spectrum quantities at long structural periods associated with the convective motion of the liquid (sloshing of the liquid). In those cases in which the high-pass filtering has been applied at periods shorter than the one mentioned above, this should explicitly be mentioned to the structural engineer. When no spectral ordinates are provided at the long periods mentioned above - due to the limited amount of strong motion recordings available for the derivation of the NLRS as described in **[2]**- the corresponding ordinates of the NLRS can effectively be assumed equal to zero.

Fitting of the spectral ordinates to obtain the continuous NLRS

When the values of the NLRS to be applied in the design are not provided as a continuous function of the structural period T of the oscillator [Sa(T)], i.e. are given at a subset of structural periods and in the form of a discrete spectrum instead (T=0.01; 0.3; 1.0; 2.0 seconds), then the continuous NLRS can be obtained by suitable fitting (interpolation) of the spectral ordinates provided at the discrete periods to a characteristic acceleration spectrum for the region provided by KNMI. By suitable fitting the following is understood; the characteristic spectrum should be scaled accordingly to minimise the difference between the final NLRS (continuous spectrum) and the discrete values (misfit) for the structural periods of interest.

Basic information needed by the structural engineer

For the seismic analysis of the structural elements the following input need to be provided at each location: (i) contour maps with the peak ground acceleration ("shake maps" at T=0.01s); (ii) non-linear response spectra (NLRS) at the several locations (derived as described above); and (iii) the soil material properties, i.e. dynamic modulus of elasticity, Poisson's ratio and density, of the upper (few) soil layer(s) at the relevant strain level expected during the considered earthquake event as used for the derivation of the NLRS by KNMI. Any alternative representation of the soil material constants that can lead to the aforementioned basic quantities as, for example, the *Lamé* coefficients for the soil description, is also relevant. The information of the soil properties for this study should be gathered by appropriate SPT and/or SCPT performed for the site under consideration in accordance with the relevant international and/or national codes.

Correction of the KNMI response spectra for different values of the critical damping ratio ξ

In order to be applied by the engineer for the design of new structures (or the verification of the capacity of existing structures) according to the Eurocode provisions (NEN-EN 1998-4:2007 **[4]**), the NLRS should be provided for at least two values of the critical damping ratio of the structure, namely, ξ=5% and ξ=0.5%. The use of the correction factor "η" as suggested in NEN-EN 1998-1:2004 under 3.2.2.2(3) may not be applied directly in the provided NLRS since the latter are derived on the basis of non-linear soil behaviour. Thus, the derivation of response spectra for different values of the critical damping ratio of the structure may not be scaled linearly for values of ξ≠5% as it is customary in earthquake engineering. Since the scaling of the various branches of the NLRS for different values of η is generally unknown, it is recommended to provide directly the NLRS for (at least) the requested values of ξ mentioned above.

Recommendation regarding soil damping for the seismic analysis

In view of assumptions mentioned above, one may not make use of the additional material damping described under section 2.3.3 of NEN-EN 1998-4:2006 for the soil, since the non-linear soil behaviour is already exploited in the derivation of the NLRS. Due to the inherent uncertainties in the process of the derivation of site-specific spectra (and the deviation from the usually adopted procedure given in NEN-EN 1998-1:2004), it is advisable:

i) not to consider any additional damping for the soil (physical) non-linearity (as described in NEN-EN 1998-5:2004 **[5]**, section 4.2.3); and
ii) to consider only limited additional damping for the radiation of elastic waves into the soil in case that significant exploitation of the non-linear soil response is to be expected. This second component may need further investigation at a later stage in order to justify the amount of radiation damping to be applied in the design together with the use of the NLRS.

In those cases in which there is not sufficient data to justify the choice of additional soil radiation damping to be used in the analysis, the ordinates of the NLRS should be applied as are without further modification.

2.3 Additional notes

It is important to mention here that the derivation of a NLRS which considers the (physically) non-linear behaviour of the soil, does not necessarily lead to reduced seismic loads when it comes to structural design. In other words, and although the provided spectral quantities may be significantly reduced with respect to their equivalent ones obtained by considering a fully linear soil behaviour, the resulting seismic action to be considered in the design for the Ultimate Limit State (ULS defined in section 3 below) may not (necessarily) be reduced. Although seemingly paradoxical, this can happen due to several reasons as explained below.

Consideration of additional damping for the soil according to NEN-EN 1998-5:2004

First, the provisions in the NEN-EN1998-5:2004 allow for significant amounts of damping ratios to be used (once justified) due to the physically non-linear behaviour of the soil beneath the foundation and the radiation of energy away from the vibrating structures (damping ratios up to ξ_{max}=25% are allowed). Thus, for seismic events (yielding considerable strain levels in the soil), the combination of elastic spectra together with behaviour factors that incorporate such high percentages of damping may (very well) lead to design spectra ordinates less conservative that the ones obtained in the NLRS.

Significance of the constant acceleration plateau of the response spectrum

Second, the exploitation of the non-linear soil response together with the vertical wave propagation over a soil column may result in NLRS that concentrate the energy in a very narrow band of structural periods, i.e. a single peak (or limited constant acceleration plateau) in the acceleration response spectrum in contrast to a wider constant acceleration plateau usually adopted in the design (NEN-EN 1998-1:2004). The presence of such sharp peaks depends very much on the choice of the soil column characteristics, for example, its material properties and depth. A larger depth may lead to a wider constant acceleration plateau in the NLRS and smaller values of the response quantities. Thus, the choice of the depth of the soil column becomes critical, especially when non-linear behaviour is exploited. A sensitivity analysis with respect to this point is recommended as a further research item should NLRS be adopted as are in future designs.

For the design engineer, the presence of a single sharp peak in the NLRS may turn to be crucial for the determination of the seismic loads. Since the majority of the design methods rely on the use of the response spectrum method of analysis, it is important to look beyond the fundamental modes of vibration, since higher modes of vibration may turn out to be critical for the correct determination of the seismic load (especially when the fundamental mode is positioned away from the peak of the

NLRS). In other words, the fundamental mode with high participation mass may not be sufficient to determine the design loads as higher modes (with reduced participation mass) may yield higher seismic loads if their natural periods are positioned closer to the peak (or *narrow* constant acceleration plateau) of the spectrum.

Significance of the soil-structure interaction effects

Third, the presence of a structure on top of the soil may alter the position of the peak of the NLRS (and in fact the shape of the spectrum itself) because the fundamental period of the structure-soil system differs from the one of the structure alone. In traditional seismic design, which makes use of the classical elastic response spectra, this fact may not be that crucial because the shifting of the natural period of the structure-soil system may still lie within the relatively wide constant acceleration plateau. However, when this constant acceleration plateau vanishes altogether, a small modification of the natural period of the coupled system may yield a completely different seismic load. This issue needs further investigation and could be a point of further research should NLRS become standard practice in the seismic design.

Recommendation for future research

For all the reasons mentioned above, it is advisable to perform as an additional research item at least one analysis (at a later stage) in which the traditional design method based on the linear response spectrum is applied together with the necessary reduction to account for soil damping instead of the use of NLRS as seismic input.

3. Definition of the design limit state

3.1 Design limit state for controlled release of contents

The limit state to be checked is defined according to the Ultimate Limit State (ULS) given in section 2.1.2 of NEN-EN 1998-4:2007 as follows:

For particular elements of the network, as well as for independent units whose complete collapse would entail severe consequences, the ULS is defined as that of a state prior to structural collapse that, although possibly severe, would exclude brittle failures and would allow for a controlled release of the contents. When the failure of the aforementioned elements does not entail severe consequences, the ULS may be defined as corresponding to total structural collapse.

3.2 Damage limitation state (DLS) and required input

No Damage Limitation State (DLS) can be checked at the current stage with the input provided by KNMI as this would entail the need to perform a separate analysis to obtain a new set of acceleration response spectra corresponding to a different seismic scenario of a lower return period for the new design seismic event (DLS). The method proposed in NEN-EN 1998-4:2006 (section 2.2 under point (3)) for the reduction of the design seismic action with a factor of v=0.4~0.5 cannot be applied in this case since the derivation of the input acceleration response spectra is based on a non-linear soil response which cannot be scaled linearly with a constant reduction factor as proposed in NEN-EN 1998-4:2007.

To be able to check the DLS, KNMI should provide additional seismic input relevant to the correspondent seismic event which is not the maximum credible earthquake expected in the region but a seismic event of a higher probability of exceedance. The satisfaction of the DLS, should this be required for controlling the damage and costs, could be checked in the future. This is not part of the verification proposed in this GBoD.

4. Seismic analysis of structures (simple method)

4.1 Basic method of analysis

Based on the seismic input provided by KNMI as described above, the basic method of analysis, when hand calculations are not applicable according to the provisions of EN 1998-1 (2004) **[1]**, should be the **Modal Response Spectrum method of Analysis (MRSA)**.

Structural models for the verification of the design criteria for the Ultimate Limit State (ULS)

The basic model to be used (when hand calculations are not applicable according to the provisions of EN 1998-1 (2004) **[1]**), should be a linear Finite Element (FE) model of the structure. In particular, for the case of liquid storage tanks, one of the three approaches described below should be followed based on the availability and type of FE models developed.

<u>Model I</u>

When the FE model of the structure **includes** the dynamic motion of the liquid, i.e. a dynamic FE model of the coupled tank-liquid system is available, then the modal response spectrum method of analysis can be carried out in accordance with:

a. NEN-EN 1998-1:2004 (specifically section 4.3.3.3);
b. NEN-EN 1998-4:2007 (section 4, considering additionally the specifications defined above regarding the modification of the soil damping);

For this analysis an eigenvalue problem of the tank-liquid system needs to be solved first followed by the modal response spectrum method of analysis which should be carried out in the usual way for obtaining the final stresses acting in the various parts of the shell structure.

The verification of the seismic capacity for the corresponding ULS should then be carried out in accordance with NEN-EN 1998-4:2007 taking into account the relevant parts of NEN-EN 1998-5:2004 and NEN-EN 1993-4-2:2007 **[6]**. In particular, the following should be checked:

a. Global stability of the liquid storage tank;
b. Yielding of the steel shell taking into account the special provisions given in NEN-EN 1998-4: Annex A.9 (for uplifting when required) as well as the provisions in NEN-EN 1998-4: Annex A.10;
c. Buckling in shear;
d. “Elephant foot” buckling and elastic buckling (Annex A.10);
e. Failure of the anchors (only for anchored tanks) and ductility checks;
f. Failure of foundation beneath the tanks;
g. Connections with adjusted piping and equipment based on the relative displacements to be expected;
h. Checking of wave sloshing height.

<u>Effects of soil liquefaction in the above verifications should be considered accordingly when relevant.</u>

Model II

When the FE model of the structure **does not** account explicitly for the dynamic motion of the liquid, i.e. a FE dynamic model of the coupled tank-liquid system is not available, then the modal response spectrum method of analysis can be carried out in accordance with:

a. NEN-EN 1998-1:2004;
b. NEN-EN 1998-4:2007. In particular, the specifications of section 4 hold for the liquid storage tanks. The dynamic pressures of the liquid at the inner side of the shell structure can be calculated according to NEN-EN 1998-4: Annex A. The calculation of the dynamic pressures to be applied at the inner side of the tank should account for the deformability of the shell structure for tanks composed of thin steel walls. For tanks made of thick concrete walls the deformability of the outer shell may be neglected.

For this analysis, the dynamic loads as a result of the liquid motion are applied as equivalent static loads at the inner side of the shell with predefined spatial distribution and an amplitude defined by the corresponding NLRS provided by KNMI. Subsequently, the seismic load cases are combined with the rest of the load cases relevant to the static situation. From that point on, the verification of the tank is done in accordance with standard practices commonly used in static design cases and the verification criteria are the same as described above.

Inclusion of higher order modes and modal truncation

Due to the specific shape of the seismic input provided by KNMI in which the constant acceleration plateau of the design spectrum is largely missing, it is advisable to account for more than one rigid impulsive and flexible modes in the seismic analysis, especially in those cases in which the first rigid and flexible impulsive modes are positioned at the right-side (at longer periods) of the peak of the acceleration response spectrum. In such cases, it can happen that the higher order modes, although of much smaller participation mass, can contribute significantly to the total response due to larger spectral accelerations assigned to them. Thus, it is highly recommended to always check the contribution of the higher modes prior to modal truncation. In general, since the constant acceleration branch is largely missing from the new seismic input, it is to be expected that the results will be highly sensitive to the period of the fundamental impulsive mode of the liquid. Given the uncertainty in the estimation of the latter, especially when considering the relatively soft soils in the region, it is highly advisable to always check the results with respect to this element to verify their validity.

Model III

When no suitable FE model of the structure is available, then the analysis can be carried out on the basis of the simplified methods as described in NEN-EN 1998-4: Annex A.3.2.2 **[7]**. Alternatively, the verification procedure can follow the provisions of the *API Standard 650: Eleventh Edition, June 2007* (Appendix E – Seismic design of storage tanks) taking into account the NLRS as provided by KNMI. However, it is recommended to investigate at least one case per facility in which a detailed FE model of the most critical elements is developed.

4.2 General remarks on structural analysis of liquid storage tanks

In all cases described previously the points described below need special attention.

Load cases

The seismic loads should be combined with the remaining load cases relevant to the accidental action effect (seismic action) in accordance with the relevant provisions of the Eurocodes.

Components of seismic input motions

When axisymmetric structures are analysed, i.e. cylindrical tanks, the earthquake load can be applied in one horizontal direction acting together with the vertical one. When two different horizontal input acceleration response spectra are provided at the same location, it is advisable to choose the most conservative one for the design, i.e. the one that results in the higher stresses at the structure. When non-axisymmetric structures are analysed all three components of the ground motion (two horizontal and one vertical) need to be considered. The superposition of the individual responses by applying the modal response spectrum method of analysis in the different directions shall be carried out in accordance with the combination rules provided in section 4.3.3.5 of NEN-EN 1998-1 considering always the most unfavourable combination rule. In contrast to other cases, it is advisable to consider always the vertical component of the seismic action in the design due to the shallow depth of the foci of the Groningen earthquakes. KNMI provides a method to obtain the NLRS in the vertical direction.

Soil-structure interaction effects

The presence of a soft soil beneath the foundation of the structures can shift the natural periods of the tank-fluid system to longer periods compared to the one supported by rigid ground. This should be considered accordingly in the design by:

a. using flexible connection to the ground, i.e. springs in both radial and tangential direction applied at the bottom of the tank, in the dynamic models of **Type I** described previously. The spring constants should be derived based on the dynamic soil properties of the upper soil layer(s) as provided by KNMI and for the correspondent strain level expected during the seismic event (should be also clarified by KNMI if requested); or
b. modifying the natural periods of the oscillations of the impulsive modes as described in NEN-EN 1998-4: Annex A.7 in the quasi-static models of **Type II** mentioned previously.

When models of **Type III** are used, the provisions of NEN-EN 1998-4: Annex A.7 apply as well.

Behaviour factor for the non-linear response of the structure

A behaviour factor of q=1.5 can be applied to obtain the pressures exerted by the impulsive modes of the liquid in accordance with NEN-EN 1998-4:2007 when **Type I** and **II** models are used. Higher values of the behaviour factor can also be applied under certain conditions, but when not sufficient data are available to justify this choice it is advisable **not to use values of the behaviour factor higher than q=1.5**[1]. When the full dynamic models of **Type I** are used, the corresponding behaviour factor should be applied only to the impulsive tank-fluid modes of the system prior to modal superposition and not to the ones corresponding to predominantly convective motions of the liquid.

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[1] According to section 2.4(2) of NEN-EN 1998-4:2007:
Use of q-factors greater than 1.5 in Ultimate Limit State (ULS) verifications is only allowed, provided that the sources of energy dissipation are explicitly identified and quantified and the capability of the structure to exploit them through appropriate detailing is demonstrated.

Reduction of NLRS provided by KNMI for q≠1

The reduction of the ordinates of the 5-percent damped NLRS with the behaviour factor q (to account for the nonlinear capacity of the structure) can be achieved as is customary for the various branches of the acceleration response spectrum. For the exact application of the q-factor, the provisions given in paragraph 3.2.2.5 of NEN EN1998-1:2004 (p.41-42) **[1]** can be used. More specifically, at the zero structural period there is no reduction of the elastic spectral ordinates. Then at the left and right corner of the constant acceleration plateau (if such a plateau exists) the elastic spectral ordinates can be divided by the q-factor. This procedure can then be repeated for longer structural periods as well. When a single peak exists in the NLRS (instead of a *constant acceleration plateau*), this peak can be reduced by the application of the q-factor in a similar manner.

Further investigation is recommended as an additional research item to determine the way of applying the behaviour factor q in cases where the inelastic response of the structural system needs to be considered together with the use of NLRS as derived by KNMI [2].

5. Seismic analysis of structures (advanced method)

For further research purposes an advanced analysis is suggested at this section. This is not part of the analysis covered in this GBoD. For such an analysis different input are required to represent the seismic action. In this section, we provide a general outline of the steps needed should such an analysis of this type be required in the future.

Seismic input required for advanced analysis methods

For an advanced method of analysis of the structures, the seismic input need to be modified in order to account for the soil-structure interaction effects. It is to be expected that the behaviour at the foundation level of the structure will be different when compared to the free-field motion. This is especially true when large structures (liquid storage tanks) are founded on relatively soft soil deposits as explained previously. In addition, the non-linear soil behaviour will be different as the stress distribution below the foundation level will change compared to the one obtained in the free-field. An advanced analysis of the structural systems should ideally account for:

i) Soil-structure interaction effects;
ii) Non-linear soil response (for seismic events that are expected to induce non-linear soil behaviour);
iii) Non-linear structural response (for seismic events that are expected to induce non-linear structural behaviour).

Modelling of the structure required for the advanced analysis methods

To be able to perform an advanced analysis the required seismic input should be provided in the form of appropriate time-histories of the seismic events to be expected under the correspondent limit state. For the ULS , this state corresponds to an earthquake for a predefined seismic hazard level in the area under investigation. The derived time histories should be provided at a reasonable depth below the structure, i.e. at such a depth that the effects of the structural response on the surrounding soil can be disregarded. The time histories should match the situation corresponding to a predefined seismic hazard at the chosen depth below the structure. The types of waves, wave propagation characteristics, and soil behaviour should match the conditions to be expected under the relevant circumstances. A suitable number of generated time histories should be provided to account for the inherent stochastic nature of earthquakes as described in NEN-EN 1998-1:2004. To properly account for the effect of soil-structure interaction the tank-liquid system has to be modelled together with the soil (or at least part of it), i.e. a tank-soil-liquid model is necessary. In this case, the classical modal response method of analysis cannot be applied and therefore suitable time histories (in terms of frequency content and amplitude characteristics) need to be provided at some level below the structure as described previously. The basic method of analysis in this case should be a non-linear time history (NLTH) analysis in accordance with NEN-EN 1998-1:2004 **[1]**. The verification criteria for global stability, yielding mechanisms and buckling failure modes remain the same as described previously.

6. References

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III

BIJLAGE: LESSONS'S LEARNED: 'LOC TOETS' IN APPLICATION TO THE INDUSTRIAL FACILITIES IN GRONINGEN [REF. 15]

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1. Summary

This document provides an overview of the most important "lessons learned" in the application of the LoC-toets during the period 2017-2018. Seismic verifications of 17 companies counting up to about 40 installations (so far) have been reported by engineering consultants and reviewed by TUD. The document reflects mainly observations on behalf of the reviewing party (TUD) and does not reflect any experience gathered by other parties which participated in the seismic verification.

The content of the document is primarily technical and focuses on two main aspects. First, a summary of the outcome of the seismic analyses in the 17 companies is given focusing solely on the most common failure modes reported per installation type. The results are reported anonymously in Tables 1 and 2 of Section 2, while no company information is revealed. Table 1 is followed by a brief summary of the main observations per type of structure analysed. This information is expected to be helpful for authorities deciding on a follow-up of the application of the LoC-toets. Second, a summary of the most common review comments is presented which reflects the items repeated several times in the review forms prepared by TUD (Section 3). This summary is meant to help improving the quality of the reports without losing unnecessary time in repeating the same comments. This information is expected to be helpful mainly for the engineers carrying out the bulk of the work in composing the *Basis of Design* documents and *calculation reports* for each installation.

2. Overview of the seismic analysis verifications

In Table 1 below, a summary of the most important installations analysed to date with the LoC-toets is given together with the cases which failed to satisfy the LoC-criteria.

Table 1: Summary of analysed objects with the LoC-toets and failures.[1]

Object	*Element*	*# Checked*	*# Failed*
Liquid storage tanks	**Total**	**11**	**4**
	Tank	11	0
	Support structure	10	1
	Foundation[2]	6	3
Buildings	**Total**	**10**	**9**
	Steel	9	4
	Concrete	6	1
	Masonry	5	5
	Foundation	9	2
Pipes supported by structural frame	**Total**	**10**	**1**
	Supporting frame	4	1
	Pipes	10	0
	Foundation	4	0
Column/chimney	**Total**	**2**	**0**
	Column	2	0
	Foundation	2	0

[1] The table content should be read as follows: per object type, the total number of analysed objects is first reported, and, subsequently, per element, the number of objects for which this element was verified. For example, 10 building objects were checked in total, of which 9 contained steel elements that were verified, of which 4 did not meet the LoC requirements. Note that in some objects, i.e. buildings, more than one structural elements are used, i.e. buildings may include both steel, concrete and masonry elements.

[2] The number of foundation elements checked per object may differ from the number of installations within the same category due to the fact that some objects are supported by the same foundation plate.

Table 1: Summary of analysed objects with the LoC-toets and failures (cont'ed)

Object	*Element*	*# Checked*	*# Failed*
Other	**Total**	**6**	**2**
	Secondary elements	2	0
	Racks	1	1
	Installations	3	1

By examining Table 1, the following observations are reported:

Liquid storage tanks

Most liquid storage tanks do satisfy the LoC-criteria. In the single reported case of a failure, not the tank itself, but the supporting structure failed at the welds of the connection with the foundation and due to buckling of the beam flange (case of an elevated tank supported by a steel frame). Given that, it seems that the tank structures are less prone to failure when compared to other types of structures in the region (at least based on the cases analysed so far). Although one should be careful with generalising this conclusion to tanks not yet being subjected to the LoC-toets (because every case is to some extent different), the reported cases so far give some degree of confidence that tank structures are less critical compared to other objects analysed[3]. This conclusion may be helpful in case one needs to take decisions aiming at prioritisation of objects to be analysed in the next phase.

Buildings

Most building structures show an LoC-scenario. Structures with masonry infills failed <u>in all the reported cases so far</u>, while the failures reported showed large unity checks both in- and especially out-of-plane (the latter substantially reduced when the NLKA method was applied but still critical). Given the fact that the LoC-toets has been developed with typical industrial facilities in mind which are primarily made of steel and reinforced concrete (check pilot studies reported in 2016[4]), this observation does not come as a surprise. For future application of the LoC-toets in <u>non-typical industrial installations containing unreinforced masonry</u>, the application of the NLKA method of analysis should be considered for seismic verification of unreinforced masonry (URM) elements to reduce some of the conservatism in EC8. However, even in this case one should still expect a significant fraction of the URM members to fail as the latter are known to be very weak in undertaking lateral deformations imposed by the seismic ground excitation. It needs to be mentioned here that URM elements were used primarily as infill walls in control/service buildings within industrial plants.

Next to URM, concrete elements failed only when used as floors of the building structure. Steel structure failures are (roughly) equally distributed among failure of members, of connections between members and of connections with the foundation.

Thus, we conclude that building structures are prone to several failure modes depending on the structural material used with structures containing URM infills being very critical.

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[3] At least in those cases in which the structures to be analysed fall within the typical tank structures supported by similar (to the analysed cases so far) soil conditions, one can be confident about the failures to be expected.

[4] Rapportage werkgroep Maatgevende aardbevingsbelasting voor de industrie: Naar een snelle, simpele, transparante en robuuste toets op de aardbevingsbestendigheid van de chemische industrie in Groningen (4-11-2016). https://www.rijksoverheid.nl/documenten/rapporten/2016/11/04/rapportage-werkgroep-maatgevende-aardbevingsbelasting-voor-de-industrie

Pipes (supported by a structural frame)

The verified pipes and pipe racks generally suffice in their current state. In one of the cases, an LoC situation was reported, due to failure of the steel frame of an adjacent structure at the location of connection with the studied pipe while the pipe itself showed sufficient capacity to undertake the seismic loads.

Columns / chimneys

Columns/chimneys met all LoC requirements in all studied cases so far. Given that, it seems that these types of structures are less prone to failure when compared to other types of structures (at least based on the limited number of cases analysed so far).

Foundations (relevant to all structures analysed)

LoC due to foundation failure occurs for different objects due to different failure mechanisms that vary between pile failure, pile carrying capacity during and after an earthquake, large displacements and failure of a concrete foundation slab. No clear pattern emerges from the reported cases so far.

Other (not categorised in the groups above)

The considered secondary elements do (usually) satisfy the LoC-criteria. The considered storage racks object failed for all considered failure mechanisms, except one. Installations were considered only regarding their connection to the supporting structure, which also failed in some reported cases.

In Table 2 below, a more detailed overview of the failure mechanisms is presented per structural typology.

Table 2: Extensive summary of the analysed objects with the LoC-toets and their common failure mechanisms. Elements that are marked with bold are the ones which are normative in the LoC seismic verifications.

Object	*Substructure*	*Element*	*# Checked*	*Failure [%]*	*Average u.c.**
Tank	Tank	Wall	11	0.0	0.45
		Bottom	1	0.0	0.04
	Support structure	Support structure	10	10.0	0.46
		Connections tank-structure	4	0.0	0.57
		Connection structure-foundation	10	0.0	0.37
Building	Steel structure	Steel members	9	**44.4**	1.78
		Internal connections	5	40.0	0.78
		Connections structure-foundation	5	20.0	0.83
		Connections to concrete members	3	**33.3**	0.83
	Concrete structure	Concrete members	4	25.0	0.83
		Concrete walls	1	0.0	-
		Concrete floors	4	25.0	0.51
		Connections structure-foundation	1	0.0	0.9
	Masonry	Masonry walls	5	**100.0**	14.91
Pipes	Pipe rack	Steel members	4	0.0	0.45
		Internal connections	3	0.0	0.24
		Connections structure-foundation	3	0.0	0.27
		Connections to adjacent structure	1	**100.0**	-
	Pipes	Pipes	10	0.0	0.38
		Connection pipe-column	4	0.0	0.14
		Column wall at pipe connection	4	0.0	0.37
Column	Column/chimney	Steel structure	1	0.0	0.79
		Steel column	2	0.0	0.72
		Connection structure-foundation	2	0.0	0.18
Foundation	Foundation	Concrete slab	12	8.3	0.65
		Concrete beams	6	0.0	0.63
		Connection pile-slab	1	0.0	0.99
		Pile moment	19	15.8	0.61
		Pile shear	8	0.0	0.39
		Pile carrying capacity during EQ	20	5.0	0.6
		Pile carrying capacity after EQ	19	5.3	0.57
		Carrying capacity during EQ (shallow)	1	**100.0**	1.2
		Carrying capacity after EQ (shallow)	1	0.0	0.74
		Pile compression and buckling	12	0.0	-
		Large displacements	14	14.3	-
Other	Secondary elmts.	Stairway	1	0.0	-
		Vestibule	1	0.0	-
	Racks	Steel members	1	0.0	0.9
		Internal connections	1	**100.0**	1.71
		Connections structure-foundation	1	**100.0**	1.2
		Stability of materials in racks	1	**100.0**	1.33
	Installations	Connections to structure	3	**33.3**	1.87

** Average based on calculated u.c.'s. Average is of maximum u.c.'s.*

3. Lessons learned – most common review comments

This section summarizes some important points of attention for the seismic assessment of selected objects in Groningen, The Netherlands. This seismic assessment is referred to as the LoC-toets" (Loss of Containment check) that has been described in Generic Basis of Design (GBoD) document.

The topics treated are divided into five sub-sections based on their relevance to different types of objects. In section 3.1, general remarks are addressed which are not *per se* related to specific types of structures. In the remaining sections, topics related to buildings, liquid storage tanks, storage racks and pipes are discussed. All topics are addressed in the following manner. First, a statement of the comment is given in *italics*. Subsequently, important points of attention are presented, or the suggested approach is explained.

3.1 General remarks

3.1.1 Modal truncation

The criterion stated in article 4.3.3.3.1(3) of NEN-EN1998-1 shall be used to determine the minimum number of modes to be included in the analysis; the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure activated in the correspondent direction.

Please note that the aforementioned rule considers the total base shear (which is proportional to the activated mass in the superimposed vibration modes) and does not per se guarantee the convergence in terms local stresses. Local modes hardly participate to the total base shear but might cause significant stress concentrations to the local members. Thus, in systems in which many local members (small beams etc.) contribute to the response, it is recommended to perform an additional check to confirm that local modes (which are found in the 10% remaining modal mass that is not accounted for if one adopts NEN-EN1998-1 alone) will not change the local stresses significantly. In that case the modal summation should be truncated based on the resulting unity checks rather than the modal mass. Please note that this criterion is meant supplementary to the one in NEN-EN1998-1; not as a substitution.

3.1.2 Non-structural elements located at high floors

Non-structural elements supported by the floors of the main structure are sometimes analysed using the maximum value of the standard 5% damped acceleration response spectrum stating this will yield conservative results. Please note that even though the use of the maximum spectral ordinate of the 5% damped response acceleration spectrum may yield an amplification factor of about 2.5 - 3.0 times the PGA at the location of interest, this does not per se guarantee a conservative approach since the amplification factor may be higher than the values mentioned above at the upper floors of a building. The explanation is given below together with the approaches that can be followed in the analysis of such elements.

Non-structural elements connected to higher floors of a supporting structure require extra attention. There are two analysis options to calculate this type of element:

1. Analytical calculation based on NEN-EN1998-1.
2. Numerical calculation based on an integral structural model.

Option 1: Non-structural elements can be checked according to NEN-EN1998-1 equation 4.24 and 4.25. These equations are the same as equations 1 and 2 below, respectively.

$$F_a = \frac{S_a W_a \gamma_a}{q_a} \tag{1}$$

where:

F_a	= Horizontal seismic force
S_a	= Seismic coefficient
W_a	= Weight of the element
γ_a	= Importance factor of element (=1.0 in the LoC-toets)
q_a	= Behaviour factor of the element (=1.5 in the LoC-toets)

$$S_a = \alpha S \left(\frac{3\left(1+\frac{z}{H}\right)}{1+\left(1-\frac{T_a}{T_1}\right)} - 0.5 \right) \tag{2}$$

where:

S_a	= Seismic coefficient
α	= Ratio between design PGA and acceleration of gravity g
S	= Soil factor (=1.0 in the LoC-toets)
T_a	= Fundamental vibration period of element
T_1	= Fundamental vibration period of supporting structure
z	= Height of the element above the level of application of the seismic action
H	= Height of the supporting structure above the level of application of the seismic action

It should be noted that the soil factor and importance factor should be taken equal to 1.0.
In addition, it should be noted that Eq.(2) above may result in a maximum amplification factor of 5.5 (if z=H and $T_a = T_1$). This is significantly larger than the maximum amplification factor of the standard 5% damped response spectrum in most of the cases.

Option 2: The integral structural model should contain both the supporting structure and non-structural element(s).

3.1.3 Connected structures

The selected object for the LoC-toets is connected to another structure that has not been selected for the LoC-toets. This connected structure has not been addressed in the report.

The other structure (not explicitly modelled for the purposes of the LoC-toets) may significantly influence the dynamic response of the selected object (for the LoC-verifications) depending on the type of connection and the mass of the other structure. Therefore, this issue should always be properly addressed in the report. In some cases, it can be argued that the influence is negligible. If this is not the case, it might be required to include the other structure in the structural model or apply appropriate dynamic boundary conditions which effectively substitute the actual structural system considered for the seismic analysis. The procedure that is followed for the derivation of the proper dynamic boundary conditions, replacing structures not explicitly modelled, should be clearly explained in the report.

3.1.4 Behaviour factor/redistribution

The internal forces in the structure should be determined based on the design response spectrum with behaviour factor q = 1.5. Subsequently, these forces are compared to the capacity of the structure and a conclusion is drawn with regard to failure of the structure or not. In some cases, when an element proves to fail, the consultant considers redistribution of forces to show that the "real" forces will not be as high as calculated.

Certain reasoning based on redistribution of forces is not valid in combination with the design response spectrum (q = 1.5) as material overstrength and plasticity (although limited) is taken into account by the use of a behaviour factor larger than 1.0. Using the design response spectrum with a behaviour factor q = 1.5, we anticipate that:

- (limited) non-linear effects are indirectly taken into account by the behaviour factor;
- the calculated internal forces are not equal to the "actual" forces in the construction.

Thus, we can only conclude one of the following:

I. when u.c. < 1.0, the system may yield but its plastic deformation stays within acceptable limits, then the "LoC-toets" is satisfied. In other words, displacement ductility demand is lower than the displacement ductility capacity.

II. when u.c. > 1.0, the system yields and its plastic deformations are unacceptable, then the "LoC-toets" is not satisfied. In other words, displacement ductility demand is higher than the displacement ductility capacity.

Conclusions, other than ones mentioned above, are not compatible with the method of analysis considered in the LoC-toets as described in the GBoD.

3.2 Buildings

3.2.1 Masonry wall loaded out of plane

The LoC-toets is meant as a simple and quick check, thereby excluding complex non-linear FE analysis. One simple method, that is only based on non-linear analysis, is the NLKA method that is used to verify masonry walls loaded out-of-plane.

To verify the structural capacity of masonry walls the NLKA method from the New Zealand building code should be used. This method is explained in NPR 9998:2017 appendix H, but it is preferred to use the New Zealand code on seismic assessment of existing buildings, part C8, as the NPR 9998:2017 contains a number of errors.

The procedure of verifying masonry walls then consists of the following steps (which can be applied in the order mentioned below):

1. Perform an MRSA of entire building including masonry and verify the assumed connection types (hinged, moment resisting, etc.).
2. Linear elastic calculation according to paragraph 6.3 of NEN-EN 1996-1-1. If the structure satisfies the LoC-criteria by applying the NEN-EN 1996-1-1 approach, then no further investigation is required.
3. *If* masonry fails in step 2 (insufficient seismic capacity according to NEN-EN 1996-1-1), then the NLKA method shall be considered.

Masonry structures that do satisfy the criteria set in step 2 are, in principle, superior to the ones that satisfy NLKA-analysis criteria alone. However, the satisfaction of the latter is a minimum requirement for the purposes of the LoC-toets.

3.2.2 Inter-storey drift

For multi-storey buildings, the inter-storey drift should be checked in order to judge whether second-order effects need to be taken into consideration. Since the description in NEN-EN1998-1 4.4.2.2 is not explicit, the method of determination of the inter-storey drift should be explained.

To calculate the inter-storey drift, the correct method is to calculate the inter-storey drift for each floor and for each mode and, subsequently, to apply a statistical combination rule (e.g. SRSS or CQC) to all relevant modes to find the total inter-storey drift to be expected. It is simply incorrect to first statistically combine the different modes to find floor displacement and to calculate the inter-storey drift from these resulting displacements at the floor level. The same holds for the calculation of other physical quantities, i.e. member stresses.

3.3 Liquid storage tanks

3.3.1 General remark

For the purpose of the LoC-toets, and specifically for the seismic verification of liquid storage tanks, a generalised document has been composed that shall be followed as the main guideline. The document is entitled:

"Generic approach liquid storage tanks: General approach liquid storage tanks for the seismic verification of industrial facilities in Groningen."

The document is in line with the GBoD dcocument. When structures satisfy the specifications described in the document above, the procedures described therein need to be followed.

3.3.2 Convective fluid mass

Different guidelines for tank verification may be used, e.g. Eurocode, API 650 or IITK-GSDMA. It is noticed that the Indian guideline gives a different, more conservative, multiplier of 1.673 to obtain the 0.5% damping spectrum compared to equation 3.6 of NEN-EN1998-1.

The Eurocode is always normative. Therefore, equation 3.6 of NEN-EN1998-1 should be used to obtain the 0.5% damping spectrum that can be applied to the convective fluid mass.

3.3.3 Combination rule

Different guidelines for tank verification might be used (e.g. API 650 or IITK-GSDMA, Guidelines for seismic design of liquid storage tanks). It is noticed that API 650 annex E uses the less conservative SRSS rule for the combination of the impulsive and convective modes.

In case the guidelines deviate from the Eurocode, the latter is always normative. The contribution of the impulsive and convective modes in the physical quantity of interest, e.g. shear force or overturning moment, should be determined using the absolute summation (ABSSUM) rule as stated in NEN-EN1998-4 annex A.2.1.6.

3.3.4 Verification of shell courses

The structural verification of steel liquid storage tanks includes among others: shell buckling, yielding and rupture of tank bottom and failure of connection (nozzle) between tank shell and in- or outgoing piping.

<u>*Local buckling*</u>

It is noticed that the simplified method in the API 650 is applicable to tanks of uniform wall thickness, which have a flat bottom and are supported at grade. The NEN-EN1998-4 provides guidance for the simplified analysis of elevated and horizontal tanks.

For tanks with <u>non-uniform wall thickness</u> a buckling verification of all shell courses is required (i.e. also the upper shell courses which are of reduced wall thickness). To verify the buckling of these shell elements, the pressure distributions can be obtained directly from NEN-EN1998-4: Annex A.2 and then integrated at the correspondent height to obtain the shear force and bending moment at that level. Subsequently, all buckling verifications can be carried out as customary for the shell course at the correspondent height given its own thickness.

This is required for the following reason: the verification of local buckling only for the bottom shell course cannot exclude the possibility that an upper shell course (of reduced wall thickness) buckles, i.e. when the wall thickness reduces with height, upper shell courses may buckle while the bottom one remains intact. This is related to the fact that the hydrodynamic pressures have a non-linear distribution with height (in contrast to the hydrostatic pressure) and, when combined with the reduced wall thickness, may cause buckling of the upper shell courses.

Connections of in- and outgoing piping

The maximum relative displacement that the connection (between the pipe and the structure) needs to accommodate to cope with the seismic loading, shall be verified against the capacity of that connection to undergo such a deformation (this data should be provided by the client or traced back for the type of connection used if this is a standardized one). A simple verification of the deformation requirement of a shell-nozzle connection is given in article 4.5.2.3 of NEN-EN1998-4. The formula in the Eurocode gives a minimum lower bound of relative displacement that needs to be considered which means that in case a more detailed calculation yields a lower value, the minimum value (Eurocode) has to be adopted; otherwise the value taken from the detailed calculation shall be used as is. This simple verification shall be minimally performed in the "LoC-toets", and the engineer should judge whether this verification suffices or that a more detailed model is required.
If the shell-nozzle connection does not meet the requirement of NEN-EN1998-4 (article 4.5.2.3), a more detailed analysis should be performed using FEM software that has been specially designed to verify these connections. Two cases should be verified:

- The forces or imposed deformations from the tank on the shell-nozzle connection.
- A MRSA of the piping system with the tank modelled as e.g. a clamped support in order to evaluate inertia forces from the piping itself.

All the above requirements are included in the document:
"Generic approach liquid storage tanks: General approach liquid storage tanks for the seismic verification of industrial facilities in Groningen."

3.4 Storage racks

3.4.1 Mass distribution

The mass in storage racks can be distributed in different configurations over the racks, both in horizontal and vertical direction. While different distributions in vertical direction are often considered, the mass in the analyses is often not varied in the horizontal direction.

Different filling configurations should be considered in the vertical as well as in the horizontal direction. An asymmetric filling in the horizontal direction can result in large torsional moments and therefore this might be governing.

3.4.2 Local failure of the containment

Objects stored in the storage racks can slip or tilt and fall out of the storage rack. This should be considered as an LoC-situation.

The analysis of a storage rack should contain checks on whether the stored containments slip or tilt. Both are easily calculated, the former assuming a realistic friction coefficient. Please also refer to section 3.1.2 for more information.

3.4.3 Pounding

Pounding occurs when a storage rack comes into contact with objects surrounding it. While in some cases storage racks are located so far from surrounding objects that a formal check is unnecessary, the consideration for pounding must always be mentioned, as well as a possible explanation for not considering it formally.

Storage racks should be checked for pounding. If pounding occurs, the structure should be considered to be in a LoC-situation, as the analysis of the subsequent behaviour of the storage rack is too complex for the current stage of structural verification. Please note that the value of the q-factor for calculating displacements should be taken as 1.0.

3.5 Pipes

Two approaches can be applied to the calculation of supported pipe lines. These approaches are explained in "General procedure for checking the above-ground pipes supported by a structural frame based on the provisions of EN1998-4:2007".

In addition to the document mentioned above, it is advised to make use of the following procedure:

- **Step 1:** Calculate the seismically induced stresses in the system by using a structural software package (e.g. SCIA) and by considering an integral model including the support structure of the pipeline. The model of the pipe in the structural software should be such that accurately describes the dynamic behaviour of the system.
- **Step 2:** Calculate the stresses resulting from temperature loads and internal pressure in a dedicated pipeline software package (e.g. CAESAR II).
- **Step 3:** Combine the results of steps 1 and 2 above to determine the final stresses. This approach is equivalent to option 1 mentioned in the document *"General procedure for checking the above-ground pipes supported by a structural frame based on the provisions of EN1998-4:2007"*.

Make sure that in the calculation procedure above, and mainly in steps 1 and 2, no loads are accounted for twice in the verification procedure.

Next to the comments above, when one is interested in the calculation of the development of actual stresses and/or displacements in the pipe (or any other structure), the q-factor should always be set equal to 1.0 in the definition of the seismic action. When calculations with q=1.5 are considered, the conclusions that one can reach are the ones mentioned under 3.1.4 above; this means that with q=1.5 no further conclusion can be drawn as to i) the actual ("true") stresses developing (below the yield stress) in the system; or ii) the true dynamic displacements expected.

IV

BIJLAGE: GENERAL PROCEDURE FOR CHECKING THE ABOVE-GROUND PIPELINES SUPPORTED BY A STRUCTURAL FRAME BASED ON THE PROVISIONS OF EN1998-4:2007 [REF. 17]

General procedure for checking the above-ground pipelines supported by a structural frame based on the provisions of EN1998-4:2007

For the modelling of the pipelines supported by a structural frame two options are possible:

- **Option 1**: *Detailed model of the structure and the pipeline in a single software*

Modelling of both the structure and the pipeline in one software package in which both structural and pipeline checks are performed in detail. This method is the preferred option in terms of accuracy since all calculations are performed in a single step. However, this would require a detailed model of both the structural components and the pipe in a single software package (CAESAR II for example) together with the strength verification checks for both the supporting structure and the pipes. Since this option may not be feasible in the short term, the second option below may be considered.

- **Option 2**: *Detailed model of the structure and the pipeline in separate software packages*

Modelling of the structure including a simplified model of the pipeline which is representative of the mass and stiffness distribution of the actual structure, i.e. a beam element representing the pipe, in a structural software package in order to perform the structural verifications of the supporting frame. Subsequently, the verification of the pipe can take place in a separate module (suitable for a detailed model and checks of the pipe alone) by the pipeline engineer using input data provided by the structural engineer. The first part of the analysis, i.e. the structural verifications of the supporting frame, is clear and shall be carried out as customary by the structural/geotechnical engineer. To be able to carry out the second part of the analysis, i.e. the verifications of the pipe, the procedure described below can be followed.

The calculation of the stresses and displacement in the above-ground pipeline should be based on the provisions of EN1998-4 (section 5). According to this standard the following seismic actions are relevant:

1. <u>Load type 1</u>: Movement of the pipeline due to the own inertia of the pipeline body activated by the acceleration imposed at the support locations;
2. <u>Load type 2</u>: Differential movement of the supports of the pipelines.

Regarding load type 2 mentioned above, the differential movement of the supports can be provided to the pipeline engineer <u>per vibration mode</u> as calculated by the structural model after applying the MRSA method. The modal displacements constitute the load type 2 mentioned above (stresses induced by the differential movement of the support locations). Subsequently, the pipeline engineer should:

i) apply the given displacements per mode in the pipe at the support locations;
ii) determine the internal stresses in the pipe as a result of the deformation pattern caused by the application of the displacements from (i) per mode; and

iii) combine the deformation patterns using usual modal combination rules (SRSS, CQA etc.) to determine the resulting stresses in the pipe as a result of the differential movement at the support locations.

Enough modes should be considered to reach convergence regarding the stresses in the pipeline caused by the differential motion at the support locations. **This procedure captures the part of the load defined in EN1998-4: section 5.3.3(2).**

Regarding load type 1 mentioned above, the inertia load represents the part of the load that is not captured by the differential motion of the support locations as defined above. In fact this load case is relevant for those modes of vibration for which the differential displacement of the supports is almost zero (same direction and same amplitude of support movement) so that the bending of the pipe (displacement of the nodes in between) is not captured correctly. This extra load case aims to capture the bending of the pipeline in between the supports caused by a uniform acceleration of the body of the pipe in the relevant direction[1]. To simulate this effect one can consider the following:

i) for the definition of a suitable horizontal load to capture the inertia of the pipe in the horizontal direction, the provisions of 4.3.5.2 of EN1998-1 can be applied;
ii) for the definition of a suitable vertical load to capture the inertia effects in the vertical direction - which can be considerable in the case of Groningen earthquakes- the inertia load generated by a uniform motion at all support locations in the vertical direction shall be considered. In the absence of more detailed calculations to determine the vertical acceleration of the whole system at the support locations of the pipeline, a uniform vertical load can be considered which is based on the maximum (spectral) acceleration obtained by the vertical response spectrum applied at all support locations. This is a conservative assumption as the first (predominantly vertical) eigenperiod of the coupled system can be located away from the peak value of the vertical acceleration response spectrum.

The load type 1 shall be applied in such a way that the combination with load type 2 (differential movement at the support locations) is always conservative[2]. To achieve this, the

[1] This load case can be altogether avoided if one can define in the pipeline software package fictitious nodes in between the support locations and provide the full displacement pattern obtained by the structural package instead of the displacements at the support locations only (provide displacement input per mode in both the support locations and the fictitious nodes in between). This way the vibration modes can be reproduced more accurately and there is no need to consider additional load cases of type 1. In fact, one should always keep in mind that the deformation patterns obtained in the structural software package include the inertia effects of the pipeline. The only reason why one need to consider the inertia effects here is the fact that the displacements are provided at the support locations only (per vibration mode) and not as continuous functions along the length of the pipeline and thus some information on the pipe deformation may be lost as explained above.

[2] If the superposition of load type 1 and load type 2 yields results that are less conservative compared to the application of load type 2 alone, then the verification should be based only on the stresses obtained by the load type 2. This is advised here since load type 1 is added here to capture possible deformation patterns of the pipes that are not captured by the differential movements at the support locations.

generation of multiple load cases may be required. **This procedure captures the part of the load defined in EN1998-4: section 5.3.2**[3].

Other code provisions:

A procedure based solely on the lateral force method (which is usually considered in practice based on other code provisions) cannot capture both effects described in EN 1998-4-section 5. In other words, the static equivalent analysis (lateral force method) of the pipeline supported by the steel frame is not directly applicable in this case for three main reasons:

i) Load type 2 (which in many cases can be the dominant one) is not captured at all when all the supports are assumed to move in-phase and subjected to a uniform acceleration. In reality, the MRSA method of the structure (including a simplistic model of the pipeline) may show that significant differential movements exist that need to be considered as they can result in significant stress concentrations in the pipes.

ii) The acceleration value applied at the level of the pipeline cannot be related directly to the Peak Ground Acceleration (PGA) in the relevant direction because the structure may amplify this ground motion significantly due to its own motion in both the horizontal and the vertical directions. For this reason, it is advisable to estimate the expected inertia load based on the provisions of 4.3.5.2 of EN1998-1 in the horizontal direction. By doing so, the (estimated) spectral acceleration is applied and not the horizontal PGA.

iii) The vertical load induced by the vertical ground motion in Groningen earthquakes can induce significant stresses in the pipe segment between the support locations. This load type needs to be considered in the verification of the pipelines. This load is inherently captured by the support movements (load-type 2 mentioned above). An additional load case is also considered to capture the case in which all supports points move in-phase due to the ground acceleration.

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[3] At the end of the process, one needs to check the deformations and forces at the interface between the structural model and the pipeline model (support locations) as these two should ideally converge to one and the same value. Apart from this, it is advisable to perform the following simple check when one adopts **option 2** (i.e. separate check of the pipes and the structure in two different software packages) for the analysis of pipelines supported by structural components. In those cases in which the resulting deformations at the support locations of the pipes per mode are all different (which means that they will cause bending in the pipe segments in between the supports) then imposing only the displacements at the supports per mode (load case 2) may be sufficient. This can be checked by comparing the deformation patterns per mode given in the structural software package with the deformation patterns obtained in the pipeline software package, i.e. CAESAR II, after imposing the displacements of a few modes at the support locations of the pipe. If we assume for example that one performs the structural analysis in SCIA and the pipeline analysis in CAESAR II, a comparison between the SCIA modes and the correspondent (per mode) CAESAR II deformation patterns will clearly show the degree of correspondence. This will already built up confidence in the results without the need to complicate the analysis with extra load cases (load type 1 given above). If the overall deformation patterns of the modes considered match in the two software packages, then the extra load case of type 1 (uniform inertia loads) can be omitted altogether and the analysis can be based on load type 2 alone. If the overall deformation patterns do not match, then one can consider adding the extra load cases of type 1 as described above.

Concluding remark:

The procedure described above can be applied to all pipelines supported by a structural frame or a building (or partially by a building and partially by a supporting frame) by proper consideration of the modes of the various interconnected structural systems. In those cases in which the inertia of the pipes is negligible (small secondary pipes) and the pipes move together with the supporting structural system, the analysis can be based on load type 2 alone as mentioned above (after completion of the structural analysis) since this will be sufficient to capture the induced stresses in the pipe accurately.

V

BIJLAGE: GENERIC APPROACH LIQUID STORAGE TANKS - GENERAL APPROACH LIQUID STORAGE TANKS FOR THE SEISMIC VERIFICATION OF INDUSTRIAL FACILITIES IN GRONINGEN [REF. 18]

Generic approach liquid storage tanks

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

23 April 2018

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1

INTRODUCTION AND STARTING POINTS

1.1 Introduction

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

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

1.2 Seismic verification method

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

1.3 Scope of document

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

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

---

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

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

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

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

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

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

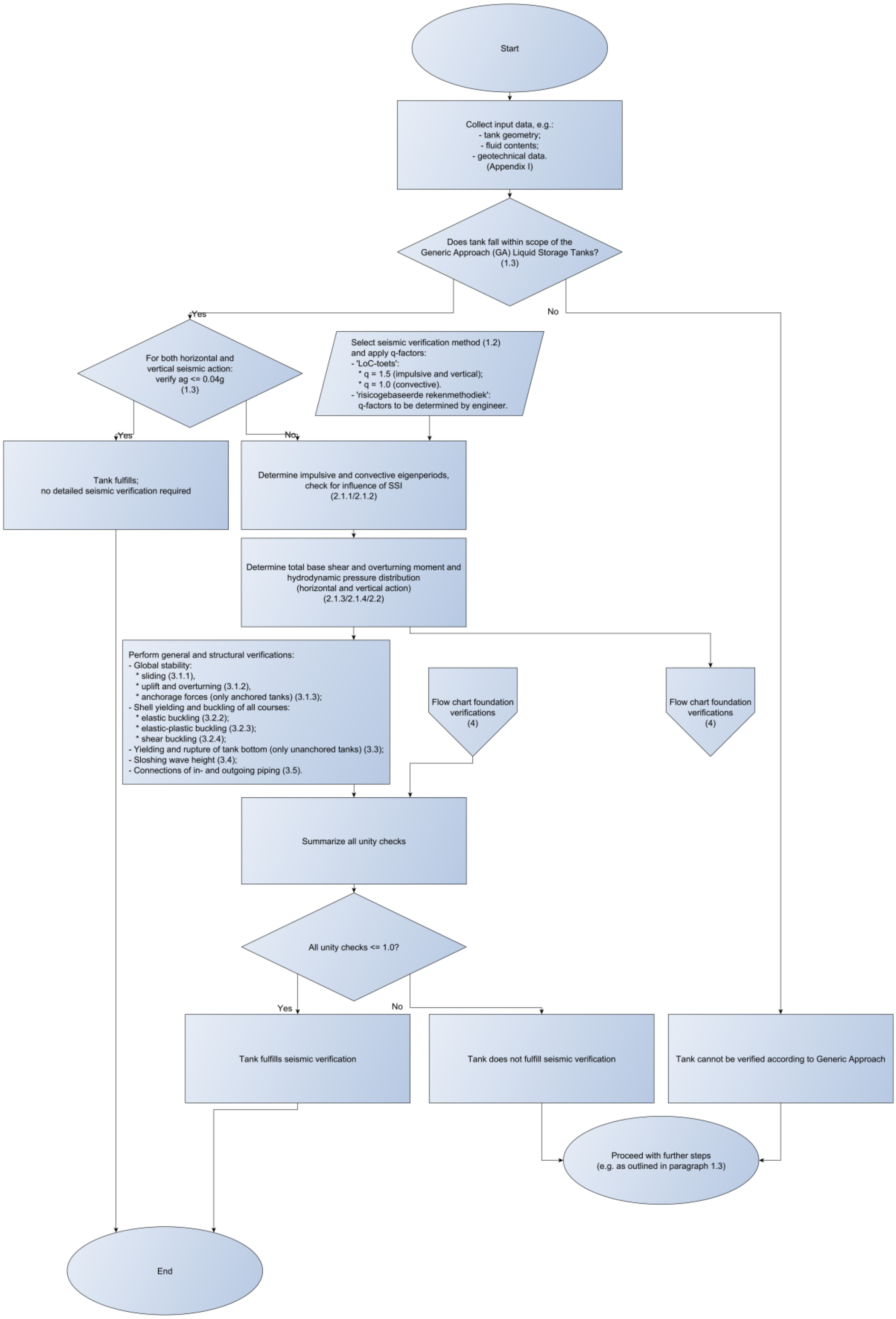
1.4 Flow chart

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

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

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

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



1.5 Standards, guidelines and other documents

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

General documents

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

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

Dutch standards and Eurocodes

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

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

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

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

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

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

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

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

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

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

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

Other standards, guidelines and literature

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

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

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

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

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

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

[20] Gazetas, G. (1983), Analysis of machine foundation vibrations: state of the art, Journal of Soil Dynamics and Earthquake Engineering, Vol. 2, No. 1.

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[22] Idriss, I.M., Boulanger, R.W. (2008), Soil liquefaction during earthquakes, Monograph EERI MNO 12, Earthquake Engineering Research Institute.

[23] Yoshimine, M., Nishizaki, H., Amano, K., Hosono, Y. (2006), Flow deformation of liquefied sand under constant shear load and its application to analysis of flow slide of finite slope, Soil Dynamics and Earthquake Engineering, Elsevier, 26: 253-264.

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

1.6 Failure mechanisms which are elaborated in this document

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

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

1.7 Fluid content volume

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

1.8 Example calculations

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

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

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

2

MODAL RESPONSE SPECTRUM METHOD OF ANALYSIS (MRSA)

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

2.1.1 Fundamental eigenperiod for tanks on rigid foundations

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

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

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

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

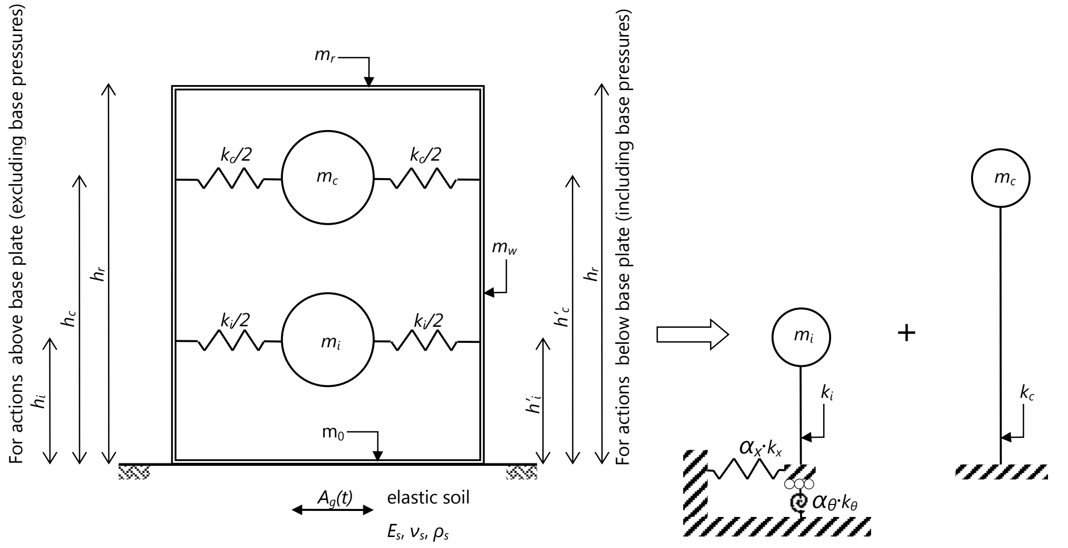
The simplified model as described in annex A.3.2.2 assumes a rigid foundation and therefore does not include the period elongation caused by soil-structure interaction (SSI). For Groningen, the soil conditions cannot be described as rigid and SSI period elongation shall be included to better define the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

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

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

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

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



2.1.3 Total base shear and overturning moment and combination rule

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

2.1.4 Hydrodynamic masses and pressure distributions

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

2.2 Vertical seismic action and combination rule

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

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

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

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

2.3 Behaviour factors

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

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

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

3

GENERAL AND STRUCTURAL VERIFICATIONS

3.1 Global stability of anchored and unanchored (uplifting) tanks

3.1.1 Sliding

Unless special measures against sliding have been taken, the sliding resistance should be calculated as the lowest coefficient of friction in the system times the acting vertical load. The vertical acceleration component shall be included in this verification. One is referred to equation 6.2 of NEN 9997-1 [12]:
$H_d \leq R_d + R_{p;d}$

in which:

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

3.1.2 Uplift and overturning

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

3.1.3 Anchorage forces in case of anchored tanks

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

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

3.2 Yielding and buckling of shell courses

3.2.1 Introduction

Types of shell buckling

Three types of shell buckling can be identified when evaluating liquid storage tanks:

- Elastic buckling.
- Elastic-plastic buckling ('elephant foot' buckling).
- Shear buckling.

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

Buckling verifications in higher shell courses

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

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

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

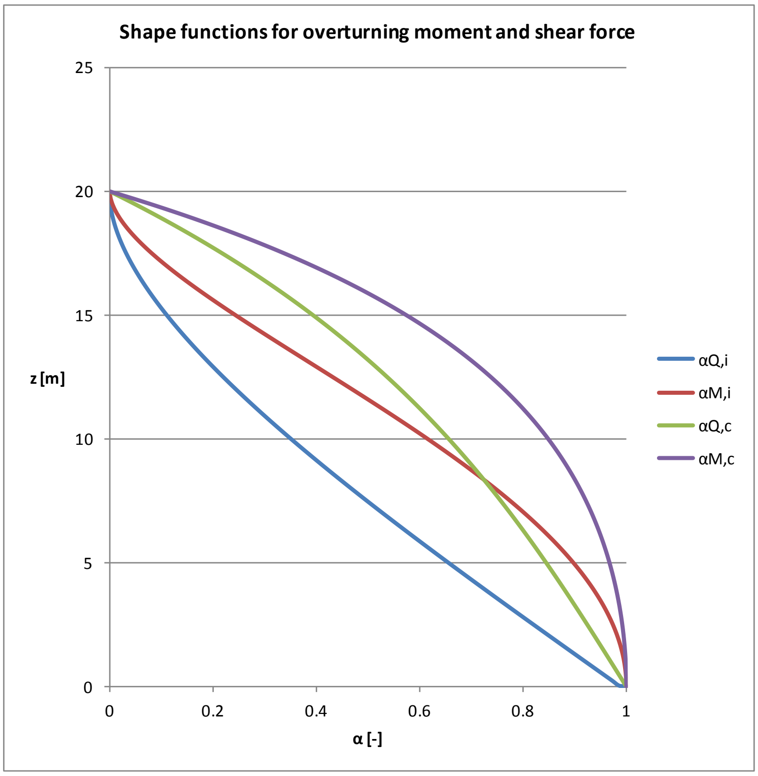
This also holds in the case in which the design of the wall thickness is based on the linear hydrostatic pressure (decrease linearly with height) since the latter is non-compliant with the highly non-linear hydrodynamic pressure distribution. Clearly, for the case of shell of constant wall thickness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

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

---

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

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



Shear and moment distributions along the height:

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

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

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

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

In which:

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

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

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

Partial factor on buckling γ_{M1}

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

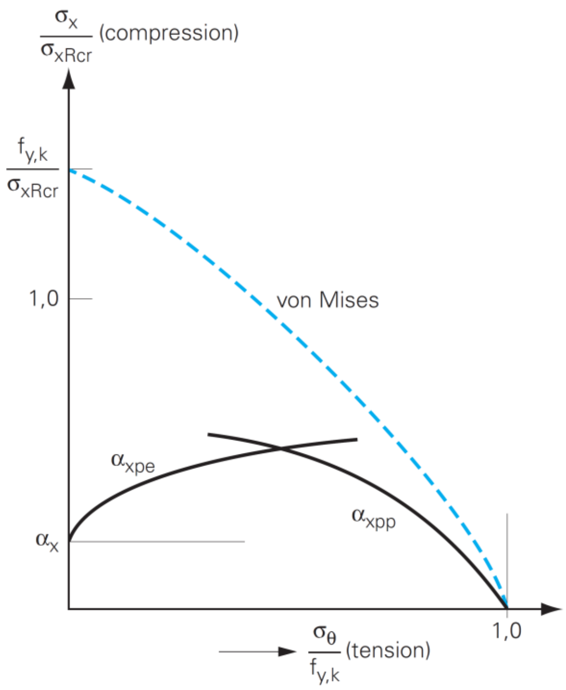
[1] As mentioned in annex D.2.2 of NEN-EN 1993-1-6

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

Effect of internal pressure and vertical seismic action on buckling verifications

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

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



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

In summary:

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

In which:

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

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

3.2.2 Verification of elastic buckling

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

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

3.2.3 Verification of elastic-plastic buckling

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

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

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

3.2.4 Verification of shear buckling

Eurocode provisions

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

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

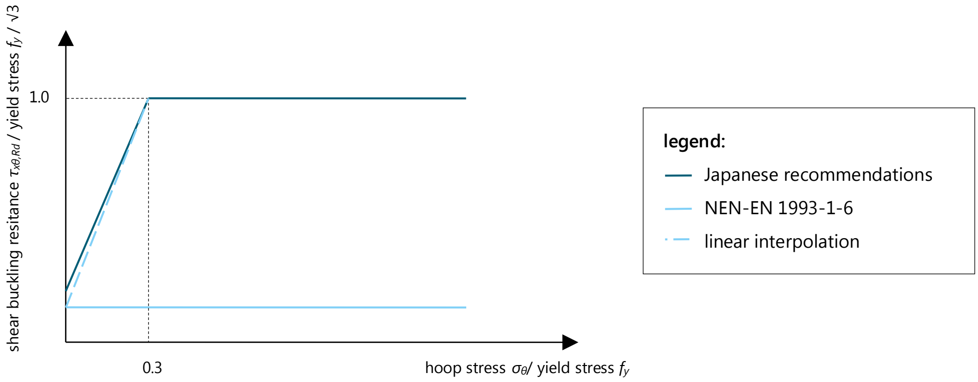
Japanese code provisions and shear buckling capacity

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

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

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

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



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

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

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

3.4 Sloshing wave height with respect to available freeboard

The tank should have sufficient available freeboard f to prevent overflow and/or damage to the roof structure. The freeboard freeboard f can be estimated by equation A.15 of NEN-EN 1998-4. Please note that this expression calculates the wave amplitude d and not the wave height H (which is twice the amplitude). Contributions from higher order modes can be calculated as follows:

$d_1 = 0.837 \cdot R \cdot S_e(T_{c1}) / g$

$d_2 = 0.073 \cdot R \cdot S_e(T_{c2}) / g$

$d_3 = 0.028 \cdot R \cdot S_e(T_{c3}) / g$

The unity check follows from:

Unity check = d_{max} / f

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

3.5 Connections of in- and outgoing piping

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

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

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

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

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

4

TANK FOUNDATION VERIFICATIONS

4.1 2-step calculation procedure

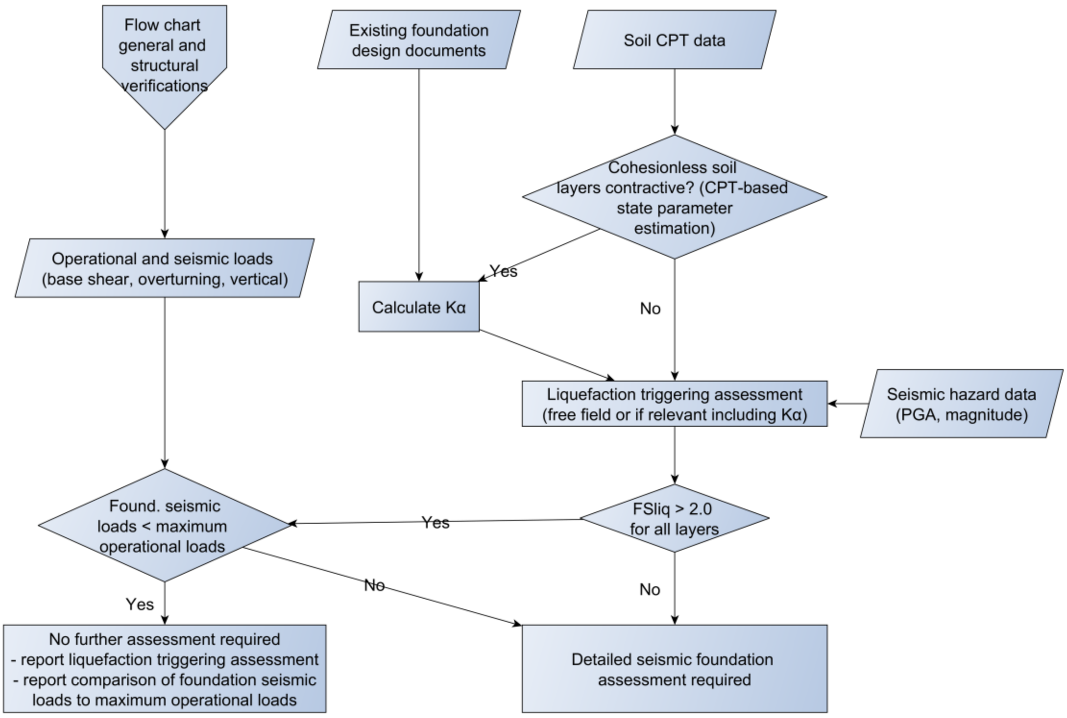
The assessment framework for liquid storage tank foundations in Groningen follows a 2-step calculation procedure. Both steps comprise quantitative calculations (not only screening) and are therefore sufficient to conclude on the expected LoC of a tank. The aim of a 2-step procedure is to limit the effort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

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

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

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

Figure 4.1 Flow chart for tank foundation assessment



4.2 Step 1: basic tank foundation calculations

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

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

4.2.1 Evaluation of foundation loads level

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

4.2.2 Liquefaction triggering assessment

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

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

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

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

4.3 Step 2: detailed tank foundation calculations

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

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

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

Table 4.1 Summary of foundation assessment methods

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

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

4.3.1 Rigid plate foundations

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

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

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

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

4.3.2 Tank pad foundations

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

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

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

4.3.3 Piled raft foundations

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

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

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

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

Appendices

I

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

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

General

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

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

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

VI

BIJLAGE: 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 [REF. 19]

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

Reference: BC7415I&BRP1809141342

Status: 0.3/Final

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Classification
Open

MANAGEMENT SYSTEM CERTIFICATION
DNV·GL
ISO 9001 = ISO 14001
ISO 45001

Royal HaskoningDHV

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

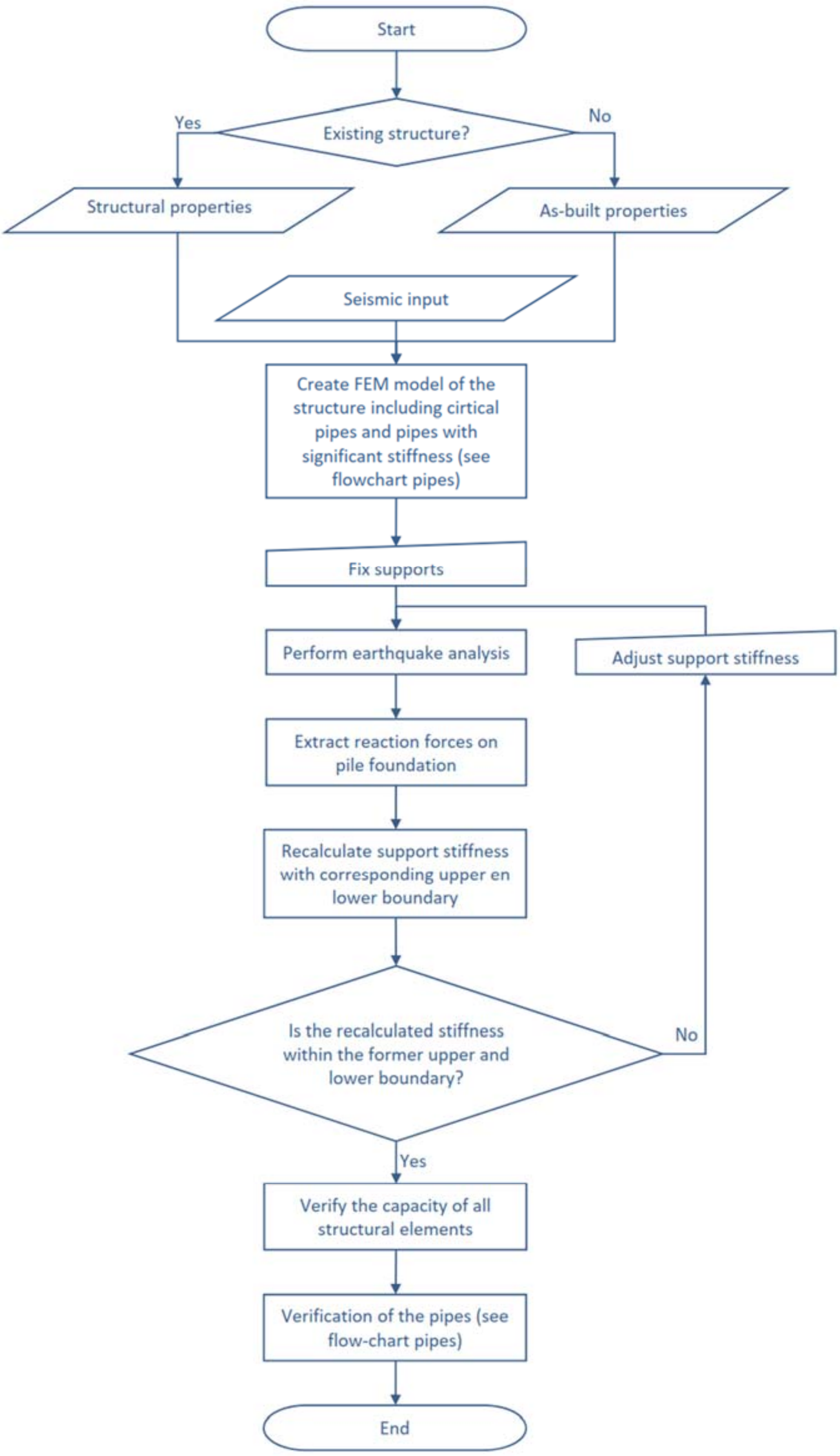

Figure 1-1: Work flow of the seismic verification of pipelines on steel structures with a pile foundation

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:

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

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).

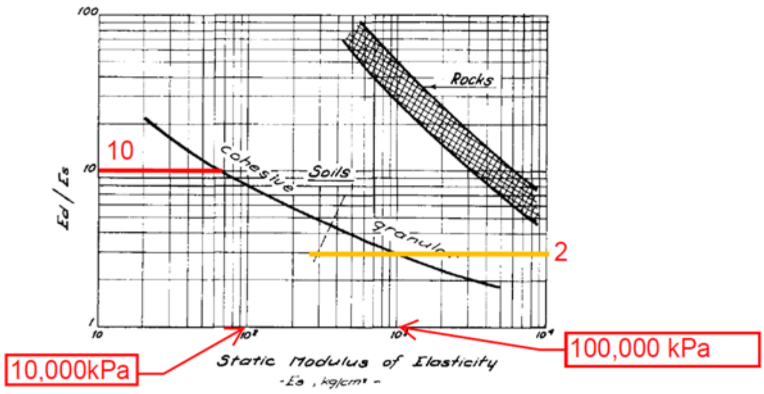

Fig.26. Dynamic and static moduli of elasticity.

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 $\gamma_L < 0.5$ use $r_{u;d} = 1$ (assuming full liquefaction);
     - If $0.5 < \gamma_L < 1.0$ determine the value for $r_{u;d}$ through interpolation in the range $0.5 \le r_{u;d} < 1$ (partial liquefaction);
     - In all other case, ($\gamma_L > 1.0$), 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.

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.

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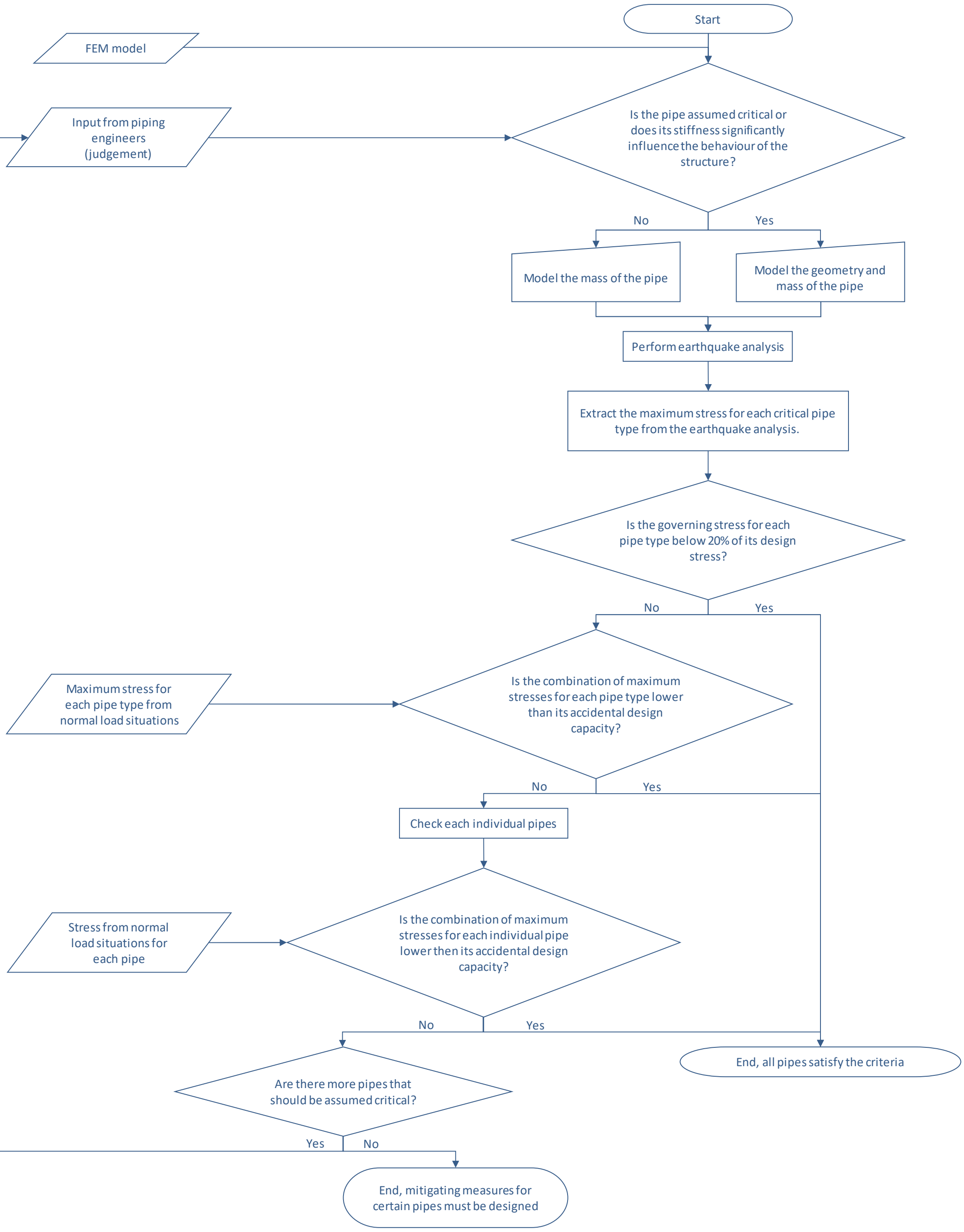

Figure 2: Work flow for pipe systems in case two separate models are used for the structural and pipe verifications

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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.

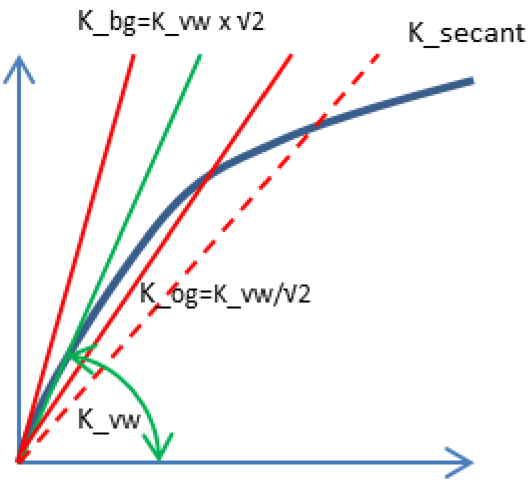

Figure 10-1 Definitions for the lower- and upper bound values for the spring stiffness

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).

VII

BIJLAGE: SEISMIC VERIFICATION OF FOUNDATIONS OF INDUSTRIAL ASSETS IN GRONINGEN - PERFORMANCE CRITERIA AND ASSESSMENT METHODS [REF. 20]

Seismic verification of foundations of industrial assets in Groningen

Performance criteria and assessment methods

Nationaal Coördinator Groningen

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1

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).

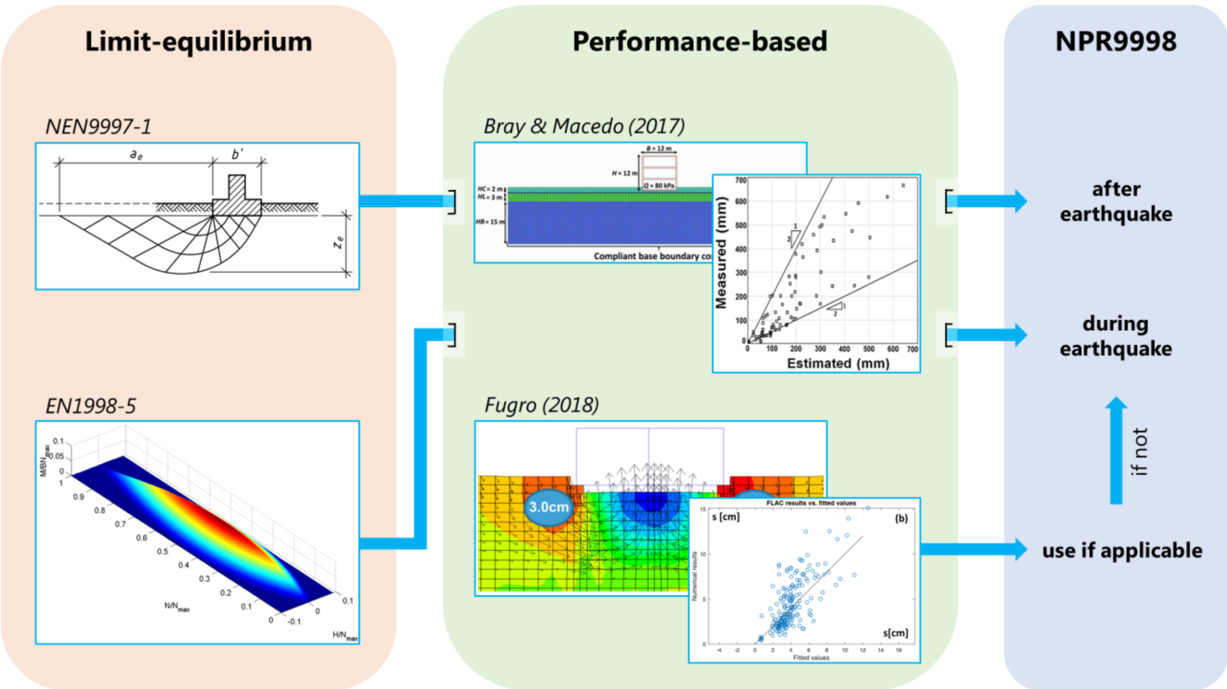
3

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 undrained 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:

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

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).

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

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:

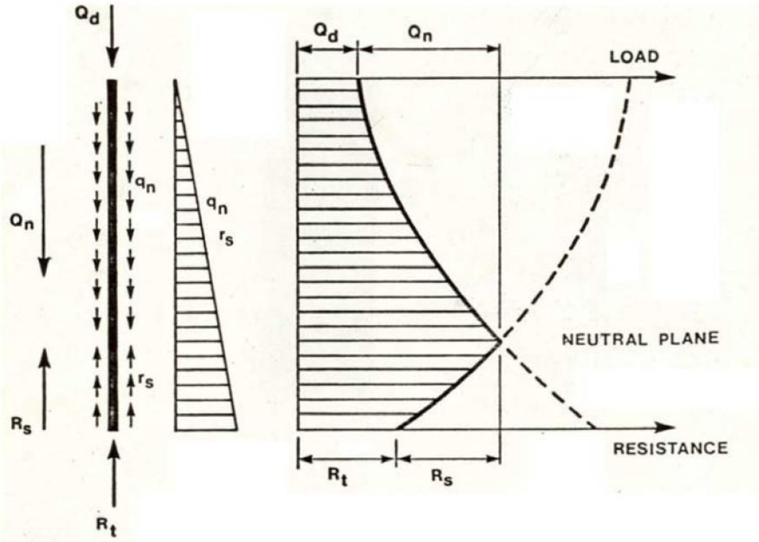
1. Is tension capacity of piles required for global overturning stability?
2. Are pile settlements due to liquefaction (GEO limit state) acceptable from a near-collapse perspective?
3. Is axial structural capacity of piles maintained for the design seismic load?

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}.

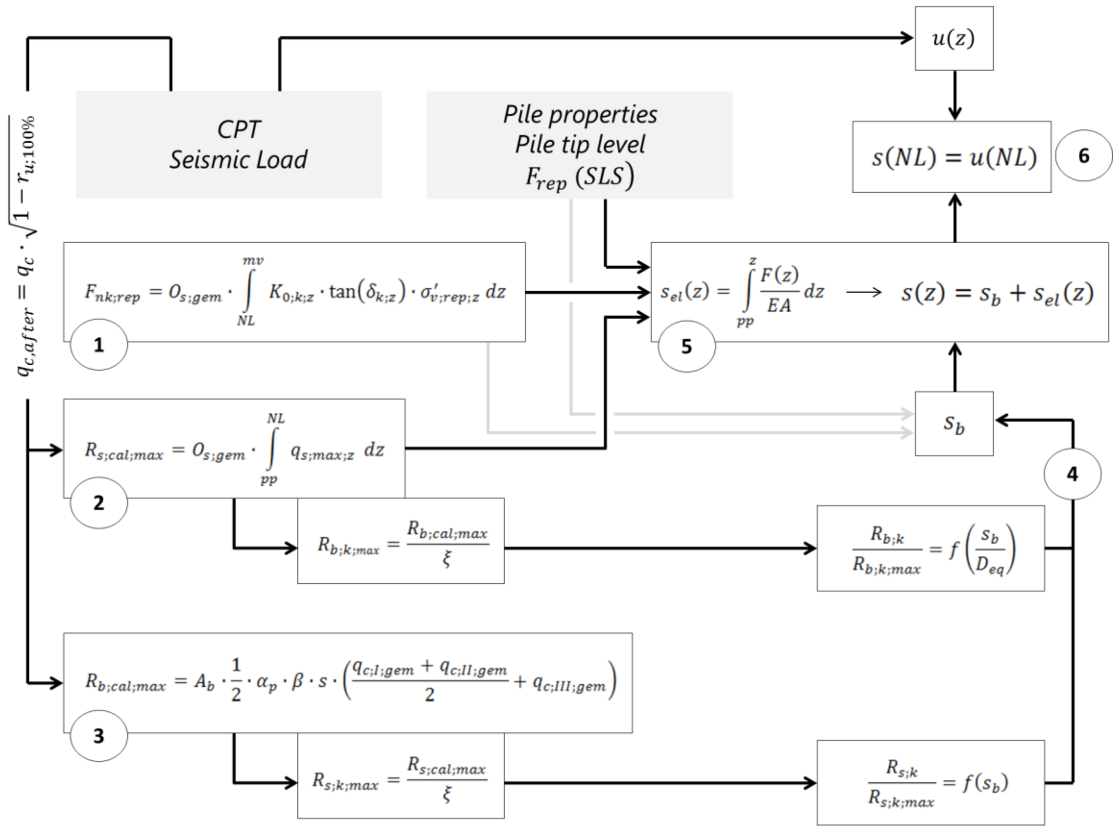
4.2.2 Pile settlement after the earthquake

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.

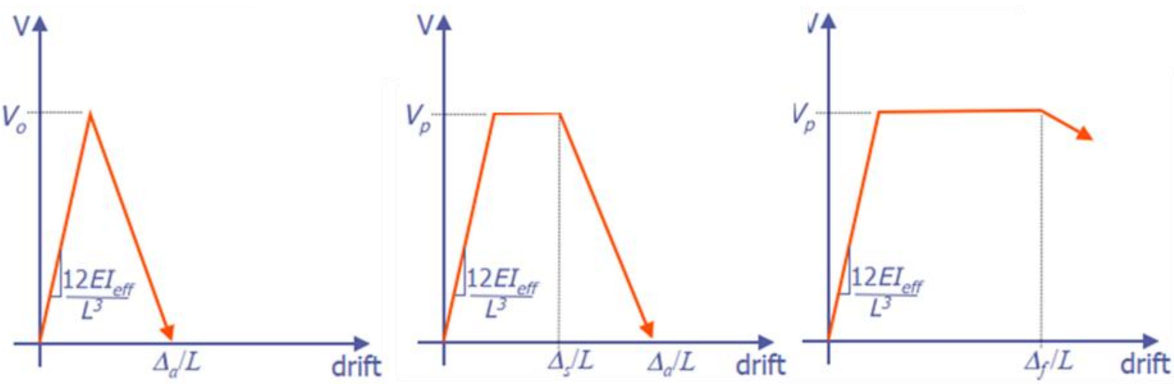
Insufficient axial residual load-bearing capacity of piles occurs if horizontal displacements become too large. NEN-NPR9998 does not set specific requirements for horizontal (permanent) displacements, other than the requirements for relative displacement of building storeys (inter-story drift limits). 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:

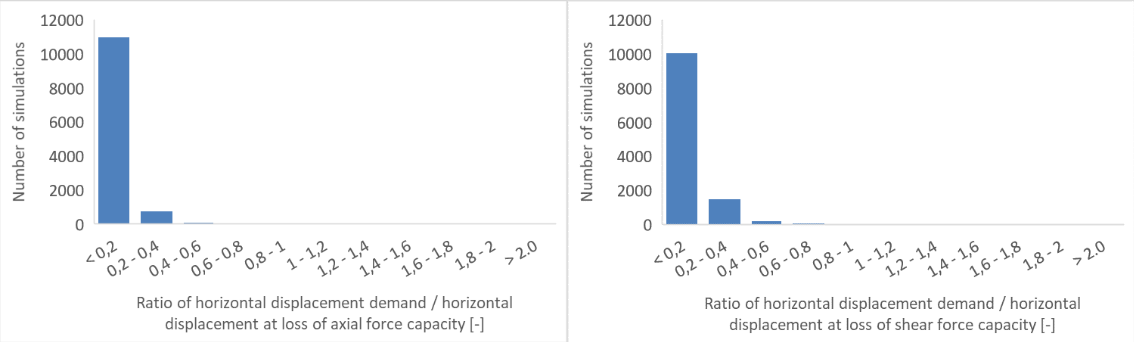
For a design seismic load level below 0.15 g for pile foundations of buildings in consequence classes CC1a and CC1b, only a GEO limit state verification is required. Verification of pile structural capacity is not required for under these conditions.

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