Reliability based code making for seismic assessment under gas extraction

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In the Netherlands shallow earthquakes are induced due to large scale gas extraction from the Groningen gas field. Question is how safe the existing building stock is and if strengthening is necessary. A reliability based assessment of the buildings is needed both on the loading and on the resistance since many uncertainties play an important role. The target safety level defined by the government is an Individual Risk level of 10^{-5} per year. In this paper the probabilistic calculation of the individual risk is developed taking into account the various building collapse states. A simplified format is derived, which is the basis of the safety philosophy applied in the Dutch Seismic Code NPR 9998:2020.

Key words: Induced earthquakes, reliability assessment, individual risk, seismic hazard, existing structures, probabilistic design

1 Introduction

Until recently earthquakes were not taken into account in the Dutch Building Decree and other Dutch building regulations. Although in 1992, an earthquake happened in Roermond with a magnitude of 5.8 on the Richter scale which damaged buildings, this was not seen by the law makers as an argument for the introduction of regulations for earthquake resistant structures and the seismic assessment of existing structures. On August 16, 2012 an induced earthquake occurred in the north of the Netherlands near the village of Huizinge in the municipality of Loppersum. The moment magnitude of the event was estimated to be M = 3.6, by the KNMI. The strength of that earthquake is the largest event in the region until present, with effects at the surface strongly felt by the population. The induced earthquakes occur at very shallow depth (about 3 km), which makes that accelerations and velocities are much larger than those of tectonic earthquakes with the same magnitude. After this seismic event it became clear that for the Groningen region regulations were necessary for seismic design and assessment. Basis of the assessment is the target individual risk of 10^{-5} per year as defined by the Meijdam Committee (Meijdam, 2015). Due to the nature of the target safety level and uncertainties in the seismic loading and the resistance of the existing often masonry houses, a reliability based approach is necessary. This will enable decision making under the present large uncertainties. In this light, in this paper, the reliability backgrounds of the Dutch guideline NPR 9998, 2020 are discussed.

2 Safety philosophy

In order to check structures for sufficient reliability, information is needed on loads, resistance, failure modes, consequences of failure and safety criteria.

2.1 *General*

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 we may formulate the decision problem as:

$$\begin{aligned} &\text{Min } C_{\text{tot}} = C_S + P(F)C_F \quad \text{in the lifetime of the structure, } P(F) \text{ in the lifetime} \\ &\text{Sub } P(F) < P(F)_{\text{limit}} \quad \text{per year, } P(F) \text{ per year} \end{aligned} \tag{1}$$

Where C, P, S and F respectively refer to costs, probability, structure and failure. Here we neglected the discount rate. If we would include the discount rate γ, the first equation in (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 Group Risk (GR). This limit value should be understood as the expected value of the failure probability. For existing structures human safety criteria are almost always dominant over economic criteria.

2.2 *Target reliability based on economic optimisation*

The target reliability index is defined as a substitute for the failure probability $P(F)$, defined by:

$$\beta = -\Phi^{-1}(P(F)) \tag{2}$$

where Φ^{-1} is the inverse standardized normal distribution. *E.g.* a reliability index equal to 3.8 represents a probability of $7 \cdot 10^{-5}$. 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 1.

Table 1. Tentative target reliabilities related to one year reference period and ultimate limit states, based on monetary optimization (ISO 2394 (2015))

Relative cost of safety measures	Consequences of failure					
	Minor		Moderate		Large	
Large (A)	$\beta = 3.1$	$P(F) \approx 10^{-3}$	$\beta = 3.3$	$P(F) \approx 5{\cdot}10^{-4}$	$\beta = 3.7$	$P(F) \approx 10^{-4}$
Normal (B)	$\beta = 3.7$	$P(F) \approx 10^{-4}$	$\beta = 4.2$	$P(F) \approx 10^{-5}$	$\beta = 4.4$	$P(F) \approx 5{\cdot}10^{-6}$
Small (C)	$\beta = 4.2$	$P(F) \approx 10^{-5}$	$\beta = 4.4$	$P(F) \approx 5{\cdot}10^{-6}$	$\beta = 4.7$	$P(F) \approx 10^{-6}$

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 fib Model Code (2010). A 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 the Eurocodes in most of the cases the lowest row (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 1 (Large).
However this economic optimization is bounded by considerations for human safety; this will be discussed in the next section.

2.3 *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, see Meijdam (2015). In the next chapter this individual risk concept is translated to the seismic assessment of buildings.

3 Calculation of seismic individual risk taking into account various collapse states of buildings

The probability $P(d)$ that a person dies in one year at a certain location due to structural failure under earthquake load, can be calculated as follows:

$$P(d) = P(F) \cdot P(d\,|\,F) \tag{3}$$

Here, $P(d\,|\,F)$ is the conditional probability of casualty given the structural failure and $P(F)$ the probability of failure. This requires an estimation of probability of fatality given a certain type of collapse and a check on the collapse capacity of the structures. In principle the total risk is built up by risks following from, on one hand, the various types of global failure and, on the other hand, the risk following from local failures (falling objects). Although collapse will occur gradually and in a continuous way, for the purpose of engineering guidelines, global failure is often split up into categories. Generally (see *e.g.* Coburn *et al.*, 1992) it is subdivided into three discrete collapse states: CS1, CS2 and CS3, see Figure 1. Local failures can be the failures of local elements such as inner walls and chimneys sometimes collapsing before parts of the global load bearing structure collapse. The sequence of collapse (global vs local) depends on the seismic capacity of each of the building parts.

CS1 20% of volume *CS2 50% of volume* *CS3 100% of volume*

Figure 1: Three categories of volume loss due to global collapse of building, from Coburn et al., 1992. Local collapses of internal walls or chimneys can occur separately before global collapse or can part of the global collapse; chimneys are not visible in the picture but the effect of chimneys on Dutch houses is accounted for in the present paper.

For the purpose of the NPR 9998, global failure is split up into three categories: 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. Important to note that a specific type or typology of buildings can show a different behaviour in terms of collapse states and volume losses, the above values chosen in NPR 9998 can be seen as reasonable nominal choices.

Important is that in engineering guidelines, such as the NPR 9998, in accordance with the normal Eurocode EN 1998 calculation, the engineer will do only one calculation to check the global seismic resistance although in Figure 1 three collapses states are defined. A check of the structural reliability of the global structure using semi probabilistic calculations done by the engineer has been chosen to be executed for the failure mode with 20% volume loss; the reason is that this collapse state connects most closely to the Near Collapse limit state in EN 1998-1 and it is not too demanding for computational calculations in terms of being able to model progressive collapse. The consequence is that in the strictness of the applied criterion some failure probability space should be reserved for the non-checked failure mechanisms (50% volume loss and 100% volume loss). As far as the local failures are concerned for an average house the presence of 2 large walls and a large chimney is assumed. These local failure mechanisms will be checked in a semi-probabilistic assessment by the structural engineer, separately from the global assessment.

According to TNO report R 10254 (2018a), a probability of being killed of 2% for a collapsing wall and 1% for a collapsing chimney is a reasonable choice.

In this way we can elaborate the basic risk requirement as follows:

$$IR = \sum P(F_{G,i})\, P(d \,|\, F_{G,i}) + \sum P(F_{L,j})\, P(d \,|\, F_{L,j}) < 10^{-5} \tag{4}$$

where

F …… Failure

G …… Global

L …… Local

and

i ……. Volume loss class

$$\begin{aligned} &i = 1\text{: } V = 20\% \quad P(V = 0.2 \,|\, F_G) = 0.90 \quad P(d \,|\, V = 0.2) = 0.10 \\ &i = 2\text{: } V = 50\% \quad P(V = 0.5 \,|\, F_G) = 0.09 \quad P(d \,|\, V = 0.5) = 0.30 \\ &i = 3\text{: } V = 100\% \quad P(V = 1.0 \,|\, F_G) = 0.01 \quad P(d \,|\, V = 1.0) = 0.50 \end{aligned} \tag{5}$$

j ……. Falling object

$$\begin{aligned} &j = 1\text{: walls} \quad P(d \,|\, V) = 0.02 \\ &j = 2\text{: chimney} \quad P(d \,|\, V) = 0.01 \end{aligned} \tag{6}$$

The engineer will check the global collapse state with 20% volume loss and will check the locally falling objects. For these checks, a design value of the seismic resistance and a return period of the seismic action has to be derived such that for all the collapse states the individual risk criterion is satisfied, this means that for the collapse states with the large volume losses some risk budget has to be reserved. This will be elaborated further in Section 5. In order to do so, in Chapter 4 we will discuss probabilistic seismic risk assessment.

4 Probabilistic seismic risk analysis

4.1 Probabilistic description of hazard and fragility

The annual probability exceeding a certain collapse state (Fig. 1) of the structure under earthquake load can be calculated according to:

$$P(F_{G,i}) = \int F_R(x)\, f_S(x)\, dx \tag{7}$$

where

$f_S(x)$ ………. Probability density function (of random variable x) of the annual maximum hazard expressed in the chosen intensity level at the location of the structure.

$F_R(x)$ .... Fragility function (of random variable x) of the structure under consideration for the specific collapse state CS_i with the intensity measure on the horizontal axis.

The probabilistic seismic hazard assessment as applied in Eurocode 8 and NPR 9998 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 probabilistic seismic hazard analysis (PSHA) is based on the following steps (Cornell, 1968):

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

The seismic statistics can be presented as an Intensity Measure-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:

$$P(a_g > a_0) = \sum_{i=1}^{N} \lambda_i \left\{ \int_M \int_R P(a_g > a_0 \mid m, r)\, f(m)\, f(r)\, dm\, dr \right\}_i \tag{8}$$

where

$P(a_g > a_0)$ ... Annual probability that the acceleration a_g will exceed a_0 at a certain location.

$f(m)$ .... Probability density function for the magnitude M of an arbitrary earthquake with parameters M_{min} , M_{max} , a, and b in zone i

$f(r)$ .... Pdf for the distance R from the epicenter in zone i to the building site.

λ_i .... Annual number of seismic events with $M > M_{min} = 1.5$ in zone i

λ .... $\sum \lambda_i$ is the total number of seismic events in all N zones in one year.

N .... Number of zones

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.

Earthquake ground motions are provided in terms of a Uniform Hazard Spectrum (UHS) for different return periods. 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.

A fragility function represents the cumulative distribution function of the capacity of a structure to resist an undesirable limit state, for each of the collapse states CS1, CS2 and CS3 (Fig. 1). 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 peak ground acceleration (PGA) has been chosen to characterize seismic ground motion level, then capacity is also expressed in terms of PGA. Often a different intensity measure such as a spectral acceleration or a combination of spectral accelerations better describe the fragility. The capacity of the structure, is generally supposed to be log-normally distributed, see *e.g.* Pitilakis *et al.* (2014). 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.).

*4.2 **Typical example for full probabilistic analysis***

In this section, a typical case for Groningen will be discussed, for which a probabilistic seismic risk assessment is performed. It will give the necessary insight for the calibration of the semi-probabilistic format in section 5.

In the typical example a global collapse state of the building is studied with $P(d\,|\,F) = 0.10$, which considered to a typical average value from literature, see *e.g.* Jaiswal (2009) and Spence (2011). This leads to: $P(F) = \int F_R(x)\, f_S(x)\, dx = 10^{-4}$. Calculation of the risk can be performed by considering the seismic hazard $f_S(x)$ and the fragility $F_R(x)$ of the structure.

For the seismic hazard, the *PSHA* carried out by KNMI (Dost and Spetzler, 2015) was used for the location with the largest hazard in the Groningen region. This version served as the background for the reliability calibration of the NPR 9998, 2020. As intensity measures the spectra accelerations at $T = 0.5$ s and $T = 1$ s were chosen. Currently, newer versions of the PSHA are available and the effect of using these should be analysed.

The typical fragility curve is assumed to be log-normal. The curve can be defined by its median (θ) that gives the 50th percentile and a dispersion (λ) which is the standard deviation of the underlying normal distribution. For the dispersion a value of 0.6 is used. The use of $\lambda = 0.6$ comes from the US practice in FEMA (Luco *et al.*, 2007) and was adopted as typical for NPR 9998. Many fragility functions in literature for (partial) collapse have more or less 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. It should be mentioned that the choice of $\lambda = 0.6$ was made based on FEMA practice. However for a specific building typology in the seismic area in the Netherlands the dispersion could be quite different, this has been neglected in the NPR 9998 since it was chosen to work with one value of the return period for the seismic action indifferently of the specific building class.

If a dispersion of $\lambda = 0.6$ is assumed, the median θ can be determined by requiring a failure probability of $(F) = 10^{-4}$ per year using equation (7). This can be achieved by iteratively repeating the calculation with different estimates of θ until the desired value of (F) is reached. The result of that calculation is shown in Figure 2.

In Figure 2a) the hazard curve is shown. In Figure 2b) the fragility function is shown such that an annual failure probability of $P(F) = 10^{-4}$ is obtained. This corresponds to $IR = 10^{-5}$ and $P(d\,|\,F) = 0.10$ as mentioned above. Around the mean of the integrand in equation (7) the risk contribution is the largest, this is shown in Figure 2c). The mean of the integrand corresponds to the 5% fractile in the fragility function and a return period of approximately T = 2475 year in the hazard function. So the 5% fractile of the fragility and the T = 2475 year intensity measure have the largest contribution to the risk and are therefore the most important fractiles in the hazard and fragility curves, they are called design point values. The differences in the Figure 2a)-c) are small for the spectral accelerations of 0.5 s and 1 s, so this has not further been investigated.

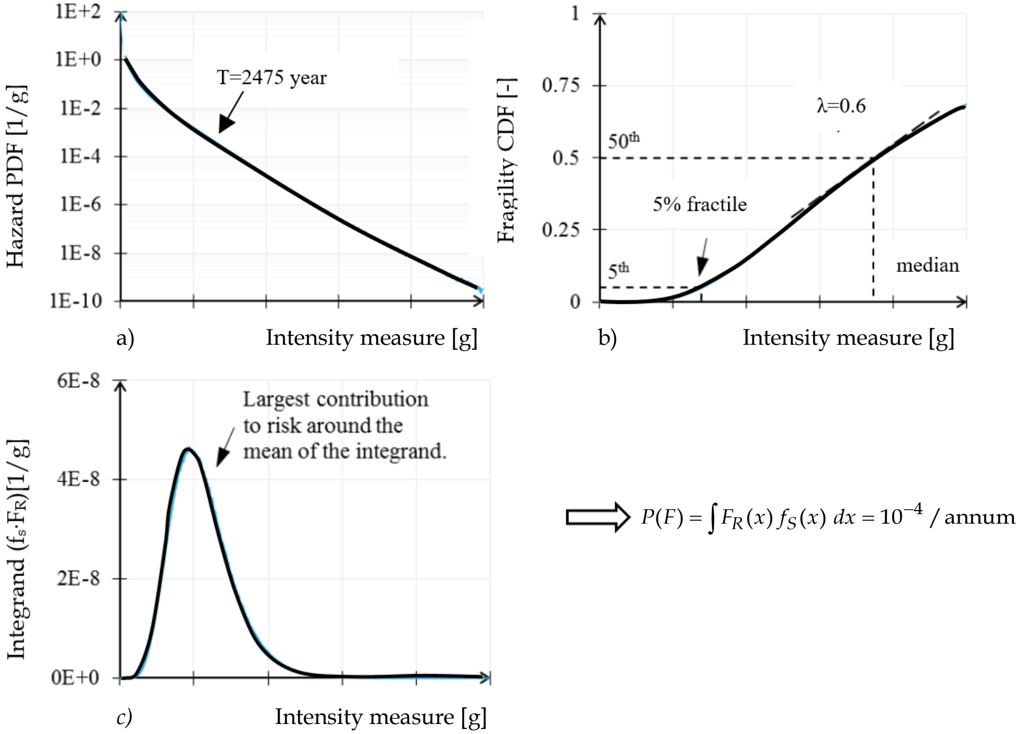

Figure 2. Typical example of a full probabilistic seismic assessment

For the probabilistic calculations shown in Figure 2 a typical probabilistic influence factor for the resistance of α_R = 0.48 was found and a typical probabilistic influence factor for the seismic load of α_S = -0.88 (see also TNO report TNO 2018 R 10254 (2018a)). The value of α_S is larger than assumed for non-seismic design in Eurocode EN 1990, however here the uncertainty in the hazard is very large and it dominates the failure probability. These α-values lead to the 5% fractile as the design value for the seismic resistance and a return period for the design value of the seismic action of T = 2475 year.

5 Calibration of semi-probabilistic format

In this chapter the design values of the seismic resistance and the seismic load are calibrated for the semi-probabilistic format proposed in NPR 9998:2020.

*5.1 **Design value of the seismic capacity***

For the calibration of the semi-probabilistic assessment the following design value of the seismic capacity is used: the capacity of the structure is such that, loaded by the design ground motions defined in the code, there is a 5% probability that the structure will collapse. So the design value of the seismic resistance is taken as the 5% fractile in the distribution function of the seismic resistance. As shown in section 4.2 this fractile results from the typical probabilistic calculation. Figure 2 confirms the choice for the 5% value: it makes sense to test the structure in an analysis against the seismic capacity corresponding to the 5% fractile; since here the risk contribution is the largest. It is also a value motivated by several studies in literature. In the US the seismic assessment is based on a $2 \cdot 10^{-4}$ 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 $1 \cdot 10^{-5}$ 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.

*5.2 **Return periods for design value of the seismic action***

In the international practice a return period of T = 2475 year is often chosen for the collapse limit state. It is the recommended return period for Near Collapse in Eurocode NEN-EN 1998-3 (2005) and in FEMA 440 (2005). It also follows from a typical full probabilistic calculation as shown in Section 4.2.
In this section we check if this standard value satisfies the requirement in expressions 4-6 for the risk contribution of the various failure scenarios. This is done as follows:

- Starting from a return period for the seismic load intensity of 2475 year, we calculate the corresponding αβ-value from the annual exceedance probability and the normal distribution: $\Phi(-\alpha\beta) = 1/T$ (T in annum)

- Based on the calculated values in section 4.2, we use the typical influence factor α = 0.88 for the seismic loading. This means that we use as point of departure that the design value of the seismic resistance is the 5% fractile. In a risk calculation we then find the corresponding target β and annual failure probability: $P(F) = \Phi(-\beta)$

- Given the assumed probabilities for a certain volume loss P(V) and finding death $P(d \mid V)$ in section 3 we calculate the contribution the three global risk scenario's to the individual risk: $IR = P(F)\, P(V \mid F)\, P(d \mid V)$

This procedure is shown in Table 2 for a return period T = 2475 year to be used for the check of both the global and local collapse states.

Table 2. Calibration of the return period for the design seismic action using T = 2475 year

Collapse state	T [a]	$\alpha\beta$	α	β	$P(F)$	V	$P(V)$	$P(d \mid V)$	IR (upper)	IR (lower)
global CS1	2475	3.35	0.88	3.81	$6.9\ 10^{-5}$	20%	0.9	0.1	$6.3\ 10^{-6}$	$6.3\ 10^{-6}$
global CS2	2475	3.35	0.88	3.81	$6.9\ 10^{-5}$	50%	0.09	0.3	$1.9\ 10^{-6}$	$1.9\ 10^{-6}$
global CS3	2475	3.35	0.88	3.81	$6.9\ 10^{-5}$	100%	0.01	0.5	$3.5\ 10^{-7}$	$3.5\ 10^{-7}$
chimney	2475	3.35	0.88	3.81	$6.9\ 10^{-5}$		1	0.01	$6.9\ 10^{-7}$	0
wall	2475	3.35	0.88	3.81	$6.9\ 10^{-5}$		1	0.02	$1.4\ 10^{-6}$	0
wall	2475	3.35	0.88	3.81	$6.9\ 10^{-5}$		1	0.02	$1.4\ 10^{-6}$	0
Total risk									$1.2\ 10^{-5}$	$8.5\ 10^{-6}$

In Table 2 it is shown that using T = 2475 year for all collapse states, the risk contribution of CS1-3 is $IR = 8.5 \cdot 10^{-6}$. The risk contribution of the falling objects (2 inner walls and 1 chimney) is $IR = 3.5 \cdot 10^{-6}$. Whether or not the falling objects are already in CS1, which will depend from building typology to another, we have to account for this $IR = 3.5 \cdot 10^{-6}$. They may already be part of the volume losses counted for the global collapse but this is not known on beforehand in the semi-probabilistic framework. Therefore we work with an upper and lower bound risk; in general, reality will be between the lower and upper bound and can only be assessed in full probabilistic framework.

So, the lower value of the IR is $IR = 8.5 \cdot 10^{-6}$ and the upper value is $IR = 1.2 \cdot 10^{-5}$.

Considering all inaccuracies in the calculations this is acceptable and the value T = 2475 year for the return period of the design seismic action satisfies by good approximation the target risk requirement in equation (4). Roughly speaking the global collapse takes 70% and the local collapse 30% of the *IR* and there is enough failure probability budget available for the non-checked collapse states with the larger volume loss.

Using the same return periods for global and local collapse could not be the optimal failure probability budget distribution. If the structure has a large capacity over demand ratio for global collapse and has some weaker local elements it makes sense to use larger return periods for the global collapse limit states and smaller return periods for the local collapse states. This could be the optimal solution in the case of low seismicity. An example is given for this alternative failure probability budget distribution in Table 3. We start with a higher demand for the global failure: the return period is 3800 year. If that is the case we may lower the requirement for the structural non seismic members to 1000 year which leads to a lower value of the seismic load. However other optimal solutions might be possible depending on the building typology and the seismic load. Note that in Table 3 for the lower *IR* we take for the local walls the risk contribution of that wall minus the CS1 risk contribution since due to the different return periods the risk contribution of the local wall is larger than the risk contribution of the global CS1.

Table 3. Calibration of the return period for the design seismic action using different return periods for global and local collapse

Collapse state	T [a]	$\alpha\beta$	α	β	$P(F)$	V	$P(V)$	$P(d \mid V)$	IR (upper)	IR (lower)
global CS1	3800	3.47	0.88	3.94	$4.1\ 10^{-5}$	20%	0.9	0.1	$3.7\ 10^{-6}$	$3.7\ 10^{-6}$
global CS2	3800	3.47	0.88	3.94	$4.1\ 10^{-5}$	50%	0.09	0.3	$1.1\ 10^{-6}$	$1.1\ 10^{-6}$
global CS3	3800	3.47	0.88	3.94	$4.1\ 10^{-5}$	100%	0.01	0.5	$2.0\ 10^{-7}$	$2.0\ 10^{-7}$
chimney	1000	3.09	0.88	3.51	$2.2\ 10^{-4}$		1	0.01	$2.2\ 10^{-6}$	0
wall	1000	3.09	0.88	3.51	$2.2\ 10^{-4}$		1	0.02	$4.5\ 10^{-6}$	$7.8\ 10^{-7}$
wall	1000	3.09	0.88	3.51	$2.2\ 10^{-4}$		1	0.02	$4.5\ 10^{-6}$	$7.8\ 10^{-7}$
Total risk									$1.6\ 10^{-5}$	$6.5\ 10^{-6}$

5.3 Some considerations with respect to the semi-probabilistic framework

In the sections 5.1 and 5.2 design values for the seismic resistance and load were derived for the purpose of an engineering guideline such as the NPR 9998 where engineers will check only one global collapse state and a few falling objects. However some assumptions and simplifications were made in order to make this possible. These are summarized and discussed below.

- For engineering purposes only one global collapse state was chosen to assess the design seismic capacity with the design seismic load: the collapse state with 20% volume loss. For the risk contribution of the other collapse states with larger volume loss some failure probability budget was reserved, but it is not proved that the chosen values are correct for all building typologies in Groningen. A full probabilistic calculation using fragility curves for all collapse states and summing up the risk over all collapse states would overcome this problem.
- The choices for the volume loss for the collapse states and probability of dying per collapse state are reasonable estimates based on literature and are seen as average values over several building typologies since in the NPR the choice has been to assess all buildings with the same return period of the seismic action. A differentiation per building typology would give a more accurate estimate of the individual risk.
- The calibration of the design values (α values) was done based on probabilistic calculations using a 2015 version of the KNMI hazard. More recent and improved models are available and the influence of these models need to be studied.
- The calibration of the design values (and α values) was done based on probabilistic calculations using typical fragility curve with a dispersion coefficient of 0.6 (see section 4.2). This value was based on international literature, however for the typical Groningen building typologies this coefficient will vary from typology to typology. This means that the design values are more or less averaged over the various typologies. More accurate risk estimates can be obtained using building typologies with their specific fragility curves.
- Using the same return period T = 2475 year for the check of both global and local collapse states does not necessary give the optimal failure probability budget distribution. This could be overcome by using a full probabilistic assessment per building typology.

6 Conclusions

In this paper the background is shown of the safety philosophy in NPR 9998; 2020. The method chosen is reliability based taking an individual risk level of $IR = 10^{-5}$ as the basis. A full probabilistic procedure was derived taking into account the risk contribution of the various global and local collapse mechanisms of buildings. A semi probabilistic framework was proposed based on the typical properties of seismic load and resistance. For the collapse states that are addressed by the engineer, a return period of $T = 2475$ year for the design value of the seismic load and a design value of the seismic resistance corresponding to the 5% fractile in the fragility functions are shown to lead to a sufficient risk level. This safety framework has been implemented in the NPR 9998, 2020. However in the derivation of the semi-probabilistic framework, a number of simplifications and assumptions were adopted, these are discussed in section 5.3. Instead of using this semi-probabilistic procedure for the assessment of the building safety, using a full probabilistic assessment per building typology will provide a more refined risk estimation.

References

Cornell C.A., Engineering seismic risk analysis, Bulletin of the *Seismological Society of America* 1968; 58(5): 1583–1606.

Dost B. and Spetzler J.; Probabilistic Seismic Hazard Analysis for Induced Earthquakes in Groningen; Update 2015., KNMI, October 2015.

Eindadvies Handelingsperspectief voor Groningen, Commissie Meijdam, December 2015

EN 1990:2002. Basis of structural design; European Committee for Standardization.

fib Model Code for Concrete Structures 2010, fib, Lausanne, 2013.

HAZUS multi-hazard loss estimation methodology earthquake model, FEMA 366, Washington, 2003.

ISO 2394, General principles on reliability for structures, International organization for standardization, 1998.

Jaiswal, K.S., Wald, D.J., Earle, P.S., Porter, K.A., Hearne, M.; Earthquake casualty models within the USGS prompt assessment of global earthquakes for response (Pager) system., Second International Workshop on Disaster Casualties University of Cambridge, UK, 2009.

JCSS Probabilistic Model Code, Joint Committee on Structural Safety, Zurich, 2001.

Luco N, Ellingwood BR, Hamburger RO, Hooper JD, Kimball JK, Kircher CA (2007) "Risk-targeted versus current seismic design maps for the conterminous United States". In: *SEAOC* 2007 convention proceedings

Martins, L., Silva, V., Crowley, H., Bazzurro, P., Marques, M. (2015), Investigation of Structural Fragility for Risk-targeted Hazard Assessment, 12th International Conference on *Applications of Statistics and Probability in Civil Engineering*, ICASP12, Vancouver, Canada

Melchers, R., *Structural Reliability Analysis and Prediction*, 2nd edn., Wiley, Chichester, 2001.

NEN 8700 (2009) Dutch Code for the assessment of existing structures.

Pitilakis, K., Crowley, H., Kaynia, A.M., SYNER-G: *Typology Definition and Fragility Functions for Physical Elements at Seismic Risk*, Springer, 2014.

Silva V., Crowley H., Varum H., Pinho R. and Sousa R. (2011) Evaluation of analytical methodologies to derive vulnerability functions, *Earthquake Engineering and Structural Dynamics*, 43(2), pp. 181-204.

Spence, R., So, E., Scawthorn, C., *Human Casualties in Earthquakes, Progress in Modelling and Mitigation*, Spinger, 2011.

Steenbergen, R.D.J.M., Sýkora, M., Holický, M., Diamantidis, D., Economic and human safety reliability levels for existing structures, *Structural Concrete*, September 2015.

TNO report 2018 R 10254, Background Report NPR 9998:2018 Part A: Terminology and Safety Philosophy; https://www.nen.nl/media/PDFjes/ Background_document_ NPR_9998-018 _TNO_Terminology_and_Safety_Philosophy_2018-10.pdf.