Nationaal Coördinator Groningen

Aardbevingen Groningen - Bouwkundig versterken

Applicatiedocument Beoordeling Seismische Capaciteit (ABSC)

Version 1.4

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Preface

The intention of this Application Document Assessment Seismic Capacity, in Dutch: Applicatiedocument Beoordeling Seismische Capaciteit (ABSC), is to guide engineering contractors/ consultants in the process of producing uniform seismic assessments and upgrading measures for the building stock/structures in the Groningen area, in compliance with the NEN-NPR9998 and under assignment of Nationaal Coördinator Groningen (NCG).

This ABSC guides engineers in applying, interpreting and clarifying the NEN-NPR9998 to make reasonable decisions and assumptions in line with the mind-set of NEN-NPR9998. The document is intended to be read and applied in conjunction with the NEN-NPR9998 and therefore has been set up to directly correspond with the chapters, sections and numbering of the NEN-NPR9998, wherever possible.

It is NCG's intention to update the ABSC regularly and adjust the document based on the requirements of the engineering contractors/consultants working for NCG.

Contents

1 General

This document is the Applicatiedocument Beoordeling Seismische Capaciteit (ABSC) and is part of NCG's technical specifications for engineering contractors/consultants working for NCG on the Groningen seismic assessment and structural upgrading project.

"Engineering contractors/consultants" are those companies that have received a Purchase Order (opdrachtbrief) from NCG to provide services related to the seismic assessment and structural upgrading of existing buildings.

The ABSC aligns starting points on how existing buildings in the Groningen region of the Netherlands are to be seismically assessed in compliance with the NEN-NPR9998 [1]. Where buildings are non-compliant, they will have to be structurally upgraded in order to meet the specified seismic hazard levels for the region. This is documented in a "Technische Versterkingsadvies" (TVA).

The ABSC provides a decision framework on how engineering assumptions are to be made and implemented when the required input information is insufficient, ambiguous or unavailable.

In case the NEN-NPR9998, NCG's Vraagspecificatie (VS) and VA-template specification and/or other critical Dutch design guidance documents are updated this document will remain valid until a revised version of the ABSC is issued by NCG. All VA reports should comply with the vraagspecificatie version stated in the contract belonging to a specific scope of work.

Whenever in this document "*NEN-NPR9998"* is mentioned in relation to the seismicity in the Groningen area, it is in direct reference towards the latest version of the *'NEN-NPR9998 2020*(Published 4 January 2021) – *Praktijkrichtlijn – Beoordeling van de constructieve veiligheid van een gebouw bij nieuwbouw, verbouw en afkeuren – Geïnduceerde aardbevingen – Grondslagen, belastingen en weerstanden'.*

1.1 Subject and field of appliance

NEN-NPR9998, Section 1.1 provides an overview of the appliance and demarcation of the document. The following bullets clarify the scope of the ABSC.

- The ABSC is only applicable to general building structures (residential houses, apartments, schools, hospitals, elderly homes, shopping centres, town halls, administrative government buildings, general healthcare facilities, general religious buildings, agricultural buildings).
- Monumental buildings are to be treated on a case by case basis to be agreed with NCG in writing.
- This ABSC is not applicable for the following types of structures:

  Tunnels, road or railway bridges, buried or overhead utilities, roads, canals, dams, dikes, levies, jetties, power stations, industrial chimneys, petrochemical facilities, oil refineries, data centres, LNG terminals, oil platforms, pipelines, grandstands, temporary structures, silos, tanks, cranes and machinery, storage racks, mobile phone masts, wind turbines, nuclear facilities, military or naval or similar facilities, prisons, uninhabited farm buildings, structures not being a building, etc… (i.e. this is not an exhaustive list of exclusions)
- Garden walls, boundary walls or similar non-building structures are not part of assessments unless specifically requested by NCG in writing.
- In the decision if a building or building part with a use different from above should be assessed the actual use is decisive. Buildings that have the possibility of a (sustainable) stay of people for over two hours should be assessed. Refer to 'beslisboom afbakening inspecties bijgebouwen' on request available with NCG. Exclusion of building parts for the assessment and upgrading must always be done in agreement with NCG.
- Individual addresses are to be assessed as part of the smallest interconnected or touching structural unit which dynamically interact with each other. This grouping of addresses will be tagged with a Seismic Unit ID (SUID), see section 1.4.3 of this document. This is in addition to NEN-NPR9998, Section 1.1.2.5.

1.2 References

NEN-NPR9998, Section 1.2 refers to a list of (mandatory) Dutch codes and Dutch Guidelines which are relevant during seismic assessment and/ or structural design.

1.2.1 Dutch Codes (mandatory)

NEN-NPR9998 is to be used in conjunction with the basic structural building regulations in the Netherlands to which buildings need to conform to. Buildings that need to be upgraded seismically in order to satisfy NEN-NPR9998 will also need to satisfy the general requirements of the Dutch building regulations for the upgraded components only.

The NEN-EN 1998-series are not mandatory because Dutch National Annexes have not (yet) been developed for them. Still, these documents are considered useful guidance.

1.2.2 Use of International Codes and Standards

The following international codes may be used to supplement gaps in knowledge and approaches not currently accounted for in NEN-NPR9998:

1. NTC 2008 (2008) Decreto Ministeriale 14/1/2008. Norme tecniche per le costruzioni. Ministry of Infrastructures and Transportations. G.U. S.O. n.30 on 4/2/2008 (in Italian)
2. Circolare n. 617 del 2/02/2009: Istruzioni per l' applicazione delle Nuove Norme Tecniche per le costruzioni di cui al D.M. 14/01/2008, G.U. n. 47 del 26/2/2009 (in Italian)
3. ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers.
4. NZS 1170.5 (2004). Structural design actions – Part 5: Earthquake Actions – New Zealand. Including amendment from 2016.

1.3 Starting points

In addition to the assumptions, clarifications and exclusions in NEN-EN1990, NEN8700 and NEN-NPR9998, Section 1.3 the following applies:

- Any alterations to building elements that are relied upon for strength and stiffness (i.e. lateral and vertical stability) have an interactive impact on the existing structure. After any alterations, the structural performance of the existing building should not be less than prior to the alterations. For this reason, it should be clearly identified on plans and elevations of seismically upgraded buildings that these elements cannot be altered unless a new seismic assessment, in Dutch: Beoordeling Seismische Capaciteit (BSC) is undertaken according to the requirements of NEN-NPR9998.
- The assumptions in the engineering process are inevitable and the engineer is responsible to justify the assumptions accurately represent the reality(justification by additional analysis and calculation, evaluation of the impact of different choices on the assessment). Once a project is accepted by the engineer, it is the engineer's responsibility to deliver the TVA with a final conclusion that does not rely on unverified assumptions. It should be a key consideration of the engineer to communicate with the client from the beginning of the project according to the impact of the assumption as it may result in the return of the project or further investigations in the building.
- Critical assumptions must be documented in the TVA. After evaluation of the structure for possible different scenarios, the level of deviations(minor or major) for the near-collapse are observed and documented by the engineer. If the observed deviations are significant, change the need for seismic upgrading and further knowledge can not be obtained by any other means; consequently the minimum requirements for these should be listed, to be verified in situ. The issues should be recorded as well in the issue log appendix for traceability.
- Where assumed building elements are not found to be present after an inspection for which a BSC had already been performed, the BSC will need to be reconsidered and or the TVA updated in light of the missing items, unless the missing items are installed (upgrading measure).
- A BSC should account for the physical observations from the building such as poor status of structural components, damaged parts, or missing parts. If necessary, the structural upgrading of these parts (poor status, damaged, or missing) should be performed and costs should be determined in the cost calculation of the required measures.

1.4 Terms and definitions

The following definitions are additional to NEN-NPR9998, Section 1.4.

1.4.1 Global coordinate system

The coordinate system used in the Netherlands is the "Rijksdriehoeksstelsel" New (RD New) which in GIS is known as Spatial Reference ID (SRID) 28992. RD New is the standard for all official records in the Netherlands and all BAG records are supplied in RD New. RD New is a Cartesian 2D system, with axes easting and northing (X and Y, Latitude and Longitude) with the unit of measure being in meters.

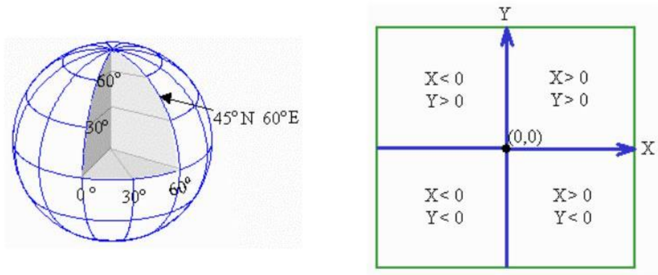


Figure 1: Geographic and projected global coordinates (X,Y)

1.4.2 Local coordinate system

For numerical models, a local tri-axial orthogonal coordinate system is to be generally applied, with two horizontal axes (x and y) and one vertical axis (z). For regular buildings in plan, and unless stated otherwise, the local x-axis is to be set parallel to the front and back façade of the buildings and the local y-axis is to be set parallel to the side façade or partition walls (in general, the front façade of the building is the façade that faces towards the street that determines the address of the building). The local z-axis is to be set for the vertical direction, where the positive z-axis is pointing upwards.

The local coordinate system has a relationship to the global RD New coordinate system defined by the rotation of the local building x axis to the global X axis and the local y axis to the global Y axis.

Figure 2: Relationship between local (x,y) and global (X,Y) axis

1.4.3 *Seismic Unit ID*

The Seismic Unit ID is the term used to describe the smallest interconnecting building block that would need to be assessed as one project. Buildings are assumed to be touching if the distance between adjacent buildings is less than 2,5% of the smaller height of adjacent structures.

When starting a new project, the assigned addresses are to be cross checked by the engineering contractor against the SUID. Should there be discrepancies between the allocation and the grouping of addresses within the SUID then this will be notified to NCG such that structurally meaningful units are assessed.

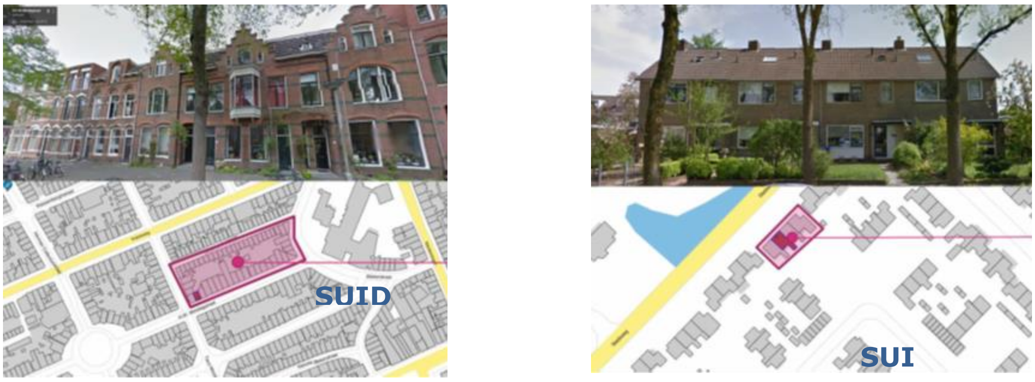


Figure 3: Touching addresses / buildings which would fall under the same SUID (two examples)

1.4.4 *Shared elements*

Shared elements that bridge across unconnected structures should be identified during inspections. Where shared elements exist the BSC needs to evaluate the impact of the shared elements to the dynamic behavior of the structures which are connected through the shared element(s). Local seismic upgrading measures may need to be considered for shared elements.

1.5 Symbols and Acronyms

The following list of Acronyms and list of Symbols are a repetition of or in addition to NEN-NPR9998, Section 1.5.

List of Acronyms:

ABSC	Applicatiedocument Beoordeling Seismische Capaciteit
ADRS	Acceleration Displacement Response Spectra
AHN	Actueel Hoogtebestand Nederland (current height map for the Netherlands)
API	American Petroleum Institute
ASCE	American Society of Civil Engineers
BAG	Basisregistratie Adressen en Gebouwen
BDWF	Beam on Dynamic Winkler Foundation
BEM	Boundary Element Method
BKCL	Building Knowledge Check List
BSC	Beoordeling Seismische Capaciteit (seismic capacity assessment)
CD	Concept Design
CDs	Construction Documents
CPT	Cone Penetration Test
CSM	Capacity Spectrum Method
DD	Detailed Design
DL	Dead Load
EN	European Norm
EVS	Extensive Visual Screening
EVS-2	Extensive Visual Screening 2
FEM	Finite Element Method
FEMA	Federal Emergency Management Agency
FOS	Factor of Safety
GIS	Geographic Information System
GSAT	Groningen Seismic Assessment Tool
I&E	Inspections & Engineering
IL	Imposed Load
IP	In-Plane (behavior of masonry)
IR	Individual Risk
KL	Knowledge Level
LD	Lateral Displacement
LDI	Lateral Displacement Index
LKA	Linear Kinematic Analysis
LFA	Lateral Force Analysis
LNG	Liquefied Natural Gas
MDOF	Multi Degree of Freedom
MEP	Mechanical, Electrical, Plumbing
MRSA	Modal Response Spectrum Analysis
NC	Near Collapse
NCG	Nationaal Coördinator Groningen (the organization and its Director)
NEN	Nederlandse Eenheidsnorm
NLKA	Non Linear Kinematic Analysis
NLPO	Non Linear Push Over
NLTH	Non Linear Time History
NPR	Nederlandse Praktijkrichtlijn

NTC	Norme Tecniche per le Costruzioni
NZS	New Zealand Standards
NZSEE	New Zealand Society for Earthquake Engineering
OCR	Over Consolidated Ratio
OOP	Out-of-Plane (behavior of masonry)
OSB	Oriented Strand Board
PC	Precast/Prefab Concrete
PCR	Peak Cracking Resistance
PI	Plasticity Index
PGA	Peak Ground Acceleration
PRBE	Potential Risk Building Element
RC	Reinforced Concrete
RD	Rijksdriehoek
RM	Reinforced Masonry
SA	Seismic Assessment
SD	Significant Damage
SDL	Superimposed Dead Load
SDOF	Single Degree of Freedom
SI	International System of Units
SLaMA	Simple Lateral Mechanism Analysis
SRA	Site Response Analysis
SRID	Spatial Reference System Identifier
SRSS	Square Root Sum of the Squares
SSI	Soil Structure Interaction
SUID	Seismic Unit ID
TO	Technisch Ontwerp (Technical Design / Detailed Design)
UDL	Uniformly Distributed Load
UO	Uitvoeringsgereed Ontwerp (Execution ready Design / Construction Design
URM	Unreinforced Masonry
VA	Versterkingsadvies
VS	Vraagspecificatie – Versterkingsadvies I&E programma
VO	Voorontwerp (Conceptual Design)
VWM	Virtual Work Method

List of symbols:

$a_{g;d}$	Design value of the peak ground acceleration
$a_{g;}S$	Value of the peak ground acceleration at surface level (including the soil factor)
$a_{g;ref}$	Reference peak ground acceleration
$a_{g;EO;NC}$	Peak ground acceleration at elastic outcrop (reference level) for the return period appropriate
$a_{g;EO;NC;n}$	Input ground acceleration for set n
d_t*	Target displacement for the equivalent SDOF system
d_{et}*	Target displacement of the structure with period T* and unlimited elastic behavior
g	Gravity
f_{ck}	Characteristic strength
f_{yk}	Yield strength
f_s	Sleeve friction
K	Pre-consolidation cone factor
M	Component demand modification factor
p-y	Lateral pile stiffness
p_a	Atmospheric pressure
q_a	Behavior factor for the non-structural item
q_t	Corrected cone tip resistance
q_{t1}	Normalized cone resistance
q_{t1net}	Normalized net tip resistance
q_u	Ductility
t-z	Skin friction pile stiffness
X	Local x-axis
Y	Local y-axis
Z	Local z-axis
z_{max}	Thickness of liquefiable layer
A_v	Vehicle footprint
CF_i	Confidence Factor "i" (See also Knowledge Factor ☐)
F_a	Horizontal seismic force acting on a non-structural element (appendage)
F_o	Maximum spectral amplification factor
G	Shear modulus
G_o	Small strain shear modulus
$G_{k;j}$	Characteristic value of permanent actions
G_v	Vehicle weight
H	Depth of the free face
K_{DR}	Aging factor
L	Distance between the free face and the examined location
N_{kt}	An empirical factor
$Q_{k;i}$	Characteristic value for variable actions
Q-z	End bearing stiffness
R_f	CPT friction ratio
S	Soil factor
S	Post-liquefaction settlement
S	Slope gradient
$S_a(T)$	Response spectrum (design spectrum) at the depth where the soil layers have a shear wave
S_D	Spectral displacement plateau
$S_e(T)$	Elastic horizontal ground acceleration response spectrum also called "elastic response
S_{MS}	Design values of the spectral accelerations for short periods
T	Vibration period of a linear single degree of freedom system

T^*	The period of the idealized equivalent SDOF system
T_a	Fundamental period of vibration of a non-structural element (appendage)
T_1	Fundamental period of the building in the horizontal direction of interest
T_B	Numerical value of the lower limit of the vibration period for which the spectral acceleration
T_C	Numerical value of the upper limit of the vibration period for which the spectral acceleration
V_s	Shear wave velocity
W_a	Weight of a non-structural element
X	Global x-axis
Y	Global y-axis
Z	Height of mass m_i above the level of application of the seismic action
α	Ratio of the design ground acceleration to the acceleration of gravity
α_A	Spectral amplification factor
γ_a	Importance factor for the non-structural element
γ_{max}	Maximum soil shear strain
γ_n	Factor relating to the number of ground motion sets used
γ_{RP}	Scaling factor to transform 475 year return period bedrock time signals to other return periods
ε_v	Vertical soil strain
ε_{vol}	Liquefaction-induced volumetric strain
η	Damping correction factor
κ	Knowledge factor (See also Confidence Factor CF_i)
$\mu_{strength}$	Ratio of elastic strength demand to yield strength coefficient
μ_{max}	Maximum strength ratio
ξ	Viscous damping ratio (in percent)
ρ	Unit weight of soil
ρ_w	Unit weight of water
σ_v	Total vertical stress
σ'_v	Effective vertical stress
ϕ'	Effective friction angle
$\psi_{2,i}$	Combination coefficient
$\psi_{E,i}$	Combination coefficient for a variable action *i*
φ	Combination coefficient

1.6 SI-Units

NEN-NPR9998, Section 1.6 refers to the SI system for consistent units as per ISO 1000. Dynamically consistent units as shown in Table 1 shall be used for all documents.

Table 1 Preferred Units

Unit	SI-System symbol (Preferred Units)	System mm-t-s		System mm-kg-ms	
		Symbol	Conversion from SI Units	Symbol	Conversion from SI Units
Length	m	mm	10^3	mm	10^3
Mass	kg	t	10^{-3}	kg	1
Time	s	s	1	ms	10^3
Temperature	K	K	1	K	1
Energy	J	mJ	10^3	J	1
Acceleration	m/s^2	mm/s^2	10^3	mm/ms^2	10^{-3}
Area	m^2	mm^2	10^6	mm^2	10^6
Frequency	Hz	Hz	1	ms^{-1}	10^{-3}
Velocity	m/s	mm/s	10^3	mm/ms	1
Volume	m^3	mm^3	10^9	mm^3	10^9
Density	kg/m^3	t/mm^3	10^{-12}	kg/mm^3	10^{-9}
Stress	N/m^2	N/mm^2	10^{-6}	kN/mm^2	10^{-9}
Force	N	N	1	kN	10^{-3}
Moment	Nm	Nmm	10^3	kNmm	1
Stiffness	N/m	N/mm	10^{-3}	kN/mm	10^{-6}

1.7 Information sources

All information available for a project is to be listed in the Building Knowledge Check List (BKCL) available in NCG Vraagspecificatie and has to state the adequacy of the information and necessary assumptions to complete a BSC. Possible information sources include:

1. Address list database
2. GESU portal database
3. Architectural and structural drawings typically from municipality archives
4. Geotechnical reports (factual and/or interpretative)
5. Existing archive based engineering calculations
6. Inspections and inspection reports
7. Google maps and Street View (to confirm that the drawings match the current building)
8. Where available laboratory test results for materials

1.8 Inspections

Inspections are to be carried out to collect as-built building data and site information to minimize the amount of assumptions, for the physical details of the building, to be made. NEN-NPR9998, Appendix A provides an inspection protocol for the assessment of existing buildings.

Additional to the inspection protocol the NCG-work instruction EVS2 inspection process [2] is to be followed to submit a complete set of documents including an adequately completed building data list. When archive building data is found to be outdated (eg. due to modification carried out after construction) deviations should be summarized and reported in the BKCL, inspection report and appropriate drawings. All modifications should be taken into account for the BSC.

A BSC with assumptions(should be noted in the issue log) for the physical details of the building will require an appropriate field inspection in the construction phase to confirm that the assumptions are valid. The engineer is responsible to evaluate the sufficiency of the newly gained knowledge for the aim of the assessment outcome.

2 Performance requirements and criteria

This chapter comprises the performance requirements and criteria for the assessment of structural safety of buildings in case of erection, reconstruction and disapproval, subject to induced seismicity in the Groningen area.

2.1 General

No additional information to NEN-NPR9998, Section 2.1.

2.2 Limit states and fundamental requirements

The required performance objective (the hazard level together with the limit state) is the Near Collapse (NC) limit state for all existing CC1b, (none monumental) CC2, CC3 and CC4 buildings as specified by NEN-NPR9998, Table 2.1.

More stringent performance objectives may be appropriate on a case by case basis. In this case NCG shall confirm in writing what the required performance objective and associated acceptance criteria should be.

For each building that needs to be seismically assessed the engineering contractor shall inform NCG of the following:

1. The consequence class (CC1b, CC2, CC3 or CC4) for the building.
2. The target performance objective (likely to be the NC performance objective).

2.2.1 Consequence classes and return periods for Barns

There are 3 different categories of barns to distinguish when determining the consequence class and return period:

- Detached barn
- Barn attached to the house and not influencing the collapse of the house
- Barn attached to the house and essential for the stability of the house

An agricultural barn should be classified in a consequence class depending on the actual usage. In Table 2 below the classification of the consequence classes and the return periods to be applied is shown for all three categories of barns.

Table 2 Gevolgklasse Groninger schuur volgens de NPR9998, NEN.

<table>
<tr><th>AFKEUR BESTAANDE BOUW</th><th colspan="5">Gebruik</th></tr>
<tr><td rowspan="2"></td><td>Normaal (aan te houden tenzij zeer duidelijk ander gebruik)</td><td colspan="4">Gebruik anders dan normaal (slechts in aangetoonde uitzonderingssituaties te gebruiken)</td></tr>
<tr><td>Normaal gebruik</td><td>Beperkt in de tijd door slechts enkele personen</td><td>Beperkt</td><td>≥20 personen gelijktijdig gedurende langere tijd</td><td>Als woonvertrekken</td></tr>
<tr><td>vrijstaand</td><td>Geen toets</td><td>Is het normale gebruik</td><td>T=95 jaar</td><td rowspan="3">T=2475 i.c.m. γ_I=1.1</td><td rowspan="3">T=2475 jaar</td></tr>
<tr><td>aan woning, instorten leidt niet tot instorten woning</td><td>T=95 jaar</td><td>Geen toets</td><td>Is het normale gebruik</td></tr>
<tr><td>aan woning, nodig voor stabiliteit</td><td>T=2475 jaar</td><td>T=2475 jaar</td><td>T=2475 jaar</td></tr>
</table>

Explanation of the usage:

- *Beperkt in de tijd door slechts enkele personen*: There are a few people **for less than 2 hours a day present** (storage of materials)(refer to checklist Groninger Schuren). The first two categories of barns with this occupancy are not required to be assessed (CC1a).
- *Beperkt*: There are a few people **for more than 2 hours a day present.** The first two categories of barns with this occupancy should be assessed with a return period of T=95 years (CC1b).
- *> 20 personen gelijktijdig gedurende langere tijd*: There are more than 20 people present for more than 2 hours during a day. In this situation, the consequence class is CC2 with a return period of T=2475 years regardless of the barn category.
- *Als woonvertrekken*: When there are living spaces built into the barn it should be assessed with a return period of T=2475 years (CC1b).

2.3 Definition of Building Elements

Table 3 provides a graphical interpretation of the alignment between the definitions of structural and non-structural elements between NEN-EN1998 and the NEN-NPR9998.

Table 3 Classification of elements according to Eurocode 8 and NEN-NPR9998

<table>
<tr><th>Eurocode 8</th><th>NEN-NPR9998</th></tr>
<tr><td>Primary Seismic (structural)</td><td>Primary Seismic (structural)</td></tr>
<tr><td>Secondary Seismic (structural)</td><td>Secondary Seismic (structural)</td></tr>
<tr><td rowspan="2">Non-structural</td><td>Non-seismic Structural</td></tr>
<tr><td>Non-structural</td></tr>
</table>

2.3.1 *Primary seismic elements*

Primary seismic elements are the elements in a structure that contribute to the seismic resistance of the building and could initiate (progressive) collapse in case of their failure/collapse, see also NEN-NPR9998, section 1.4.2.13. For the purposes of carrying out the required SAs and SUs the following elements are considered primary seismic members:

1. Floors, roofs and members providing both lateral and vertical support to floors and roofs.
2. Partitions that provide vertical and/or lateral strength and stiffness which the building relies upon (intentional or unintentionally).
3. Members that provide restraints to floors and their vertical supports (i.e. cores, braces, ties, compression restraints etc.). Compression restraints to columns, floor beams and roof beams. This is important so as to recognize the role of building components irrespective of whether they were originally considered to be part of the primary system or not.
4. All connections that are critical to holding the building together and whose failure would lead to disproportionate collapse.
5. Foundations

It is most likely that the definition of primary element would typically encompass the majority of the building’s supporting elements.

2.3.2 *Secondary seismic elements*

Secondary seismic elements are elements of the building’s supporting structure that do not contribute to resisting seismic actions but carry vertical building loads, see also NEN-NPR9998, section 1.4.2.16. Failure of the secondary seismic element may lead to progressive collapse of the structure in case of their failure/collapse. Secondary seismic elements should be able to follow the drifts of the building while not being relied upon to provide lateral resistance.

Examples of secondary seismic elements are: intermediate gravity column pinned at both ends.

2.3.3 *Non seismic structural elements*

Non seismic structural elements are elements that do not contribute to resisting seismic actions, do not transfer vertical building loads to the foundations other than those arising from the consequence of their own weight and shape, while local failure of the non-seismic structural element should not lead to disproportionate collapse or global building failure or collapse. See also NEN-NPR9998, section 1.4.2.11.

Their mass will either be explicitly accounted for or be deemed to be part of the super imposed dead load or imposed load. Examples of secondary seismic elements are:

- Infill panels and partitions that are not engaged with the building (vertically or laterally) other than for the purpose of maintaining their own stability.
- Walls that only carry their own weight and do not interact structurally with the building.
- Hangers (i.e. steel rods supporting stairs etc.)
- Gables (not carrying vertical roof loads)
- Parapets
- (Slender) chimneys

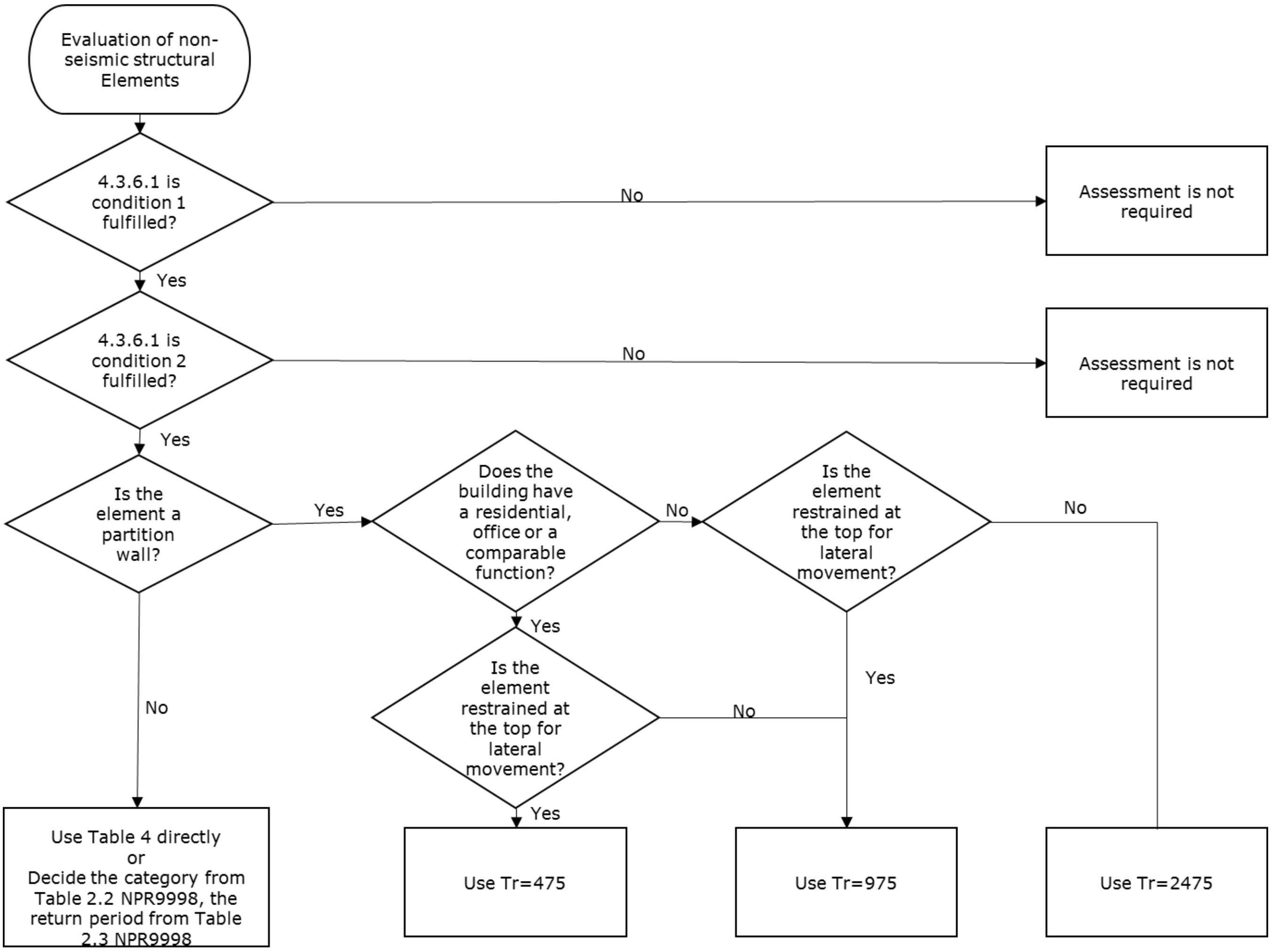
- Façade elements (i.e. large precast façade panels, heavy ornaments)
- Stairs
- Canopies
- Separated balconies that are only attached to the floor but do not contribute to its strength and stiffness.
- Masonry dormers

The interpretation of NC limit state by engineering contractors results in different outcomes and conservatism levels. The major identified differences are in the assessment of outer leaves of masonry, gable walls, and partition walls. When a building reaches NC limit state, according to NPR 9998 20% of the risk is contributed by the non-seismic structural elements. Non-seismic structural elements' performance is to be evaluated only if they exceed a weight threshold and their location is populated by people. The conditions of causing fatality by these components should be decided according to section 4.3.6.1. Unless these conditions are valid, the local failure of non-seismic structural elements is not capable to impose a considerable fatality risk and they do not need an assessment. The Figure 4 below provides a tool to the engineer to identify the requirements for assessing different types of non-seismic structural elements.
When a gable wall or outer leaf facade is evaluated as risky according to the NPR 9998 conditions, the assessment process for this component should be carried out with T_{LS}=2475 years. If the cumulative debris with contribution from gables and outer leaves (identified as risky by NPR conditions) exceeds 20% then it is assumed NC limit state is reached. For gables or outer leaf components that support a part of the building(not a non-seismic structural element anymore), the consequences are more and regardless of the given conditions, the assessment should be carried out with T_{LS}=2475 years. The seismic performance check of non-seismic structural elements in NPR 9998 is sort of adjusted according to their contribution to the individual risk and not as straightforward as the check for seismic structural elements. For this reason, the engineer should take particular care to ensure that these elements are suitably addressed. Partitions are a good example of this. The standard return period of ground motion for the assessment of 100 mm thick non-load bearing partition walls with a height less than 3m is T_{LS}=475 years(see Figure 4). If the wall is free to move at the top, then the performance should be checked with T_{LS}=975 years due to higher consequences. If the same wall has the risk of falling into a densely populated area, performance should be checked with T_{LS}=2475 years. An exception according to section 2.2.3 of NPR 9998 is that if the demand/capacity ratio is calculated lower than 0.8 for all seismic elements, the return period for the assessment of non-seismic structural elements should be lowered one level.

Table 4 The return periods for non-seismic structural elements

		ELEMENT TYPES			
LOCATION OF FALL		**Less Hazardous Elements (stairs in single family home, light glazing)**	**Small Size Elements (chimneys, parapets with a surface less than 3m²)**	**Medium Size Elements (chimneys, parapets)**	**Large Size Elements (gable, outer leaf, any other elements with a minimum surface 10m²)**
	Publicly accessible outdoor areas	95	475	975	2475
	Well frequented publicly accessible outdoor areas	475	975	2475	2475
	Escape route/internal space	475	2475	2475	2475

Figure 4 Non-seismic structural element assessment flowchart



2.3.4 Non-structural elements

Non-structural elements are architectural, electrical or mechanical elements, systems or components which, due to their lack of strength and stiffness or the way they are connected to the structure, are not considered in the seismic design as load carrying elements. Examples of these elements are air-conditioning and ventilation units, cables, pipes, false ceilings, electrical cabinets, elevators, domestic boilers, ordinary household sized book shelves etc. See also NEN-NPR9998, section 1.4.2.10.

3 Soil conditions, hazard and loads

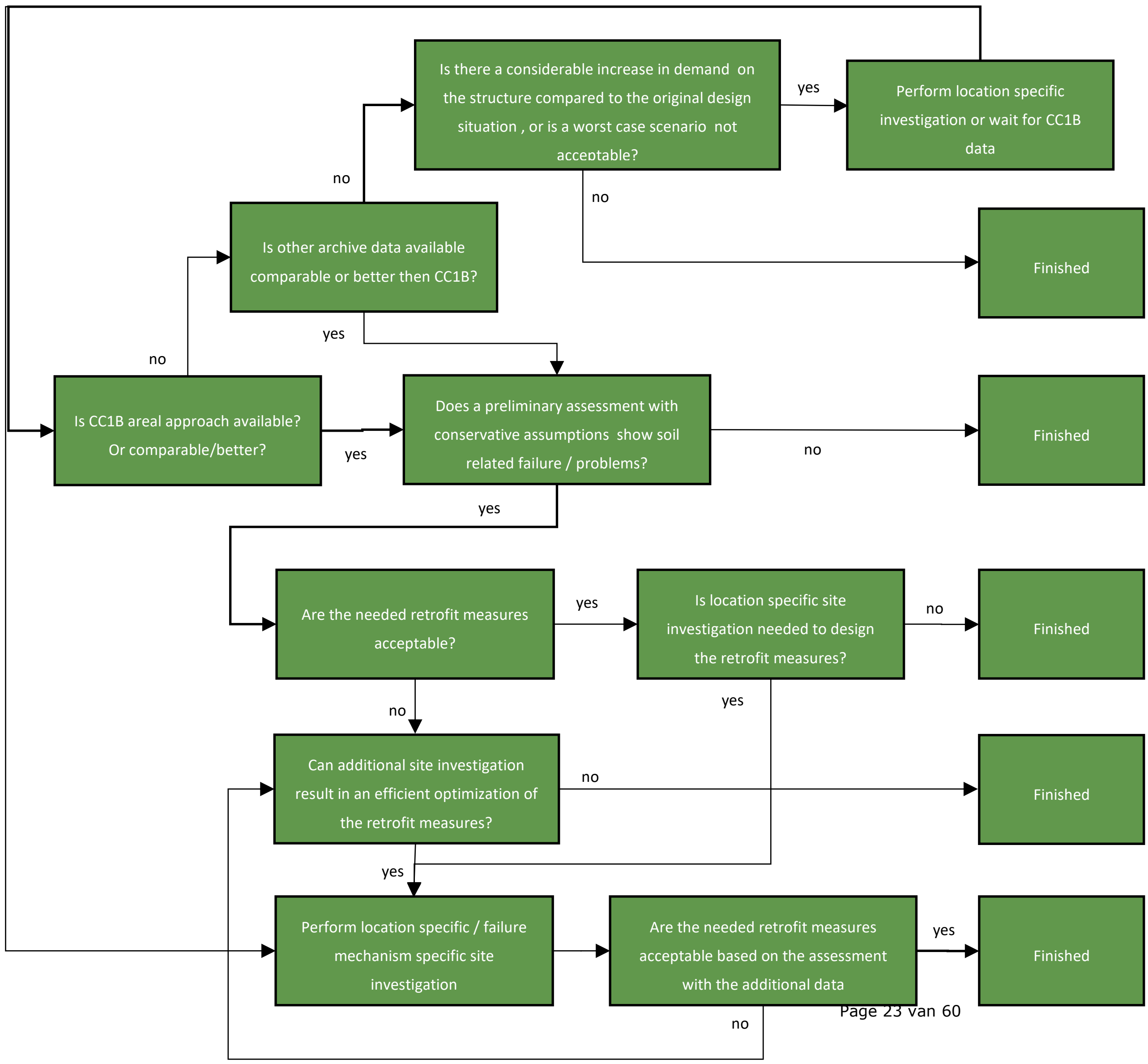
The NEN webtool available on https://seismischekrachten.nen.nl/map.php should be used for the hazard input.

3.1 Soil conditions

To perform an assessment according to NEN-NPR9998 information about the subsoil is needed to perform foundation and geotechnical checks. In line with the NEN-NPR9998, NCG has performed site investigation based on an aerial approach to gather information about the subsurface in an efficient way. The already available collected site investigation is shared via a GIS portal, access can be requested via NCG. The seismic performance of buildings is evaluated addressing the soil-related hazard with simplified and practical approaches. Depending on the contribution of the foundation behavior to the Near Collapse criterion of the superstructure it can be necessary to obtain (more) information about the subsurface. In standard practice, the soil investigations are to be performed after the TVA in the phase of the Detailed Design with the possibility of exceptional cases at the TVA phase in communication with NCG.

In the figure below a flow chart is shown which can be followed to determine if additional site investigation is required. The engineering contractor must use the BKCL to explain the need for (additional) site investigation, to agree the path forward with NCG.

Flowchart 1: Soil condition assessment



3.2 Seismic action

3.2.1 Basic ground acceleration

The NEN-NPR9998 webtool can be found using the following link:
http://seismischekrachten.nen.nl/map.php.

The dataset to be used for assessment in compliance with NEN-NPR9998 is GMMv6 d.d. 2020-07-01 Maaiveld – Timeframe T5.

3.2.2 Surface response spectrum

The surface response spectrum can be taken from NEN-NPR9998 for sites with normal or special soil conditions as defined in the NEN-NPR9998.

If there is adequate knowledge of the soil conditions at the site and a special need for the project, the surface spectrum can be derived by a site-specific non-linear site response analysis under the seven or eleven bi-axial input motion sets scaled as required by the NEN-NPR9998 using hazard definition compatible ground motions. This should be agreed with NCG before the execution of the site response analysis.

3.2.3 Ground motions (NLTH)

Ground motions at surface and depth for NEN-NPR9998 are available in the NEN-NPR9998 webtool. Where any future changes in seismic input take place, until the availability of the new ground motion set in the webtool, the engineering contractor is required to agree with NCG to perform the analysis with an adequate ground motion input.

3.3 Loads

NEN-NPR9998, Section 3.2.4 specifies the combination of seismic loads with other loads.

This section provides reference guidance for both gravity (dead, superimposed and live) and seismic loads. The reference values provided below for gravity loads should be used in the absence of specific loading information from original design documentation or site investigations. Deviations from the reference values are permitted whenever more appropriate as built knowledge/information is available.

Only seismic actions and expected gravity loads are included. Wind, snow and thermal actions are excluded.

Seismic load combinations are to be taken from NEN-EN1990, Section 6.4.3.4 as specified in NEN-NPR9998.

3.3.1 *Dead loads*

With regard to the buildings actual material properties the default densities of structural materials to be adopted are shown in the table below. Excluding for masonry, these are derived from EN 1991-1-1, Appendix A. Masonry densities are based on values provided in NEN 6702 Table C.1 for clay and calcium silicate masonry.

Table 5 Densities of structural materials

Structural materials	Density (kg/m^3)
Masonry (solid clay bricks)	1950
Masonry (calcium silicate bricks)	1850
Concrete (unreinforced)	2400
Concrete (reinforced)	2500
Structural steel	7850
Steel reinforcement bars	7850
Sawn softwood timber C14 (pre 1945)	350
Sawn softwood timber C18 (post 1945)	380
Sawn hardwood timber D18	570
Oak	750
Plywood sheathing (softwood)	450
Plywood sheathing (hardwood)	650
OSB sheathing	550
Chipboard/particle board sheathing	750

For masonry typically used in Dutch construction, with solid bricks, the density ranges between 1700 and 2000 kg/m^3 (NEN 6702 Table C.1). For the purpose of this document, the values provided are intended to bring consistency to the assumptions rather than give exact values. Higher or lower values may be used based on the specific characteristics of the masonry under consideration.

Non loadbearing masonry walls, that cannot be simply moved without construction activity are to be considered as permanent loads and should not be reduced by Ψ_2.

The following are assumed weights of prefabricated floor systems commonly found in existing buildings in the Groningen region.

Table 6 Assumed weights of prefabricated floor systems

Floor system	UDL (kN/m^2)
Hollow core floor (without topping) t = 150mm	2,70
Hollow core floor (without topping) t = 200mm	3,10
Hollow core floor (without topping) t = 260mm	3,80
Hollow core floor (without topping) t = 320mm	4,30
NeHoBo floor t = 120mm	1,60
NeHoBo floor t = 140mm	2,00
NeHoBo floor t = 150mm	2,20
Beam and block floor	3,00
PS-insulation floor	1,90

Beam and block floors and PS-insulation floors are ground floors, consisting of pre-stressed concrete beams with either concrete or Polystyrene blocks between them as formwork and a concrete top layer.

3.3.2 Superimposed dead loads

The following superimposed dead loads (SDL) are considered in the analyses:

Table 7 Assumed weights for common SDL

Element/system	Density (kg/m^3)
Reinforced concrete toppings on floors	2500
Screeds (normal weight)	2000
Aerated concrete	800
Gypsum boards	900
Glass (windows/partitions)	2500
Element/system	**UDL (kN/m^2)**
Ceilings	0,15
Bitumen only on flat roofs	0,10
Bitumen and gravel on flat roofs	0,60
Roof tiles (concrete)	0,50
Roof tiles (clay)	0,40
Tiled roof (full build-up)	0,65

3.3.3 *Imposed loads*

The following imposed loads apply, following NEN-EN 1991-1-1 (+ NB):

Table 8 Imposed loads

Location	UDL (kN/m^2)
Living and domestic use	1,75
Stairs	2,00
Balconies	2,50
Roofs	1,00
Office spaces	2,50
School buildings	2,50
Public spaces (with tables and/or fixed seats)	4,00
Public spaces (including public stairs)	5,00
Shops	4,00
Garages (vehicle weight less than 25 kN)	2,00
Garages (vehicle weight 25-120 kN)	5,00
Garages (vehicle weight bigger than 120 kN)	G_v / A_v*
* G_v the vehicle weight and A_v is the vehicle footprint	

Living and domestic use includes rooms in houses, wards in hospitals, sleeping rooms in hotels, kitchens and toilets. The above excludes dedicated MEP rooms, archives, storage facilities, libraries, data centre/computer rooms, etc... For these spaces, actual imposed loads shall be determined based on a case by case basis and agreed with NCG.

3.3.4 *Non-masonry partitions*

Moveable non masonry partition walls (that can be readily re-configured in layout by a building user) may be considered as uniformly distributed imposed loads if not shown on the drawings. Imposed load values for non-masonry partitions can be assumed based on the values shown in Table 9.

Table 9 Partition loads

Location	UDL (kN/m^2) (floor area)
Partition walls (light, less than 1,0kN/m' wall length)	0,50
Partition walls (middle, between 1,0 and 2,0 kN/m' wall length)	0,80
Partition walls (heavy, between 2,0 and 3,0 kN/m' wall length)	1,20
The previous Dutch code (NEN6702) stated that a maximum 2,5 meters width is to be taken in account to spread the partition wall weight over the floor area. This is based on a typical floor to ceiling height of not more than 3,0 meters.	

General partition walls in CC1b buildings consisting of 70mm aerated concrete up to 3,0 meters in height shall be modelled as an imposed load of 0,80 kN/m^2.

3.3.5 *Seismic mass*

In accordance with NEN-NPR9998, the inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\sum G_{k;j} + \sum \psi_{E,i} \cdot Q_{k;i}$$

where: $\psi_{E;I}$ is the combination coefficient for transient action *i*

The combination coefficients $\psi_{E;i}$ for the calculation of the effects of the seismic actions, shall be computed from the following expression:

$$\Psi_{E;i} = \varphi \cdot \Psi_{2;1}$$

The values to be ascribed to φ shown in Table 10:

Table 10 φ factors for different load categories

Load category	φ
for roofs	1,0
for other floors	0,6*
for storage	1,0
* from NEN-NPR9998	

The combination coefficients $\psi_{2,i}$ are derived from NEN 8700 Table A1.1 (for existing buildings) and are presented in Table 11:

Table 11 $\psi_{2,I}$ factors for different load categories

Load category	$\psi_{2,i}$
Residential and office space:	0,3
Schools	0,6
Roofs	0,0
Public spaces and shops	0,6
Stairs (multi-storey buildings)	0.6
Storage areas	0,8
Spaces for traffic (like garages) for vehicles < 30 kN	0,6
Spaces for traffic for vehicles between 30 and 160 kN	0,3
Snow loads	0,0
Loads from rain water	0,0
Wind loads	0,0
Thermal loads	0,0*
* Where seismic upgrading is proposed this value may need to be adjusted	

From these combination coefficients it can be seen that of all imposed loads, only imposed loads on floors have to be considered in the seismic design situation. Imposed loads on roofs are discarded.

4 Design, re-design and assessment of buildings

NEN-NPR9998, Chapter 4 provides guidance for building assessment and acceptance criteria.

4.1 General

The description of the NC limit state in NEN-NPR9998, Section 2.2.1 should be considered as the general definition applicable for all methods of seismic analysis. The acceptance criteria in NEN-NPR9998 broadly aligns with the definition of the NC limit state (like Ed < Rd and provided drift limits in Annex G). The acceptance criteria based on the method of analysis is given in Table 12 below.

Table 12 Acceptance criteria based on analysis method

Analysis method	Acceptance criteria
LFA MRSA	See section 4.4.2 of NEN-NPR9998 for force capacity guidance. See Annex G.6 of NEN-NPR9998.
NLPO/NLKA	See Annex G.6 for NLPO and Annex H for NLKA of NEN-NPR9998.
NLTH	See section F.6 and G.6 of NEN-NPR9998 without considering the building volume loss criteria(in standard application with exceptions).

* NEN-NPR9998, Table G.2 provides global limit state values for interstory drifts, regardless the number of stories and provides limits for the drifts at the effective height, only up to two story buildings. Caution should be given in judging the compliance of buildings higher then two stories. The given value for a two story building can be used as an upper limit for the assessment of a buildings higher then two stories, judgement should be used if this value should be reduced, case by case.

According to NPR 9998, the NC compliance of NLTH analysis results can be evaluated with indirect and explicit criteria. In micro modelling(brick and mortar are not meant to be separately modelled) based finite element models, the failure of components is included in the models through the constitutive algorithms. When the failure criteria are exceeded, in advanced models, elements are removed from the model. This allows a direct evaluation of failure for the explicit check. The direct method should evaluate the compliance explicitly based on a volume loss of 20%. The indirect method evaluates the compliance based on the global limit state drift criteria. Regardless of the compliance criteria, the level of micro modelling detail and refinement should be similar to the more advanced micro-models where all significant explicit failures are modelled and checked with explicit criteria. The detail of modelling should include the function to capture the diaphragm behaviour, the failure of floor/wall, wall/wall connections, the contribution of non-load bearing walls(if applicable).
The combination of each modelling approach(micro modelling, macro modelling) and the analysis approach requires different checks to achieve the limit state criteria of NPR 9998. Table 13 lists common combinations that are used with NLTH analysis in Groningen. The NPR provided failure criteria of global limit state for pushover analyses can be grouped as (1) exceedance of the storey and effective height global drift limits or 50% loss of total base shear capacity for models with continuum finite elements, (2) exceedance of the component drift limits or 50% loss of total base shear capacity for models with macro equivalent frame models and mechanism-based SLaMA.

The results of the global analyses should be examined rather than in component failures, to what extent the failure of components impacts the global collapse. If a primary or secondary seismic element exceeds the acceptance criteria and additional analyses can be performed to demonstrate that the failure of these elements will not result in a disproportionate extent of collapse, it should be considered. Where a 2d wall line analysis is performed by SLaMA, if 50% strength reduction of one of the gridlines is used in judging the limit state, a further and more accurate investigation should be carried out to demonstrate this failure criteria is also reached in building level. Due to the nature of SDOF based pushover assessment, regardless of the sophistication of modelling the local collapse of the walls should be independently evaluated based on the local collapse assessment by NLKA of Annex H unless a special study based on NLTH on component level is performed. Where the local component model developed for OOP failure of components can be demonstrated to have greater accuracy in dynamic response simulation, the results from this model can be used as displacement demands on the component. For the local response, the accuracy level of results should be at a similar level to obtaining the local response from the NLTH on micro models of Table 13. For this case, the same displacement limit criteria(obtained from instability) of the micro modelling Table 13 should be used in the verification of the component NC limit state compliance.

Table 13 Overview of Modelling&Analysis Type and Acceptance Criteria

Modellling & Analysis Type	Description and Criteria
NLTH on advanced micromodels	• The aim is to predict the response with greater accuracy by the combination of accurate modelling with accurate analysis type and prevent over-conservatism. • All modes of failure(including local failures) that have a contribution to collapse are simulated. The model is not only capable of simulating the global instability but also capable of simulating localized failures and collapses such as failure of a wall anchorage or OOP collapse of a wall. • The underlying behaviour of the building components shall be explicitly modelled. The model shall provide a direct evaluation of failure for elements. • The model should use failure induced element erosion and should be able to detect contact and impact between two components and generate the forces by this contact. • The model is validated for its specific intention which is explicit collapse simulation. The mode and quantity of damage accumulation at collapse state are some of the useful parameters helpful for validation against experimental data. • The compliance to the NC limit state should be verified according to the explicit check of NC. • NLTH is performed for each record. If the results due to any of the records are classified as collapse, the NC limit state is unverified. • Response quantities such as story drifts shall be demonstrated although explicit check does not enforce limit state criteria. As most of the computations are embedded in the software, an effective review can only be performed in this way. • The response quantities are to be taken as the largest of maximum responses to individual records. • Any behaviour of any element and connection without inelastic deformation capacity should be classified as force-controlled behaviour. At this sophistication level of analysis, the failure of these components should also be explicitly modelled. • According to the explicit check of such advanced models(as described here), the failure of a single component with a limited consequence would not indicate a NC state.
NLTH on micromodels	• This approach provides relatively simplistic representations and does not simulate some of the complex aspects of the building components or connection behaviour. • All modes of failure(including local failures) that have a contribution to collapse are evaluated but are not always explicitly simulated. Examples of these modes can be OOP failure of walls, fracture in the connections of roof beams. • Structural components are mostly modelled with continuum elements but for some of the components, the underlying behaviour is not explicitly modelled. For these components, the demand is extracted from the simulation and compared with appropriate limit state criteria. This approach is an approximation to the real behaviour rather than a direct evaluation of failure for elements. • The model is validated for the nonlinear response simulation but not for explicit collapse simulation. • The analysis may be performed either with an explicit solver or an implicit solver as a numerical technique.

	• The compliance to the NC limit state should be verified according to the implicit check of NC. • A NLTH is performed for each record. The limit state check for the global performance of the building should be done with the global drift criteria from NEN-NPR9998, Table G.2. • The response quantities are to be taken as the average of maximum responses to individual records. The exceedance of the global drift limit criteria in a single record does not violate the NC limit state. • The global drift criteria limits the collapse possibility in the building components but it does not necessarily guarantee the verification of all the building components. The failure status of building components should be evaluated either explicitly in the simulation or implicitly. • According to the implicit check, the failure of a single component with a limited consequence would not indicate NC state unless it is demonstrated that it leads to a significant collapse or significant life threat to the person. • If the failure of force-controlled elements is not explicitly modelled, the response quantities for these elastically modelled elements should not be averaged and the extreme value obtained from all records is conservatively taken. • OOP failure mode of walls is a good example for implicit verification. The analysis results are used to obtain the average displacements from all records. The verification of the component is checked by comparing the average displacement with the limit state criteria. The limit state criteria is 60% of instability displacement for vertically spanning walls.
NLTH on equivalent frame models(macro)	• The modelling operation has significant simplifications. This approach represents the response of structural building components by idealized deformation relationships assigned to frame elements. • Structural components are modelled with one-dimensional line type elements. While these models may define a three-dimensional geometry in space, they have some limitations with complex wall geometry when three-dimensional behaviour is important to simulate. For example, more advanced models may be needed for the accurate prediction of response in orthogonally interlocked walls. • The model is validated for the nonlinear response simulation but not for explicit collapse simulation. • The compliance to the NC limit state should be verified according to the implicit check of NC. • A NLTH is performed for each record. The limit state check for the global performance of the building should be done with the global drift criteria from NEN-NPR9998, Table G.2. • The response quantities are to be taken as the average of maximum responses to individual records. The exceedance of the global drift limit criteria in a single record does not violate the NC limit state. • The global drift criteria limits the collapse possibility in the building components but it does not necessarily guarantee the verification of the building components. The failure status of components that are not simulated in the model should be checked with additional analysis. • According to the implicit check, the failure of a single component with a limited consequence would not indicate NC state unless it is demonstrated that it leads to a significant collapse or significant life threat to the person. • NLKA of NEN-NPR9998 is the baseline method for assessing OOP. OOP failure mode of walls should be verified for NC limit state independently from the NLTH on global model by implementing NLKA for each wall. For unverified walls with NLKA, more advanced and accurate approaches are possible with a special study based on NLTH on component level(see last paragraph in Section 4.1).

4.2 Characteristics of earthquake resistant buildings

The non-load bearing elements, such internal partitions are acknowledged to add mass to the system, their structural stiffness and strength are generally ignored in simplified modeling practice. In reality, some of the partition walls can play a structural role in the overall seismic behavior of buildings. The studies reveal that the capacity of the building is well underestimated when the non-intended contribution of these walls is ignored in the structural models. In such cases, the partition walls should be included in the model. The research by NEN was carried out to investigate the possibility of finding correlations between different analyses. Some of the outcomes of this study can be used here to give guidance on when the impact of the non-load-bearing walls should be included in the model. The key requirements to evaluate the inclusion of these walls in modeling includes (1) flange function of non-load bearing wall acting as a flange with a connection to an orthogonal wall, (2) the existence of a sufficiently strong connection to the load-bearing in-plane wall where vertical shear stresses can develop along the connection, (3) consideration of the different behavior by the loading direction of the flange due to the flange contribution effectiveness only under compression, (4) existence of a load path to the ground.
No additional guidance is provided to NEN-NPR9998, Section 4.2.

4.3 Structural Analysis

Verification of building compliance to NEN-NPR9998 is performed by using various analysis methods. Each method has its own limitations and constraints. An engineer with relevant experience and skills shall be responsible for the consequences of choosing a specific analysis type and modelling approach. At the start of the project, NCG and the engineering contractor shall agree on the analysis method on a case by case basis. In choosing a certain analysis and modelling approach, the engineering contractor shall be able to demonstrate their criteria of selection.

Seismic hazard and building vulnerability are two main components that impact the outcome of seismic performance assessment. Where the combination of two components results in a favorable outcome for the building, an NLTH with an advanced modeling approach could be a viable option. For example, a building known to be relatively more seismic-resistant experiencing a PGA of 0.13g has a higher chance to be compliant in its as-is condition via NLTH than other possible combinations. Instead, at combinations where a seismic upgrading is highly likely, NLPO analyses with simplified modeling approaches could be a viable option considering time, cost, expertise and capacity.

The analysis of an accurate response of Groningen buildings is subject to uncertainties in the input parameters such as connection properties, material properties, diaphragm stiffness, etc. Beyond the uncertainties of the input parameters, there are also various sources of uncertainties including different modelling techniques, theoretical assumptions. Simply picking up the most conservative choices for each matter is not a proper accounting strategy and would ruin the efforts for a sophisticated assessment. Every parameter value that is not known perfectly does not necessarily have a significant effect and the same effect on the collapse of the building. Sensitivity studies have a key role in identifying the effects of varying the properties, assumptions on the collapse results. In order to improve the confidence in NC prediction and account for the impact of uncertainties on the results, a more consistent approach can be followed by undertaking the following steps:

1) A list of parameters that can influence the results of the analysis is made based on the engineering experience, literature, and gained knowledge in Groningen. It can be related to mass, stiffness properties, material properties, damping coefficients, position of elements, etc. For example, during the seismic impact, there are possible complexities that modify the effective friction coefficient. These can include, painted and unpainted surfaces, erosions on the surfaces. If the engineer estimates the friction coefficient as an

influential parameter and finds out a significant variation evidence in the literature, the friction coefficient becomes part of this list.

2) To reduce the number of parameters to a more manageable number, the engineer can use the knowledge already gained in the assessment of Groningen buildings. For example, if the impact of foundation spring stiffnesses was already investigated in a previous sensitivity study, and its impact was judged to be limited for a similar building.
3) For the rest of the parameters, sensitivity analyses should be performed at a component or global level to investigate the impact of the parameter variability on the level of collapse damage. Once the results are obtained, engineered conclusions can be drawn about the influence of parameters and assumptions.
4) When a parameter is found to have a critical role in collapse safety (small changes in the parameter change the end decision), a realistic variation around the best estimate value should be considered and an appropriate number of standard deviation about the best estimate(mean) value of input parameter can be used in the final assessment.
5) For other parameters that are identified to have less impact on the collapse damage results, it could be more appropriate to use the mean values rather than extreme values unless the opposite is justified by the engineer.

Rather than recommending the engineer one particular value to use, the appropriateness of the value is left to the engineer. It is recommended that the engineer considers the level of already implemented conservatism from the sources of modeling, analysis type, acceptance criteria, etc. in the project. The value choice of an engineer implementing a sophisticated model far from simplifications in geometry and behavior with an explicit check may be one standard deviation away from the mean, while with another modeling and analysis type the same parameter may be assigned a mean value.

In some projects, the consideration of the uncertainty in a broader context may be necessary. The engineer may face a lack of information about a relevant physical property and with a responsible decision may decide to continue the assessment without a further investigation. Hereby an example scenario of how the engineer considers the lack of knowledge about a physical property is provided:

1) The details of connectors between two building components are unknown.
2) The direct impact of this lack of knowledge is evaluated. For instance, its direct effect is on the shear force transfer.
3) The possible options of connectors commonly used in this type of building stock are listed.
4) The related quantity, in this case, the shear force capacity is calculated.
5) The demand for the connector is independent of the unknown physical detail of the connector for this case.
6) If the lower bound capacity obtained from the connectors exceeds the demand, in practice the lack of knowledge does not impact the assessment result.
7) If the upper bound capacity obtained from the connectors is less than the demand, it can be concluded the connector fails.
8) If the demand is in between the lower and upper bound values, the interpretation of the engineer is essential. Based on the bandwidth of possible capacity values and the demand value’s relationship with it, the engineer may decide to go for an inspection.
9) The impact of the connector failure on the NC limit state is evaluated. Each failure type does not necessarily have a significant effect to violate the NC limit.

4.3.1 Lateral Force Analysis

This method describes the most simplified way of evaluating the effects of dynamic, seismic action on a structure by using a static approach. While it is not restricted for the assessment of existing buildings, it is more appropriate for the design of new built structures. For buildings with higher mode contributions, with a highly nonlinear response, with significant horizontal and vertical irregularities this method can’t be applied. It is a useful approach as a sanity check on the results of more advanced analysis types. For further details of the method NEN-NPR9998, Section 4.3.4.2.

4.3.2 *Modal response spectrum Analysis*

As the lateral force analysis, Modal Response Spectrum (MRS) analysis is a method to be used in force-based assessment approaches. The basis of the analysis is constructed on an elastic analysis model. The actual nonlinear response of the building is not taken into account. It is well known that it is inaccurate to determine the demands on a building with pronounced nonlinear behaviour by using MRS. For this reason, in the evaluation of NC limit state and development of retrofit strategy, unnecessarily conservative results may be the outcome for these buildings. For buildings where the lateral force resisting systems are hybrid, a MRS approach becomes less accurate in evaluating the structural performance of the building. Response spectrum analysis can also be considered as an initial step to understand the structural behavior before moving towards a NLTH analysis. For the definition of general applicability of the analysis method reference is made to NEN-NPR9998, Section 4.4.3.3 and for the detailed steps of verifying structural safety, reference is made to the "Uitgangspuntenrapport MRS analyse" document by VIIA [4], available at request at NCG.

4.3.3 *Non-linear Push over Analysis*

Software packages and tools used for the nonlinear static assessment of Groningen buildings shall be verified by showing good agreement with laboratory test results and/or verified software results.

According to NEN-NPR9998, Annex G, seismic assessments are to be carried out using the capacity spectrum method (CSM) or the NEN-EN1998 N2 method. It is clearly stated that the capacity spectrum procedure is the preferred option of NEN-NPR9998. Both procedures (CSM and N2) start with obtaining displacement vs base shear curve of the building. The difference between the assessment procedures lies in the derivation of SDOF capacity curve and the so-called performance point. A comparison of building performances with both methods was carried out for a group of selected buildings in Groningen. As expected, the methods did not produce equivalent results. It is a valuable exercise for the engineering contractors/consultants making a decision on the pushover based assessment method on a case by case basis.

With a NLPO procedure, the failure mode of buildings, where the response is dominated by a fundamental mode of vibration, can be estimated sufficiently. In NEN-NPR9998, G.2, guidance is provided on the applicability of the method with given consideration on higher mode effects, irregularities. When certain conditions of G.2 are not satisfied, further actions and analysis to compensate the limitations are performed based on the recommendations of G.4.7-G.4.9.

The seismic assessment techniques implemented in Groningen range from simplified mechanism-based assessment to numerical modelling based pushover methods. In NEN module 3, research about the results of different modelling approaches with nonlinear dynamic and nonlinear static analysis on case studies was performed. SLaMA and 3Muri macro model based pushover analyses generally underestimate the force capacity of the nonlinear force-displacement response significantly. Even under controlled research conditions for the analyses, the factor of underestimation can be as high as 2.5. This underestimation results in an unintentionally downscaled capacity curve and it leads to a shift of the identified performance point to the higher displacement zone. For this reason, the PGA level causing the collapse displacement is inaccurately underpredicted in respect to other analyses. At lower intensity ground shaking, the outcome(pass/fail) of in-plane assessment is not sensitive to this outcome but in higher intensity shaking(for example PGA higher than 0.2g) the assessment can be concluded with unnecessary upgrading. In addition, for limited ductility governed buildings, the underestimation may lead to an in-plane failure mode as the outcome of the analysis. For this kind of buildings, a micro model based pushover assessment can be a better choice to improve the in-plane response prediction.

The key elements of NLPO procedures in the guidance of NEN-NPR9998 are:

- The seismic demand is represented in the form of a spectrum that should be obtained from the NEN-NPR9998 webtool (http://seismischekrachten.nen.nl/map.php). When specific surface spectra derived from a soil column response are believed to give more conservative results, NCG is contacted to discuss the method of seismic demand input.

- Loading patterns are to be in accordance with G.4.6.
- Where SSI increases the spectral acceleration of the building, the effects of SSI must be taken into account.
- All primary elements have to be included in the modelling. Secondary seismic elements are preferably included in the modelling as well, as long as their compatibility is not verified separately.
- For accurate performance prediction of the assessed buildings, an accurate damping prediction is necessary. The hysteretic damping of structures is related to the energy dissipation of building elements during cycles of motion. The cycles of deformation during an earthquake are a complex phenomenon and the displacement amplitudes are moving about less than the amplitude at peak response. Theoretically, a damping ratio can be obtained at each cycle of displacement demand. This phenomenon is assumed to be incorporated conservatively in the derivation of the hysteretic damping formula (NEN-NPR9998, Table G.3).
- The hysteretic damping formula can also be used for cases where a mixture of failure types occur simultaneously. For example, sliding shear and rocking.
- Damping phenomena have to be evaluated for each principle direction.
- Beneficial effects of soil and foundation damping above 5% are to be used only when the calculations from G8.2 indicate as such.
- In the existence of flexible diaphragms, the pushover analysis with flexible diaphragms or/and walls line based pushover analysis shall also be considered. Though the difficulties due to the limitations(for example, unreliable results if the higher modes are excited) of a flexible diaphragm pushover, a case-specific approach with the existence of sensitivity checks may provide extra confidence to the engineering contractor in the elimination of a rigid diaphragm assumption.

4.3.4 *General limitations of NLPO*

There are several limitations of the NLPO approach which make the results of an analysis less reliable than an NLTH. The capability and limitations of NLPO are discussed in NEN-NPR9998. Some of the key limitations of assessments with NLPO are considered as having a fixed base structural model, assumptions about horizontal and vertical irregularities, higher mode effects and flexible diaphragms. For further guidance on the limitations and practical ways to deal with the limitations under aforementioned conditions, the reader is referred to annex G. The following recommendations provide some key issues related to the NLPO:

- Pushover analyses mostly assume that the building has a rigid foundation and does not consider SSI. Where the foundation flexibility is considered to change the response drastically, foundation flexibility can be added by using foundation springs or modelling soil and structure together in modelling.
- Where the vertical component of the ground motion is considered to change the structural response drastically, a proposal to change the analysis method can be discussed with NCG.
- Where the building has an immediate descending response upon yielding due to 2nd order effects, a proposal to change the analysis method or provide an additional way of assessment, can be discussed and forwarded for approval to NCG.

4.3.5 *SLAMA for NLPO*

In NEN-NPR9998, Annex G.6.2 and G.10 the use of the SLaMA method is mentioned as one of the nonlinear static seismic assessment methods. As the method is a simplified technique for determining the deformation mechanisms and associated strengths, the limitations of NLPO is also applicable for this method. The effect of horizontal irregularities on the performance of the building are not particularly considered in this analysis method. When certain conditions of G.2 are not satisfied, further actions and analysis to compensate the limitations are performed based on the recommendations of G4.7-G4.9. NCG encourages the use of this simplified nonlinear assessment method for simple building configurations with limited torsional response.

4.3.6 *NLKA and VWM for OOP*

The Non-Linear Kinematic Analyses (NLKA) method for out-of-plane (OOP) behavior of masonry has recently been a standard practice to perform the compliancy check of the wall elements in Groningen. This method is only valid whenever (rigid) block behavior of the masonry and the assumption of the initial cracked status can be justified for walls experiencing one-way bending. In the latest publication of NEN-NPR9998, Annex H [15], a virtual-work-method based formulation is included to estimate the two-way bending resistance of URM walls. This section applies to URM walls that are under the effect of out of plane loading without any differentiation of primary seismic, secondary seismic or non-seismic structural walls with detailed guidance in NEN-NPR9998, Annex H.

The latest annex states 3 methods with an increasing accuracy to perform the seismic safety assessment for out of plane direction. The engineer plays an important role to choose between the 3 different tiers. Up to now, experience has shown that Tier 2 predicts the acceleration demands more accurately. Therefore, the engineer may directly skip Tier 1 and perform Tier 2 based assessment.

The seismic behaviour of URM walls are highly dependent on the boundary conditions and vertical load. Without the presence of any evidence, the permanence of boundary conditions of a wall and its vertical load during a seismic activity is difficult to judge and most of the experimentally available tests were performed keeping these variables as constant. The fifth paragraph of annex paragraph states: *Bij zowel de NLKA-methode als de MVA is de juiste schematisering van de randvoorwaarden een belangrijk aspect, evenals de juiste bepaling van de verticale belastingen die op de te analyseren wand aangrijpen. Hierbij moet rekening worden gehouden met de mogelijkheid dat de randvoorwaarden die bij een statische belasting gelden, tijdens een aardbeving veranderen. Ook kan het gebeuren dat de verticale belasting op de wand niet meer samenvalt met de statische zwaartekrachtsbelasting door de herverdeling van krachten die bepaald wordt door de globale, in het vlak werkende, weerstandsmechanismen in de constructie. Het effect kan ook op andere wijzen worden bepaald, bijvoorbeeld met NLTH.* Therefore the engineer shall be aware of the uncertainty about the validity of the assumed conditions of the wall during the seismic activity. To prevent any too optimistic assumptions, the engineer should be able to explicitly demonstrate the validity of the assumed boundary and loading conditions with its reason for each wall. The choices should be reasonable and should not lead the engineer to over-conservative assessments either. The NEN-NPR9998 is not able to identify all the variable scenarios regarding the assumptions but as an illustration, the text in section H.1 continues with a few examples and with Annex H, Figure H.2 as a description.

The section H.1 affirms a number of bullet points for the safety assessments of cavity walls. It is understood that to take into account the negative effect of the outer leaf on the inner leaf, proportionally the mass of the outer leaf is introduced in the annex regardless of the condition of the cavity ties. It is suspected that the committee's intention is to direct the engineers to a conservative evaluation in terms of the mass interaction between 2 leaves. As the guidance on how much mass from outer leaf should be introduced to the inner leaf is unclear in the annex, a maximum of 50% is recommended here to limit too much mass transfer. With an explicit technical rationale stated in the TVA, the engineer still has the choice not to include the mass effect of the outer leaf in special cases. The conditions for the strength contribution of the outer leaf is also covered in section H.1.

Per NEN-NPR9998, the new rules about the spectral value calculation for the components and walls at tier 1 and tier 2 are contained in section H.3. The calculation of the spectral value is now based on 2 formulations. The slight difference between the 2 formulations simply comes from the insertion of the period coefficients in place. The introduction of spectral value lower bound limits in the formulations, $S_{Ea;d} \geq S_e(T_A)$, seems to eliminate the underestimation of spectral acceleration values for the walls at floor levels. By this rule, the spectral values for the wall are not allowed to be predicted less than ground response spectra values. At tier 2, there are two different approaches to determine the building specific floor spectra based on the global response of the building.

With the latest annex, a new way of calculating the seismic resistance is introduced for two-way spanning walls. The new chapter categorizes the resistance of the walls based on the spanning direction of the walls. One-way spanning walls refer to vertical bending along the horizontal failure plane. The content of the section about the resistance of vertically spanning walls is nearly identical to the previous version's resistance section with small adjustments. A guidance is given about the applicability of one way spanning walls to gables, walls, parapets, and chimneys. A requirement is introduced about the boundary conditions to evaluate gables either as a cantilever mechanism or as a gable mechanism. In the absence of criteria to evaluate the adequacy of the roof restraint, the minimum of half of the thickness of the restrained wall and 2.5% relative drift can be considered the maximum allowable relative displacement of the flexible diaphragm. The non-linear kinematic analysis based resistance estimations for URM walls in the form of graphics are still the basis of resistance calculations. Before attempting to choose the right graphic for each URM wall component, the engineer is supposed to evaluate the type of mechanism as a vertically spanning, gable or cantilever wall. After the evaluation of the eccentricities and the inter-story drift, an appropriate graphic is chosen. As it is clearly seen from the provided curves for each graphic, the estimation of the vertical load on the wall element has a major effect on the seismic resistance prediction of the wall element. The structural engineer should account for the uncertainties about the vertical load on the wall element, evaluate if the wall has a vertical load transfer from the roof, slab, etc. and clearly state these findings in the TVA.

The assumed static deformed status at the time of static instability is shown in Annex H, Figure H6 with the relevant parts of the equation of equilibrium to provide some guidance for the key parameters of NLKA. The new Annex H in its updated section H.4.1 has new recommendations for the assessment of cavity walls with NLKA. A distinction is made for the calculation of the parameters a, b, and J for cavity walls based on the level of interaction and independency of the two leaves. As J represents the rotational inertia of masses, the outer leaf's J is always accounted for according to the Annex H requirements unless the independency of the leaves is proven by the engineer. Where the two leaves behaviour is evaluated to be strongly dependent and strength contribution of the outer leaf is present, also the parameters a and b of each leaf are combined. In addition to the requirements of the new Annex H, for the peace of mind of the engineer, it is recommended to support his/her calculations with a separate calculation for each leaf.
The text of the annex does not explicitly state how the spanning direction of the walls is decided and defined but Annex H, Figure H.1 provides examples to support conditions and diagonal cracking patterns. It is interpreted that for a wall to be considered as 2 way spanning at least one vertical edge shall be supported.
Section H.4.2 of the annex is dedicated to one-way horizontal bending and two-way bending. At first look, it is uncommon for a wall element to undergo pure horizontal bending under out of plane loading but it is possible for portions of a wall to act in horizontal bending. As a practical example, for a wall that is restrained at the vertical edges, the part of the wall located above a large opening lacking top restraint, experiences a horizontal bending. After the introduction of a new resistance calculation methodology, the code committee adds a simplified flowchart for the seismic assessment of out of plane behaviour for the practical implementation of the engineers. As NLKA for one way bending has potentially a larger margin of safety against collapse compared to the virtual work method for two-way bending, the flowchart follows a sort of tiered approach. Regardless of the vertical boundary conditions at the side of the wall, safety verification is recommended to be performed based on the NLKA. It means that the engineer does not need to consider the presence of vertical supports in the implementation of NLKA. Unless the verification is satisfied, a more accurate calculation is recommended starting with a VWM (if conditions exist) until the engineer is convinced that a more accurate calculation will not be a benefit anymore.

Wall elements that are under a combination of vertical and horizontal bending provide sharing of the load within the wall as cracks develop. Two-way bending is therefore a more desirable behaviour than one-way vertical bending. As shown in the idealized cracking patterns Annex H, Figure H.1, diagonal cracks form as long as the wall is supported from at least one side and the strength of the wall is strongly influenced by the bending capacity along crack lines. The method assumes that the vertical crack lines can form through masonry units or through the mortar joint. Thus, the equation H.18 is formulated in 2 versions to indicate the horizontal bending moment capacity per unit crack length. The flexural strength along the diagonal crack is expressed as diagonal bending moment capacity and is strongly influential on the overall strength. As a general parameter to predict the load capacity of the wall element, the peak cracking resistance is defined in the annex. The two-way bending calculations are also applicable for the wall elements with openings. It is understood from the equations that the masonry above and below the openings is not taken into account to calculate the PCR. Note that the new annex gives recommendations and references for estimating the value of R_{f1}, R_{f2} factors that are needed for the calculation of the k_1 factor. While the guidance of the committee is helpful, the engineer could consider also the other options and evaluate the effect of the chosen value on the resistance. The ultimate judgement belongs to the engineer as long as the engineer shows that the requirements of the annex are fulfilled in reaching a meaningful value.

4.3.7 NLTH

The NLTH procedure implies the use of 7 or 11 ground motions to run a nonlinear time history analysis with further guidance in NEN-NPR9998, Annex F. Other than the previous analysis and assessment methods, NLTH is the most suitable type of analysis to evaluate the performance of irregular and complex buildings. However, it is recommended that an NLTH analysis is preceded by a simplified method, such as NLPO in general or the SLaMA spreadsheet type of crosscheck in particular, to properly evaluate the performance of the building. It is noted here this crosscheck is not intended to make direct comparisons with NLTH results but to gain insight about the behavior and weakness of the building for the reasonableness of advanced NLTH simulation results.

With the acceptance of the typology-based assessment approach which is the application of HRA in its model chain calculation by TNO, the demand for having approximately similar performance predictions from NPR 9998 based assessments is increasing. Though the safety goals of the two approaches are in terms of 10^{-5} personal fatality risk, the way of and also the verification of achieving that safety level is significantly different. However, consistent end results in terms of seismic upgrading requirements are needed.

The micro model based NLTH analyses with an explicit check generally give slightly more realistic results in terms of global in-plane failure mode compared to the indirect check results. The main observed difference in micro model based NLTH results in respect to other approaches is due primarily to the local out of plane failures. As the out of plane response of the walls is accurately modelled for an explicit check and the explicit check is not bounded by the NLKA based displacement limits(60% for vertically spanning, and 30% for cantilever), the prediction of the performance is less bounded by the assumed displacement limits. In addition to this, due to the increased OOP collapse criteria in typology approach based assessments, explicit NPR based results can give more consistent results. For these reasons, in NLTH projects explicit modelling of OOP behaviour and check is encouraged only with the demonstration of relevant research and reliability. It should be noted that the validation of the accuracy of the software and modelling technique in predicting the out of plane response up to the collapse point should be part of the validation process. Due to the complex simulation of the interaction of in-plane and out of plane response at large displacements, the reliability of the out of plane behaviour simulation in explicit checks should be additionally controlled. Where the achieved displacements of the considered wall in the out of plane directions exceed the displacement limit enforced by implicit check, the engineer should also verify and report the capability of simulating large displacement response of the wall and the amplification factor to cause the explicit collapse of the component wall in the TVA.

An NLTH analysis can be performed with or without implementing SSI by suitable documentation of the reason of decision and consistency with seismic assessment and retrofit philosophy. The most advanced/time-consuming modeling choice of boundary conditions between structure foundation and soil can be chosen if the engineering contractor shows the major effect on the end conclusion of the individual building(retrofit decision, quantity of retrofit). For NLTH analysis the ground motions are provided in the NEN-webtool at surface level. The engineering contractor/consultant is responsible for assuring that the requirements in Annex F.2 are carried out. In this respect, through validation it has to be clearly shown that the software package(s) used for the NLTH analyses, have the ability to replicate the non-linear dynamic behavior of typical buildings from the Groningen region (considering the Dutch way of building with slender cavity wall structures and different floor and roof systems).

Typically the masonry building stock in Groningen may show high natural frequencies where SSI effects could make a difference for the force and deformation demands relative to a fixed base analysis. It has been observed that lack of experience and overconfidence with the SSI modelling may lead to incorrect results and upgrading measures. NCG therefore states that it is mandatory that the consultant performing SSI in conjunction with NLTH, will continuously develop their skills and show to be sufficiently skilled in this type of analyses and the interpretation of the outcome/results.

The key elements of an NLTH procedure in the guidance of NEN-NPR9998 are:

- The structural model should be three dimensional and its discretization has to be considered/proven adequate for the dynamic analysis mentality of capturing the frequency range of interest. Structural modelling/detailing is to be achieved through Annex F.3
- The ground motion effects have to be considered in two horizontal directions and vertical direction simultaneously according to Annex F.1.
- Unless otherwise stated, intrinsic damping will not be taken more than 5%.
- For buildings that experience a premature collapse in the analysis, the response of the existing building is not observed thoroughly. For these collapse vulnerable buildings, the engineering contractor should scale down the intensity of the ground motion in proportion to a sufficient intensity in order to display the response. The decision of the intensity can be justified by the engineering contractor, NCG recommends to use 70% scaling.
- For performing an explicit NLTH, an agreement with NCG about the technical validity of the potential simulations are required. Where an explicit check is performed, the engineering contractor has more flexibility and is not limited to the drift criteria of NEN-NPR9998. For an explicit check, the important consideration is the capability of the simulation in capturing the collapse process, the presence of an agreement between parties about the level of detail and complexity of the computational modelling.
- The ground motion factor (γ_n) is assumed to be applied to the time history results, not to the reference depth input.
- The variation of foundation ground motion from the surface ground motion is considered to be significant when the size of the foundation is relatively large. For buildings with a raft (mat) foundation and the building's main contributing modal periods (T) are less than 0.5 seconds, free field motions can lead to conservative results. Possible effects have to be considered as part of the final reporting.
- The demands on foundation piles can be significantly amplified due to the soil structure interaction (SSI). To obtain the final displacement and force effects with indirect SSI, the absolute value of displacement and force effects shall be added from kinematic and inertial calculations when the peak ground acceleration exceeds 0.15g.
- The displacement demands under the shallow foundation should remain within reasonable limits and in the definition of NEN-NPR9998, a differential deformation less than 20 mm/m is considered as negligible. For soft clays and saturated cohesionless loose soils, the stiffness and strength loss should be considered in calculating the displacement demands. A numerical study by the joint effort of Fugro and BICL [11] was performed for Groningen soil conditions and typical housing. The displacements and rotations at the foundation interface between soil and a shallow foundation system, are not expected to be high, due to the flexibility in the soil. The engineering contractor/consultant is encouraged to make use of this study and the directions in NEN-NPR9998, Section 10.3, Figure 10.1 and perform fixed base NLTH analysis together with the required foundation checks.
- Where a SSI approach is required, the horizontal dimensions of the soil model on each side of the foundation should not be less than three times the building width.
- Where a SSI approach is required, the ground motion is generally defined at bedrock level. In the Groningen case, the bedrock is found at a very high depth. Due to this limitation Annex F.3.8.2.3 recommends soil modelling to continue to a depth where the shear wave velocity (Vs) reaches 300m/s.

- Other indirect SSI approaches with p-y, t-z and q-z springs are not included in NEN-NPR9998 and the necessary guidance is not provided. If other methods are used by the engineering contractor/consultant, it has to be communicated with NCG before commencing of the analyses.
- The SSI approach has to be considered for buildings with deep subterranean (basement) levels.

4.3.8 NLTH Limitations

The following limitations, areas of concern towards the use of the NLTH approach require attention:

- Sensitivity analyses have to be performed until enough confidence is gained. Evidence of this study/these analyses have to be part of the final reporting.
- The analyses results are totally reliant on appropriate ground motions.
- The proper location in the model to apply the ground motions.
- Detailed study of analyses results (post-processing) require sufficient time and effort. Limited analyses or short cuts may have to be well described as their results may lead to inaccurate conclusions.
- Due to its complexity the use of the NLTH approach often requires (additional) independent review.

4.4 Assessment of structural safety

If the results of simplified seismic performance evaluations ensure compliance with near collapse state, no further action is required. For buildings requiring a more detailed evaluation, the compliance of the building and its components can be sought with a more comprehensive method.

4.4.1 Knowledge level and Confidence factor

NEN-EN 1998-3 guides the Engineer to define the “Knowledge level” that is valid for an individual building and then assigns “Confidence Factors” (CF_{KLi}) which are used to reduce element properties, referring to the capacity/strength and stiffness of materials/components. Within NEN-NPR9998, Section 4.6.1, the knowledge factor (κ) is set to 1,0. In annex A, the inspection protocol for existing buildings is provided. NCG confirms the general use of a knowledge factor of 1,0 for cases that satisfy the requirements of Annex A.1 and Annex A.2. In case of concerns about specific building knowledge, as listed below, it is proposed to reduce material properties using a confidence factor larger than 1,0.

- Where non-compliance exists with the information needed in Annex A.1 and Annex A.2
- Where a large amount of conflict exists between the construction documents and inspection documents
- For materials not covered specifically in NEN-NPR9998

The confidence factors for NEN-NPR9998 assessments to be used are summarized in Table 14. Guidance on the use of confidence factors is given in Table 15.

Knowledge factor assumptions are to be documented, communicated and agreed in writing with NCG as part of the Building Knowledge Check List (BKCL) procedure. All seismic upgrading measures will be implemented with a CF of 1,0 and designed using conventional new build design capacities, unless agreed otherwise with NCG.

Table 14 Knowledge levels and confidence factors for NEN-NPR9998 assessments

Knowledge level	Confidence factor to reduce existing component material properties
KL_1	CF_{KL1} = 1,35 (in case of great specific concerns)
KL_2	CF_{KL2} = 1,20 (in case of specific concerns)
KL_3	CF_{KL3} = 1,00 (general cases)

Table 15 Guidance on the use of the confidence factor

Component material	Confidence factor (CF_i)		Commentary
	Strength	Stiffness	
Soil	No [1]	No [1]	Not applicable as it is handled via the lower bound, best estimate and upper bound soil properties approach
Masonry	Yes	Yes [2]	Masonry stiffness is affected by strength
Timber	Yes	Yes [2]	Timber stiffness is affected by strength
Concrete	Yes	Yes [2]	Concrete stiffness is affected by strength
Structural Steel and steel reinforcement bars	No [3]	No	Steel stiffness is not sensitive to strength.
Bolts, Anchors, Nails etc…	No [3]	No	
[1]	The NEN-webtool (http://seismischekrachten.nen.nl/map.php) provides surface spectra for the defined grid areas. Where localized soil conditions are highly variable, a more cautious approach is required.		
[2]	If stiffness reduction elongates the buildings main periods of vibration such as to reduce seismic demands, then the Confidence Factor (CF) shall only be applied to reduce a components capacity (i.e. stiffness shall not be reduced). If the stiffness reduction increases the demands, then CF shall be applied to both the strength and stiffness.		
[3]	A cautious approach should be adopted for steel manufactured before 1945. Presence of structural cast iron (gietijzer) should be treated on a case by case basis.		

4.5 Base Isolation

By implementing base isolation as an upgrading measure, a (masonry) building is isolated from the seismic excitation by using (steel, rubber, pendulum) isolators. Where due to structural weakness or monumental value a seismic upgrading by the means of base isolation is chosen, it can be an alternative solution to demonstrate the compliance with NEN-NPR9998. Base isolation is not considered as an ordinary upgrading measure in Groningen case and the implementation as a seismic upgrading measure in exceptional cases requires communication and agreement with NCG before the start of engineering(as a first step). At this moment and until the next release of the ABSC, NCG encourages the use of the following procedure/criteria:

- MRS and NLTH analysis are allowed for the design of base isolated buildings.
- If the building is highly irregular in shape/structure and the vertical mode of vibration has a period exceeding 0.1 seconds, the use of a NLTH analyses is encouraged.
- The minimum requirement for the through base isolation upgraded building, is to satisfy the requirements of Near Collapse (NC) limit state. The preference however, is to have a better performance than the minimum requirement.
- The maximum allowed behaviour factor to be used in MRS analyses is 1.5.

4.6 Assessment and measures for existing buildings

4.6.1 General

For the verification of upgraded buildings, the analysis methods given in NEN-NPR9998, Section 4.3.3 are to be used. The elements comprising the upgrading measures have to be designed in compliance with all relevant Dutch codes and standards. After the minimum required amount of upgrading measures has been determined to satisfy the NC limit, the model with proposed measures has to be run again. If the upgraded model satisfies the performance criteria, the engineering contractor/consultant must evaluate the outcome and investigate a possible reduction of the set of measures in order to come to an optimized solution. This process may require 1 or more (re-)runs.

The NEN-NPR9998 does not specify the criteria for upgraded buildings. By lack of information (at this time) the criteria (drift limits in most cases) used to define the NC limit state are to be applied also for the upgraded building structures, until further notice.

4.6.2 Assessment of an individual building

In addition to NEN-NPR9998, Section 4.6.2 the Building Knowledge Checklist (BKCL) should be completed and agreed upon with NCG before commencing the BSC. The BKCL is available with NCG as part of the documents/templates to be supplied with the NCG Vraagspecificatie.

4.6.3 *Possible measures for existing buildings*

The amount of existing masonry buildings in the Groningen building stock is high. In order to prove compliance with the NEN-NPR9998, with respect to the NC performance criteria, the engineering contractors/consultants have to consider all deficiencies that may lead to (progressive) collapsing mechanisms. For example, damage to foundation elements can be acceptable provided this does not lead to a collapse of the building. In a specific study made by Arup and Fugro [5],[6] it has been demonstrated that with the existence of sufficient number of piles to maintain the vertical load capacity of the structure, shear failure of a single pile can be tolerated.

Possible ways of seismic upgrading measures for building structures are summarized in NEN-NPR9998, Section 4.6.4. The analyses of upgraded masonry buildings are to be performed within the framework of NEN-NPR9998, Chapter 9.

The use of the Groningen Maatregelen Catalogus (GMC) portal provides an efficient way of reducing engineering effort in implementing upgrading measures. The GMC contains conceptual (and generic) structural upgrading measures for seismic deficiencies within masonry structures. Where a building has several deficiencies, engineering contractors/consultants are required to recommend a list of upgrading measures with a priority ranking considering the overall level of safety related to each deficiency. The intention of the use of the GMC and its listed set of minimum required upgrading measures, is to reduce the risk of earthquake damage to masonry buildings, not to prevent earthquake damage.

In the investigation of the as-built performance of the building, the engineering contractor is responsible to make representative assumptions. For example, if a certain connection is assumed to perform sufficiently/insufficiently and is simplified or idealized in the model, then the sufficiency/insufficiency of this connection should be numerically verified. In principle, this verification can be based on a straightforward approach by implementing a demand/capacity ratio for the component. Where the component size, spacing are not known perfectly, a prediction based on the normal design practice at the time of the original design by accounting for aging can be implemented. If the capacity of the component is shown to not satisfy the demand, then an upgrading of the component is required.

5 Specific rules for concrete structures

5.1 Concrete properties

Material properties for concrete shall be taken from NEN-EN 1992-1-1, Table 3.1 for the mean cylinder strength, which is equal to the characteristic cylinder strength plus 8MPa. Additional material properties (e.g. Young's modulus and strain limits) can be found in NEN-EN 1992-1-1, Table
] 3.1. For the recommended stress-strain relation for compression and tension refer to NEN-NPR9998, Section 5.1.2.

The concrete strength for a particular building follows from original design documentation (drawings, existing calculations, specifications) or can be determined by testing. If no information is available, a default characteristic cylinder strength of 20MPa can be used (C20/25 according to NEN-EN 1992-1-1, Table 3.1) with a default density of 2400kg/m^3 for unreinforced concrete and 2500kg/m^3 for reinforced concrete. This should be clearly stated in the BKCL.

For older qualifications of concrete grades (such as the K-types for the pre-1974 era) reference can be made to Ref. [7] Table 2.2. The characteristic cylindrical compressive strengths, derived from this source, is shown in Table 16 below.

Table 16 Qualification of concrete grades

Quality	f_{ck} [N/mm^2]	Quality	f_{ck} [N/mm^2]
K150	8,0	B12,5	10,0
K160	9,0	B15	12,0
K200	11,0	B17,5	14,0
K225	13,0	B22,5	18,0
K250	13,5	B25	20,0
K300	19,0	B30	25,0
K400	28,0	B35	28,0
K450	32,0	B37,5	30,0
K500	33,0	B45	35,0
K600	40,0	B52,5	47,5
		B55	45,0
		B60	50,0
		B65	53,0

Concrete material properties develop over time due to ongoing hydration and may be increased by using the formulations provided in NEN-EN 1992-1-1. For ease of reference the percentage of increase of concrete properties, for a concrete material in good condition and known to be at least 10 years old, shall be taken as per Table 17.

Table 17 Percentage of change in concrete properties (good condition, > 10 years old)

Parameter	Percentage of change
Compressive strength	+15%
Tensile strength	+10%
Young's Modulus	+5%

5.2 Reinforcement

The steel grade for the reinforcement bars should follow from structural drawings, existing calculations or specifications. If no information is available, the following grades will be assumed:

- Pre 1965: Grade 220 (f_{yk} = 220MPa)
- 1965 – 1980: Grade 400 (f_{yk} = 400MPa)
- Post 1980: Grade 500 (f_{yk} = 500MPa)

With the default E = 210,000 N/mm^2 and ν = 0,3 for all grades.

The mechanical properties for reinforcing steel should be taken from NEN 6008 and NEN-EN 1992-1-1, Section 3.2.7 and Table C.1 of Annex C. The expected mean strength shall be derived from the stress-strain relation of reinforcement steel according to the schematized diagram in NEN-EN 1992-1-1, Section 3.2.7 and with the modifications provided in NEN-NPR9998, Section 5.1.2.

Material properties for steel reinforcing bars may be taken from Table 18 (source: Table 2.6, Ref.[7])

Table 18 Quality of reinforcement grades

Quality	f_{yk} [N/mm^2]	Ductility	Quality	f_{yk} [N/mm^2]	Ductility
1.B	220	B	QRn32	320	A
St. 37	220	B	QRn36	360	A
L. St. 52	340	B	QRn40	400	A
Sv 36	360	B	QRn42	420	A
Sv 48	480	B	QRn48	480	A
QR22	220	B	QRn54	540	A
QR24	240	B	FeB220	220	B
QR30	300	B	FeB400	400	B
QR32	320	B	FeB500	500	B
QR36	360	B	FeB500 HKN	500	A
QR40	400	B	FeB500 HWN	500	A
QR42	420	B			
QR48	480	B			

5.3 Level of deterioration and damage

If there are visual signs of (significant) deterioration due to rusting, rot, concrete spalling or similar for reinforced concrete components, the engineering contractor/consultant is to advise NCG on measures to treat or stop the deterioration in such way that a reliable seismic assessment of the building is possible.

Measures can include the replacement or local demolishment and rebuild of the affected sections that are part of the seismic load path of the building. For other deteriorated or damaged parts which are proven not to be part of the seismic load path, the engineering contractor/consultant will make a notification and if needed check if the building is safe using the NEN8700.

For minor deterioration the engineering contractor/consultant may advise to treat the deterioration locally, to stop progressive deterioration and use reduced material properties or section dimensions in the seismic assessment. For concrete, deterioration affects the material strength(s) as well as the stiffness(es) which are derived from the material strength.

All advice on deterioration or damaged parts should be part of the final VA-report of the building.

6 Specific rules for steel structures

6.1 Material properties

Material properties for structural steel shall be taken from NEN-EN 1993-1-1 Table 3.1 for hot rolled structural steel (The nominal values of material properties provided should be adopted as characteristic in the design.)

The mean yield strength is taken as 1.1 times the characteristic yield strength, as per ASCE 41-17, Table 9.3 where lower bound and expected strength are intended as characteristic and mean strength respectively.

For non-linear analyses, NEN-EN 1993-1-5 Figure C.2 is to be followed, with a strain hardening equal to E/100 ($f_{u;i} = f_{y;m} + E/100 \cdot \varepsilon_i$). The ultimate capacity of a steel cross section is defined by either the ultimate (tensile) stress capacity or a strain of 5%.

The steel grade for a particular building should follow from original design documentation (drawings, existing calculations, specifications) or can be determined by testing. If no information is available, a default grade S235 can be used.

6.2 Level of deterioration and damage

Steel, deterioration (i.e. corrosion) implies that the net section is less than the original section. Therefore the section thickness needs to be reduced whilst maintaining the actual material stiffness (E-modulus) properties.

In case significant deterioration or damage is observed and the structural integrity is at stake, the engineering contractor/consultant is to advise what measures have to be applied to rehabilitate the structural integrity of the steel component. For other deteriorated parts which are proven not to be part of the seismic load path, the contractor will make a notification and check if the building is safe using the NEN8700.

All advice on deterioration or damaged parts should be part of the final VA-report of the building.

7 Specific rules for steel-concrete structures

See sections for Concrete and Steel structures. No additional guidance to NEN-NPR9998, chapter 7.

8 Specific rules for Timber structures

8.1 Material properties

Material properties for timber shall follow from NEN-EN 1995-1-1. Timber strength classes can be found in NEN-EN 338. Only characteristic values for strength properties are typically specified for timber.

Guidelines indicate a conversion factor (characteristic to mean values) of between 1,3 to 1,5 for solid timber sections. Although connections nearly always govern the assessment of timber frames, if there are any circumstances where material strength governs, it should be recognized that the mean material values derived using the above recommended conversion factor are only approximate and they could have relevant deviations.

The mean strength of timber material properties may be taken equal to 1,3 times the characteristic values, as per NEN-NPR9998, Section 8.2.1.

The timber grade for a particular building should follow from original design documentation (drawings, existing calculations, specifications) or can be determined by testing. If no information is available, a default grade C18 can be used for post 1945 buildings and C14 for pre 1945 buildings.

According to NEN-NPR9998, Section 8, timber only shows ductile (failure) behavior when loaded in compression. Under flexure, shear and tension, timber shall be considered as being brittle without ductility or plastic capacity.

8.2 Timber sheeting

Material properties for timber sheeting can be obtained from local suppliers' catalogues in the absence of material tests or relevant properties listed in building codes or standards applicable to the year of construction of the building. Characteristic values for modern timber panels/sheeting can be obtained from:

- BS EN 12369-1:2001 for Orientated Strand Board (OSB), particleboards and fiberboards
- BS EN 12369-2:2011 for plywood

Assumed expected material proprieties for some common timber sheeting are presented in Table 19.

Table 19 Expected material properties for common timber sheeting

Material Property	Birch Plywood Sheeting	Spruce Plywood Sheeting	OSB Sheeting
Shear Strength [N/mm^2]	13,3	-	9,5
Density [kg/m^3]	650	450	550
Young's Modulus [N/mm^2]	8000	4000	3500
Shear Modulus [N/mm^2]	620	-	1080
Poisson's Ratio	0,3	0,3	0,3

For the complete limit state verification of buildings with timber floors, the verification of the response of timber components is required and the availability of simplified diaphragm stiffnesses provides a starting point. In NEN-NPR9998, G.9.5.3.4, the shear stiffness formulas dependent on the geometry of flexible timber floor diaphragms (planks) are provided for existing diaphragms that respond in the direction perpendicular and parallel to the joists. In-plane experimental tests were conducted to investigate the failure modes, load resisting capacity, stiffness and strength degradation of the specimen. The force-displacement response of the tested diaphragms is given in TU Delft test report[14]. In both directions, the test specimens did not show any strength degradation while in the direction longitudinal to the joists, significant stiffness degradation was observed with increasing displacements up to 65 mm. The current knowledge from the assessments and experiment shows that the floor upgrading decisions are rather than strength more displacement driven. In the case of cantilever roof diaphragm response, the shear stiffness values which are independent of the geometry are provided for small displacement and large displacement scenarios. It should be understood that the number of experimental tests is limited and the useful recommendations of NPR 9998 for diaphragm stiffness calculation may not cover all types of roof structures present in Groningen. For structural systems that differ from the reference cases, the engineering contractor can make use of the experimental test results from literature and advanced simulation for predicting the realistic response of the component.

Additional provisions for the use of timber sheeting for the primary seismic systems are given in NEN-NPR9998, Section 8.2.3. These provisions may be interpreted as follows: when the timber sheeting, part of the lateral system, does not comply to formula 8.3, then don’t use this system in the horizontal load path or change the system. If the system complies to formula 8.3, but does not comply to table 8.5, the use of DCH is not allowed. If the system complies to table 8.5, the use of DCH for the q-factor is allowed.

For renovation and assessment of existing buildings DCL should be used according to NEN-NPR9998, Section 8.3.

8.3 Level of deterioration and damage

Timber deterioration (rot, woodworm, etc.) should be treated on a case by case basis depending on the cause and extent of the deterioration. The engineering contractor/consultant is to advise on measures to treat the deterioration in such way that a reliable assessment of the building is possible.

Measures can include the replacement or local demolishment and rebuild of the affected sections that are part of the seismic load path of the building. For other deteriorated parts which are proven not to be part of the seismic load path, the contractor will make a notification and check if the building is safe using the NEN8700.

For minor deterioration the engineering contractor can advise to treat the deterioration locally, to stop progressive deterioration and use reduced material properties or section dimensions in the assessment.

All advice on deterioration or damaged parts should be part of the final VA-report of the building.

9 Specific rules for masonry structures

9.1 Material properties

Material properties for masonry can be obtained using one of the four methods described in NEN-NPR9998, Section 9.3.2.1.

For both NLPO and NLTHA the mean value of material properties of masonry are as per NEN-NPR9998, Table F.2. Besides the material properties for the four types of masonry constructions provided in the NEN-NPR9998, the following additional parameters are assumed for analyses purposes:

Table 20 Additional material properties for masonry

Material Property	Clay brickwork (pre 1945)	Clay brickwork (post 1945)	Calcium-silicate brickwork (~ 1960-present)	Calcium-silicate blocks with thin layer joints (~1985 – present)
Density [kg/m^3]	1950	1950	1850	1850
Poisson's Ratio[1]	0,25	0,20	0,21	0,25
[1] The Poisson's Ratio is assumed equal to; $PR=E_m/2G_m-1$				

The density of perforated bricks has to be calculated or assumed based on the brick geometry or the values from the manufacturer shall be assumed (if available). Appropriate surface area should be taken into account for the assessment.

9.2 Wall ties

In case the condition of the wall ties is not known/specific building information is not available, the default wall tie distribution/spacing and quality shown in Table 21, is to be adopted.

Table 21 Assumed wall tie spacing, diameter and quality (in case of no relevant information)

Year	spacing [$ties/m^2$]	nominal diameter (mm)	Quality, probability of corrosion	Reference for wall tie spacing
Pre-1965	1	4,0	High	-
1965 - 1979	1,67	4,0	High	"Modelbouwverordening" 1965-1989 [8]
1980 – 1990	1,67	4,0	Low	"Modelbouwverordening" 1965-1989 [8]
Post-1991	4* / 6**	4,0	Low	D6791
*building height < 11m **11m < building height < 20m Tie requirements for buildings above 20m in height need to be considered on a case by case basis.				

In masonry cavity walls built before 1980 there is a high possibility that the wall ties, most probably made of plain or galvanized steel with poor corrosion protection, are now either completely corroded or in an advance stage of corrosion. This assumption is corroborated by the findings of survey studies commissioned by VROM (Ministerie van Volkshuisvesting, Ruimtelijke Ordening en Milieubeheer) to report on the reduced functionality of masonry wall cavity ties because of corrosion for building constructed between 1945-1980. Refer to VROM-Inspectie: "Constructieve veiligheid gevels en glazen overkappingen"[9].

9.3 Level of deterioration and damage

During the initial inspections (detailed inspections or less detailed validation inspections) it is important that the engineering contractors/consultants acquire knowledge of the level of deterioration and existing damage of the buildings' structural materials, as they influence building behavior. Table 22 and Table 23 are complementary. Table 22 gives information about the consequences of the material quality selection for engineering. The following approach is proposed for the reduction of material properties of masonry based on the visual inspection of the masonry quality:

Table 22 Engineering approach masonry quality after visual inspection

Level	Engineering approach with regard to material properties
Excellent	This refers to an element that is still in constructed new conditions. This condition is satisfied almost only with recently constructed buildings. The BSC shall be undertaken assuming 100% of the material properties.
Good	This refers to an element where simple defects that would not affect the overall performance are existent. The element is capable to carry its structural function. The BSC shall be undertaken assuming 80% of the material properties.
Fair	This refers to an element where the defects are clearly visible and maintenance type of remedial action is necessary. The poor state of maintenance starts to affect the surrounding elements. The element is only partially capable to carry its structural function. Replacement or enhancement is required to have a better level of state. The BSC shall be undertaken assuming 60% of the material properties.
Poor	This refers to an element where severe defects are visible. The combination of the poor state of maintenance and experienced vibrations cause the enlarged cracks. The element is only partially capable to carry its structural function. Breaking out and replacing sections of the element is required. The BSC shall be undertaken assuming 40% of the material properties.

Some key points related to the use of deterioration factors are:

- The approach is not aimed to assess the safety level of earthquake damaged buildings. For earthquake-damaged buildings that have significant deflections, broken door and window frames with its load-carrying elements losing their integrity; a more refined approach should be agreed based on the unique situation.
- The reduction is not to be applied to the friction coefficient of URM to other components.
- Deterioration factor assumptions are to be documented, communicated and agreed in writing with NCG as part of the BKCL list process.
- New materials applied to a structure as part of an upgrade will be assumed to have nominal material properties in line with new constructions.
- For masonry, deterioration affects the material strength(s) as well as the stiffness(es) which is derived from the material strength.
- All advice on deterioration and underpinning of the masonry properties above shall be part of the Versterkingsadvies (VA) of the building.

In Table 23 guidance is given how to assess/categorize the level of deterioration and damage of unreinforced masonry.

Table 23 Guidance on the level of deterioration and damage for URM

Level	Commentary
Excellent	Masonry that is masoned recently with the best possible practice and no visible cracking.
Good	Masonry found during condition assessment to have mortar and units intact with minor mortar cracks up to 1mm.
Fair	Masonry found during condition assessment to have mortar and units intact but with cracking up to 10 mm through mortar or bricks. If the number of cracks are extensive, it may require the engineer to go one level below. To evaluate this, an engineering judgement is needed taking into account the type of crack, crack patterns, the origin of the crack, etc. The masonry would typically show signs of ageing in excess of what would be considered normal ageing of the material based on building construction year (or year of construction of the URM element considered) possibly associated with average/low general maintenance.
Poor	Masonry found during condition assessment to have degraded mortar, degraded masonry units, or significant cracking. Typical crack widths are above 10 mm but it depends on the number of number cracks as well. For example, a wall covered with several of 8 mm cracks could also be classified in poor condition. The masonry would typically show extensive areas where mortar has shrunk away from the joints (powdered or eroded) and/or cracks are not local and they affect the mortar and / or the units. The erosion of the mortar often causes the deterioration of the surrounding bricks as well. The masonry would typically have very evident signs of ageing in excess of what would be considered normal ageing of the material based on building construction year (or year of construction of the URM element considered) possibly associated with poor maintenance.

10 Foundations

10.1 General

NEN-NPR9998 definition of near collapse limit state does not ban damage to foundations, if the foundation damage has a negligible effect on the probability of collapse of the building. In order to have a consistent assessment approach with superstructure evaluation of NEN-NPR9998, several studies have been done by the instruction of NEN. The relevant studies have been used by NEN to derive simplified methods especially for the assessment of shallow foundations. The engineering contractor shall be familiar with studies performed about reinforced concrete pile capacities and foundation assessments. The relevant reports are also referenced at the end of the ABSC document.

10.2 Criteria for the assessment of Liquefaction

The potential for liquefaction shall be assessed when peak ground acceleration ($a_{g;d}$) is greater than 0.125g (at a 2475 year return period), using the method specified in NEN-NPR9998, Section 10.2. The impact of liquefaction on shallow foundations is considered in accordance with NEN-NPR9998, Figure 10.1. The engineer is directed to the use of LPIish maps for a stepwise assessment approach. LPIish is to provide information about the liquefaction induced damage. The latest maps are available on http://seismischekrachten.nen.nl and conclude that liquefaction induced damage is limited with LPIish lower than 5 in almost all locations for the time period of T4,T5 and at all locations for the time period of T6 [13].

10.3 Shallow foundations

For the assessment of shallow foundations the method described in NEN-NPR9998, Section 10.3 should be followed. When a situation is encountered that does not comply with the range of validity of this method, this shall be reported to NCG.

First step in the foundation assessment is to check the building for any signs of foundation related damages. The simplified assessment procedures is based on the assumption of buildings without foundation related problems. Foundations problems can be summarized as buildings that are tilted or extensively cracked due to foundation settlements. For buildings with existing foundation related damages, a unique analysis and assessment approach shall be discussed with NCG.

For general information regarding some of the parameters used in the method, please refer to statements noted below:

Q: The expected best estimate value without factors shall be used.

Cu: The undrained shear strength can be calculated with the formula from NEN-EN 1997-2, section 4.3.4.1. (3)

When no information is available, it can be estimated by back-calculating the undrained shear strength from the static foundation pressure.

(3) If the sample analytical method for bearing resistance of Annex D in EN 1997-1:2004 is used, the undrained shear strength of fine soil, (c_u) may be determined for a CPT from:

$$c_u = \frac{q_c - \sigma_{v0}}{N_k} \tag{4.1}$$

Or, in the case of a CPTU, from:

$$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{4.2}$$

where

q_c	is the cone penetration resistance
q_t	is the cone penetration resistance corrected for pore water pressure effects;
N_k and N_{kt}	are coefficients estimated from local experience or reliable correlations
σ_{vo}	is the initial total vertical overburden stress at the depth under consideration;

Nk Cone factor – Nk can be taken as 14 (average) as a first estimate. Judgement on the outcome can be used to verify the need of more accurate data.

10.4 Pile foundations

The assessment of pile foundations in terms of liquefaction, structural and geotechnical limit states are clearly stated in a scheme in the background document of NEN-NPR9998 [16]. The reader is recommended to get familiar with the document. The geotechnical capacity of pile foundations is to be calculated in accordance with NEN EN-1997:2017 and NEN-NPR9998, Section 10.4. Partial factors (loading and material) for soil and foundation capacity do not need to be applied when the building is assessed to the near collapse (NC) limit state. Definition of the NC limit state for below-ground structural parts may pose a challenge. For this reason, [16] gives a more consistent definition of NC state and the contribution of foundation elements to the limit state. According to the latest insight, unless the PGA(T_{LS}=2475 years) is greater than 0.15g, a structural assessment of pile foundations for NC limit state is not required. For locations with higher seismic shaking, a scheme for the structural assessment of piles are provided in Achtergrondrapport NPR9998 Funderingen.

When pile foundation stiffness is modelled together with a building, the stiffness shall be assessed using Q-z, t-z and p-y springs (end bearing, skin friction and lateral respectively). If assessed separately, standard lateral pile analysis software can be used.

Kinematic interaction between soil and piles occurs due to the incompatibility of the free-field motion and the rigidity of the piles that oppose to the motion of the ground. The piles experience additional deformation, bending, axial and shear stresses. Kinematic demands on the piles shall be assessed by combining inertial loads and relative soil displacements along the pile length when all the following occurs simultaneously:

1. V_s,30 < 180 m/s and the pile is embedded through layers of sharply differing stiffness. For example, a transition between a liquefying and non-liquefying layer is a sharp stiffness change
2. The surface PGA exceeds 0.1g and
3. The building is classified as consequence class of CC2 or greater.

The peak demand can reasonably be estimated by the following load combinations:

- 100% kinematic displacement ± 50% inertial load; and
- 100% inertial load and 50% kinematic displacement.

Available methods for the analysis of kinematic soil-pile interaction are:

- Numerical approaches;
- Winkler methods and
- Simplified formulations such as closed-form expressions [10]
- ARUP 2017b Groningen Earthquakes – Structural Upgrading, Reinforced Concrete Pile Capacity Investigation – Stage 2. [5]
- Fugro 2018b Numerical investigation of the post-failure axial capacity of reinforced concrete piles in Groningen area The Netherlands. [6]

10.5 Impact of soil-structure interaction on building performance

SSI is normally considered to be beneficial to the performance of buildings due to the potential for period elongation and additional 'radiation' damping through the flexibility of the soil domain, at the cost of increased displacement. However, this precept is based upon the assumption that the structural foundation for the entire building is 'rigid' and that the seismic deformation of the soil results only in additional and very small rigid body motion of the whole building. For masonry buildings in the Groningen region this is not always a valid assumption, since local (differential) foundation deformations due to the softness of the soil which can introduce different damage mechanisms compared to those that would arise if the buildings had a truly rigid foundation footprint, as demonstrated in the figure below.

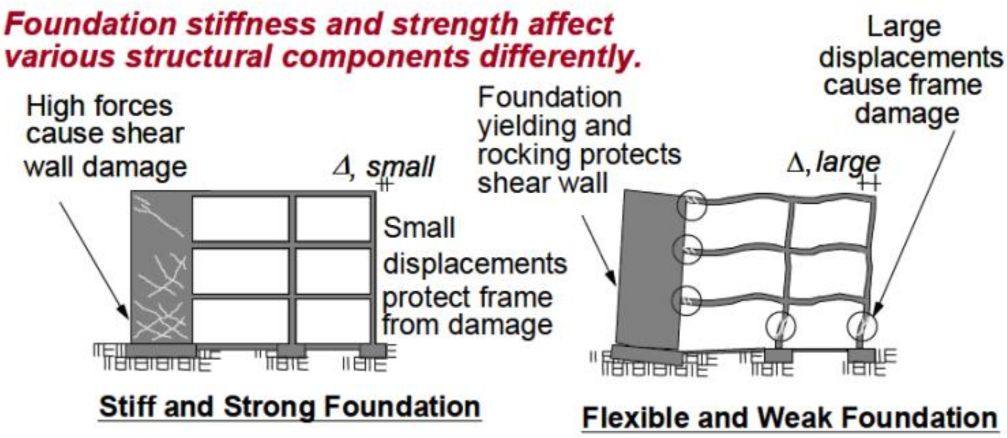


Figure 5: Diagrammatic representation of foundation flexibility on failure mechanisms, reproduced from (NZSEE, 2017).

A 3D nonlinear soil structure finite element model is capable of simulating the ground motions at the foundation soil interface(via site-specific site response) and the dynamic interaction effect on the building. In the development of Groningen ground motion model, site response calculations were performed using the regional site properties in Groningen. The ground motion model-based surface UHS(webtool) and surface ground motions are not a replacement of the products of a site-specific site response because it is not produced for a single site profile. However, for the intensity of shaking in Groningen and for the typical size and weight of the buildings, the surface ground motions are satisfactory to be used as an input to mimic the influence of soil nonlinearity. Soil structure interaction is a complex topic that depends on site-specific parameters such as foundation conditions, as well as assumed model parameters, ground motions and boundary conditions. Simulation of SSI by finite element programs should be validated similar to the validation of simulation of the above-ground structure. NAM has investigated the impact of SSI on collapse prediction by implementing different modeling approaches. For buildings with shallow foundations, the influence of the SSI on the collapse of the index buildings is found to be generally negligible and beneficial. When SSI is included in the simulation, the appropriate range of values

applied for soil properties to take into account uncertainties, the way of modeling, the sensitivity studies should be thoroughly documented. Considering the unknown level of accuracy, the extra time required for the analysis, postprocessing and documentation time, the experience from reference projects in Groningen, the reasonable prediction of GM model; the inclusion of direct SSI in the simulation of buildings with shallow foundations are not encouraged. As a practical matter, acceleration can be achieved in a group of buildings, keeping a sufficient accuracy in the results.
In exceptional cases where the engineer expects a significant increase in the base shear or displacement demand(changing the assessment result), the contractor may decide to include SSI in the simulation.

11 References

References to Dutch and European codes and guidelines per NEN-NPR9998, Bibliografie. Additional references from NCG-ABSC are presented below:

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[11] Evaluation of liquefaction-Induced settlements of typical residential buildings on shallow foundations Groningen, the Netherlands – Fugro, august 2018.

[12] NPR-9998 for liquefaction and foundations, background document for NPR2018 – 1208782-007- GEO-004 Deltares, 20-08-2018.

[13] Addendum for Report NPR 9998 for liquefaction and foundations -11204802-007-GEO-0001 Deltares, 28-07-2020.

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[16] NEN, Achtergronddocument bij H3 en H10_Funderingen van NPR 9998:2020 (module 5), January 2021

[17] NEN, Rapportage resultaten en toepassing verschillende berekeningsmethoden NPR 9998 (module 3), January 2021

[18] NEN, Module 4: Final summary report on differences between NPR9998 and Hazard & Risk Assessment, December 2020