## Contents

1 Introduction work programmes task groups ........................................... 3
   1.1 Introduction work Programme Task Group .................................. 3
   1.2 Terminology ............................................................................. 4
   1.3 Safety Philosophy ..................................................................... 5
   1.4 Outline of report ...................................................................... 5

2 Terminology ..................................................................................... 6
   2.1 Elements ................................................................................. 6
   2.2 Limit states .............................................................................. 7
   2.3 Risk Metrics ............................................................................ 9

3 Safety Philosophy ............................................................................ 10
   3.1 Connection with general safety philosophy of Eurocodes .......... 10
   3.2 Target reliability based on economic optimisation ............... 10
   3.3 Target reliability based on human safety arguments .......... 11
   3.4 Application to NPR 9998 ....................................................... 12
   3.5 Importance factors in NPR 9998 ............................................ 14

4 Return periods of seismic action ....................................................... 16
   4.1 Reliability requirements ......................................................... 16
   4.2 Probabilistic seismic hazard analysis and fragility functions ..... 16
   4.3 Calibration of a semi-probabilistic assessment ...................... 18
   4.4 Procedure to obtain the return period of the design seismic action ............... 18
   4.5 Calculation of return period design seismic action .............. 19

5 Capacity modelling and drift limits .................................................. 22

6 References ...................................................................................... 23

7 Signature .......................................................................................... 25
1 Introduction work programmes task groups

1.1 Introduction work Programme Task Group

The purpose of this Part A of the report is to provide the background of NPR 9998:2018 with respect to new terms used in this version of the NPR in addition to the terms used in NPR 9998:2015, and the safety philosophy. It is noted that the additional Part B of this report deals with falling objects. The drafting of those rules are based on the activities of the NEN Working Group Earthquakes (committee 351 001 01 117).

According to the project plan/tender document of NEN dated 31 October 2016, Paragraph 3.4, “Module 1 - Mathematical determination of the seismic resistance of building structures and option to improve”, the following scope and objectives have been defined:

To design and analyze structures in earthquake areas the following four calculation methods are available:
   a. Lateral force method of analysis
   b. Modal response spectrum analysis
   c. Non-linear pushover analysis
   d. Non-linear time history (dynamic) analysis

Methods a) and b) are based on elastic behavior. They are especially suitable for the design of newly built structures. The fact that these methods give mostly conservative outcomes, is not expected as negative and the additional costs are minor. In addition the structure will get an higher degree of robustness which is favorable.

Methods c) and d) take into account the effects of damage which occurs during an earthquake. In principle these methods are more accurate and specially applicable to analyze existing structures for which the determination of more precisely bearing capacity is of great benefit.

For existing structures the non-linear behaviour (e.g. in case of cracks in wall and floors) has a major impact on the seismic resistance. Only the two non-linear methods given by the NPR, the non-linear pushover analysis (method c) and the non-linear time history analysis (method d), are able to simulate this non-linear behaviour within a calculation procedure. The most advanced methods of those tow is the non-linear time history analysis d). But this is also the most laborious and expensive method.

It is known that discussions are held in practice if these calculation methods can be used or not within various frameworks. Also the modeling of the underground which follows from soil probes or other tests, and effects from modeling the underground by these calculation methods should be described in more detail, resulting into more unambiguous methods.

Further development of these calculation methods and formulation of user friendly instructions, will be based on existing knowledge (and possible knowledge that will be developed in other research projects on short terms). To be able to make use of the input from those parties, the quality,
the generic applicability has to be judged and the translation of this input to texts for the NPR standard has to be done. It is assumed that the knowledge developed by NAM as part of will become available to be able to improve the NPR.

The following outcomes are anticipated:

a. Improved reliability of the results of various calculation methods for the situation of Groningen by improved frameworks, rules and guidance for users.

b. Improved estimation of the structural safety of an existing structure which can be used as the basis of a decision to strengthen the structure and how to do this in an optimal way.

It is noted that in this report only background regarding terminology and safety philosophy are considered. The background of other issues regarding the developed of the calculation models will be reported by other parties involved in the task group dealing with “Module 1 - Mathematical determination of the seismic resistance of building structures and option to improve”.

Draft versions of this report have been discussed in the task group 351 011 01 117 TG1, with the following members:

- Lurvink, M. (NEN)
- Mulder, Y., secretary (NEN)
- Allaix, D. (TNO)
- Beazly, P. (BECA)
- Bekkering, A.J. (NCG)
- Besseling, F. (Witteveen+Bos)
- Bijlaard, F.S.K. (TU-Delft)
- Burggraaf, H. (TNO)
- Groot, W. de (SHR)
- Vries, R. de (Arup)
- Hendriks, M. (TU-Delft)
- Hicyilmaz, K. (Arup)
- Jorissen, A. (SHR)
- Kraus, J. (TNO)
- Messali, F. (TU-Delft)
- Muris, V.L.C. (NCG)
- Philippart, M.A. (NCG)
- Pruiksma, J. (TNO)
- Rots, J.G. (TU-Delft)
- Schmersal, E.J.W. (NCG)
- Scholten, N.P.M. (ERB)
- Steenbergen, R.D.J.M., chair (TNO)
- Velis, R. (NCG)
- Walraven, J.C. (TU-Delft)
- Wijte, S.N.M. (Adviesbureau Hageman)

1.2 Terminology

At the beginning of the project a discussion was held about the terminology for elements in NPR 9998:2018 and those of the Eurocodes. Further information is
given in Chapter 2 of this report including a discussion of the limit states and risk metrics considered in the NPR 9998:2018.

1.3 Safety Philosophy

In this Part A of this report the safety philosophy applied in the NPR 9998:2018 is based on the Individual Risk concept and is discussed in detail. In addition the return periods of seismic actions are determined for the global failure (primary and secondary seismic elements). The return periods of seismic actions for local failure (non-seismic structural elements) are determined in detail in Part B of this report. In addition the capacity modelling and drift limits of the calculation procedure of Annex G of NPR 9998:2018 are given in Part A of this report.

1.4 Outline of report

This report provides the following:

- Background of the terms used to classify elements, of terms used to classify limit states and of risk metrics used (Chapter 2).
- Background of the general safety philosophy of the NPR 9998:2018, considered target reliabilities and how to apply these for global structural collapse and local structural collapse (Chapter 3).
- Determination of return periods seismic actions for global structural collapse, importance factors and capacity modelling (Chapter 4).
- Determination of the capacity modeling and drift limits (Chapter 5).
2 Terminology

At the beginning of the project a discussion was held about the terminology in the NPR 9998:2015 and those of NEN-EN 1998-1 (2005). New or modified terminology was introduced regarding elements, limit states and risk metrics.

2.1 Elements

In case of global structural behaviour NEN-EN 1998-1 (2005) distinguished primary and secondary members with the following definitions:

**primary seismic members (according to NEN-EN 1998-1)**

members considered as part of the structural system that resists the seismic action, modelled in the analysis for the seismic design situation and fully designed and detailed for earthquake resistance in accordance with the rules of EN 1998

**secondary seismic members (according to NEN-EN 1998-1)**

members which are not considered as part of the seismic action resisting system and whose strength and stiffness against seismic actions is neglected

It is decided to use the term elements instead of members and to modify the definitions and notes, with small changes within the explaining texts:

**primary seismic element**

structural element that has been considered as part of the structural system which resists the seismic actions

NOTE: In the structural model, primary seismic elements are the primary contributors to human safety during seismic actions.

**secondary seismic element**

structural element that transfers the vertical loads from its own weight and from adjacent elements to the foundations and that does not contribute to the resistance against the horizontal seismic action

NOTE: E.g. intermediate columns.

In case of local structural behaviour a distinction was proposed for the term for non-structural elements as used in NEN-EN 1998-1 (2005), with the following definition:

**Non-structural element (according to NEN-EN 1998-1)**

architectural, mechanical or electrical element, system and component which, whether due to lack of strength or to the way it is connected to the structure, is not considered in the seismic design as load carrying element
The elements which have to be considered as structural elements according to the Dutch Building Decree, are given the term non-seismic structural elements, while for the elements which are not considered as structural elements according to the Dutch Building Decree, are given the term non-structural elements.

For the term non-seismic structural elements the following definition and accompanying note is drafted:

**non-seismic structural element**

structural element that is not intended to transfer the seismic action to the foundations, other than the consequence of its own weight, with local failure of the structural element not leading to global failure

**NOTE 1:** Non-seismic structural elements are structural elements according to 1.5.1.7 of NEN-EN 1990 as laid down in the Dutch 2012 Building Decree. Examples are chimneys, gables, non-load bearing partition walls, non-vertically load bearing leaves of cavity walls, outer leaves of cavity walls, balconies, galleries, balustrades, parapets, decorative structural elements, pinnacles, windows (glazing). It should be noted that if apparently non-seismic elements contribute to the resistance of the seismic action, these elements should be considered as primary seismic elements. This may be the case, for example, with stairwells or façades that are part of the structural system.

For the term non-structural element the following definition and accompanying note is drafted:

**non-structural element**

architectural, installation or electrical element, system or component which is not intended to bear any loads other than its self-weight

**NOTE 1:** Non-structural elements do not offer any structural resistance to the progressive collapse of the structure; they are not part of the structural system. Examples are modular ceilings, piping, cabinets, storage racks, equipment, elevator cabs, etc. Non-structural elements are not part of clause 2.1 of the Dutch 2012 Building Decree. If deemed applicable, the rules for non-seismic structural elements can be applied to non-seismic elements for detailed safety verifications.

### 2.2 Limit states

In the NPR 8889:2018 the following limit states related to the global structural behaviour are used:

- \( DL \) = Damage Limitation
- \( SD \) = Significant Damage
- \( NC \) = Near Collapse

Definitions in the NPR refer to the transition from one global state of damage according to the NEN-EN 1998-1 (2005) to another more serious one (see Figure 2.1 of this report):

- The global limit state \( DL \) may be considered as the boundary between damage states \( DS1 \) and \( DS2 \) (sometimes between \( DS2 \) and \( DS3 \)).
- The global limit state \( SD \) may be considered as the boundary between damage states \( DS3 \) and \( DS4 \).
- The global limit state \( NC \) may be considered as the boundary between damage states \( DS4 \) and \( DS5 \).
These global limit states are primarily intended to be used for the system of the primary and secondary seismic elements. For this case the NPR 8889:2018 assumes that casualties are only considered to be present after exceeding the global limit state NC, that is after entering the damage state DS5.

Figure 2.1  Global limit states NC, DS and DL according to the NPR 9998:2018 and damage stated DS1 to DS5 according to the Eurocode

Just for information it is noted that the following definition of NC is given in the NPR 9998:2018 (Section 2.2.1):

*The load bearing structure that ensures the stability of the building is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-seismic structural elements and non-structural elements may have failed. Large permanent deformations are present. The load bearing structure is near collapse and would probably not survive another earthquake, even of moderate intensity.*

The for draft NPR 9998:2017 intended NC global limit state is further defined in section 3.4.

For non-seismic structural elements another definition of the limit states related to local structural behaviour, may be more useful. Taig (2016A, B and C) proposes the local damage states (non-seismic damage state, NDS) for chimneys according to Table 2.1 of this report, by considering the degree of damage, for instance the relative amount of bricks which falls down.

Table 2.1 – Definition of local damage stages (NDS) for chimneys

<table>
<thead>
<tr>
<th>Local damage state</th>
<th>Degree of damage</th>
<th>Type of damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>NDS$_1$</td>
<td>1-3 %</td>
<td>only a few bricks come down</td>
</tr>
<tr>
<td>NDS$_2$</td>
<td>3-10%</td>
<td>lightly damaged</td>
</tr>
<tr>
<td>NDS$_3$</td>
<td>10-30%</td>
<td>heavily damaged</td>
</tr>
<tr>
<td>NDS$_4$</td>
<td>30-70%</td>
<td>partial collapse</td>
</tr>
<tr>
<td>NDS$_5$</td>
<td>70-100 %</td>
<td>full collapse</td>
</tr>
</tbody>
</table>
It is assumed that a similar classification can be applied for other (masonry) non-seismic structural elements. According to the considerations made in Section 3.1 of Part B of this report, the local damage state $\textit{NDS}_5$ is used as definition for the $\textit{NC}$-local limit state in Table 2.3 of NPR 9998:2018.

### 2.3 Risk Metrics

The following risk metrics are distinguished in this report.

**Individual risk (IR)**

The probability of death for an specified person as a consequence of a specified cause (in this report an earthquake in the Groningen region) in the period of one year.

- **Note 1:** The IR may differ from person to person, depending on his/her activities and whereabouts.
- **Note 2:** The IR may also be interpreted as a frequency with unit [\# / year].

**Local Personal Risk (LPR)**

The annual probability (or frequency) of death per year from specified causes for a hypothetical person located for 100% of the time at a specific location.

**Object related Individual risk (OIR):**

The Object related Individual Risk is the contribution of one object to the IR. It is noted that the safety philosophy of draft NPR 8889:2017 is in principle based on IR only. Effects on larger groups of persons (Societal (Community) Risk) is considered on qualitative terms and is discussed for global structural behaviour in chapter 4 and for local structural behaviour in Part B of this report. In principle the following risk metrics are distinguished:

- **Group Risk (GR):** The annual probability (or frequency) of occurrence of an event involving $N$ or more fatalities among a specified population from a specified cause. This can be done with use of FN-curves.
- **Community Risk (CR):** The average number of deaths per year expected among a specified population from a specified cause.
- **Object related Community risk (OCR):** The Object related CR is the FN-curve related to damage to an object caused by an earthquake in the period of one year.

Relations between the various metrics may be found in Annex A.
3 Safety Philosophy

3.1 Connection with general safety philosophy of Eurocodes

In order to check structures for sufficient reliability information is needed on loads, resistance, failure modes, consequences of failure and safety criteria. Consequences of failure may be related to aspects of human safety as well as to economic losses. The same holds for the safety criteria. The safety criteria for economy require insight into structural costs (or strengthening measures) and the possible losses in case of failure. Also intangibles like the value of human life or the feelings of unsafety might be taken into account. The safety criteria for human life in itself have also ethical aspects.

In (strongly simplified) mathematical terms the decision problem might be formulated as:

\[ \begin{align*}
\text{Min } C_{\text{tot}} &= C_S + P_{F} C_F, & \text{in the lifetime of the structure} \\
\text{Sub } P_{F,\text{a}} &< P_{F,\text{limit}}, & \text{per year}
\end{align*} \]

Where \( C, P, S \) and \( F \) respectively refer to costs, probability, structure and failure. Here the discount rate \( \gamma \) has to be included, the first equation in Equation 1 changes into:

\[ C_{\text{tot}} = C_S + \int_0^T P_F C_F e^{-\gamma t} dt. \]

The limit value \( P_{F,\text{limit}} \) may follow from notions as Individual Risk (IR) or Societal (Community) Risk. This limit value should be understood as the expected value of the failure probability.

The above system has been elaborated for new structures in Eurocode EN 1990 (2011) and the corresponding Dutch National Annex for new structures and in NEN 8700 (2011) for existing ones. Only rough economic criteria (partly based on calibration to old codes) and Individual Risk criteria have been taken into account. Societal (Community) Risk has not been considered explicitly, but is considered to be accounted for using the different consequence classes. For new structures almost always economic criteria are dominant over human safety criteria.

For the assessment rules for existing structures affected by induced earthquakes the NPR 9998:2018 connects to the present Dutch safety philosophy for existing buildings, which is anchored in the 2012 Building Decree through NEN 8700 (2011).

3.2 Target reliability based on economic optimisation

The target reliability index is defined as a substitute for the failure probability \( \rho \), defined by:

\[ \beta = - \Phi^{-1}(\rho) \]

where \( \Phi^{-1} \) is the inverse standardized normal distribution. E.g. a reliability index equal to 3.8 represents a probability of \( 7 \times 10^{-5} \). In ISO 2394 (2015) the target reliability index is related not only to the consequences but also to the relative costs of safety measures as shown in Table 3.1 of this report.
Table 3.1. Tentative target reliabilities related to one year reference period and ultimate limit states, based on monetary optimization (ISO 2394 (2015))

<table>
<thead>
<tr>
<th>Relative cost of safety measure</th>
<th>Consequences of failure</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minor</td>
<td>Moderate</td>
</tr>
<tr>
<td>Large (A)</td>
<td>$\beta = 3.1 \left( P_f \approx 10^{-3} \right)$</td>
<td>$\beta = 3.3 \left( P_f \approx 5 \cdot 10^{-4} \right)$</td>
</tr>
<tr>
<td>Normal (B)</td>
<td>$\beta = 3.7 \left( P_f \approx 10^{-4} \right)$</td>
<td>$\beta = 4.2 \left( P_f \approx 10^{-5} \right)$</td>
</tr>
<tr>
<td>Small (C)</td>
<td>$\beta = 4.2 \left( P_f \approx 10^{-5} \right)$</td>
<td>$\beta = 4.4 \left( P_f \approx 5 \cdot 10^{-6} \right)$</td>
</tr>
</tbody>
</table>

According to ISO 2394 (2015) the target level for existing structures is lower than for new structures as it takes relatively more effort to increase the reliability level compared to a new structure. Consequently for very expensive safety measures one may use the values of one category higher, i.e. instead of “moderate” consider “high” relative costs of safety measures. This is in agreement with the recommendations of the new fib Model Code (2010). Similar recommendation is provided in the Probabilistic model code by the Joint Committee on Structural Safety (2001) and in Steenbergen et al (2015). Recommended target reliability indices are also related to both the consequences and to the relative costs of safety measures.

In Europe (e.g. Eurocodes) in most of the cases the lowest line (Small) is used. In NEN-EN 1990 (2011) the classification in ‘low’, ‘moderate’ and ‘high’ are specified in consequence classes. It seems, from an economical point of view, logical to use a reduction in the case of earthquakes as there the costs are high for the realization of a high safety level. One could even think of a reduction to the first line in Table 3.1 (Large) of this report.

However this economic optimization is bounded by considerations for human safety; this will be discussed in the next section.

3.3 Target reliability based on human safety arguments

Limits for human safety play an important role for design and assessment of structures. The annual probability of failure may not exceed requirements based on individual human safety. The probability, for an arbitrary healthy (relatively young) person to die as a result of for instance an accident in daily life is about $10^{-4}$ per year in developed countries. It is certainly not accepted in society that the probability to become the victim of structural failure is larger than the normal probability to die as a result of an accident. A value between $10^{-5}$ and $10^{-6}$ would be an appropriate requirement for the individual risk for structures, see Melchers (2001).

In the Dutch Code for existing structures NEN 8700 (2011) the limit value for the IR (maximum acceptable probability that a person dies in one year as result of a collapsing structure) has been taken as $10^{-5}$, see Steenbergen and Vrouwenvelder (2010) and Vrouwenvelder et al (2011). For the NPR 9998 also a value of $10^{-5}$ has been prescribed by the government.
The probability $P_d$ that a person dies in one year at a certain location due to structural failure under earthquake load, becomes:

$$P_d = P_f \cdot P_{df}$$  \hspace{1cm} (4)

Here, $P_{df}$ is the conditional probability of casualty given the structural failure and $P_f$ the probability of occurrence of an earthquake.

This thus requires an estimation of probability of fatality given collapse and a check on the collapse capacity of the structures. The former has been observed to be highly correlated with the collapse mechanism and the latter is difficult to explicitly model with most software. In the literature several studies to his conditional probability $P_{df}$ are available.

3.4 Application to NPR 9998

The basic requirement is that the $IR$ is less than $10^{-5}$.

It is assumed that the total risk is built up by risks following from global failure and risk following from local failures (falling objects).

Global failure is split up into three categories (see Figure 3.1 of this report): failure with 20% volume loss, 50% volume loss and 100% volume loss. For each category there is a corresponding probability of occurrence (given failure) and a corresponding probability of being killed.

A check of the structural reliability using semi probabilistic calculations is only done by the engineer for the failure with 20% volume loss; in the strictness of the applied criterium some space is reserved for the non-checked failure mechanisms.

As far as the local failures are concerned we assume the presence of 2 large walls and a large chimney. According to Chapter 4 in Part B of this report, we have a probability of being killed of 2% for a collapsing wall and 1% for a collapsing chimney.

This way we can elaborate the basic requirement as follows:

$$IR = \sum P(FG_i) P(d|FG_i) + \sum P(FL_j) P(d|FL_j) < 10^{-5}$$
Where:

\( F = \text{failure} \)
\( G = \text{global} \)
\( L = \text{local} \)

\( i = \text{volume loss class:} \)

\( i = 1: V = 20\% \quad P(V = 0.2|FG) = 0.90 \quad P(d|V = 0.2) = 0.10 \)
\( i = 2: V = 50\% \quad P(V = 0.5|FG) = 0.09 \quad P(d|V = 0.5) = 0.30 \)
\( i = 3: V = 100\% \quad P(V = 1.0|FG) = 0.01 \quad P(d|V = 1.0) = 0.50 \)

\( j = \text{falling object} \)

\( j = 2\text{ walls} \quad P(d|V) = 0.02 \)
\( j = 1\text{ chimney} \quad P(d|V) = 0.01 \)

In principle the budget for the various failure scenarios is as in presented in Table 3.2 of this report, upper part:

- Starting from a return period for the seismic load intensity of 2500 year, we may calculate the corresponding \( \alpha\beta \)-value from the annual exceedance probability and the table for the normal distribution:
  \[
  \Phi(-\alpha\beta) = \frac{1}{T} \quad (T \text{ in annum})
  \]

- Based on the calculated values in Table 4.1 of this report, we use the average influence factor \( \alpha=0.88 \) for the seismic loading we find the corresponding target \( \beta \) and annual failure probability:
  \[
  P(F) = \Phi(-\beta)
  \]

- Given the assumed probabilities for a certain volume loss \( P(V) \) and finding death \( P(d|V) \) we calculate the contribution the three global risk scenario's to the individual risk:
  \[
  IR = P(F) \cdot P(V|F) \cdot P(d|V)
  \]

The upper and lower bound values are the same in this case. In the last two columns we find the sum of the three contributions.

For the three local risk scenarios the upper bound value follows from:

\[
IR = P(F) \cdot P(d|F)
\]

The lower bound has been put on zero. The point is that the collapse of the primary and secondary seismic members may give additional (impact) loads on the non-seismic members that they will collapse anyway. They may already be part of the volume losses counted for the global collapse. In general reality will be between the lower and upper bound given.
Given the result in Table 3.2 of this report that is between 0.8 and 1.2 times $10^{-5}$. Considering all inaccuracies in the calculations the is fully acceptable. Roughly speaking the global collapse takes 80% and the local collapse 20 % of the IR.

In the lower part of Table 3.2 of this report an alternative failure probability budget distribution is given. Here we start with a much higher demand for the global failure: the return period is 3800 year, corresponding to a 25% higher load (or a unity check for the normal load of maximum 0.8). If that is the case we may lower the requirement for the structural non seismic members to 1000 year.

Table 3.2: Failure probability budgets

| T     | ab | a   | b   | P(F) | V  | P(V) | P(d|V) | IRupper | IRlower |
|-------|----|-----|-----|------|----|------|-------|---------|---------|
| global | 2500 | 3.35 | 0.88 | 3.81 | 6.9E-05 | 10%  | 0.1 | 6.3E-06 | 6.3E-06 |
|       | 2500 | 3.35 | 0.88 | 3.81 | 6.9E-05 | 50%  | 0.09 | 1.9E-06 | 1.9E-06 |
|       | 2500 | 3.35 | 0.88 | 3.81 | 6.9E-05 | 100% | 0.01 | 3.5E-07 | 3.5E-07 |
| chimney | 2500 | 3.35 | 0.88 | 3.81 | 6.9E-05 | 1   | 0.01 | 6.9E-07 | 0       |
| wall   | 2500 | 3.35 | 0.88 | 3.81 | 6.9E-05 | 1   | 0.02 | 1.4E-06 | 0       |
| wall   | 2500 | 3.35 | 0.88 | 3.81 | 6.9E-05 | 1   | 0.02 | 1.4E-06 | 0       |

| T     | ab | a   | b   | P(F) | V  | P(V) | P(d|V) | IRupper | IRlower |
|-------|----|-----|-----|------|----|------|-------|---------|---------|
| global | 3800 | 3.47 | 0.88 | 3.94 | 4.1E-05 | 10%  | 0.1 | 3.7E-06 | 3.7E-06 |
|       | 3800 | 3.47 | 0.88 | 3.94 | 4.1E-05 | 50%  | 0.09 | 1.1E-06 | 1.1E-06 |
|       | 3800 | 3.47 | 0.88 | 3.94 | 4.1E-05 | 100% | 0.01 | 2E-07   | 5E-06   |
| chimney | 1000 | 3.09 | 0.88 | 3.51 | 0.00022 | 1   | 0.01 | 2.2E-06 | 0       |
| wall   | 1000 | 3.09 | 0.88 | 3.51 | 0.00022 | 1   | 0.02 | 4.5E-06 | 7.8E-07 |
| wall   | 1000 | 3.09 | 0.88 | 3.51 | 0.00022 | 1   | 0.02 | 4.5E-06 | 7.8E-07 |

It should be noted that according to NPR 9998:2018 only the global limit state for the 10 (or 20) % volume loss is actually checked; the numbers for the 50 and 100% volume losses are generic numbers and have been presented only to enable a corresponding defendable total risk estimate. For the local limit states full collapse is always the basis for the reliability check.

3.5 Importance factors in NPR 9998

The consequence class according to the Eurocodes express the effect of Societal (Community) Risk. For CC1B, CC2 and CC3 the importance factors $\gamma_i$ as given in Table 3.3 of this report, are proposed.
Table 3.3 – Overview of importance factors for existing structures

<table>
<thead>
<tr>
<th></th>
<th>( \gamma_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( CC1B )</td>
<td>1.0</td>
</tr>
<tr>
<td>( CC2 )</td>
<td>1.1</td>
</tr>
<tr>
<td>( CC3 )</td>
<td>1.2</td>
</tr>
</tbody>
</table>

For new structures it is usual to have slightly larger reliability levels, because of economic arguments. This leads to the values given in Table 3.4 of this report.

Table 3.4 – Overview of importance factors for new structures

<table>
<thead>
<tr>
<th></th>
<th>( \gamma_i ), existing</th>
<th>( \gamma_i ), new</th>
</tr>
</thead>
<tbody>
<tr>
<td>( CC1B )</td>
<td>1.0</td>
<td>1.1</td>
</tr>
<tr>
<td>( CC2 )</td>
<td>1.1</td>
<td>1.2</td>
</tr>
<tr>
<td>( CC3 )</td>
<td>1.2</td>
<td>1.3</td>
</tr>
</tbody>
</table>

The rational of these rather rough choices is not well expressed and has to be worked out in more detail in future. The values given here are applied in NPR 9998:2018.
4 Return periods of seismic action

4.1 Reliability requirements

In this chapter the reliability requirement is elaborated for global failure using a full probabilistic assessment in the calibration of a semi-probabilistic assessment with design values of the earthquake load and the seismic resistance of the structure.

Reasoning from an Individual Risk perspective, in Section 3.4 of this report the check for local structural collapse (primary and secondary seismic elements) has been stated, which would not be allowed to have an annual probability of partial collapse greater than $0.8 \cdot 10^{-5}$.

It is noted that the return periods of seismic actions in case of local structural partial collapse (non-seismic structural elements) are derived in Part B of this report.

4.2 Probabilistic seismic hazard analysis and fragility functions

The probabilistic seismic hazard assessment as applied in NEN-EN 1998-1 (2005) and NPR 9998:2018 is related to the prediction of the strong ground motion likely to occur at a particular site and the subsequent response by the structure. The most widely-used characterisation of the strong ground motion is the maximum amplitude on the acceleration time series, the peak ground acceleration (PGA). The probabilistic seismic hazard analysis (PSHA) is based on the following steps (Cornell method) as illustrated in Figure 4.1 of this report:

1) Sources: Identification of the independent sources of seismic activity and determination of the magnitude model from contribution of each source.
2) Recurrence: Attenuation relationship on peak ground motion parameter, classified according to the soil category.
3) Ground motion: Calculation of the probability distribution of the peak ground motion parameter at the site.
4) Probability of Exceedance: The calculation of the structural response to earthquakes with given peak ground acceleration.

The seismic statistics can be presented as a Peak Ground Acceleration (PGA) Return Period relation for each relevant location in the Groningen area. The required models are:

- a set of seismic active zones
- the statistics for the magnitude $M$ for each zone
- attenuation models.

In the elaboration care has to be taken of the statistical uncertainties in the distribution for $M$ as well as the model uncertainties in the attenuation law. The basic equation for the evaluation of the seismic load can be written as:

$$P(a_g > a_0) = \sum_{i=1}^{N} \lambda_i \left\{ \int_m \int_r P[a_g > a_0 | m, r]f(m)f(r)dmdr \right\}$$  \hspace{1cm} (4.1)
where:

\[ P(\cdot) \] is the annual probability that the PGA value \( a \) will exceed \( a_0 \) on a certain location.

\[ f(m) \] is the probability density function for the magnitude \( M \) of an arbitrary earthquake with parameters \( M_{\text{min}}, M_{\text{max}}, a, \) and \( b \) in zone \( i \).

\[ f(r) \] is the probability distribution function for the distance \( R \) from the epicenter in zone \( i \) to the building site.

\[ \lambda_i \] is the annual number of seismic events with \( M > M_{\text{min}} \) in zone \( i \).

\[ \lambda \sum \lambda_i \] is the total number of seismic events in all \( N \) zones in one year.

\( N \) is the number of zones.

**Figure 4.1: Steps in a PSHA**

Using collections of ground-motion recordings, empirical equations have been developed, relating PGA to variables like the magnitude and the distance between the earthquake and the site of recording. These relationships are generally called ground-motion prediction equations, or GMPEs. Based on this method, distribution functions of the PGA (annual exceedance probabilities) are obtained per location. Seismic hazard maps are derived with PGA contours for 0.2% annual probability of exceedance.

Earthquake ground motions are provided in terms of a Uniform Hazard Spectrum (UHS). The UHS provides the response spectrum requirements for structures as a function of vibrational period, where the response spectrum is the maximum response of a single-degree-of-freedom oscillator. UHS spectra provide the spectral accelerations for a range of periods but for a uniform level of hazard. The shape of the response spectrum may depend largely on the local ground conditions. Probabilistic site response calculations should be carried out to character the spectra.

A fragility function represents the cumulative distribution function of the capacity of a structure to resist an undesirable limit state. Capacity is measured in terms of the degree of environment excitation at which the asset exceeds the undesirable limit state. For example, a fragility function could express the uncertain level of shaking that a building can tolerate before it collapses. The chance that it collapses at a given level of shaking is the same as the probability that its strength is less than that level of shaking.
The fragility of a structure (or component) is determined with respect to "capacity". Capacity is defined as the limit seismic load before failure occurs. Therefore, if PGA has been chosen to characterize seismic ground motion level, then capacity is also expressed in terms of PGA. In what follows, and in order to simplify the notations, it is considered that PGA has been chosen to characterize seismic ground motion. The capacity of the structure, is generally supposed to be log-normally distributed, see e.g. Pitilakis et al (2014).

4.3 Calibration of a semi-probabilistic assessment

The Individual Risk requirement is formulated as:

\[ IR = P(d|F)P(F) < 10^{-5} \]  

Here, the annual probability of collapse of the structure under earthquake load can be calculated according to:

\[ P(F) = \int F_R(x) f_{sa}(x) \, dx \]  

where:

- \( f_{sa}(x) \) is the probability density function (of random variable \( x \)) of the annual maximum hazard expressed in spectral acceleration at the location of the structure.
- \( F_R(x) \) is the fragility function (of random variable \( x \)) of the structure under consideration for NC with the spectral acceleration \( S_a \) on the horizontal axis.

To calibrate to a \( IR = 0.8 \cdot 10^{-5} \), fragility functions are needed that account for uncertainties in record-to-record variability, within-building uncertainties (e.g. material properties, connection details), and model uncertainty (e.g. whether degradation is explicitly modeled, if models are calibrated to experimental tests etc.).

4.4 Procedure to obtain the return period of the design seismic action

In order to determine the return period for the design seismic action (expressed in the spectral acceleration, UHS spectrum) at a particular site that will achieve the target risk, the following procedure is used:

- A trial return period for design seismic action is selected.
- The spectral response acceleration at this return period for the site is determined from the site's hazard curve (from NAM or KNMI PSHA).
- A lognormal fragility function having a 5 percent probability of collapse at this spectral response acceleration and a dispersion of 0.6 is constructed (the curve can be defined by its median (\( \Theta \)) that gives the 50th fractile and a dispersion (\( CoV \)) which is the standard deviation of the underlying normal distribution.). The use of \( CoV = 0.6 \) comes from the US practice in FEMA (Luco et al., 2007). Many fragility functions in literature for partial collapse have comparable dispersion coefficients. This dispersion coefficient accounts for all possible uncertainties in the seismic capacity: record to record variability, material uncertainties, geometrical uncertainties, eccentricities, inclinations, uncertainties in boundary conditions, not
modelled effects in e.g. boundaries and all other model uncertainties. The record to record variability is the dominant uncertainty.

- The hazard and fragility curves are integrated to produce an annual collapse probability.

This process is repeated until the return period results in a target annual collapse probability. As derived in chapter 3 this target is equal to $10^{-4}$ for global partial collapse.

The calibration of the semi-probabilistic assessment hinges on an important assumption: the capacity of the structure is such that, loaded by the design ground motions defined in the code, there is a 5% probability that in reality the structure will collapse. In the US the assessment is based on a $2 \times 10^{-4}$ annual probability of collapse assuming 10% probability of collapse under the MCE ground motions (Luco et al., 2007). In Europe the acceptable annual probability of collapse is found to be around $1 \times 10^{-5}$ but there are various opinions on what the probability of collapse under the design ground motions should be (Silva et al., 2011). In order to check this, Martins et al. (2015) designed buildings to Eurocode 8, produced fragility functions and then calculated the probability of collapse under the design ground motions and found probabilities of a few percent for buildings designed to low levels of PGA (Martins et al. 2015). Hence, the 5% seems reasonable, though could be checked in the future in a similar manner.

The use of $CoV = 0.6$ comes from the US practice in FEMA (Luco et al., 2007). Many fragility functions in literature for partial collapse have comparable dispersion coefficients.

4.5 Calculation of return period design seismic action

A PSHA using the Groningen specific GMPE v2 (Bommer et al, 2015) was carried out; hazard curves for Loppersum and Delfzijl have been derived for two periods: $T = 0.5$ and $T = 1$ s. These periods are considered to be the dominant periods for low rise structures. Two $M_{\text{max}}$ models were used: $M_{\text{max}} = 5.0$ (KNMI according to Dost and Spetzler, 2015) and $M_{\text{max}}$ is uniformly distributed between 5 and 6.5 (NAM Winningsplan 2016). See Figure 4.2 of this report for the hazard curves.
Figure 4.2: Hazard curves for Loppersum and Delfzijl based on two $M_{\text{max}}$ assumptions (right graph $M_{\text{max}} = 5$ and left graph $M_{\text{max}}$ uniform between 5.0 and 6.5)

The procedure given in Section 4.4 of this report was applied and the resulting return periods for the design seismic action are given in Table 4.1 and Table 4.2 of this report. For informational purposes also the FORM influence factors for both seismic load and resistance are given.

In the case we use $P(d|F)=0.1$ (for the global partial collapse check) and the return periods $T_R$ of the design seismic action and matching weighting factors $\alpha_R$ and $\alpha_S$ (for respectively the resistance and the loading) are shown in Table 4.1 of this report. It is observed that the return period is rather insensitive to the choice of $M_{\text{max}}$ and to the exact location in the Groningen area.

A general value of $T = 2475$ year is chosen. This value is confirmed by international practice since it is exactly the recommended return period for Near Collapse in Eurocode NEN-EN 1998-3 (2005) and in FEMA 440 (2005).
Table 4.1: Return periods for global partial collapse check

<table>
<thead>
<tr>
<th>location</th>
<th>$M_{\text{max}}$</th>
<th>period [s]</th>
<th>Return period [year]</th>
<th>$\alpha_R$</th>
<th>$\alpha_S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>'Delfzijl' 'KNMI'</td>
<td>0.5</td>
<td>2535</td>
<td>0.49</td>
<td>-0.87</td>
<td></td>
</tr>
<tr>
<td>'Delfzijl' 'KNMI'</td>
<td>1</td>
<td>2367</td>
<td>0.42</td>
<td>-0.91</td>
<td></td>
</tr>
<tr>
<td>'Delfzijl' 'NAM'</td>
<td>0.5</td>
<td>2592</td>
<td>0.5</td>
<td>-0.87</td>
<td></td>
</tr>
<tr>
<td>'Delfzijl' 'NAM'</td>
<td>1</td>
<td>2521</td>
<td>0.39</td>
<td>-0.92</td>
<td></td>
</tr>
<tr>
<td>'Loppersum' 'KNMI'</td>
<td>0.5</td>
<td>2679</td>
<td>0.52</td>
<td>-0.85</td>
<td></td>
</tr>
<tr>
<td>'Loppersum' 'KNMI'</td>
<td>1</td>
<td>2459</td>
<td>0.47</td>
<td>-0.88</td>
<td></td>
</tr>
<tr>
<td>'Loppersum' 'NAM'</td>
<td>0.5</td>
<td>2744</td>
<td>0.54</td>
<td>-0.84</td>
<td></td>
</tr>
<tr>
<td>'Loppersum' 'NAM'</td>
<td>1</td>
<td>2513</td>
<td>0.48</td>
<td>-0.88</td>
<td></td>
</tr>
</tbody>
</table>

Similar calculations were performed using the Groningen specific GMPE v4 (Bommer et al, 2017). This leads to very similar return periods and therefore this is not studied further.
5 Capacity modelling and drift limits

To model the capacity of the structure, the design value of the resistance has to be determined based on the product of the influence factor \( \alpha_R \) and the reliability index \( \beta \). To determine the resistance on basis of calculation rules, mean values for material and geometric properties are applied. Then, for a structure (with mean properties) loaded by the design seismic load as explained in section 4.4 of this report, the acceptance criteria should be defined in the model to calculate the resistance such that the structure has a 5% probability of global collapse.

The calculations rules with drift limits as proposed in Annex G of NPR 9998:2018, therefore should therefore lead to a design value of the global capacity leading to 5% probability of global collapse. We refer to the background document on drift limits (Messali et al, 2017, 2018a en 2018b).
6 References


Fib (2013) Model Code for Concrete Structures 2010, fib, Lausanne

ISO 2394 (2015), General principles on reliability for structures, ISO


NEN 8700 (2011), NEN 8700:2011, Assessment of existing structures in case of reconstruction and disapproval - Basic Rules, NEN.


7 Signature

Delft, October 2018
Prof. Dr. Ir. R.D.J.M. Steenbergen
Author

TNO
Dr. Ir. I.J.J. van Straalen
Reviewer

Dr. P.C. Rasker
Research manager
Structural Reliability
Annex A: Notes on and Relationships between Risk Metrics

1. Risk metrics can be defined either as probabilities or as frequencies. In the relationships below I have adopted frequency throughout as this simplifies the inter-relationships.

2. The general equation for LPR at a given point associated with a given set of events is
\[ \text{LPR} = \sum \text{fi Pdi} \]  
(A1)
where fi is the frequency of event i, Pdi is the probability of death for the specified person at the specified location, given that event i occurs and \( \sum \) the sum is made over all possible events i.

3. Note that when dealing with falling objects LPR will vary very strongly with exactly where a person is, and with their characteristics. For example in the event of a 1.2 m high brick wall collapsing, the probability of death for a small child standing next to the wall could be quite high, whereas that for a 2 m tall person standing next to the wall could be very small. The probability of death, and hence the LPR, for either will decrease rapidly with distance from the wall.

4. LPR and Individual Risk are related by:
\[ \text{IR} = \sum \text{LPR}_j \times \%T_j \]
(A2)
where LPRj is the LPR at location j (with the sum made over all locations) and \( \%T_j \) is the proportion of a similar individual’s time spent at location j.

5. Community Risk is equal to the sum of individual risk over all members of a specified community:
\[ \text{CR} = \sum \text{IR}_k \]
(A3)
where IRk is the individual risk of community member k.

6. Group Risk is a set of values defined for each value of N, the number of fatalities occurring in an event, up to the maximum possible value of N:
\[ F(\geq N) = \sum \text{fi P(\geq N)i} \]
(A4)
where P(\( \geq N\))i is the probability of N or more fatalities occurring in event i.

7. Community Risk can be defined similarly:
\[ \text{CR} = \sum \text{fi Ni} \]
(A5)
where Ni is the number of people killed in event i.

8. Community Risk and Group Risk are related:
\[ \text{CR} = \sum \text{fN N} \]
(A6)
where fN is the frequency of events killing exactly N people; \( \sum \) the sum is over all possible values of N and note fN = \( F(\geq N) - F(\geq N+1) \).

Community Risk is sometimes described as the “Area beneath” or “Integral of” the \( F(\geq N) \) curve (or similar), but is not in fact either of these quantities.