Reliability based assessment of buildings under earthquakes due to gas extraction

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In the northern part of the Netherlands over de last decades shallow earthquakes are induced due to large scale gas extraction from the Groningen gas field. Earthquakes occur due to the compaction of the reservoir rock, which leads to subsidence at surface and strain build-up in the reservoir rock and existing faults. The induced earthquakes differ from the better known tectonic earthquakes all over the world, caused by movement of the earth at large depths. The structures affected by the earthquakes however have never been engineered to withstand earthquakes. Layout of the walls and standard constructional details can lead to dangerous partial collapse like fall of walls, opening of cracks and even to total collapse. In the Dutch Building Decree safety criteria for human life have been established and quantified in target level for the Individual Risk. Based on this requirement, in this paper, a reliability based method is developed for the design of new structures and assessment of existing structures for the area affected by the Groningen gas field. This method is adopted in the actual Dutch Guideline NPR 9998 which will be after some years be replaced by the Dutch National Annex to the Eurocode Earthquakes.

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 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 as a cause for the introduction of regulations for earthquake resistant structures and the seismic assessment of existing structures. An important argument of the government for this was that in most areas of the Netherlands structures are calculated with a high wind load, which, was often assumed, in the vast majority of cases would dominate over moderate earthquakes. However this was never proven by calculations or experiments and we also remark that earthquake loads differ essentially from wind loads since the mass of the structure is involved. Starting from 2013, the impact of earthquakes on the built environment was reassessed. This awareness process originated in the earthquakes that happen currently in the North-Eastern part of the Netherlands due to gas extraction. 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. More than 2000 damage reports have been received by the company responsible for the gas production (NAM). The induced earthquakes occur at very shallow depth (about 3 km) with makes that accelerations and velocities are much larger than those of tectonic earthquakes with the same magnitude. Several cases of earthquakes induced by gas production have been recorded in literature. A possible analogue for larger earthquakes in the Groningen area is the 2004 earthquake of magnitude M = 4.4 in the Rotenburg gas field (Germany), which also produces from the Rotliegend. The production from the field causes the reservoir pressure to decline. This results in compaction of the reservoir rock, which leads to subsidence at surface and strain build-up in the reservoir rock.

When in 2012 it became clear that for the Groningen region regulations were necessary for seismic design and assessment of the existing stock, the first idea was to set up the EN 1998 (Eurocode 8) National Annex. This code provides the general European rules, with the possibility for fine-tuning the content in the National Annex by national determined parameters for the Dutch situation. It was estimated that a well-founded set of the national parameters needs a period of three years. The acute situation in Groningen however asked to act more speed. The Ministry of Economic Affairs indicated that a regulation should be available within one year.

To address this, it was decided to develop an NPR (Dutch Practice Guideline),under the supervision of a committee of experts set up by NEN working on NPR 9998 "Design and evaluation of earthquake-resistant buildings in construction, renovation and disapproval - induced earthquakes (NEN, 2015). This NPR 9998 offers clients, designers and contractors a (technical) guidance in calculations methods for new construction and rehabilitation of buildings.

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.
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:

$$\begin{array}{ll} \text{Min } C_{\text{tot}} = C_s + P_F C_F & \text{in the lifetime of the structure} \\ \text{Sub } P_F < P_{F,\text{limit}} & \text{per year} \end{array} \qquad (1)$$

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, 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.

The above system has been elaborated for new structures in Eurocode EN 1990 and the corresponding Dutch National Annex for new structures and in NEN 8700 for existing ones. Only rough economic criteria (partly based on calibration to old codes) and Individual Risk criteria have been taken into account. Group or Societal 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 connects to the present Dutch safety philosophy for existing which is anchored in the 2012 Building Decree through NEN 8700.

In the fundamental requirements for structures under earthquakes we can distinguish three main limit states:

- The structure is at the edge of collapsing (Near Collapse, NC);
- Significant damage (Serious Damage, SD);
- Damage (Damage Limitation, DL).

In the NPR the Minister of Economic affairs asked primarily for a further elaboration of the NC level because it has a direct relationship with preventing victims which is a primary task for the government.

In earthquake engineering a convenient approach is to make a subdivision into five damage limit states called DS1 to DS5 ranging from small damage to full collapse. For life safety only DS4 and DS5 are of importance. DS4: Significant damage (in Eurocode 8-3 referred to as passing the limit state SD) DS5: Near Collapse (in Eurocode 8-3 referred to as passing the limit state NC). After the passing of the SD limit there is quite an amount of economic damage (the structure is usually beyond repair) but the number of casualties is believed to be small (according to relevant HAZUS studies (HAZUS, FEMA 2013)). In the case of passing the NC limit the economic damage is not much larger than SD, but the number of casualties may be much larger, depending on the type and use of the structure. Given this distinction in consequences we may reformulate (1) as

$$\begin{aligned} &\text{Min } C_{\text{tot}} = -C_s + P_F(\text{SD})C_F && \text{in the lifetime of the structure} \\ &\text{Sub } P_F(\text{NC}) < P_{F,\text{limit}} && \text{per year} \end{aligned} \tag{2}$$

Both parts of the equation are discussed in the following sections.

2.1 Target reliability based on economic optimisation

In ISO 2394 (1998) the target reliability index is given for the working life and related not only to the consequences but also to the relative costs of safety measures as shown in Table 1.

Table 1: Target reliability index (life-time) in accordance with ISO 2394 (1998)

	Consequences of failure			
Relative costs of safety measures	small	some	moderate	great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

According to ISO 2394 (1998) the target level for existing structures decreases 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), in ISO 2394 (2015) 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. EN 1990) in most of the cases the lowest line (Low) is used. In EN 1990 the classification in 'low', 'moderate' and 'high' are specified in building 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 (High). However this economic optimization is bounded by considerations for human safety; this will be discussed in the next session.

2.2 ***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 (see e.g. ISO 2394 (1998), Annex E.4).
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 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). This value is meant to be applied in exceptional cases. An important question is if IR = 10^{-5} would be acceptable for application on a large scale for existing structure since it means a significant reduction in safety level with respect to newly built structures. For the NPR a preliminary value of 10^{-5} has been prescribed by the government; however from other viewpoints like the group risk this value could be changed in the future.

2.2.1 *Individual risk*

The probability P_d that a person dies in one year at a certain location due to structural failure under earthquake load becomes

$$P_d = P_f P_{d|f} \tag{3}$$

Here, $P_{d|f}$ is the conditional probability of casualty given the structural failure.
In the literature several studies to his conditional probability $P_{d|f}$ are available.
Jaiswal *et al.* (2009) performed an analysis of collapses due to earthquakes worldwide. For collapses in the USA Jaiswal *et al.* (2009) take the 'fatality rates given collapse' from HAZUS (NIBS-FEMA, 2006, 'with injury severity level 4 at the complete damage state'). For earthquakes in other countries Jaiswal et al (2009) works with 'injury category-5 (deaths) associated with damage grade D5 (partially or totally collapsed)'. The results from the study of Jaiswal *et al.* (2009) are shown in Table 2. In Spence *et al.* (2011) comparable value are found based on various earthquake damage databases.

Table 2: Fatality rates given structural collapse (FR), Jaiswal et al. (2009)

Building type	$P_{d\|f}$
Adobe buildings	0.06
Mud wall buildings	0.06
Non-ductile concrete moment frame	0.15
Precast framed buildings	0.10
Block or dressed stone masonry	0.08
Rubble or field stone masonry	0.06
Brick masonry with lime/cement mortar	0.06
Steel moment frame with concrete infill wall	0.14

Design and assessment of structures will be done in the NEN-EN 1990 and NEN 8700 framework. Therefore the values from Table 2 have to be translated to values that can be used in the consequence classes CC1-2-3 that are being used in these codes. In NEN 8700 CC1 is split into CC1A for structures where no human lives are at risk and CC1B for normal residential houses. For the definition of the consequence classes we refer to NEN-EN 1990 and NEN 8700.

For the Groningen area, CC1B consists out of mostly masonry houses, 'brick masonry' with $P_{d|f}$ = 0.06). Some of the CC1B buildings will be other types from Table 2 e.g. 'framed buildings' with $P_{d|f}$ = 0.14 or $P_{d|f}$ = 0.10). We look for a characteristic $P_{d|f}$ that is characteristic for the houses, the majority consists of masonry houses, therefore for CC1B the value $P_{d|f}$ = 0.07 is a pragmatic choice.
CC2 consists of larger, more important buildings like schools (e.g. 'framed buildings' from Tabel 2). It seems that in Table 2, here the conditional probability of casualty given the structural collapse is somewhat larger. Therefore for CC2 we assume $P_{d|f}$ = 0.15.
CC3 structures are not explicitly mentioned in Table 2. For CC2 $P_{d|f}$ appears to be somewhat larger than for CC1B. Therefore for CC3 we choose a value that is larger than for CC2: $P_{d|f}$ = 0.5. Further research is needed to confirm this value.
For the time being, it is still uncertain if the Dutch (masonry) buildings behave better or worse under earthquake load in terms of the conditional probability of casualty given structural collapse.
Considering Equation (3) and the proposed $P_{d|f}$ values the annual target collapse probabilities for structures in NC become (all numbers on an annual basis):

$$P_f P_{d|f} < 10^{-5}$$

CC1B: $P_f \leq 1.43 \cdot 10^{-4} \rightarrow \beta \geq 3.6$

CC2: $P_f \leq 6.67 \cdot 10^{-5} \rightarrow \beta \geq 3.8$

CC3: $P_f \leq 2 \cdot 10^{-5} \rightarrow \beta \geq 4.1$

The target failure probabilities related to a reference period t_{ref} (in years) are obtained as follows:

CC1B: $P_f \leq t\, 1.43 \cdot 10^{-4} \rightarrow \beta \geq \Phi^{-1}\{ t_{\text{ref}}\, 1.43 \cdot 10^{-4} \}$

CC2: $P_f \leq t\, 6.67 \cdot 10^{-5} \rightarrow \beta \geq \Phi^{-1}\{ t_{\text{ref}}\, 6.67 \cdot 10^{-5} \}$

CC3: $P_f \leq t\, 2 \cdot 10^{-5} \rightarrow \beta \geq \Phi^{-1}\{ t_{\text{ref}}\, 2 \cdot 10^{-5} \}$

In Table 3 for reference periods of 1, 15 and 50 year the reliability index resulting from the IR = 10^{-5} criterion are summarised.

Table 3: Reliability index based on IR $= 10^{-5}$

Consequence class	Reference period		
	1 year	15 year	50 year
CC1B	$\beta \geq 3.6$	$\beta \geq 2.9$	$\beta \geq 2.4$
CC2	$\beta \geq 3.8$	$\beta \geq 3.1$	$\beta \geq 2.7$
CC3	$\beta \geq 4.1$	$\beta \geq 3.4$	$\beta \geq 3.1$

2.2.2 *Group risk*

Authorities in many cases would to avoid accidents where large numbers of people could die simultaneously, this is also described in ISO 2394 (1998). This code provides a group (GR) or societal risk metric to be applied to one single building:

$$P_{\mathrm{GR}} \leq A N^{-a} \text{ per year} \tag{4}$$

where N is the expected number of casualties in a single event; A and α are constants with recommended values A = 0.01 or 0.1 and α = 2. In the Netherlands often the following criterion is applied (e.g. for tunnels)

$$P_{\mathrm{GR}} \leq \frac{10^{-2}}{N^2} \text{ per year} \tag{5}$$

Here we therefore apply A = 0.01 and α = 2 and see how the IR and GR criteria relate. Question is what the expected number of casualties N in one building is as a results of an earthquake. Tanner and Hingorani (2010) did an analysis of more than 100 collapses of buildings and the number of casualties; all observed buildings were built according to Western building codes and many of the studied collapses were due to earthquakes. Tanner and Hingorani derived the following empirical relations between the expected number of casualties N and the collapsed area A_{col}

$$\text{Non-densely occupied:} \quad N = 0.27 \; A_{\mathrm{col}}^{0.50} - 1 \tag{6}$$

$$\text{Densely occupied:} \quad N = 0.59 \; A_{\mathrm{col}}^{0.56} - 1 \tag{7}$$

Note that the formulas may result in negative values for N, which then of course should be neglected. Based on expressions (5), (6) and (7) we can now plot the reliability index based on the group risk criterion depending on the collapsed area and compare with the values from Table 3. This is shown in Figure 1.

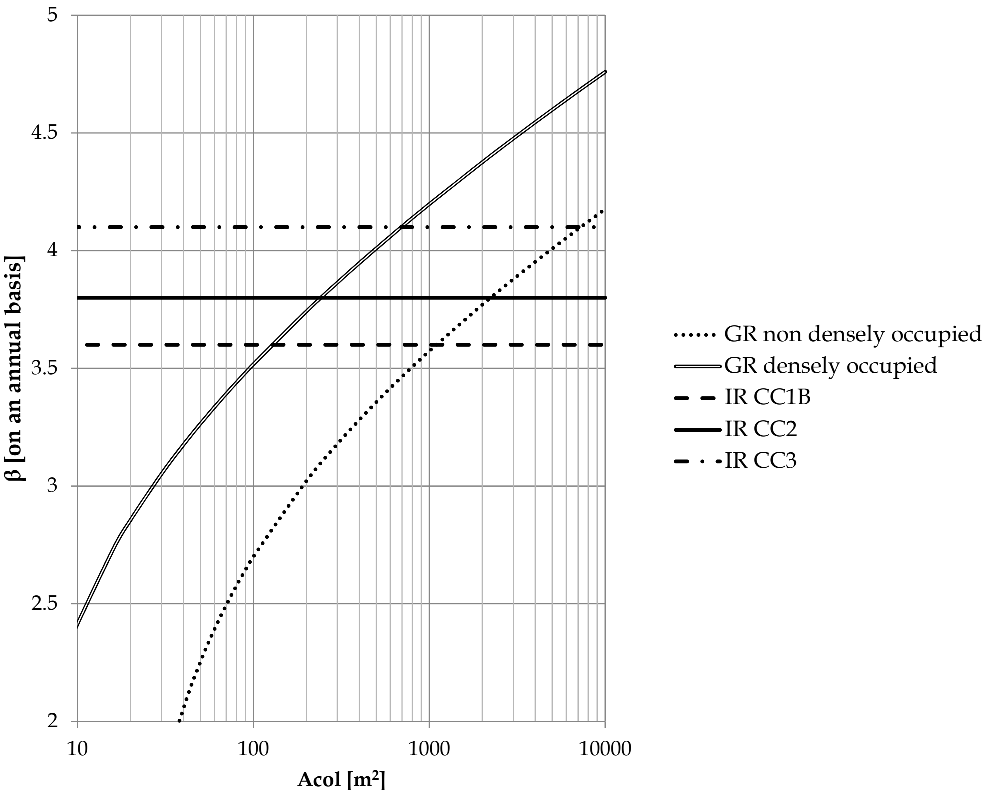

Figure 1: Relation between IR and GR

Figure 1 makes clear that especially in the case of densely occupied buildings (schools, public buildings, hospitals, churches) the GR criterion might be governing already from areas larger than 200 m^2 for e.g. CC2. Also in the case of high correlation of the collapse of single houses (e.g. rows of houses with a joint stability system) this might be an issue. Vrijling *et al.* (1998) give a method to quantify the confidence bounds in the GR calculation; these should be observed while calculating the GR. Presently in NPR 9998 no GR criterion is incorporated however; this should be a point of attention for the future.

2.3 *Reliability levels NPR 9998*

Reliability levels for both new and existing structures are derived based on the values in section 2.2 since it is assumed that human safety is governing over economic optimisation. For new structures we use a reference period of 50 year according to NEN-EN 1990. For existing structures, the assessment can result in the acceptance of an actual state or in the upgrade of a structure; two reliability levels need to be specified - the minimum level β_0 below which the structure is unreliable and should be upgraded, and the target level β_{up} indicating an optimum upgrade strategy. In NEN 8700 a minimum reference period of 15 year is prescribed, here therefore the reliability index will be given for this reference period. Based on the theory in section 2.2 for existing structures (upgrading and disapproval) the reliability index is given for t_{ref} = 15 year. In the following sections the reliability requirement is elaborated using a full probabilistic assessment of using a semi-probabilistic assessment with design values of the earthquake load and the seismic resistance of the structure.

3 Probabilistic seismic hazard analysis and fragility functions

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 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):

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

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

where

$P(..)$	=	the annual probability that the PGA value a_g will exceed a_0 on 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	=	$\Sigma\, \lambda_i$ 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 (KNMI, 2013 and KNMI, 2015). These relationships are generally called ground-motion prediction equations, or GMPEs. Based on this method, we obtain per location distribution functions of the PGA (annual exceedance probabilities). 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, we will consider 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 Probabilistic and semi-probabilistic assessment

The individual risk requirement can be formulated as:

$$IR = P(d \mid F)\, P(F) < 10^{-5} \tag{9}$$

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

$$P(F) = \int F_{\mathrm{R}}(x) f_{\mathrm{PGA}}(x)\, dx \ , \tag{10}$$

where $f_{\mathrm{PGA}}(x)$ is the probability density function of the annual maximum hazard expressed in PGA at the location of the structure and $F_{\mathrm{R}}(x)$ is the fragility function of the structure under consideration for NC.

This requirement can be translated in a Load and Resistance Factor Design (LRFD) semi probabilistic reliability calculation using partial load factors (importance factors γ_{I}) and resistance factors γ_{R} related to the limit state NC. This semi probabilistic procedure has to be calibrated on the basis of a full probabilistic calculation.
The design value of the seismic action is defined as the importance factor γ_{I} times the action with a return period of T = 475 year (corresponding to the 0.2% mentioned before),

according to NEN-EN 1998-1. The resistance factor γ_M is meant to be applied on the global resistance calculated via linear of non-linear calculations. In general for design values the following possibilities are possible (see Fig. 2):

1. a dominant high value
2. a non-dominant high value
3. a non-dominant low value
4. a dominant low value.

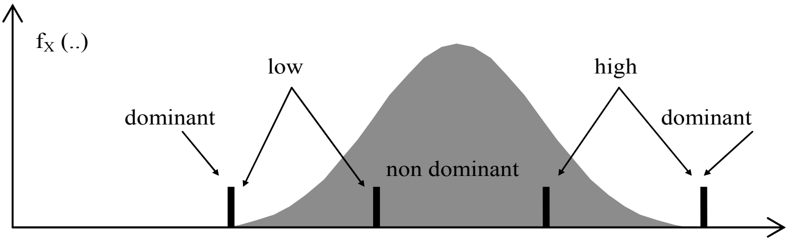

Figure 2: Dominant and non-dominant design values in a semi-probabilistic approach

For variables with distribution function $F_X(..)$ the value of the design point X_d can be found via $F_X(X_d) = \Phi(-\alpha\beta)$, with $\Phi(..)$ the distribution function of the normal distribution. The value for the probabilistic influence coefficient α follow from a full probabilistic calculation, but for general purposes NEN-EN 1990 and ISO 2394 give the values in Table 4 based on experience and theoretical arguments.

Table 4: Standard values for α, according to ISO2394 and NEN-EN 1990

X	α
Dominant resistance parameter	0.8
Other resistance parameters	0.4 x 0.8 = 0.32
Dominant load parameter	- 0.7
Other load parameters	- 0.4 x 0.7 = - 0.28

Table 5 provides the results of the calculation of the return periods *T* for the PGA with an α-factor of 0.7 - 0.75 for the different consequence classes from NEN-EN 1990 and NEN 8700. Based on section 2.2 the reliability index is calculated for the reference period (50 or 15 year).

From full probabilistic calculations it follows that the standard α-factor from Table 4 is too low; it should be in the order of magnitude of 0.9. In the NPR it however is chosen to incorporate this in the partial factors for the resistance. The calibration of these factors is done via a full probabilistic calculation, see below.

Table 5: Derivation of return periods for the design value of the seismic action

	New structures			Existing structures		
	CC1B	CC2	CC3	CC1B	CC2	CC3
Reference period [year]	50	50	50	15	15	15
β (Section 2.2, Table 3)	2.4	2.7	3.1	2.9	3.1	3.4
α	0.7	0.7	0.7	0.75	0.75	0.75
$\alpha\beta$	1.71	1.90	2.16	2.14	2.32	2.57
Probability in the reference period that the PGA is larger than the design value	0.0432	0.0288	0.0153	0.0161	0.0102	0.0050
Return period design ground acceleration						
$T_{calculated}$ [year]	1158	1738	3276	932	1466	2982
$T_{rounded}$ in NPR [year]	1200	1800	3600	800	1500	3000

The rounding off value $T_{rounded}$ in NPR for CC1B has been chosen to be 800 years, this is a little too low, this is corrected for in the calculation below for γ_M. The importance factors γ_I follow from:

$$\gamma_I = \mathrm{PGA}(T) \ / \ \mathrm{PGA}(475 \text{ year}) \qquad (11)$$

These important factors can be calculated from the hazard curves resulting from the PSHA. In Table 6 the importance factors γ_I resulting from the 2013 KNMI model are shown.

Table 6: Importance factors γ_I for NC based on the 2013 KNMI model

Consequence class	New Structures		Existing Structures	
	T [year]	γ_I [-]	T [year]	γ_I [-]
CC1B	1200	1.3	800	1.2
CC2	1800	1.5	1500	1.4
CC3	3600	1.7	3000	1.6

The design values of the PGA obtained in this way are too small because of the α-value that should be larger. This is compensated for using partial factors fort the resistance γ_R. To establish these factors a full probabilistic calculation is carried out; it consists of the following steps:

1. Define the seismic load on the structure as the PGA.
2. Determine the design value of the PGA from the hazard curve as described above.
3. Take the design value of the resistance equal to the design value of the load, expressed in the ground acceleration at which the structure globally collapses (NC).
4. Determine the characteristic value of the resistance by dividing by an assumed partial factor γ_M for the resistance.
5. Take as starting point that the calculation of the seismic resistance provides a 0.05 fractile.
6. Use a coefficient of variation of 0.3 for the seismic resistance of new structures and 0.5 for existing structures.
7. Determine the fragility curve $F_R(x)$ using a lognormal distribution.
8. Determine the failure probability in the reference period via $P_f = \int F_R(x) f_{PGA}(x)\,dx$, with $f_{PGA}(x)$ the probability density function of the maximum PGA in the reference period, this can be derived from the hazard curve.
9. Determine the annual failure probability.
10. Determine the individual risk as $IR = P_f P_{d|f}$.
11. Compare this with the target $IR \leq 10^{-5}$.
12. Repeat steps 4 to 11 until the target IR is reached.

The partial factor for the resistance factor γ_M should not be understood as a material factor γ_m but a partial factor fort the seismic resistance expressed in the PGA where collapse occurs. A condition for the application of this factor is that the calculation of the seismic resistance delivers the 0.05 fractile in the fragility functions; if calculations would e.g. provide the mean value of the resistance it should be converted to a 0.05 fractile.
In Table 7 the result is shown in the case the KNMI 2013 model is used for the hazard curves.

Table 7: Calculation γ_R based on the 2013 KNMI model

		T [year]	$V(R)$	γ_M	γ_I	IR/10^{-5}
new	CC1B	1200	0.3	1.1	1.3	0.9
	CC2	1800	0.3	1.2	1.5	1
	CC3	3600	0.3	1.3	1.7	1.1
existing	CC1B	800	0.5	1.1	1.2	1.3
	CC2	1500	0.5	1.2	1.4	1
	CC3	3000	0.5	1.3	1.6	1.2

A combination of the PGA values from a PSHA analysis for T = 475 year with the importance factors and partial factors for the resistance from Table 7 satisfies the requirement with respect to individual risk. In the derivation of these factors a full probabilistic calculation is used, avoiding the conservative assumptions of a level I probabilistic calculation. In the same way, for the KNMI (2015) model, the values for γ_M and γ_I are calculated and shown in Table 8.

Table 8: Calculation γ_I and γ_M based on the 2015 KNMI model

		T [year]	$V(R)$	γ_M	γ_I
new	CC1B	1200	0.3	1.1	1.4
	CC2	1800	0.3	1.2	1.6
	CC3	3600	0.3	1.3	1.9
existing	CC1B	800	0.5	1.1	1.2
	CC2	1500	0.5	1.2	1.5
	CC3	3000	0.5	1.3	1.8

The differences are caused by the fact that in the KNMI (2015) curves the tails of the frequency-magnitude relationship are less favourable. The non-linear site effects are not taken in consideration in the KNMI (2015) study while in the KNMI (2013) there are taken from measurements in Southern European countries. It is expected that the correct probabilistic implementation of the non-linear site response for the Groningen soft soils will lead to a decrease of γ_I and γ_M. Recommendation is to implement this as soon as possible.

5 Conclusions

In this paper the background is shown of the safety philosophy in NPR 9998. The method chosen is a reliability based design and assessment of buildings under earthquake load. In this way for the inhabitants of Groningen clear insight can be provided in the risks. At this moment the target safety level is coupled to an individual risk criterion. For the future also group or societal risk criteria and cost optimization procedures should be taken into account. Based on a full probabilistic approach partial factors (importance factors and global resistance factors) can be derived. They depend largely on the shape of the hazard curve, here the probabilistic implementation of the non-linear site response for the Groningen soft soils is of large importance.

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