Seismic Assessment of a Typical Dutch Rijtjeshuis

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Title

Seismic Assessment of a typical Dutch Rijtjeshuis

By

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An electronic version of this thesis is available at http://repository.tudelft.nl/.
The present thesis is the outcome of research done in TNO, in order to obtain my Master Degree in Civil Engineering at Delft University of Technology, with a specialization in Structural Engineering. The objective of this report is the seismic assessment of a typical Dutch Rijtjeshuis in the area of Groningen, by the derivation of fragility curves and taking into consideration the variability in both the material properties and the ground motions.

The research was carried out under the guidance of TNO and TU Delft. TNO has provided me the license for the DIANA software as well as workplace and resources to perform this research.

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SUMMARY

A critical issue that is raising concern among the scientists and engineers during the last decade in the Netherlands is the occurrence of relatively small earthquakes in the North part of the Netherlands (Groningen, Drenthe and North Holland). These earthquakes that have a non-tectonic origin and are related to the gas-field depletion, have caused feelings of anxiety among the residents of the area. Thus, investigation of the seismic vulnerability of the structures in the affected area is crucial and the necessity of it is increasing when taking into account the dominating type of buildings: unreinforced masonry structures.

The present Master Thesis Project is focusing on the seismic assessment of the Dutch “Rijtjeshuis”. A seismic analysis of a series of unreinforced masonry houses in Groningen is executed, which indicates the response of the structure to seismic actions and identifies the near collapse state for a range of seismic scenarios. The assessment of the performance of existing buildings is usually done by either fully dynamical procedures (non-linear time history analyses) or static pushover methods. Structural response is defined in the present study by non-linear static pushover analyses, using finite element software DIANA (Version 9.6). The capacity spectrum method, using the inelastic demand spectra, is followed after considering variability both in the material properties (thus capacity curves) and the demand spectra, with ultimate purpose the derivation of the fragility curves for this typology of structures. The fragility curves are necessary to allow for a reliability based judgment of the structure. The distribution of the seismic resistance is built up from several parameters the most important of which are the ground motion variability, within building variability, and building- to building variability.

Masonry as a material is characterized by high rigidity, low tensile and shear strength, low ductility and low capacity of bearing reverse loading. These are the main reasons for the frequent collapse of masonry buildings during earthquakes often responsible for a considerable number of casualties. In the majority of real cases building properties are not well known due to the variability in building materials and building techniques.

The masonry structure examined in the present study is supposed to be representative of a class of buildings with similar structural characteristics, mechanical parameters have been considered as random variables. The material variability can be regarded as the most important source of uncertainty in the determination of structural response, with all other sources either deriving directly from its effects or being insignificant in size compared to it.

After the variability in the material properties is taken into account and the pushover analyses have been executed, the behavior of this typology of structures is known with the pushover and capacity curves. The reliability of the model used for the pushover analyses, is based on the verification of the finite element model with an existing one made by TNO in the framework of the project “NPR 9998 - Rekenvoorbeeld betonconstructie (TNO pushover analyses in DIANA)”. The first model that is developed in the Finite Element Program DIANA FX+ (version 9.6) is made of reinforced concrete as the one made by TNO. After the model is certified, the material properties of the concrete wall are going to be replaced with the mean material properties of masonry (Calcium-silicate brickwork, typical approximately 1960-1985). All the material properties of the structure (referring to masonry only) are assumed to be random variables, to which normal probability distributions are associated based on realistic ranges of variation. The sensitivity study is done based on the mean values and the coefficient of variation of a dominant random variable which is chosen to be the tensile strength of masonry. The pushover curve of each analysis will be extracted with ultimate goal the derivation of the family of pushover curves for the different material properties and further the capacity displacement for each case.

Following, the demand from different earthquake scenarios are examined: after a probabilistic seismic hazard assessment, the necessary knowledge of the likely earthquake actions at a subject site have been determined. After using Jayaram’s method for the derivation of the acceleration response spectra and accelerations-
displacement spectra, the demand of each earthquake can be computed. The inelastic displacement demand, that needs to be calculated because of the energy dissipation observed during the performance of the structures under seismic loads, is derived using Lin and Miranda’s method, which is an accurate and non-iterative procedure.

Finally, since both the capacity of the structure and the inelastic demand of each earthquake is known, the comparison between these values will reveal information about the performance of the structure for the examined limit state. Since the variability in both the material properties and in the ground motions have been considered, the probability of collapse for each earthquake can be calculated. The repetition of this procedure for all possible earthquakes and structures results in the fragility curve.

As a conclusion, the fragility curve that came out has the expected behavior and is comparable to ones found in literature for unreinforced masonry structures. The assumptions made in the present study are discussed further and recommendations for future studies are done.
1 INTRODUCTION

The Netherlands is a country where no earthquakes occur as it is not affected by the convergence of the plates which lead to seismic action. Only exception is a fault in the south part of the country between Brabant and Limburg, which gives light earthquakes. However, in the last few decades the occurrence of relatively small earthquakes in the North part of the Netherlands (Groningen, Drenthe and North Holland) has raised concern among the residents of the area and the scientists all over Holland. These earthquakes are related to the gas-field depletion. The Groningen field, the largest gas field of Europe, has been in production since 1964 while the first tremors with a magnitude higher than 2.4 were observed in 1990. Since then a multidisciplinary study was initiated with a goal to find the relationship between gas production and earthquakes. The main result was that the earthquakes had non-tectonic origin and were induced by the reservoir depletion (i.e. gas production). In order to have a clearer view of the phenomenon a borehole seismometer network was installed in 1995 which can provide the location of the tremors and a quantification of their magnitude.

![Man-Made Quakes | As a gas field depletes, tension causes tremors](image)

In the last few years an increased frequency of earthquakes with higher magnitude is observed and their effect on the structures becomes more pronounced: more and more cracks have appeared in the buildings raising uncertainty and insecurity among the residents. Thus, a further research is decided to be executed, which will focus on the behavior of the structures in the North Netherlands.

The necessity for investigation of the seismic vulnerability is increasing when taking into account the type of buildings dominating the area: unreinforced masonry structures. The buildings are typically two floor terraced houses (typical Dutch Rijtjeshuis) of unreinforced masonry construction, where a row of identical or mirror-image houses share side walls. They are considered less as individual buildings, but as part of each city block.
Unreinforced masonry structures in North Netherlands seem to be really vulnerable to seismic actions, due to their layout, having only load bearing walls in one direction and the fact that no seismic rules have been applied during their design. The Dutch “Rijtjeshuis” are also characterized by great variability in the structural behavior depending on the constituents (blocks and mortar) and the dimensions and shape of the blocks, the interlocking in the external leaves and the transversal connection through thickness.

The present Master Thesis Project is a separate independent research that focuses on the seismic assessment of the Dutch “Rijtjeshuis”. A seismic analysis of a series of unreinforced masonry houses in Groningen will be executed, which indicates the response of the structure to seismic actions and identifies the near collapse state for a range of seismic scenarios. Structural response is defined by non-linear pushover analyses, using finite element software DIANA (Version 9.6). A difference in the seismic behavior in the two horizontal directions is expected and thus the weaker one will be examined. The capacity spectrum method, using the inelastic demand spectra, will be followed after considering variability both in the material properties (thus capacity curves) and the demand spectra, with ultimate purpose the derivation of the fragility curves for this typology of structures. The fragility curves are necessary to allow for a reliability based judgment of the structure, something not possible with one Finite Element Model calculation. The distribution of the seismic resistance is built up from several parameters the most important of which are the ground motion variability, within building variability, and building to building variability. All the above will be analyzed further in the following chapters.

1.1 Objectives of this Thesis
The main objective of this Master Thesis Project is the seismic assessment of a series of typical Dutch Rijtjeshuis by deriving the fragility curves for a variety of material properties and a variety of seismic scenarios. As described previously, the capacity spectrum method will be used for inelastic demand spectra and capacity curves obtained by non-linear pushover analyses using finite element software DIANA (Version 9.6).

In order to successfully derive the primary objective of this study, the definition of several secondary objectives is crucial:

- Review of the state of art of structural analysis methods commonly used for the assessment of the response of structures to seismic actions.
- Review of the methods used to examine masonry structures
- Obtain plans of an unreinforced masonry structure in Groningen. Thus, the choice of the appropriate finite element model will be ensured in order to examine a representative of the typical Rijtjeshuis in the area.
- Find the mean and extreme values for the material properties of masonry in Groningen from literature
- Select the appropriate material properties for the concrete in the floors
- Choose the type of wall-floor connections taking into account the geometrical uncertainties
- Create a finite element model of the building using DIANA (Version 9.6). The model must well represent the true form of the building. The model’s material will be reinforced concrete as one already made by TNO. The two models will be compared, until the model created is verified.
- When the model for concrete is verified, the material properties of the bearing wall will change to the ones of unreinforced masonry. For the non-linear finite element analysis, the element type and discretization, material behavioral models, and non-linear solution procedures must be carefully selected to ensure representative and trustful results. In this model only global failure will be examined.
- Execute non-linear pushover analyses for the mean values of the material properties of masonry for the created model.
- Definition of limit state. Especially the definition of the ultimate limit state in terms of global widespread damage is very difficult and often subject of subjective evaluation introducing additional uncertainty in the result of the evaluation
- Execute non-linear pushover analyses for all the different material properties and derive the capacity curves (top drift vs. base force) until the drift limit decided previously.
Computing the equivalent bilinear capacity curves using N2 method

A transformation of the obtained graph to spectral accelerations and spectral displacements will provide the capacity spectrum, something achieved after using the single degree of freedom system for the equivalent period.

Derive the demand spectrum, which is obtained by highly damped elastic spectra (Lin & Miranda, 2008), in an Acceleration-Displacement Response Spectrum (ADRS) format, where the radial lines represent the equivalent periods $T_{eq}$ of the structures.

Using the Lin & Miranda method the calculation of the inelastic displacement demand of each earthquake will be calculated.

Apply the capacity spectrum method, where the intersection of the two graphs (capacity spectrum and demand spectrum) is an indication of the performance of the structure.

Derive the fragility curves by following the procedure described further:

- Compare the nonlinear displacement response with the ultimate displacement capacity ($S_{nu}$) to identify if the building exceeds the collapse limit state, or not.
- Repeat for all response spectra and estimate the probability of collapse for the selected capacity curve.
- Repeat for all capacity curves, and then estimate the mean probability of collapse, for a given direction.
- Plot the probability of collapse against the level of PGA.
- Give an estimation of the sufficiency of the response of the structures to seismic actions.

1.2 Outline of the Thesis

The present thesis constitutes of an introductory chapter (Chapter 1), that gives a description of the tasks that have been carried out, and two other parts, which include the literature study and the implementation of it in a specific case study respectively. A final part (part 3) describes the conclusions of the study as well as recommendations for future studies.

The first section consists of five chapters: Chapter 2 gives a short description of the behavior of the material use, Masonry. Chapter 3 is a summary of the techniques that exist in literature regarding the seismic assessment of structures. Chapters 4 and 5 introduce the methods for the derivation of the inelastic displacement demand from an earthquake and the state of art of the seismic hazard analysis respectively. Finally, chapter 6 gives a description of the theoretical background of Fragility curves.

The second section is about the case study examined in the present thesis. Chapter 7 gives a description of the exact steps that are going to be followed in the study. Chapter 8 is about the analyses that were carried out and chapter 9 gives the sensitivity study in the material properties of masonry. In chapters 10 and 11 the application of Probabilistic Seismic Hazard Assessment and Lin & Miranda methods are presented respectively. Chapter 12 includes the application of the capacity spectrum method and the derivation of the fragility curves which were the target of the thesis. In chapter 13 an additional sensitivity study is done in order to examine three of the important assumptions that have been done in this thesis.

Finally comes the third part of the thesis that includes the conclusions and recommendations for future studies and chapter 15 in which the literature can be found. Following Appendixes A, B and C can be found which include respectively the MATLAB script used for the production of the response spectra, the inelastic displacement demand and the capacity spectrum method (A), figures from the important results coming from the pushover analyses (B) and the drawings of the building in Groningen that is examined (C).
PART 1: LITERATURE STUDY

2 MASONRY

2.1 INTRODUCTION

Masonry structures have been very popular all over the world among the years. They are a much diffused type of construction which can be built rapidly, cheaply and often without any plan or particular technical competence. Thus, this is one of the most common housing types built across the world. Masonry also represents the structural type of a large architectural heritage that needs to be preserved. Moreover, old unreinforced masonry buildings constitute the large majority of urban aggregates in several countries which can be affected by earthquakes or not.

Masonry buildings have mechanical properties which need to be examined since the behavior of masonry structures needs to be interpreted. Masonry as a material is characterized by high rigidity, low tensile and shear strength, low ductility and low capacity of bearing reverse loading. These are the main reasons for the frequent collapse of masonry buildings during earthquakes often responsible for a considerable number of casualties.

2.2 GENERAL CHARACTERISTICS OF MASONRY MODELLING

Masonry as a material is characterized by distinct directional properties due to the mortar joints which act as planes of weakness. The numerical representation of masonry structures can vary based on the level of accuracy needed. The following modelling strategies can be used:

- Detailed micro-modeling: continuum elements represent units and mortar in the joints, whereas their interface is represented by discontinuous elements;
- Simplified micro-modelling: expanded units are represented by continuum elements, while their interface is lumped in discontinuous elements;
- Macro-modelling: units, mortar and interface are smeared out in the continuum.

The above methods of masonry modelling are presented in Figure 2.

FIGURE 2: MODELING STRATEGIES OF MASONRY STRUCTURES: (A) MASONRY SAMPLE; (B) DETAILED MICRO-MODELING; (C) SIMPLIFIED MICRO-MODELING; (D) MACRO-MODELING (LOURENCO (1996))
The masonry mechanical properties depend on many parameters, such as the material properties of units and mortar, the arrangement of bed and head joints, anisotropy of units, dimensions of units, joint width, quality of workmanship, degree of curing, age of construction and environment.

2.3 ASPECTS OF SOFTENING BEHAVIOR

Softening is the gradual decrease of mechanical resistance under a continuous increase of deformation forced upon a material specimen or structure. It is a salient feature of quasi-brittle materials like clay brick, mortar, ceramics, rock or concrete, which fail due to a process of progressive internal crack growth. Such a mechanical behavior is commonly attributed to the heterogeneity of the material due to the presence of different phases and material defects, like flaws and voids.

The phenomenon of softening has been well identified both in tensile and shear failure of masonry. In compression, softening behavior depends on the size of the specimen and the boundary conditions in the experiments.

In Figure 3-Figure 4 some typical stress-displacement diagrams are presented for quasi-brittle materials in uniaxial tension and compression. The assumption that inelastic behavior both in tension and compression can be described by the area under the $\sigma$-$\delta$ diagram is used in the present study, and thus the quantities $G_f$ and $G_c$ (fracture energy for tension and compression) are dealt with as material properties of masonry.

![FIGURE 3: TYPICAL BEHAVIOR OF QUASI-BRITTLE MATERIALS UNDER UNIAXIAL TENSION AND DEFINITION OF FRACTURE ENERGY (LOURENCO (1996))](image1)

![FIGURE 4: TYPICAL BEHAVIOR OF QUASI-BRITTLE MATERIALS UNDER UNIAXIAL COMPRESSION AND DEFINITION OF FRACTURE ENERGY (LOURENCO (1996))](image2)
2.4 PROPERTIES OF UNIT, MORTAR AND THEIR INTERFACE

Masonry mechanical properties are strongly dependent on the properties of units and mortar that is used in the construction. The properties of each of the two constituents are obtained from series of experiments and can be combined, based on different codes, to give the properties of the composite material, masonry.

The weakest point in a masonry composite is often observed at the interface between unit and mortar, as cracking is usually concentrated in these areas. Thus, the non-linear behavior of this interface highly affects the deformation capacity of masonry. The behavior of this interface depends on various parameters such as the water retention capacity, porosity of mortar, absorbency of the units, amount of binder and curing conditions. Two different failure modes have been observed related to the interface unit-mortar, Mode I failure and Mode II failure, which are related to tensile and shear failure respectively.

2.4.1 MODE I FAILURE

As a result from deformation controlled test, exponential tension softening curves appear. The phenomenon which is observed is the development of micro-cracks into macro-cracks. The fracture energy in this type of failure, type I, which represents the amount of energy needed in order to form a complete crack along the interface, ranges from 0.005 to 0.025 J/mm$^2$.

\[ \sigma = f(\varepsilon) \]

\[ \varepsilon = g(\sigma) \]

\[ \varepsilon = h(\sigma) \]

\[ \sigma = i(\varepsilon) \]

\[ \sigma = j(\varepsilon) \]

\[ \sigma = k(\varepsilon) \]

\[ \sigma = l(\varepsilon) \]

\[ \sigma = m(\varepsilon) \]

\[ \sigma = n(\varepsilon) \]

\[ \sigma = o(\varepsilon) \]

\[ \sigma = p(\varepsilon) \]

\[ \sigma = q(\varepsilon) \]

\[ \sigma = r(\varepsilon) \]

\[ \sigma = s(\varepsilon) \]

\[ \sigma = t(\varepsilon) \]

\[ \sigma = u(\varepsilon) \]

\[ \sigma = v(\varepsilon) \]

\[ \sigma = w(\varepsilon) \]

\[ \sigma = x(\varepsilon) \]

\[ \sigma = y(\varepsilon) \]

\[ \sigma = z(\varepsilon) \]

What was also observed at the specimens was that the bond area is not the full cross-section but a part of it, as shown in Figure 6. This phenomenon could be interpreted as a result from the process of laying units in the mortar and shrinkage of mortar.

\[ \text{FIGURE 5: TYPICAL EXPERIMENTAL STRESS-CRACK DISPLACEMENT RESULTS FOR SOLID CLAY BRICK MASONRY: ENVELOPE OF THREE TESTS (LOURENCO (1996))} \]

\[ \text{FIGURE 6: TYPICAL NET BOND SURFACE FOR TENSILE SPECIMENS OF SOLID CLAY UNITS (LOURENCO (1996))} \]
2.4.2 Mode II Failure
The results from tests which were executed in order to examine this type of failure revealed that shear behavior is characterized by a gradual decrease in strength up to a constant non-zero stress level, as shown in Figure 7. What is also observed, is that there is progressive linear relationship between the confining stress and the mode II fracture energy, which is a Coulomb type of friction.

![Stress-Displacement Diagram](https://via.placeholder.com/150)

**FIGURE 7: STRESS-DISPLACEMENT DIAGRAM FOR DIFFERENT NORMAL STRESS LEVELS: ENVELOPE OF THREE TESTS (LOURENÇO (1996))**

2.5 Properties of Composite Material
The material properties of masonry are strongly connected with the material axes that are examined, since the loading angle has a strong influence, as shown in Figure 8.
2.5.1 **Compressive Behavior**

The compressive strength of masonry is defined by the execution of series of tests, with a compressive force in the direction normal to the bed joints.

2.5.2 **Tensile Behavior**

Tensile failure is generally related to the failure of the joints. The lowest value of the bond strength between unit and joint and the tensile strength of the unit is chosen as the tensile strength of masonry. Thus, the tensile failure can have two types: a zigzag crack through head and bed joints and a vertical crack through unit and mortar, as shown in Figure 9.
2.5.3 Biaxial Behavior

Biaxial test are identical for the examination of the complete behavior of masonry. Full stress vector or the combination of principal stresses and rotation angle \( \theta \) is needed for the derivation of the biaxial strength envelope.

2.6 Structural Capacity Assessment

Unreinforced masonry structures should be designed (and thus also examined) considering vertical and horizontal loads, since the types of failure that can occur are the in-plane and out-of-plane failure mechanisms. Observed failure in unreinforced masonry structures from past earthquakes reveal that the two types of failure are independent and thus can be examined separately.

2.6.1 In-Plane Failure Mechanism

When in-plane behavior is examined, the actual behavior of masonry walls is as Shear walls, which have been thoroughly examined by many researchers such as Anthoine and Magonette, and Kikuchi et al. Two general types of failure can be observed:

1. Diagonal Failure: Cracks develop through unit-mortar interface and the unit itself or through the former as it is a case of biaxial tension compression state. Low aspect ratios and lower axial load characterize this failure.
2. Flexural Failure: Here tension and compression failure are combined, since exceedance of tensile bond strength that results in a crack in the mortar-brick interface, is followed by loss of the resisting section resulting in compressive crushing (known as toe crushing).
2.6.2 Out-of-plane Failure Mechanism

Out-of-plane failure mechanisms are important for the overall structural behavior of the masonry building. Such type of failure is associated to wall spandrels not being well restrained by structural elements and thus fail with a rocking mechanisms when earthquake loads are present. Possible out-of-plane collapse mechanisms are presented in Figure 10.

The force-displacement curve corresponding to out-of-plane failure can be seen in Figure 11:

In Figure 11 the line connecting the points $\lambda W$ (force corresponding to the activation of the rocking mechanism) and $\Delta$ (displacement where instability occurs under static loads) assumes that the system is cracked before the activation of the motion and the parts of wall panels participating to rocking behave as rigid bodies. The dashed line, however, is a better representation of the real behavior, as it assumes that the wall can be initially uncracked and the point around which the rocking is activated has finite dimensions. A more detailed description of the phenomena is given in Doherty et al. (2002).
3 STATE OF ART OF SEISMIC ANALYSIS

The seismic design of structures according to the Performance-Based Design (PBD_ Performance Levels) has its basis on the principle of the determination of the acceptable level of damage (performance level) in combination with the probability of occurrence of the design ground acceleration and thus, the determination of the pursued seismic capacity target. In detail, the method examines the real behavior of the structure in different seismic design levels and the corresponding expected level of damage. The multi-level approach is then the basis of performance-based design and is the natural extension of Limit State Design (LSD) to the new-generation earthquake engineering. In this classical LSD method structural safety is examined by comparing the design force ($S_d$) to a design strength ($R_d$). Respectively, in the PBD the Demand ($D$) is compared to the Capacity ($C$), where demand is the effect of the design earthquake and capacity is the level that the structure resists to this earthquake, which means that demand and capacity are related to each other. Because of the high number of uncertainties included in the PBD method (seismic hazard, site response, structural materials and capacity models), only an estimation for the behavior of the structure is made rather than a determination. In that way, a cost optimization is tried to be achieved in the design of the structure based on human safety and seismic action.

However, the classical design method of the seismic rules (force controlled method) only examines the behavior of the structure up to the point where damage starts to occur (elastic response) and not further. The minimum required safety level (ductility) is approached by the use of behavior factors used in the design. The behavior factors are determined by the knowledge that has been acquired from former earthquakes and the experimental and analytical research executed. However, this method of design can be inaccurate and not conservative (such as in case of highly non-normal buildings), as most structures will respond in the inelastic range during a design earthquake, with the response being affected by the dynamic nature of the seismic action.

The performance based design method is mainly applied for the control, strengthening and repair of existing structures and forms the basis for the international repair regulations (Eurocode 8 – Part 3, FEMA 356, ATC-40, etc). In new structures the method can only be applied for the checking of an already dimensioned structure, as design solutions based on PBD method are not yet reliable and advantageous from an economic point of view.

Necessary prerequisite for the use of the PBD method is the prior knowledge of the behavior of the structure further of the elastic zone (ductility and displacement capacity), which means after the initiation of the damage until the structural failure (collapse). Thus, the method is used in combination with non-linear (inelastic) analyses, either static (pushover analyses) or dynamic (time-history analyses).

3.1 SEISMIC CAPACITY TARGETS

3.1.1 GENERAL

The seismic capacity targets of a structure are combinations of the performance level (state of damage in the structure) and a specific level of seismic action, which is usually determined by the acceptable exceedance probability in the life time of the structure or the equivalent return period. Thus, each seismic capacity target determines an acceptable damage limit state for a specific ground motion. After the desired seismic capacity target is determined, the assessment of an existing structure or the design of a new one can be done. Seismic capacity targets concern both bearing and non-bearing structures. The overall target for the structure results from the combination of the targets for the bearing and non-bearing system, as described in Paragraph 3.1.3.

In table 2.1 the design targets are displayed according to the performance levels and the return periods according to Eurocode 8 – Part 3, following the description of the limit states described in 3.1.2. Indicatively, it can be mentioned that the usual design (Ultimate Limit State) with the current design regulations (Eurocode 1998-1) corresponds to design target B2.
### Required Performance

<table>
<thead>
<tr>
<th>Hazard (Return Period of Design Spectrum)</th>
<th>TX=225 years (20% in 50 years)</th>
<th>TX=475 years (10% in 50 years)</th>
<th>TX=2475 years (2% in 50 years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limit State of Damage Limitation (DL)</td>
<td>A1</td>
<td>A2</td>
<td>A3</td>
</tr>
<tr>
<td>Significant damage (SD)</td>
<td>B1</td>
<td>B2</td>
<td>B3</td>
</tr>
<tr>
<td>Near Collapse (NC)</td>
<td>C1</td>
<td>C2</td>
<td>C3</td>
</tr>
</tbody>
</table>

**TABLE 1: SEISMIC DESIGN TARGETS ACCORDING TO EUROCODE 8 – PART 3**

The determination of the design target depends on the desired combination of human safety targets and costs (cost-benefit analysis), after taking into account the importance of the structure.

### 3.1.2 PERFORMANCE LEVELS FOR BEARING AND NON-BEARING SYSTEMS

All the regulations define, with small differences, three basic performance levels for the bearing system and the non-bearing elements, according to the damage level. In several regulations intermediate performance levels exist (e.g. ATC-40, FEMA 356). Following, the states of damage in the structure according to Eurocode 8 – Part 3 are described and can be seen in Figure 12:

A. Limit State of Damage Limitation (DL)

As far as the bearing structure is concerned (level A), the damage level is such that no function is disrupted during or after the earthquake. Significant yielding is prevented and the strength and stiffness properties are retained. That practically means that only sparse capillary cracks caused by bending, which do not affect the capacity of the structure to bear both vertical and horizontal loads just like before the occurrence of the earthquake. In addition, the risk of human injury is practically negligible.

As far as the non-bearing elements (e.g. partitions and infills) are concerned (level A), only small damage is allowed, which is not affecting basic functions. The exits and the safety systems (stairs, doors, elevators, fire safety systems etc) should stay in service.

Capacities shall be based on yield strengths for all structural elements, both ductile and brittle, and on mean interstorey drift capacity for the infills.

In Eurocode 8 – part 3, the first performance level is *Limit State of Damage Limitation* does not completely coincide with the first level *Immediate Occupancy* of FEMA 356, but refers to somehow larger damage. For the design of usual structures, the level Immediate Occupancy is associated with seismic vibration with return period 72 years (50% exceedance probability in 50 years) while the level Limited Damage for a seismic vibration with return period 225 years (20% probability of exceedance in 50 years).

B. Limit State of Significant Damage (SD)
As far as the bearing system is concerned (level B), some damages are expected which will be repairable and will not lead to loss of the structural stability or serious human injury (small injuries, not fatal, can occur) or important damage of objectives being into the structure. Some residual strength and stiffness is observed and vertical elements are able to sustain vertical loads. For further use of the structure after the earthquake, repair of the damage is needed, which is likely to be uneconomic.

For the non-bearing elements (level B) some damaged are expected, which, however, do not put humans in or out of the structure in risk, either because of object fall or secondary causes.

Capacities shall be based on damage-related deformations for ductile elements and on conservatively estimated strengths for brittle ones.

C. Limit State of near Collapse (NC)

In the bearing system (level C), important, extended and non-repairable damages are expected. The bearing system has still the capacity to carry the vertical loads, but the horizontal stiffness and the resistance towards horizontal forces have dropped significantly and as a result, the structure cannot provide safety towards partial or total collapse. Thus, the danger of collapse because of the aftershocks arises. The danger of serious injuries of humans because of falling objects is large, either in or out of the structure. The use of the building after the earthquake is only possible after extended repair, while it is possible that repairs can be technically impossible or not cost-optimal.

In the non-bearing elements (level C), important damage is expected, which can cause their fall. Exception can be the non-bearing elements or attachments of high danger which must be safely secured in order to avoid them falling in areas where humans gather.

Capacities shall be based on ultimate deformations for ductile elements and on ultimate strengths for brittle ones.
The performance level of the structure is determined by the performance level of the bearing and non-bearing elements. A combination of such a performance level is given in the following table, where A, B and C refer to the performance levels of the bearing system and a, b and c to the respective levels of the non-bearing system.

<table>
<thead>
<tr>
<th>Performance Level of the Bearing System</th>
<th>Performance Level of the Non-Bearing Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limited Damage (A)</td>
<td>Recommended for high importance buildings</td>
</tr>
<tr>
<td>Significant Damage (B)</td>
<td>Recommended for usual importance buildings</td>
</tr>
<tr>
<td>Near Collapse (C)</td>
<td>Not recommended</td>
</tr>
</tbody>
</table>

3.2 Assessment of Performance of Existing Buildings

The assessment of the performance of existing buildings is usually done by either fully dynamical procedures (non-linear time history analyses) or static pushover methods. A comparison between those methods, concerning the maximum interstorey drift and maximum drift demand, can distinguish the advantages and disadvantages of the various types of the static pushover methods and the incremental dynamical analysis. The choice of the analysis to be executed in any case is determined by several parameters some of which are the importance of the structure, the performance level, the characteristics of the structure (e.g. frequency properties, regularity and complexity) and the amount of data available for the structural model. The methods used in earthquake engineering are described in this paragraph and are summarized in Figure 13.
3.2.1 Dynamic Analysis

In order to obtain a reliable estimation of the seismic risk, it is desirable to perform full dynamical analyses that describe the effective transmission and dissipation of the energy coming from the ground motion into the structure. Dynamic analysis is the most natural approach towards the assessment of earthquake response, but is significantly more demanding than static analysis regarding the computational effort and the interpretation of results. In some cases, a building requires an explicit dynamic lateral force procedure, which may be either a response spectrum analysis or a time history analysis. While these procedures add aspects of dynamics to the design procedure, resulting forces are generally scaled to match the lateral force used in the static procedure. Also, components are still evaluated for serviceability in the elastic range of strength and deformation.

Dynamic analysis can be executed by several different methods, as shown in Figure 14, which can be employed either in the time or in the frequency domain. In order to examine earthquake loads the basic methods used are the modal, spectral and time-history, which are analyzed in the following paragraphs.
3.2.1.1 **Modal and Spectral Analyses**

Modal analysis is based on the decomposition of the coupled equations of motion of the MDOF system into equations describing the motion of each of the modes individually in the time domain. An algebraical combination of these equations can give the response of the MDOF system. Modal spectral analysis refers to the case where the various modal maxima are calculated under the effect of a response spectrum representing the transient signal and the maxima are combined to give an upper bound of the maximum response of the MDOF system. Obviously, both methods (modal and spectral) have their base on the principle of superposition, fact that makes them valid only for linear elastic systems. The coupled equation of motion for the MDOF system is the following:

\[ M\ddot{\mathbf{x}} + C\dot{\mathbf{x}} + K\mathbf{x} = -MI\ddot{x}_0 \]

Where: \( M \) and \( C \) are the mass and damping matrices, 

\( K \) is the stiffness matrix,

\( I \) is a vector of influence coefficients, i.e. the \( i^{th} \) component represents the acceleration at the \( i^{th} \) degree of freedom due to a unit ground acceleration at the base. For simple structural models with degrees of freedom corresponding to the horizontal displacements at storey level, \( I \) is a unity vector.

\( \ddot{x}_0 \): the acceleration of the ground

The steps to execute a modal analysis are the following:

i. Make an assumption about the shape of the displacement vector:
\[ x = \Phi Y(t) \]

Where \( \Phi \): modal matrix

\( Y(t) \): vector of modal coordinates

ii. Formulate the eigenvalue problem:

\[ K\Phi_i = \omega_i^2 M\Phi_i \]

iii. Compute the N eigenvalues and eigenvectors using the equation:

\[ (K - \omega^2 M)x = 0 \]

The shape modes are characterized by orthogonality with respect to mass \( M \) and stiffness \( K \) matrices.

iv. Assume mode-proportional damping:

\[ \Phi_i^T C \Phi_j = 2\omega_i \xi_i \delta_{ij} \]

v. Formulate the equations of motion with respect to normal coordinates \( Y_i \):

\[ Y_i + 2\xi_i \omega_i Y_i + \omega_i^2 Y_i = -L_i x_B \]

Where:

\[ \omega_i = \sqrt{\frac{K_{ii}}{M_{ii}}} \]

is the angular frequency for the \( i \)th mode

\[ \bar{M}_i = \Phi_i^T M \Phi_i \]

is the generalized mass

\[ \bar{K}_i = \Phi_i^T K \Phi_i \]

is the generalized stiffness

\[ L_i = \frac{L_i}{M_i} \]

is the modal participation factor and gives a measure of the degree to which the \( i \)th mode participates to the global dynamic response, where \( L_i = \Phi_i^T M I \)

vi. Derive the solutions of the system of N uncoupled equations in normal coordinates given in v. The convolution (Duhamel) integral can be used to express the response of the \( i \)th mode of vibration at any time \( t \).

vii. Compute the total elastic restoring force:

\[ R = K \Phi Y(t) = \sum_{i=1}^{N} \frac{L_i}{M_i} A_i(t) \Phi_i \]

viii. Compute the total seismic base shear \( V_B \):

\[ V_B = \sum_{i=1}^{N} \frac{L_i^2}{M_i} A_i(t) \]

ix. Compute the relative displacement with respect to the base of the structure corresponding to the \( i \)th mode of vibration:

\[ x_i = \Phi Y_i(t) = \frac{L_i}{M_i} A_i(t) \Phi_i \]

### 3.2.1.2 RESPONSE HISTORY ANALYSES

Time-stepping techniques can be used for the calculation of the response of MDOF systems to a transient signal. In this method series of coupled equations of motion are solved as static equilibrium systems, while inertia and damping effects are included. The relatively large deformations of the structure that are caused by the strong earthquake motions, lead to geometric nonlinearities. The analysis of such inelastic systems, characterized by nonlinearities, includes changes in stiffness and thus the periods of vibrations, which also affect the solution characteristics. Because of the unavailability of the superposition principle in the inelastic systems, the necessity to directly integrate the coupled equations of dynamic equilibrium arises.
The response history is divided into time increments $\Delta t$ and a sequence of individual time-dependent force pulses $\Delta F(t)$ are applied on the structure. During each $\Delta t$ the structure is assumed to be linear elastic, with characteristic material and geometry components of the system stiffness matrix which reflect the current state of deformation. Thus, by re-computing the stiffness of all individual members at each time increment and iteration within the time increment, the non-linear behaviour of the structure is approached. This requires considerable computing resources for large structural systems. The steps used to execute the response history analysis on MDOF systems are the following:

i. Derive the incremental form of the equation of motion for the discretized structure:

$$M\Delta \ddot{x} + C\Delta \dot{x} + K(t)\Delta x = \Delta F(t)$$

Where: $K(t)$ : stiffness matrix for the time increment

$\Delta x$ : displacement increment during $\Delta t$

$\Delta F$ : time-dependent force pulse, derived from the ground motion under which the structure is subjected

ii. Using a numerical integration scheme integrate the incremental form of the equation of motion for each time step

iii. At each time step, evaluate the increments of displacements, velocity and acceleration

iv. Update the displacement, velocity and acceleration at the beginning of the interval to derive the corresponding quantities at the end of the time step interval

v. At the end of each time step, evaluate stress states which correspond to the total displacements

vi. If necessary, update the tangent stiffness matrix $K(t)$

When executing a time history analysis on a structure, the derivation of peak acceleration, velocity and displacement of the structure’s response to a ground motion becomes possible. The repetition of this analysis for a series of single degree of freedom structures, each having a different period $T$, can give the data for the production of graphs correlating the peak response acceleration, velocity and displacement with the period of the structure. The resulting graphs form the response spectra for each characteristic factor.

### 3.2.1.3 Incremental Dynamic Analysis

Incremental dynamic analysis (dynamic pushover) is also a method to compute the seismic response of a structure, from elasticity to yielding and finally to collapse. The method is based on subjecting a structure under one or more ground-motion records, each scaled to multiple levels of intensity. The response of the analyses executed are plotted on graphs versus the record intensity level. The resulting curves give an indication of the system performance at each level of excitation, just as the load-displacement curve which is obtained by the pushover analysis. The steps followed in this method are the following:

i. Selection of a suitable ground motion matching the design scenario

ii. Define a monotonic scalable ground-motion intensity measure (e.g. PGA, PGV, PGD etc)

iii. Define a damage measure or structural state variable: force, displacement or energy based parameters, depending on the purpose of the analysis and the system considered

iv. Define a set of scale factors to apply for the selected intensity measure in ii

v. Scale the sample record to generate a set of records in order to test the structure from elastic response to collapse

vi. Perform response history analysis using the scaled accelerogram at the lowest intensity measure

vii. Evaluate the damage measure in iii corresponding to the scaled intensity measure in ii.

viii. Repeat steps vi to vii for all the scaled intensity measures
3.2.2 Static Pushover Analysis

Static pushover analysis has given engineers the benefit of fast and reliable results compared to the non-linear dynamical analysis. The static pushover analysis assumes that the response of the structure (multi-degree-of-freedom-system) can be related to the response of an equivalent single-degree-of-freedom-system subjected to monotonically increasing lateral forces which represent the seismic inertia forces. As a result the examined system becomes very simple with reduced computation time while the obtained information is very important for the capacity of the structural system. A graph showing the relationship between the global base shear and the top lateral displacement $\delta_{\text{top}}$ is the most common outcome of the pushover analysis. However, the reliability of the method is reduced in cases where higher modes contribute to the response, and not only the fundamental one as the method assumes, and due to the fact that the changes in the building properties are not taken into account when the lateral forces are incremented.

![Graph showing the relationship between the global base shear and the top lateral displacement]

**FIGURE 15: CONSTRUCTION OF THE CAPACITY CURVE FROM THE PUSH-OVER ANALYSES (CHOPRA ET AL. (2002))**

There are two basic types of pushover analyses, the conventional and the adaptive pushover analysis, with their basic difference located in the pattern of the forcing function:

3.2.2.1 Conventional Pushover Analysis

Conventional pushover analysis is an inelastic static analysis method, characterized by the constant pattern of forcing function (loads or displacements) throughout the analysis. The pushover analysis commonly makes use of an incremental and iterative solution of the static equilibrium equations. Within those small increments, the behavior is assumed linear and equilibrium is described by the following equation:

$$K\Delta x = \Delta F$$

During each increment, the internal equilibrium conditions of the structure determine the resistance of a structure, while the tangent stiffness matrix $K_t$ is updated. Until at least one convergence criteria is fulfilled the out-of-balance forces are re-applied. At the point where convergence occurs, the tangent stiffness matrix is updated and the next increment of the load (force or displacement) is applied. This continuous procedure comes to its ultimate point when one of the following conditions occur: The target displacement is reached (associated with a performance level) or divergence occurs. The reliability and the effectiveness of the analysis depend on parameters such as the number of applied load steps, the iterative strategy and the convergence criteria. The steps needed to execute the conventional pushover analysis are as follows:

i. The gravity loads are applied in one step
ii. Apply a lateral load pattern (displacement or force)
iii. Select a displacement node for control
iv. Determine the vertical distribution of lateral forces $V_i$ or the vertical displacement distribution according to the load pattern that has been chosen (displacement or force respectively).
v. Calculate the incremental-iterative solution of the static equilibrium equations until the target performance level is reached

vi. For non-symmetric structures in the plan perpendicular to the applied loads, apply the chosen load pattern in both positive and negative direction

vii. Determine the storey shear $V_i$, the storey drift $\delta_i$, the base shear $V_{\text{base}}$ and top displacement $\delta_{\text{top}}$

viii. Plot the graphs base shear $V_{\text{base}}$ versus top displacement $\delta_{\text{top}}$ (pushover curves).

### 3.2.2.2 ADAPTIVE PUSHOVER ANALYSIS

The conventional pushover analysis proposes a time-invariant pattern of horizontal loads (forces or displacements) which may not totally agree with the inelastic response characteristics of the structure. Thus, other methods were developed to overcome this issue, such as the adaptive pushover analysis, in which the distribution of inertial forces can be changed, as described in Figure 16. The calculation of the increment size in lateral forces or displacements is based on the changing inelastic modal characteristics of the structure. The step sizes are determined by monitoring the total displacement against the total energy demand of the imposed forces or displacements at each step (Kalkan and Kunnath, 2007). Each step and the contribution of each mode is then incorporated in the final pushover curve.

![Figure 16: Changes of the distribution of inertial forces in a regular framed building (adaptive force distribution) (Antoniou (2002))](image)

The steps required to execute an adaptive pushover analysis are the following:

i. The gravity loads are applied in one step

ii. Perform an eigenvalue analysis for the current stiffness state of the structure starting from the elastic stiffness

iii. Determine the modal participation factors $\Gamma_j$ for the $j^{th}$ mode

iv. Compute the modal storey forces for all the stories, for the $N$ modes which satisfy mass participation of about 85-90% of the total mass, as follows:

$$F_{i,j} = \Gamma_j M_i \Phi_{i,j} g$$

Where: $M_i$: the seismic mass of story level $i$

$g$: the acceleration of gravity

$\Phi_{i,j}$: the modal matrix

v. Perform a static pushover analysis based on the forces calculated in iv. for each mode independently
vi. Estimate local or global forces and displacements using square root sum of the squares (SRSS) combinations of each modal quantity for the \(k^{th}\) step of analysis by adding it to the respective quantity of the \((k-1)^{th}\) step.

vii. Compare the values obtained from step vi. with the values corresponding to the target performance level.

viii. Repeat i.-vii. until the target is reached.

### 3.2.3 Applicability of Pushover Curves

Pushover analysis has become a key tool used in many methods which are developed to assess the seismic performance of a structure. Based on the Performance based Design, which is described in Chapter 3.1, seismic demand should be estimated for each limit state of interest. The accurate predictions of the actual seismic performance require the consideration of non-linear response of the structure in this estimation. Possible procedures which can be employed for this purpose, which are based on the pushover analysis are the displacement coefficient method (ASCE-41), the capacity spectrum method (ATC-40) and the N2 method of the Eurocode EC8.

#### 3.2.3.1 The Displacement Coefficient Method (ASCE-41)

When applying the displacement coefficient method, the nonlinear static procedure is implemented. The target displacement (displacement at the top of the structure during a specified seismic event) is given by the following formula:

\[
d_t = C_0 C_1 C_2 S_a(T_e) T_e^2 \frac{T_e^2}{4 \pi^4} g
\]

Where:
- \(d_t\): target displacement
- \(T_e\): effective fundamental period of the structure
- \(S_a(T_e)\): spectral acceleration at the effective period
- \(C_0\): modification factor that converts the spectral displacement of a single degree of freedom system to the multi-degree of freedom roof displacement (elastic)
- \(C_1\): modification factor used to estimate the maximum displacement on an elastic-perfectly plastic single degree of freedom system as a function of that experienced by the same single degree of freedom system with elastic linear behavior; the factor is calculated using the following formula:

\[
C_1 = \begin{cases} 
1.0 & \text{for } T_e \geq T_c \\
1.0 + (R-1) \frac{T_e^2}{R} & \text{for } T_e < T_c
\end{cases}
\]

Where \(T_c\) is a lower bound period of the constant velocity branch of the response spectrum and \(R\) is the strength reduction factor given as \(R = S_a / V_y W\) where \(S_a\) is defined above, \(V_y\) is the yield strength calculated by the nonlinear static procedure, \(W\) is the effective seismic weight and \(C_m\) the effective mass factor taken from...
<table>
<thead>
<tr>
<th>No. of Stories</th>
<th>Concrete Moment Frame</th>
<th>Concrete Shear wall</th>
<th>Concrete Pier-Spandrel</th>
<th>Steel Moment frame</th>
<th>Steel Concentric Braced Frame</th>
<th>Steel eccentric Braced frame</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>3 or more</td>
<td>0.9</td>
<td>0.8</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**TABLE 3: VALUES FOR EFFECTIVE MASS FACTOR $C_m$ ($C_m$ SHOULD BE TAKEN AS 1.0 IF THE FUNDAMENTAL PERIOD $T$ IS GREATER THAN 1.0 SEC).**

$C_2$: modification factor that takes into account the pinched hysteric shape, stiffness degradation, and strength deterioration

$C_3$: amplification factor that takes into account the dynamic P-Delta effects, which depend on the post-peak response of the single degree of freedom model.

### 3.2.3.2 CAPACITY SPECTRUM METHOD

The theoretical approach of the capacity spectrum method is to compare the capacity of a structure with the demands of earthquake ground motion on it, by means of a graphical procedure. The clarity of the comparison between the two graphs makes the method easy to use. The two curves describe the acceleration for various values of the displacement of the mass in a single degree of freedom system. The capacity spectrum is obtained after executing non-linear static pushover analyses, which result in a base-shear-force – roof-displacement curve, after computing the equivalent bilinear capacity curve. A transformation of the obtained graph to spectral accelerations and spectral displacements will provide the capacity spectrum, something achieved after using the single degree of freedom system. The demand spectrum, on the other hand, is obtained by highly damped elastic spectra, in an Acceleration-Displacement Response Spectrum (ADRS) format, where the radial lines represent the periods $T$ of the structures. The intersection of the two graphs is an indication of the strength and displacement of the structure.

![Capacity Spectrum Method](image)

**FIGURE 17: CAPACITY SPECTRUM METHOD (FAJAR (1999))**

However, two arguments make the foundation assumptions of the method questionable: there is no evidence that there is a stable relationship between the hysteretic energy dissipation of the maximum excursion and equivalent viscous damping and the period corresponding to the intersection of the two diagrams may not consort with the dynamic response of the inelastic system. The capacity curve obtained by the pushover analysis corresponds to monotonically increasing loading. Thus, the ultimate capacity should be reduced in order to be used for seismic design, due to cumulative damage effects and there comes the concept of equivalent ductility factors.
The seismic demand is fully described by the inelastic spectra, which follows form the ground motion equations.

3.2.3.2.1 ATC-40 Method

ATC-40 method is a variation of the Capacity Spectrum method which is analyzed in 3.2.3.2 and is based on the comparison of the capacity of a structure to resist lateral forces to the demand given by a response spectrum. The response spectrum represents the demand while the capacity curve represents the available capacity. The procedure used in the method is as follows:

i. Perform pushover analysis and create the capacity curve (base shear $V_b$ – roof displacement $D$)
ii. Use the equivalent single degree of freedom system (ESDOF) to convert the aforementioned diagram to acceleration-displacement terms. The modal mass ($A=V_b/M$) and the first mode participation factor $C_0 (D^*=D/C_0)$ are used.
iii. Plot on the same graph the capacity spectrum and the elastic response spectrum (5% damping coefficient)
iv. Choose a peak deformation demand $d^*$ and determine the corresponding pseudo-acceleration $A$ from the capacity spectrum assuming that $\xi=5\%$
v. Calculate the ductility $\mu=D^*/u_y$ and the hysteric damping $\xi_h=2(\mu-1)/\pi\mu$. The equivalent damping ratio is evaluated by the form: $\xi_{eq}=\xi_{el}+\kappa \xi_h$, where $\kappa$ is a damping modification factor that depends on the hysteric behavior of the system.
vi. Update the chosen $d^*$ using the elastic demand diagram for $\xi_{eq}$

vii. Check for convergence the displacement $d^*$ and when convergence is achieved the target displacement of the MDOF system is equal to $d_t=C_0 d^*_t$. 
FIGURE 18: CAPACITY SPECTRUM METHOD (MWAFY, 2001)
3.2.3.3 **N2-Method**

A variation of the Capacity Spectrum method is described by the N2-Method, which is implemented in the Eurocode 8. The method consists of the following steps:

i. Perform pushover analysis and create the capacity curve (base shear $V_b$ – roof displacement $D$)

ii. Use the equivalent single degree of freedom system (ESDOF) to convert the aforementioned diagram to acceleration-displacement terms. Approximate the capacity curve with an idealized elasto-perfectly plastic relationship to get the period $T_e$ of the ESDOF. In order to derive the bilinear curve, two rules should be followed:
   a. Elastic portion of the bilinear curve crosses the pushover curve at 60% of the proposed yield stress
   b. Total energy demand is the same for both curves ($\rightarrow$ equal areas under the curves) as in Figure 19.

![Figure 19: Bilinearisation of the capacity curve, N2 Method](image)

iii. Calculate the target displacement as: $d_e^{**} = S_a(T_e) \left[ \frac{T_e}{2\pi} \right]^2$, where $S_a(T_e)$ is the elastic acceleration response spectrum for the period $T_e$. Depending on the period, different expressions are suggested for the derivation of the target displacement $d_e^{**}$.
   a. Short period range ($T_e^*<T_c^*$): When $F_{y}^{*/m^*}S_a(T_e)$, the response is elastic and thus $d_e^{**} = d_m^{*}$ and $d_e = C_0d_e^{**}$. When the response is nonlinear and the ESDOF maximum displacement is given by the formula: $d_e^{**} = \frac{d_e^{**}}{q_u} \left( 1 + (q_u - 1) \frac{T_c}{T_e} \right) \geq d_e^{**}$, where $q_u = S_a(T_e) m^*/F_{y}^{*}$
   b. Medium and long period range ($T_e^* \geq T_c^*$): The target displacement of the inelastic system is equal to that of the elastic system and thus $d_e^{**} = d_m^{**}$, while $d_e = C_0d_e^{**}$ for the MDOF system.

In the present case study the Conventional pushover analysis will be used. The small computational time that this analysis needs (compared to other analyses) as well as the simplicity of the method together with the high level of reliability and effectiveness that it provides depending on parameters that can be critically chosen (such as the number of applied load steps, the iterative strategy and the convergence criteria) make the method ideal for the needs of this study, which requires small computational time and simplicity in getting the results, since many analyses for different cases will be executed. If a more accurate analysis was required, the adaptive pushover analysis would be good to apply as well as the incremental dynamical analysis which would give the most detailed and accurate response of the structure.
4  **CALCULATION OF THE INELASTIC DISPLACEMENT DEMAND**

During the assessment of the performance of a structure, the energy dissipation observed during the analysis is only modeled by considering hysteretic response of the structural components. Thus, equivalent viscous damping should be assigned to the model which is analyzed which should represent the energy dissipation that is not otherwise captured in the analysis model. The use of effective damping and period becomes necessary because of the non-detailed modelling of the structure, since energy dissipation can be observed in the structure because of components that are modeled as elastic but where limited cracking or yielding occurs; architectural cladding, partitions and finishes, that are ignored in the model; and foundations and the soil (if the soil-structure interaction effects are not modeled). Where energy dissipation components (dampers) are installed in a structural system, these components should be modeled explicitly using appropriate component models, including velocity terms to capture viscous effects and/or nonlinear springs to capture hysteretic effects.

In addition, during the calculation of the specific ground motion demand, the elastic acceleration response spectra is used, which corresponds to a damping level of 5% of the structure. Thus, in performance based seismic design and evaluation it is important to determine the maximum inelastic displacement response of nonlinear structures. The two basic approximate procedures used for that purpose are a) the displacement modification factor and b) the equivalent linearization. In the first method the maximum deformation of a nonlinear structure is determined as the product of a modification factor and its maximum linear elastic deformation, whereas in the second method as the maximum deformation of an equivalent linear structure with increased viscous damping and longer period of vibration (more flexible lateral stiffness). Applications of the aforementioned methods can be found in the displacement coefficient method (FEMA-356) and Capacity Spectrum Method (ATC-40) respectively.

The traditional Equivalent Linear System method is based on the transformation of a nonlinear system to a linear elastic equivalent system with different period and viscous damping, as described by the equations of motion of the two systems under earthquake excitations:

\[\ddot{x} + \frac{4\pi \xi_0}{T_0} \dot{x} + \left(\frac{2\pi}{T_0}\right)^2 f(x) = -\ddot{x}_g \]  
\text{nonlinear system}

\[\ddot{x} + \frac{4\pi \xi_{eq}}{T_{eq}} \dot{x} + \left(\frac{2\pi}{T_{eq}}\right)^2 = -\ddot{x}_g \]  
\text{equivalent linear elastic system}

Where: \(x\): lateral displacement of the mass relative to the ground

\(\ddot{x}_g\): ground acceleration

\(\xi_0\): inherent damping

\(T_0\): period of vibration of the system corresponding to initial stiffness \(k_0\)

\(f(x)\): restoring force of the system

\(T_{eq}\): equivalent period of vibration (usually \(T_{eq} > T_0\))

\(\xi_{eq}\): equivalent viscous damping (\(\xi_{eq} > \xi_0\))

The derivation of the equivalent viscous damping and period has been thoroughly examined by many researchers based on either harmonic excitations or random responses and depend on the ductility ratio (\(\mu=\text{maximum displacement/yield displacement}\)) (Jacobsen 1930, 1960; Rosenblueth and Herrera 1964; Jennings 1968; Takeda et al. 1970; Gulkan and Sozen 1974; Shibata and Sozen 1976; Iwan 1980; Hadjian 1982; Kowalsky 1994; Iwan and Guyader 2002; FEMA 2005).

The latest improved equivalent linear system is presented in the capacity spectrum method (ATC-40) (1996), Iwan and Guyades (2002) and FEMA (2005) in which the optimal set of the equivalent period and damping are
obtained by simultaneously minimizing the mean and standard deviation of the displacement error between the actual nonlinear system and its equivalent linear counterpart. For systems with elastoplastic hysteric behavior, the following empirical formulas are proposed, depending on the ductility factor \( \mu \).

For \( \mu < 4.0 \):

\[
\frac{T_{eq}}{T_0} = 1 + 0.111(\mu - 1)^2 - 0.0167(\mu - 1)^3
\]

\[
\xi_{eq} = \xi_0 + 0.0319(\mu - 1)^2 - 0.0066(\mu - 1)^3
\]

For \( 4.0 \leq \mu \leq 6.5 \):

\[
\frac{T_{eq}}{T_0} = 1.279 + 0.0892(\mu - 1)
\]

\[
\xi_{eq} = \xi_0 + 0.106 + 0.00116(\mu - 1)
\]

For \( \mu > 6.5 \):

\[
\frac{T_{eq}}{T_0} = 1 + 0.57(\sqrt{\mu - 1} - 1)
\]

\[
\xi_{eq} = \xi_0 + 0.19 \left[ \frac{0.73(\mu - 1) - 1}{0.73(\mu - 1)^2} \right] \left( \frac{T_{eq}}{T_0} \right)^2
\]

In the present thesis, a variation of the Equivalent Linear System method proposed by Lin, Miranda (2008) is going to be used and is explained further in 4.1. In the used method, the equivalent period and damping are calculated based on the eigenperiod of the structure, the strength ratio and the postyield stiffness with a straightforward way instead of the ductility ratio which is unknown for existing buildings and requires an iterative way.

4.1 LIN & MIRANDA METHOD

As described in paragraph 4, the equivalent period and damping are functions of the ductility factor of the system. However, in cases of evaluation, retrofit, rehabilitation and upgrading of existing structures, the ductility ratio is unknown and as a result the calculation of the maximum inelastic displacement response requires an iterative procedure until the computed displacement is within an allowable tolerance to the assumed displacement. On the other hand, the strength ratio of existing structures (\( R = \) elastic lateral strength/yield lateral strength) is known.

Non-iterative procedures have been suggested so far, by expressing the ductility ratio (\( \mu \)) as a function of the strength ratio (\( R \)) by the \( R-\mu-T \) relations, the so-called indirect method (Chopra and Goel 1999; Newmark and Hall 1982; Miranda and Bertelo 1994). The direct method on the other hand is based on the direct derivation of the equivalent linear system defined by \( R \) from statistical analysis of them. The difference in the results between the two methods is caused by the conceptual differences between them, with indirect method being a first-order approximation of the direct method. To avoid statistical bias in the results, the use of the direct method is preferable.

Moreover, from the equations (1)-(5) it is obvious that the equivalent periods and damping coefficients are independent of the period of the structure. Thus, a certain ductility factor will lead to the same changes in the period and damping regardless of the structure being stiff or soft. According to Miranda Ruiz-Garcia (2002) and Akkar and Miranda (2005), disregarding the influence of the period will significantly overestimate the maximum inelastic displacement for short period systems.

In order to eliminate the aforementioned problems of the equivalent linear system method, Lin and Miranda (2008) have presented a non-iterative procedure which takes into account the strength ratio and the period of vibration of the structure. The suggested method not only avoids iterations and improves accuracy but also gives good predictions of the mean maximum inelastic displacement of structures for systems with all period ranges (long, intermediate and short ones).
The strength ratio is given by the formula

\[ R = \frac{m S_a}{f_y C_m} \]  

(7)

Where:  
\( m \): mass of the system  
\( S_a \): pseudoacceleration spectral ordinate  
\( f_y \): lateral yield strength of systems  
\( C_m \): modal mass coefficient of systems. For SDOF systems \( C_m = 1 \)

The required values of period and damping can be derived by the following formulas:

\[ \frac{T_{eq}}{T_0} = 1 + \frac{m_1}{m_2} \left( R^{1.8} - 1 \right) \]  

(8)

\[ \xi_{eq} = \xi_0 + \frac{n_1}{T_0} \left( R - 1 \right) \]  

(9)

Where: \( m_1, m_2, n_1 \) and \( n_2 \) are constants dependent on the postyield stiffness ratio as listed in Table.

**TABLE 4: COEFFICIENTS FOR PROPOSED EQUIVALENT PERIOD AND DAMPING**

<table>
<thead>
<tr>
<th>( \alpha ) (%)</th>
<th>( m_1 )</th>
<th>( m_2 )</th>
<th>( n_1 )</th>
<th>( n_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.026</td>
<td>0.87</td>
<td>0.016</td>
<td>0.84</td>
</tr>
<tr>
<td>5</td>
<td>0.027</td>
<td>0.65</td>
<td>0.027</td>
<td>0.55</td>
</tr>
<tr>
<td>10</td>
<td>0.027</td>
<td>0.51</td>
<td>0.031</td>
<td>0.39</td>
</tr>
<tr>
<td>20</td>
<td>0.024</td>
<td>0.36</td>
<td>0.030</td>
<td>0.24</td>
</tr>
</tbody>
</table>

The accuracy of the proposed method is examined by the following formula:

\[ \overline{E}(T_0, R, \alpha) = \frac{1}{n} \sum_{k=1}^{n} \frac{\Delta_{ap}(T_{eq}, \xi_{eq})_k}{\Delta_{ex}(T_0, \xi_0, R, \alpha)_k} \]

Where: \( \overline{E}(T_0, R, \alpha) \): mean ratio of approximate \( \Delta_{ap}(T_{eq}, \xi_{eq}) \) to exact \( \Delta_{ex}(T_0, \xi_0, R, \alpha) \) maximum inelastic displacements for systems with a given period of vibration \( (T_0) \), a given strength ratio * and a given postyield stiffness ratio \( (\alpha) \).

The best estimation of the system is achieved when \( \overline{E}(T_0, R, \alpha) \) approaches 1, with underestimations when \( \overline{E}(T_0, R, \alpha) < 1 \) and overestimations when \( \overline{E}(T_0, R, \alpha) > 1 \).

An application of the method for 72 earthquake ground motions shows that all mean approximate to exact displacement ratios fall around 1.0 irrespective of the strength ratios, periods or postyield stiffness ratios, which means that the prediction of the maximum inelastic displacement is very good for all the different cases of structures (Lin & Miranda (2008)) Thus, the method can be securely applied in the present case study where the periods of the structures are relatively low (around 0.8 sec).

**4.1.1 APPLICATION OF LIN & MIRANDA METHOD**

The proposed method can be used for the evaluation of existing buildings with the following procedure:

1. Execute pushover analysis and derive the pushover curve (base shear-roof displacement)
2. Bilinearise the obtained pushover curve and derive the yield strength \( f_y \), the postyield stiffness \( \alpha \) and the strength ratio \( R \) (eq. (7)).
3. Compute the equivalent period $T_{eq}$ and damping $\xi_{eq}$ using the equations (8) and (9).

4. Construct the $\xi_{eq}$-damped elastic displacement spectrum $S_d(T, \xi_{eq})$ which can be obtained from the 5%-damped elastic design spectrum $S_a(T, \xi=5\%)$ and the damping modification factor $B$, which is calculated from Table 15.6-1 of NEHRP-2003 and is given in .

5. The maximum displacement demand ($u_j$) for the $j^{th}$ mode is the displacement corresponding to the period $T_{eq}$ in the $\xi_{eq}$-damped elastic displacement spectrum:

$$u_j = S_d(T_{eq}, \xi_{eq}) = \frac{T_{eq}^2 S_a(T_{eq}, \xi = 5\%)}{4\pi^2 B}$$

Where: $S_a(T_{eq}, \xi = 5\%)$: ordinate of the 5%-damped elastic design spectrum at the period $T_{eq}$.

6. Convert the displacement obtained from step 5 to global (roof) displacement ($u$) based on the square-root-of-the-sum-of-the-square rule as

$$u = \sqrt{\sum_{j=1}^{N} (u_j \cdot \Gamma_j)^2}$$

Where: $\Gamma_j$: modal participation factor for the $j^{th}$ mode

$N$: total number of modes considered

<table>
<thead>
<tr>
<th>Effective Damping $\beta$ (percentage of critical)</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 2$</td>
<td>0.8</td>
</tr>
<tr>
<td>5</td>
<td>1.0</td>
</tr>
<tr>
<td>10</td>
<td>1.2</td>
</tr>
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<td>20</td>
<td>1.5</td>
</tr>
<tr>
<td>30</td>
<td>1.8</td>
</tr>
<tr>
<td>40</td>
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<td>50</td>
<td>2.4</td>
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<tr>
<td>60</td>
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</tr>
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<td>80</td>
<td>3.3</td>
</tr>
<tr>
<td>90</td>
<td>3.6</td>
</tr>
<tr>
<td>$\leq 100$</td>
<td>4.0</td>
</tr>
</tbody>
</table>
5 SEISMIC HAZARD ANALYSIS _ STATE OF ART

In the present chapter, the most important methods of Seismic Hazard Analysis are described as well as a more detailed description is made of the method used in this Master thesis, which is based on Jayaram et al (2011).

5.1 INTRODUCTION

Seismic hazard is the likelihood, or probability of experiencing a specified intensity of any damaging phenomenon at a particular site, or in a region, in some period of interest (Thenhaus and Campbell, 2003). In order to design new seismic resistance structures, or examine the reliability of existing structures, it becomes necessary to have the knowledge of the likely earthquake actions at a subject site.

Earthquake actions result from ground motion in horizontal and/or vertical direction. The importance of such events can be expressed as the effect that they would have on structures and depends on the seismic magnitude $M_w$, the location and depth of the event, the seismic mechanism (Style of Fault) and the geological conditions between the examined site and the event $V_{S30}$, which is the shear-wave velocity, averaged over the top 30m of soil.

A common way of examining seismic action is by the acceleration response curve, which expresses the peak horizontal acceleration, either measured or expected, of a single degree of freedom system as a function of the structural period. A typical transformation of this curve is to compute the spectral displacements for each structural period (ADRS format) through pseudo-spectral relationships:

$$S_a = S_a\left(\frac{T}{2\pi}\right)^2$$

An example of acceleration response spectra is given in Figure 20.

![FIGURE 20: TYPICAL ACCELERATION RESPONSE SPECTRA (WEBSITE)](image)

Seismic hazard studies are conducted with several different methods. Two fundamental types exist: the deterministic and the probabilistic approach. Following, these two methods are going to be presented as well as the method proposed by Eurocode 8.

5.2 DETERMINISTIC APPROACH

In deterministic analyses the seismic hazard assessment is based on the data from past seismic events that had the maximum impact on the site. The difficulty in the method lies on the definition of the one maximum earthquake that is representative and should be considered in each case. Thus, only the closest source-to-site distance is examined combined with high values of magnitude. The application of the severest scenario makes the method a conservative seismic assessment.
5.3 Probabilistic Approach

The probabilistic seismic hazard analysis, on the other hand, considers all the possible seismic sources that could affect the examined site. All the local geological and seismological data for the full range of ground motions that affect the subject site are taken into account. First of all, the examined area is divided into seismic source zones, each defined by its observed seismicity, its geophysical, geological and tectonic characteristics and its exact location within a larger tectonic framework.

In this method, the magnitude of each earthquake on each source, the location of the earthquake in or along each source, and the prediction of the response parameter of interest are given probability distributions \( f_m(m) \), \( f_r(r) \) and \( P(\text{pga} > \text{pga}' | m, r) \) respectively. The procedure is described by Kramer in four steps and is depicted in Figure 21.

1. Identify and characterize (geometry and potential \( M_W \)) all earthquake sources which are capable of causing significant shaking (say \( M_W \geq 4.5 \)) at the site. Develop the probability distribution of rupture locations within each source. Combine this distribution with the source geometry to obtain the probability distribution of source-to-site distance for each source.
2. Develop a distribution of earthquake occurrence for each source using a recurrence relationship. This distribution can be random or time-dependent.
3. Using predictive (attenuation) relationships, determine the ground motion produced at the building site (including the uncertainty) by earthquakes of any possible size or magnitude occurring at any possible point in each source zone.
4. Combine the uncertainties in earthquake location, size, and ground motion prediction to obtain the probability that the chosen ground motion parameter (e.g., peak horizontal ground acceleration, spectral acceleration at a specified frequency) will be exceeded in a particular time period (say 10% in 50 years).

Through the aforementioned procedure, the calculation of annual rates of exceedance of ground motion parameters (e.g., spectral acceleration at a selected period) at a particular site are possible, based on aggregating the risks from all possible source zones. Thus, the calculated rate of exceedance for a given intensity value is not associated with a specific earthquake magnitude or distance, but the combinations of magnitude, distance and source that contribute most to particular values of an intensity are calculated. This process is termed de-
aggregation (or disaggregation). Hazard deaggregation requires expression of the mean annual rate of exceedance as a function of magnitude and distance of the form:

\[
\lambda_y(m_j, r_k) = P[M = m_j]P[R = r_k] \sum_{i=1}^{N_i} v_i P[Y > y|m_j, r_k]
\]


Another important ground motion variable is epsilon, \(\varepsilon\), which is defined as:

\[
\varepsilon = S_a - \theta / \beta
\]

where \(S_a\) is the computed spectral acceleration for a given probability (e.g., 2%) of exceedance in a specified time period (e.g., 50 years) and equal to 0.829g in this instance at a period of 1 second;

\(\theta\) is the median value of spectral acceleration computed by an appropriate attenuation relationship(s) for the dominant \([M, r]\) pair

\(\beta\) is the dispersion in the attenuation relationship.

All these variables vary as a function of period.

\(\varepsilon\) is an indicator of spectral shape, with a positive (negative) value of \(\varepsilon\) at a given period tending to indicate a relative peak (valley) in the acceleration response spectrum at that period (Baker and Cornell (2005, 2006)).

5.4 EUROCODE 8 RECOMMENDATIONS

According to the Eurocode, for special engineering structures, such as gas storage tanks and nuclear power plants, seismic hazard should considered where an exceedance of peak ground acceleration (PGA) is expected of 0.1g (g= 9.81 m/s^2).

For specific ranges of the structural period, the acceleration response spectra can be computed using the relations (1)-(4). The different parts of the spectra represent an area with a) constant acceleration, b) constant velocity and c) constant velocity, as shown in Figure 22.

The parametric functions for the horizontal acceleration of an elastic single degree of freedom system are as follows:

\[
0 \leq T \leq T_B \quad S_e(T) = a_g \cdot S \cdot \left[1 + T/T_B \left(\eta \cdot 2.5 - 1\right)\right] \quad (1)
\]

\[
T_B \leq T \leq T_C \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \quad (2)
\]

\[
T_C \leq T \leq T_D \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[T_D/T\right] \quad (3)
\]

\[
T_D \leq T \leq 4 \text{ sec} \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[T_D/T\right] \quad (4)
\]

\[
a_g = a_{gr} \cdot \gamma_1 ; \eta = \sqrt{10/(5 + \zeta)} \geq 0.55
\]

Where:

\(\rightarrow S_e\): elastic spectral acceleration
T: Vibration period of a linear single degree of freedom system
a_g, a_{gr}: Design ground acceleration and reference ground acceleration on rock
γ: Importance factor for the structure
S: Soil factor
ζ: Viscous damping ratio
T_B, T_C, T_D: Definition of ranges of the structural period to define the start values for the constant acceleration response (T_B), constant velocity response (T_C), and for constant displacement response (T_D).

![Figure 22: Elastic Acceleration Spectra According to EC8](image)

The values of the parameters S, T_B, T_C, T_D and the importance factors are given in the Eurocode.

According to the Eurocode, two types of spectra should be considered for each seismic analysis. Type I is representative of larger and more distant seismic actions (high values of R and M and thus high PGA), whilst Type II is representative of lower magnitude events closer to the subject site. In case where the earthquakes that contribute most to the seismic hazard have a magnitude less than M=5.5, only Type II needs to be examined.

### 5.5 Conditional Mean Spectra

Probabilistic Seismic Hazard Assessment was introduced in Chapter 5.3. A type of spectra commonly used is the Uniform Hazard Spectra, the ordinates of which are associated with the same annual frequency of exceedance.

The Uniform hazard spectrum is used in building codes and standard seismic design provisions as the target spectra to match with the scaled ground motions over a wide period of range. However, despite the fact that application of the described method is convenient and leads to conservative results, two important facts are disregarded:

1. In case that the spectral ordinates of a uniform hazard spectrum are governed by multiple scenario events, irrespective of the return period, for any of the governing events the spectral shape will not be representative (Cornell 2006, Baker and Cornell, 2006).
2. When long-return-period earthquakes are examined, the spectral ordinates of Uniform Hazard spectra are typically associated with a high value of ε (greater than 1) across a wide range of period (Harmsen, 2001). When for a given period the geomean spectral ordinate of a ground motion pair matches the uniform
hazard spectrum ordinate, the geomean spectrum of the pair is unlikely to have ordinates as large as those of the uniform hazard spectrum at other periods.

In order to overcome these problems, the concept of Conditional Mean Spectrum was introduced by Baker and Cornell (2005, 2006). In this new approach, the correlation of spectral demands (represented by values of $\varepsilon$) at different periods is applied. Conditional Mean Spectra estimate the median geomean spectral acceleration response of a pair of ground motions given an [M, r] pair and a target spectral ordinate, $S_a(T_1)$ (from which $\varepsilon_{T_1}$ is back-calculated using an appropriate attenuation relationship). The following equation was used by Baker and Cornell (2005) in order to generate conditional mean spectra:

$$CMS_{S_a(T_1)}(T_2) = \theta(M, r, T_2) \cdot \exp[\beta(M, r, T_2) \cdot \rho_{\varepsilon(T_1), \varepsilon(T_2)} \cdot \varepsilon_{T_1}]$$

Where: $CMS_{S_a(T_1)}(T_2)$: ordinate of the spectrum at period $T_2$ given that the spectral demand at $T_1$ is $S_a(T_1)$;

$\theta(M, r, T_2)$ and $\beta(M, r, T_2)$: the median and logarithmic standard deviation of spectral acceleration at $T_2$ computed using a ground motion attenuation relationship for the [M, r] pair of interest;

$\varepsilon_{T_1}$: the value of $\varepsilon$ associated with $S_a(T_1)$;

$\rho_{\varepsilon(T_1), \varepsilon(T_2)}$: the correlation coefficient for $\varepsilon$ between $T_1$ and $T_2$.

The procedure in order to construct Conditional mean spectra is described following:

Step 1. Determine the value of $S_a(T_1)$ from the probabilistic seismic hazard analysis at the desired mean annual frequency of exceedance.

Step 2. De-aggregate the hazard at $S_a(T_1)$ and identify the modal [M, r, $\varepsilon$] triple.

Step 3. Select an appropriate ground motion attenuation relationship.

Step 4. Generate the spectrum using (E-15) through (E-20) and the selected attenuation relationship for the [M, r, $\varepsilon$] triple of Step 2.

Step 5. Amplitude scale the spectrum to recover $S_a(T_1)$

Due to the advantages that the Conditional Mean spectrum has over the Uniform hazard spectrum, it will be used in the present study, following Jayaram’s et al. method described in 5.6:

5.6 JAYARAM ET AL METHOD_ A COMPUTATIONALLY EFFICIENT GROUND-MOTION SELECTION ALGORITHM

The response of a structure under seismic loading is usually predicted by dynamic structural analyses in performance-based earthquake engineering. An important aspect when executing a dynamic structural analysis is the appropriate selection of input ground motions with a target response spectrum. For that reason an algorithm which is universally approved both in a theoretical and in a computational basis is needed. Such an algorithm, that uses Monte Carlo simulation to probabilistically generate multiple response spectra from a distribution parameterized by the target means and variances, is proposed by Jayaram et al. (2011) and is used in the present study. The method makes use of the theory suggested in Akkar et al (2014a) in order to obtain the unconditional mean and standard deviation (the ground-motion prediction equations have been used) and following the theory in Akkar et al. (2014b) in order to obtain the correlation between $\varepsilon(T)$ and $\varepsilon(T^*)$.

According to Akkar et al. (2014a) the ground motion equations are given by following model:
\[
\ln(Y) = \ln[Y_{REF}(M_w, R, SoF)] + \ln[S(V_{S30}, PGA_{REF})] + \varepsilon
\]

Where:
\[
\ln(Y_{REF}) = \begin{cases} 
  a_1 + a_2(M_w - c_1) + a_3(8.5 - M_w)^2 + [a_4 + a_5(M_w - c_1)]\ln(\sqrt{R^2 + a_6^2} + a_7F_N + a_8F_R + S) & \text{for } M_w \leq c_1 \\
  a_1 + a_2(M_w - c_1) + a_3(8.5 - M_w)^2 + [a_4 + a_5(M_w - c_1)]\ln(\sqrt{R^2 + a_6^2} + a_7F_N + a_8F_R + S) & \text{for } M_w > c_1
\end{cases}
\]

And
\[
\ln(S) = \begin{cases} 
  b_1 \ln\left(\frac{V_{S30}}{V_{REF}}\right) + b_2\ln\left[\frac{PGA_{REF} + c(V_{S30}/V_{REF})^n}{(PGA_{REF} + c)(V_{S30}/V_{REF})^n}\right] & \text{for } V_{S30} \leq V_{REF} \\
  b_1\ln\left(\min(V_{S30}, V_{CON})\right) & \text{for } V_{S30} > V_{REF}
\end{cases}
\]

In the aforementioned formulas \(\ln Y\) is the median spectral acceleration, which is calculated after modification of the reference ground motion model \(\ln(Y_{REF})\) through a nonlinear site amplification function \(\ln(S)\). The values of constants \(a_1\) to \(a_9\) are period dependent and are given in the appendix and \(V_{CON} = 1,000\) m/s.

Jayaram’s method is described further in the following steps, where correlated ordinates of spectral acceleration (Sa) are randomly derived, conditional on a given level of PGA:

1. Parameterize the multivariate normal distribution of \(\ln S_a\)’s at multiple periods (means, variances and the covariances between the \(\ln S_a\)’s at all pairs of periods). These parameters should be set to their target values
2. Monte Carlo simulations to generate response spectra from the above multivariate normal distribution
3. For each simulated response spectrum, a ground motion with similar response spectrum is then selected
4. Evaluation of the similarity between the two spectra: \(SSE = \sum_{j=1}^{p}(\ln S_a(T_j) - \ln S_a^{(s)}(T_j))^2\)
5. Greedy optimization technique: further improves the match between the target and the sample mean and variances. The deviation of the set from the target parameters is estimated from: \(SSE_s = \sum_{j=1}^{p} \left[ (\ln S_a(T_j) - \mu^{(s)}_{\ln S_a(T_j)} - \sigma^{(s)}_{\ln S_a(T_j)})^2 \right] \)
6. Replacement of one ground motion at a time

An application of the proposed algorithm to estimate the seismic response of a single-degree-of-freedom and a multiple-degree-of-freedom sample, revealed that considering the response spectrum variance does not significantly affect the median response but increases the dispersion of the response in both cases, and slightly increases the mean response in the case of the single-degree-of-freedom structure.

After following the procedure described above, the result is the derivation of the conditional mean spectrum and the correlated mean spectra. The specific procedure used in the present study is explained in Chapter 10 and in Appendix A.
6 FRAGILITY CURVES

This chapter describes a general approach toward the seismic assessment of structures. Different methods and procedures are presented based on the unique site, structural, non-structural and occupancy characteristics of the examined structure. The methodology and procedures are applicable to performance-based design of new buildings and performance assessment and seismic upgrade of existing buildings.

6.1 INTRODUCTION

The frequent existence of earthquakes in several areas of the world has smaller or bigger impact on the people’s lives, which varies from simple repairable damages to huge economic losses and human casualties. The amount and significance of such consequences depends not only on the severity of the earthquake but also on the seismic behavior of the structures. Thus, the evaluation of seismic vulnerability of existing buildings becomes crucial in order to diminish such unfavorable events.

Most casualties (deaths and serious injuries) that arise in buildings during earthquakes occur as a result of partial or total building collapse. Therefore, in order to assess potential casualties it is necessary to define the probability of incurring structural collapse, as a function of ground motion intensity, and the modes of structural collapse (e.g. single story, multi-story) that can occur. These are represented in the form of collapse fragility functions, which describe the possibility of exceeding different limit states (such as damage levels) given a level of ground shaking. Fragility functions will be analyzed further in the following paragraphs.

6.1.1 FRAGILITY FUNCTION DEFINITION

Fragility functions are probability distributions that indicate the probability that a chosen characteristic part of the structure (e.g. element or system) will exceed a chosen damage state as a function of a single predictive demand parameter such as story drift or floor acceleration. It may be used to set fragility functions for either structural or non-structural components, elements or systems.

The probability of failure is associated to a predefined limit state can be defined as

$$P_f(PGA) = \int_0^\infty [1 - F_{DPGA}(a)]f_c(a)da$$

Where $F_{DPGA}$ is the cumulative density function (cdf) of the demand for a given level of PGA and $f_c$ is the probability density function (pdf) of the capacity, i.e. of the damage state under consideration.

The probability calculated by the comparison between the capacity of the structure and the demand of the ground motion can be represented by a graph, as the one presented in Figure 23.
6.2 TYPES OF PERFORMANCE ASSESSMENT

Different methodologies can be used for deriving fragility functions:

- Empirical (based on observed data)
- Expert opinion based
- Analytical (based on analytical models)
- Hybrid (combination of analytical and empirical).

Characteristic parameter of a fragility function is the Intensity Measure Type (IMT) which is a ground motion parameter (observational: MSC, MMI, MSK81, EMS98, and instrumental: PGA, PGV, RMS, $S_a(T)$, $S_d(T)$, Roof Drift Ratio) against which the probability of exceedance of a given limit state is plotted.

Three types of performance assessment are commonly used: intensity-based, scenario-based and time-based.

6.2.1 INTENSITY-BASED ASSESSMENTS

In this type of assessment, the structure’s performance is assessed by examining the response of the structure to a certain ground motion. The shaking intensity is defined through the elastic acceleration response spectra (5% damped). In this assessment type, any response spectrum can be used to assess a building’s performance.

6.2.2 SCENARIO-BASED ASSESSMENTS

In the scenario-based assessments, a specific case is chosen for examination, where the magnitude of the earthquake ($M_w$) and the exact location relative to the building site (R) are chosen. Scenario assessments can be advantageous in case decision-making is needed, since it can provide valuable information about the performance of the structure under a chosen (maybe real-historic) event. The difference between scenario-based and intensity-based assessments lies in the uncertainty in the earthquake intensity, given the scenario that is considered. The results of scenario-based assessments are performance functions with the probable performance conditioned on the occurrence of the scenario earthquake, rather than a specific intensity of shaking.
6.2.3 TIME-BASED ASSESSMENTS
In time-based performance assessments a structure’s performance is examined over a period of time, e.g. 1-year, 30-years, 50-years, considering all the possible earthquakes to occur in that period of time, and the probability that each will occur. The interests and needs of the individual decision-makers are the main objectives over which the period of time that a time-based assessment is performed is chosen. The uncertainty of the method can be found not only in the magnitude and location of future earthquakes but also in the intensity of motion resulting from these earthquakes.

6.3 DEFINITION OF DAMAGE STATES
After the seismic assessment of a structure is performed, the use of fragility curves can be more convenient when deriving analytical fragility curves. In order to do so, the definition of the damage states in terms of a mechanical parameter that can be directly obtained from the analysis is required. Masonry buildings are often examined under three different limit states:

a) Elastic (cracking) limit, where the wall displacement exceeds the elastic limit and the first significant crack forms in the wall, which results in changes in the initial stiffness.

b) Maximum resistance, determined by the shear and displacement corresponding to the point where the building reaches its maximum resistance.

c) Ultimate limit state (near collapse limit state) where the building resistance deteriorates below an acceptable limit eg 80% of the maximum resistance.

However, a real masonry building is very unlikely to collapse at the so-defined ultimate state. Although the actual collapse takes place at a lateral displacement larger than that corresponding to ultimate state, at this latter displacement the structure is already damaged beyond repair.

Other common damage states that are used for masonry structures are connected with the interstorey drift capacity, and are identified through results from experimental tests. Each limit state condition is expressed through a probabilistic distribution, something necessary when using a probabilistic assessment procedure.

a) For light damage limit state (LS1), a normal distribution is considered with an average drift of 0.13% and a coefficient of variation of 35%.

b) For significant damage limit state (LS2), a normal distribution is considered with an average drift of 0.34% and a coefficient of variation of 30%.

c) For the collapse limit state condition (LS3) determining a unique drift limit would result in great uncertainty depending on the exact material properties of the structure:

- Brick masonry with low percentage of voids (<55%): normal distribution with an average drift of 0.72% and a coefficient of variation of 35%
- Brick masonry with high percentage of voids (>55%): normal distribution with an average drift of 0.45% and a coefficient of variation of 30%
- Masonry in natural stone: normal distribution with an average drift of 0.61% and a coefficient of variation of 25%

6.4 DERIVATION OF THE FRAGILITY CURVES
Once the probability density functions of the different considered damage states have been defined from pushover analyses and the displacement demand imposed on the building by different levels of ground motion has been evaluated from the use of the corresponding spectra, it is possible to convolve these two results to obtain fragility points which will then be fitted by lognormal distributions in order to obtain analytical fragility curves.

The final step aims to the derivation of the fragility curves, which the primary purpose of the present study. The main steps of the procedure that is used, is presented:
1. Compare the nonlinear displacement response with the ultimate displacement capacity ($S_{du}$) to identify if the building exceeds the chosen collapse limit state, or not.

2. Repeat for all response spectra and estimate the probability of collapse for the selected capacity curve.

3. Repeat for all capacity curves, and then estimate the mean probability of collapse, for the examined direction (the weakest).

4. Plot the mean probability of collapse against the level of PGA.

5. Fragility functions in the present study are approximated by lognormal cumulative distribution functions, having a median value $\mu$ and logarithmic standard deviation, or dispersion, $\sigma$. Thus analytical fragility functions are derived.

The mathematical form for such a fragility function is:

$$F_i(D) = \Phi \left( \frac{\ln D + \mu}{\sigma} \right)$$

where: $F_i(D)$: conditional probability that the component will be damaged to damage state “$i$” or a more severe damage state as a function of demand parameter, $D$

$\Phi$: standard normal (Gaussian) cumulative distribution function

$\mu$: median value of the probability distribution

$\sigma$: logarithmic standard deviation
PART 2: CASE STUDY

7 CASE STUDY

The final target of the present research, namely the derivation of the fragility curves of the typical Dutch Rijtjeshuis, is going to be achieved by executing multiple pushover analyses on Finite Element Models. The model should be designed such that will be able to capture the real behavior of a series of Rijtjeshuis under the imposed actions determined by the seismic hazard analysis (Chapter 5 and 10).

7.1 CAPACITY SPECTRUM METHOD—CASE STUDY

The aim of the present chapter is to provide a clear description of Capacity Spectrum method, which is used as a tool in order to derive the fragility curves for unreinforced masonry building in Groningen. A clear sequence of the steps followed will be presented together with the assumptions and the methodology used in each step.

7.1.1 STEP 1: SEISMIC DEMAND

The seismic demand for the examined site should be determined, Groningen in this case. The methods to develop the elastic acceleration response spectra are presented in Chapter 3 and are based on Jayaram et al (2011), Akkar et al (2014a) and Akkar et al (2014b). A modification of the response curves to acceleration-displacement format is needed (ADRS—demand spectra—Figure 24), in order to make the comparison with the capacity curves in step 5.

\[ S_{ae} = \frac{4\pi^2}{T^2} S_{de} \]

Where:

- \( S_{ae} \): Elastic spectral acceleration
- \( S_{de} \): Elastic spectral displacement
- \( T \): Period of vibration of a linear single degree of freedom system

![FIGURE 24: TRANSFORMATION OF THE ELASTIC SPECTRUM TO ADRS FORMAT (CHOPRA ET AL. (2002))](image)

The demand spectra for the area of Groningen are presented in Chapter 3.

7.1.2 STEP 2: PUSHOVER CURVE

The result of the second step is the derivation of the pushover curve for the structure, which gives the relation between the top displacement of the structure and the base shear force which is a direct result of the response
of the multi-degree-of-freedom system. To obtain this curve, a non-linear analysis of the structure is needed. The load application consists of initially applying the vertical loads on the structure, and afterwards a monotonically increasing pattern of lateral loads corresponding to a specific distribution. The three most commonly used distributions are the following:

- Uniform load Distribution
- Linear load Distribution
- Distribution related to the first mode of vibration

To determine the lateral load distribution related to the first modal shape, the lateral force at each level is given by the following expression:

\[ P_i = m_i \Phi_i \]

Where:

- \( P_i \): lateral load applied at level \( i \)
- \( m_i \): mass of the structure at level \( i \)
- \( \Phi_i \): shape factor for assumed load distribution at level \( i \)

In the present study the uniform load distribution is going to be examined. However, since the second floor of the examined model is carrying not only its self-weight but also the weight of the roof, the lateral load distribution is described in Table 6:

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Uniform Distribution</th>
<th>Uniform Distribution (normalised)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second Floor</td>
<td>1.15</td>
<td>1</td>
</tr>
<tr>
<td>First Floor</td>
<td>1</td>
<td>0.870</td>
</tr>
<tr>
<td>Ground</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

A typical pushover curve for the Groningen unreinforced masonry building is presented in Figure 25:
Pushover curves of the analyses if the model are presented in Chapter 4 and 5.

7.1.3 Step 3: Capacity Spectrum Bilinearisation of the Capacity Spectrum
The first part of the third step consists of the transformation of the pushover curve to the capacity curve, which actually means the transformation of the multi-degree-of-freedom-system to a single-degree-of-freedom-system. The selected method is ATC-40 method, which coincides with the method N2 of the Eurocode and the one suggested by Goel and Chopra (2004).

After the eigenvalue analysis is executed the following formulas are used to obtain the capacity curve in an acceleration displacement ADRS format (as shown in Figure 26)

$$m^\ast = \sum m_i \phi_i$$
$$D^\ast = D/\Gamma$$
$$F^\ast = V/\Gamma$$
$$\Gamma = m^\ast / \sum m_i \phi_i^2 = \sum m_i \phi_i / \sum m_i \phi_i^2$$

Where $\Gamma$: modal participation factor

The transformation to the ADRS format is done by dividing the lateral force of the equivalent SDOF by the equivalent mass:

$$S_a = F^\ast / m^\ast$$
For the bilinearisation of the capacity curve the N2 method of the Eurocode is used. N2 method (which is described in detail in chapter 2) provides an elasto-perfectly-plastic bilinear curve based on two assumptions:

c. Elastic portion of the bilinear curve crosses the pushover curve at 60% of the proposed yield stress
d. Total energy demand is the same for both curves (equal areas under the curves) as in Figure 27.

FIGURE 27: BILINEARISATION OF THE CAPACITY CURVE, N2 METHOD

An example of bilinearisation of the capacity curve is presented in Figure 28 and the results from the bilinearisation of all the capacity curves are exhibited in Chapter 9.4.
7.1.4 **Step 4: Inelastic Displacement Demand**

The calculation of the maximum inelastic displacement response of nonlinear structures is important in order to seismically assess a non-linear structure. Thus, the elastic displacement demand should be transformed to the inelastic displacement demand. The methods used to describe this procedure are described in detail in Chapter 3 (3). In the present study, Lin and Miranda (2008) method is going to be used, which is a non-iterative procedure that takes into account the strength ratio and the period of vibration of the structure. The chosen method not only avoids iterations and improves accuracy but also gives good predictions of the mean maximum inelastic displacement of structures for systems with all period ranges (long, intermediate and short ones).

![Graph showing inelastic displacement demand](image)

**Figure 28: Example of Bilinearisation of the Capacity Curve Using N2 Method**

7.1.5 **Step 5: Derivation of the Fragility Curve**

The final step aims to the derivation of the fragility curves, which is the primary purpose of the present study. The theoretical approach of fragility curves is given in detail in Chapter 3 (3). A brief description of the followed procedure is presented:

![Diagram of demand and capacity curves](image)

**Figure 29: Translation from the Elastic to the Inelastic Demand Spectra (Chopra et al. (2002))**

The resulting inelastic displacement demand for each examined case is presented in Chapter 6.
6. Compare the nonlinear displacement response with the ultimate displacement capacity ($S_{du}$) to identify if the building exceeds the chosen collapse limit state, or not.

7. Repeat for all response spectra and estimate the probability of collapse for the selected capacity curve.

8. Repeat for all capacity curves, and then estimate the mean probability of collapse, for the examined direction (the weakest).

9. Plot the mean probability of collapse against the level of PGA.
8 Structural Analysis__Case Study

The final target of the present research, namely the derivation of the fragility curves of the typical Dutch Rijtjeshuis, is going to be achieved by executing multiple pushover analyses on Finite Element Models. The determination of the displacement capacity for a typical unreinforced masonry series of buildings becomes the aim of this analysis, as it will enable the comparison between the displacement demand and capacity, which will be used for the derivation of the fragility curves. The model should be designed such that will be able to capture the real behavior of a series of Rijtjeshuis under the imposed actions determined by the seismic hazard analysis (Chapter 10).

This chapter presents the procedure and the results of the non-linear finite element analyses for examined model. First, a reinforced concrete building will be examined in order to validate the correctness of the model. Afterwards, the same model of the structure will be examined made of unreinforced masonry bearing wall and reinforced concrete floors. The structure will be consistent with the ones being in the area of Groningen and the material properties of the structure will be based on the mean values for the masonry material properties, as will be analyzed in more detail in the following chapters. The geometry of the model and constraints are discussed and the different values and behavioral models used to describe the materials are presented.

8.1 Geometry

The analyses that will be executed concern a series of 2-floor houses made of reinforced concrete (bearing frame-floors) and unreinforced masonry (bearing and shear walls). The finite element program DIANA FX+ (version 9.6) will be used. The geometry of each house is shown in Figure 30 and is based on plans, sections and elevations of an existing building in Groningen. The building plan is provided in Appendix C. The house of the figure when repeated gives a series of houses, as in Figure 31.

The finite element model will be a part of these houses consisting of a separation wall, of 5.8 m height in total and the floors with half span, meaning 2.7m in both sides of the separation wall (5.4m in total). The described model is presented in Figure 32. The symmetry and repetition in the geometry of the whole structure gives the opportunity to create a reliable simplified model (Figure 32) which can give the behavior of the series of houses when repeated. The foundations of the structure are modeled as a hinge.
FIGURE 30: SERIES OF RIJTJESHUIS (DIMENSIONS IN MM)
The derivations of a simple model to describe a whole block of Rijtjeshuis is based on assumptions and simplifications which need to be kept in mind. The primary simplifications to the geometry and constraints are the following:

1. Separation walls with openings are totally ignored. The masonry walls can work as shear walls and provide higher resistance to lateral loading (earthquake loads). The differences in the dimensions between each building and the uncertainty involved in their behavior guarantees that ignoring their contribution will lead to more conservative results.
2. Only part of the structure is modeled, namely a wall and 2 floors of 1 m width in the direction perpendicular to the plane of the frame (Figure 32). The response of the entire structure has been represented by applying suitable boundary conditions to all elements, ensuring that the deformations of the outer points on each floor are totally fixed (same displacements and rotations). This assumption does not have any effect neither on gravity loading (gravity loads are evenly distributed at each side of the symmetry axis), nor the lateral-earthquake loading (lateral loads are applied on the center of mass). The repetition of the existing model in x and y direction will sufficiently represent the whole structure.

3. Constraints have been adopted in all elements of the model in order to ensure in-plane behavior of the model. Local out of plane wall movements are commonly observed in case of earthquake loading. Thus, out-of-plane mechanisms are prevented and the global response of the building is examined (Figure 38).

4. The floors are assumed to be safely attached to the bearing wall. During the construction process of masonry houses with reinforced concrete floors, the building sequence is as follows: 1) Building the masonry wall of the first level, 2) casting the 1st concrete floor, 3) building on the 1st floor the masonry of the 2nd floor and 4) casting the concrete of the 2nd floor. Thus, the continuity of the elements is not guaranteed. However, the big difference between the stiffness of the masonry wall and the concrete floor can lead to the conclusion that the lateral loads will bring about cracks on the masonry wall first instead of the concrete floors (something verified from the results). As a result a fixed connection between the bearing wall and the floors can be assumed.

5. The floors are assumed to be continuous reinforced concrete slabs over the adjacent houses. However, as shown in Detail 4 in Appendix C the real situation is hollow core slabs, with only in 4 cores over the length of the wall a connection with reinforcement bars. This rises the uncertainties of the created model, making it non-conservative.

6. For practical reasons the floors of the model are positioned in a different plane than the loadbearing wall. If the bearing wall and floors were modeled in one plane, plane x-z, then the reinforcement of the floors would have be passing through some elements of the wall. This could bring about problems in the running procedure of the analysis (for easier understanding see Figure 35 and Figure 36). In order to avoid such problems, the floors have been designed in the plane parallel to x-z and with a distance of 3m from it. The points where the wall and the floor at both levels are connected in the real structure, however should fixed (see assumption 4). Thus, the connection points (wall-1st floor and wall-2nd floor) are totally fixed by applying tyings, which ensure that all the deformations of the coupled nodes will be the same. Thus, the reinforcement of the concrete floors can be assigned without any problems with the mother elements in DIANA.

7. The foundations of the structure are assumed to be a hinge based on the plans, sections and elevations of the existing building in Groningen. Stability is provided to the model through the fixed connections between the extreme points of the 1st and 2nd floor respectively.
The above described model is made using DIANA FX+ and is presented in Figure 33.

**FIGURE 33: MODEL OF RITJESJUIS IN DIANA FX+**

In Figure 34 the reinforcement is also applied to all the members of the structure (It is worth mentioning that the reinforcement of the wall is only valid for the reinforced concrete model and not for the masonry, which will be unreinforced) Figure 34.

**FIGURE 34: MODEL IN DIANA FX+**
The model with the given geometry and dimensions of the elements can be found in Figure 35 and Figure 36 in 2D and 3D respectively:
The aforementioned supports and tyings are described in more detail further:

Supports

The foundation is modeled as a hinge in this model and thus a constraint in the translation in x and z direction is applied for node 1, which is the base of the wall. In order to examine the in-plane behavior of the structure, a constraint of the translation in y direction and of the rotation around the x and z direction are applied to all the nodes of the model (1-335) (see Figure 38).
Tyings

A Master-Slave relationship is given between several nodes in order to ensure the connectivity between the wall and the floors, and continuity in the model. The connected nodes are the following (number of nodes in Figure 40) (see Figure 40):

- 172-118: Translation in x and z direction and rotation around y axis
- 281-227: Translation in x and z direction and rotation around y axis
- 59-145: Translation in x and z direction and rotation around y axis
- 30-254: Translation in x and z direction and rotation around y axis

FIGURE 39: NUMBERING OF NODES CONNECTED WITH TYINGS IN THE BEAM MODEL 2D

FIGURE 40: NUMBERING OF NODES CONNECTED WITH TYINGS IN THE BEAM MODEL 3D

8.3 Finite Element Model

In order to have a reliable model, the verification of the finite element model used is of crucial importance. The first model that is developed in the Finite Element Program DIANA FX+ (version 9.6)
is made of reinforced concrete and is compared with the respective one made by TNO in the framework of the project “NPR 9998 - Rekenvoorbeeld betonconstructie (TNO pushover analyse in DIANA)”. After the model is certified, the material properties of the concrete wall are going to be replaced with the mean material properties of masonry (Calcium-silicate brickwork, typical approximately 1960-1985). All the material properties of the structure (referring to masonry only) are assumed to be random variables, to which normal probability distributions are associated based on realistic ranges of variation. Afterwards, in Chapter 9 a sensitivity study will be done based on the mean values and the coefficient of variation of a dominant random variable which is chosen to be the tensile strength of masonry. The pushover curve of each analysis will be extracted with ultimate goal the derivation of the family of pushover curves for the different material properties and further the capacity displacement for each case.

8.3.1 BEAM ELEMENT MODEL

In the first model only the concrete frame of one house is modeled. The element size is selected 100 mm both for the wall and the floors, giving a total of 166 elements.

Elements CL18B

The whole model, with geometry as Figure 32, consists of beam elements type CL18B, which belong in class III category. The CL18B element which is used in this case, is a three-node, three-dimensional class-III beam element. Basic variables are the translations $u_x$, $u_y$ and $u_z$ and the rotations $\phi_x$, $\phi_y$ and $\phi_z$ in the nodes. The strains vary linearly along the center line of the beam. By default DIANA applies a 2-point Gauss integration scheme along the bar axis, but 11 integration points in the width of the wall and length of the floors (x-direction) were chosen in order to have a more accurate view of the developing crack pattern.

The class-III beams in DIANA comprise a number of curved (higher order) elements which are numerically integrated over their cross-section and along their axis. They are based on the so-called Mindlin-Reissner theory which does take shear deformation into account. Unlike the classical beam elements of class-I and class-II, the class-III beam elements are based on an isoparametric formulation which assumes that the displacements and rotations of the beam axis normals are independent and are respectively interpolated from the nodal displacements and rotations. Because the displacement interpolation of class-III beams is compatible to the continuum elements they are very suitable for connection to those type of elements. Another advantage of the class-III beam elements compared to other classes, is that they may be curved due to the fact that they have more than two nodes, as shown in Figure 41.

8.4 VERIFICATION OF THE MODEL: APPROACH FOR A CONCRETE STRUCTURE

The wall and the floors have the same material properties, which are the properties of concrete C30/37. The model used to describe these properties is Total Strain Based Crack Model, Rotating, in which the stress-strain
relationships are evaluated in a coordinate system continuously rotating in the principal directions of the strain vector. The modulus of elasticity for concrete is $E_{cm}=33000 \text{ N/mm}^2$ (according to Eurocode 2, table 3 for concrete C30/37), and Poisson ratio $\nu=0.2$. The density of the material is 2500 kg/mm³.

For the compressive behavior of concrete the multi-linear curve is chosen (Figure 42) resulting in the multi-linear curve shown in Figure 44, based on the values of Table 7. The assumed yield stress is $38*10^6 \text{ N/m}^2$.

![MULTILINEAR MODEL FOR THE COMPRRESSIVE BEHAVIOR OF CONCRETE (DIANA MANUAL)](image)

**TABLE 7: STRESS-STRAIN VALUES FOR THE MULTILINEAR COMPRESSION CURVE OF CONCRETE**

<table>
<thead>
<tr>
<th>Stress N/m²</th>
<th>Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>-15.4E+06</td>
<td>-5.0E-04</td>
</tr>
<tr>
<td>-26.9E+06</td>
<td>-1.0E-03</td>
</tr>
<tr>
<td>-34.4E+06</td>
<td>-1.5E-03</td>
</tr>
<tr>
<td>-37.8E+06</td>
<td>-2.0E-03</td>
</tr>
<tr>
<td>-38.0E+06</td>
<td>-2.2E-03</td>
</tr>
<tr>
<td>-37.0E+06</td>
<td>-2.5E-03</td>
</tr>
<tr>
<td>-32.1E+06</td>
<td>-3.0E-03</td>
</tr>
<tr>
<td>-22.7E+06</td>
<td>-3.5E-03</td>
</tr>
<tr>
<td>0.00</td>
<td>-3.5E-03</td>
</tr>
<tr>
<td>0.00</td>
<td>-5.0E-03</td>
</tr>
</tbody>
</table>

The tensile curve is supposed linear ultimate strain based, as in Figure 43 with tensile strength $1.93*10^6 \text{ N/m}^2$ and ultimate strain 0.1%.
The aforementioned material properties of concrete are concentrated in Table 8: Concrete Material Properties:

**TABLE 8: CONCRETE MATERIAL PROPERTIES**

<table>
<thead>
<tr>
<th>Material Characteristics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ρ</td>
<td>2500 kg/m³</td>
</tr>
<tr>
<td>f_k</td>
<td>38 MPa</td>
</tr>
<tr>
<td>f_t</td>
<td>1.93 MPa</td>
</tr>
<tr>
<td>E</td>
<td>33000 MPa</td>
</tr>
</tbody>
</table>

Geometry of the elements

The geometry of the different parts of the models is based on the real dimensions of the structure. The wall is a solid rectangular with dimensions 1m in y axis and 0.25m in x axis. The two floors are modeled in y = 3m for reasons of easier understanding of the reinforcement from DIANA and are solid rectangular with dimensions 1m in y axis and 0.2 in z axis.
Reinforcement

Two types of reinforcement are used: Ø12/150 for the upper part of the floors and Ø10/150 for the bottom part of the floors and the wall. The model used for the material properties of the reinforcement steel is Von Mises Plasticity with Young modulus $2 \times 10^{11} \text{ N/m}^2$ and hardening hypothesis Work Hardening. The Hardening Diagram, which is shown in Figure 45, has maximum values of stress and strain $594 \times 10^6 \text{ N/m}^2$ and 4.703% respectively.

![Hardening Diagram of Reinforcement Steel](image)

**FIGURE 45: HARDENING DIAGRAM OF REINFORCEMENT STEEL**

8.4.1 Analysis Procedure

The initial step for a pushover analysis is to determine an appropriate lateral force distribution to be applied on each floor of the structure. Commonly, three types of lateral load distribution are used:

- A uniform distribution of loads
- A linear increase of the load as a function of the building height
- A distribution corresponding to the first mode of vibration

To determine the lateral load based on the last type of loading, a modal analysis is required. In the present study, the uniform load distribution is selected. However, for completeness reasons the eigenvalue analysis will be executed.

8.4.1.1 Eigenvalue Analysis

The modal analysis was executed for the beam model using DIANA FX+ (version 9.6) and the result gave one dominant mode of vibration. The first five modes are presented in Figure 46-Figure 49.
The modal analysis results are displayed in Table 9:

**TABLE 9: RESULTS OF THE MODAL ANALYSIS**

<table>
<thead>
<tr>
<th></th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>Mode 4</th>
<th>Mode 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency $f$ (Hz)</td>
<td>2.73</td>
<td>13.31</td>
<td>25.68</td>
<td>26.20</td>
<td>48.29</td>
</tr>
<tr>
<td>Period $T$ (sec)</td>
<td>0.366</td>
<td>0.075</td>
<td>0.039</td>
<td>0.038</td>
<td>0.021</td>
</tr>
<tr>
<td>Modal mass (kg)</td>
<td>5822.0</td>
<td>6523.1</td>
<td>1527.1</td>
<td>1494.0</td>
<td>3703.9</td>
</tr>
<tr>
<td>Participation factor $\Gamma_i$ (%)</td>
<td>0.65</td>
<td>0.72</td>
<td>0.17</td>
<td>0.17</td>
<td>0.41</td>
</tr>
</tbody>
</table>

**1.2.1.1 Pushover Analysis**

Following, the lateral load pattern should be determined.

The present pushover analysis is based on a uniform distribution along the height of the structure.

The three lateral load distributions are presented in Table 10:

**TABLE 10: LATERAL LOAD DISTRIBUTIONS**

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Modal Distribution</th>
<th>Linear Distribution</th>
<th>Uniform Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second Floor</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>
The load application consists of initially applying the vertical loads on the structure (the dead load followed by 30% of the live load), applied in 10 equal steps, 10% of the total vertical load each. Once this load has been applied, the seismic action is modeled, represented by a monotonically increasing pattern of lateral loads corresponding to one of the aforementioned distributions (uniform).

The lateral load in this case was applied in 1000 steps, with a load of 1kN and 1.15kN per step on the first and second floor respectively. The input given in DIANA are resumed in the following table:

**TABLE 11: INPUT FOR NON-LINEAR ANALYSIS IN DIANA**

<table>
<thead>
<tr>
<th>Non-linear Effects</th>
<th>Physical + Geometrical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load in Steps</td>
<td></td>
</tr>
<tr>
<td>1) Dead load + 0.3* live load: 10 steps, max no. iterations: 100</td>
<td></td>
</tr>
<tr>
<td>2) Horizontal load: 1kN per step, 1000 steps, max no. iterations: 300</td>
<td></td>
</tr>
<tr>
<td>3) Arc-length: node 117, translation x, alpha=1</td>
<td></td>
</tr>
<tr>
<td>Method</td>
<td>Newton-Raphson, Regular</td>
</tr>
<tr>
<td>Convergence Norm</td>
<td>✓ Satisfy all specified norms</td>
</tr>
<tr>
<td></td>
<td>✓ Energy</td>
</tr>
<tr>
<td></td>
<td>✓ Displacement</td>
</tr>
<tr>
<td></td>
<td>✓ Force</td>
</tr>
<tr>
<td>Convergence tolerance norm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>0.01</td>
</tr>
</tbody>
</table>

8.4.2 RESULTS

The result of the non-linear analysis is the pushover curve given in Figure 51:

**FIGURE 51: PUSHER CURVE FOR THE CONCRETE STRUCTURE**

<table>
<thead>
<tr>
<th>First Floor</th>
<th>0.7084</th>
<th>0.5</th>
<th>0.870</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
A first attempt to verify the obtained pushover curve is made by comparing it to the pushover curve given by the TNO model (Figure 52):

![Graph comparing pushover curves](image)

**FIGURE 52: COMPARISON BETWEEN PUSHOVER CURVES OBTAINED FROM TNO AND THE PRESENT STUDY**

In the non-linear analysis executed, the number of load steps were made sufficient to ensure that the analysis would stop due to non-convergence and not due to lack of load steps. The arc length control was used to ensure that the analysis would continue regardless of the unloading.

Following, some interesting results coming from the analyses will be presented, which can contribute to the understanding of the behavior of the structure up to failure. The presented results are the crack pattern along the frame of the structure for some specific points of the pushover curve and the characteristic result at the step examined k and the step k+1, in order to see the difference in the results.
The points of interest are selected as follows:

- A: concrete reaches tensile strength (step 20): the linear part of the pushover curve changes direction which means a reduction in the stiffness of the structure. The reason for such a behavior is the appearance of fully open cracks in concrete, which only happens when the applied tensile stresses exceed the tensile strength of concrete (193 MPa).
• B: concrete reaches maximum strain (step 49): During the second phase (area between points A and B) the tensile stresses are carried thanks to the cooperation of concrete and reinforcement while cracks in concrete continue to develop. At step 49 concrete reaches its maximum strain, 3 \% 

![Figure 55: Strains in Concrete for Step 49 (Left) and Step 50 (Right)](#)

• C: reinforcement yield (step 77): The stress of the reinforcement reaches the yield value, and equals to 594 MPa. After that point the pushover curve is descending.

![Figure 56: Stresses in Reinforcement for Step 77 (Left) and Step 78 (Right)](#)

• D: shear base force equal to 80\% of the maximum force (Chosen Ultimate Limit State) (step 217)
• E: concrete reaches compressive strength (step 476)
F: Formation of the plastic hinge (step 536): A plastic hinge is created on a rigid connection when the number and width of the cracks are such that do not allow for the connection to transmit any moments. Thus, the connection becomes a hinge and consequently, the moment distribution along the height of the first floor (wall between two hinges – foundation and plastic hinge) is zero.

The crack status for each of the aforementioned states, is presented in Figure 59:
FIGURE 59: CRACK STATUS FOR STAGES A-F AND LEGEND

Additionally, the following results will be presented in Appendix B for the aforementioned characteristic states of the analysis as indicated in Figure 53 (points A-E):

- The deflected shape of the structure
- Distribution of normal forces along the frame
- Distribution of shear forces along the frame
- Distribution of moments along the frame
- Strains for the concrete and the reinforcement
- Stresses for the concrete and the reinforcement

8.5 MASONRY STRUCTURE

In the second case, the material properties of the wall are changed from reinforced concrete to the mean values of a Calcium Silicate brickwork (approx. 1960-1985) masonry. The material of the floors remain the same as in Chapter 8.4, which are the properties of concrete C30/37 reinforced with Ø12/150 and Ø10/150 for the top and
bottom part of the floors respectively. The model used to describe the masonry properties is Total Strain Based Crack Model, Fixed, in which the stress-strain relationships are evaluated in a fixed coordinate system which is fixed upon cracking. The modulus of elasticity for concrete is $E_{cm} = 3500 \text{ N/mm}^2$ and Poisson ratio $v = 0.2$. The density of the material is $1900 \text{ kg/mm}^3$.

For the compressive behavior of concrete the parabolic curve is chosen (Figure 60). The assumed yield stress is $6 \times 10^6 \text{ N/m}^2$ and the fracture energy in compression $G_c = 5000 \text{ J/m}^2$.

![Figure 60: Parabolic Model for the Compressive Behavior of Masonry (DIANA Manual)](image)

(g) parabolic

The tensile curve is supposed exponential, as in Figure 61, with tensile strength $0.2 \times 10^6 \text{ N/m}^2$ and fracture energy in tension $G_{t1} = 15 \text{ J/m}^2$.

![Figure 61: Exponential Curve for the Tensile Behavior of Masonry (DIANA Manual)](image)

(f) exponential

The post-cracked shear behavior was defined by taking into account the retention factor of its linear behavior, which reduces its shear capacity according to Figure 62, where $\beta$ is the retention factor with values $0 < \beta \leq 1$ and $G$ is the shear modulus of the uncracked material. The shear retention factor, was left at the default value of 0.01, which means that the shear strength of the material will be reduced to one percent of the original shear strength when cracks form.
In addition no reduction due to lateral cracking was selected (REDCRV: NONE) and constant behavior of Poisson’s ratio due to cracking (POIRED DAMAGE).

The aforementioned material properties of masonry are concentrated in Table 12:

**TABLE 12: MASONRY, MEAN VALUES OF THE MATERIAL PROPERTIES**

<table>
<thead>
<tr>
<th>Material Characteristics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ρ Density</td>
<td>1900 kg/m³</td>
</tr>
<tr>
<td>$f_k$ Concrete Compressive strength</td>
<td>6 MPa</td>
</tr>
<tr>
<td>$G_c$ Fracture Energy in Compression</td>
<td>5000 J/m²</td>
</tr>
<tr>
<td>$f_t$ Concrete Tensile Strength</td>
<td>0.2 MPa</td>
</tr>
<tr>
<td>$G_{f1}$ Fracture Energy in Tension</td>
<td>15 J/m²</td>
</tr>
<tr>
<td>E Elastic Modulus</td>
<td>3500 MPa</td>
</tr>
</tbody>
</table>

**Geometry of the elements**

The geometry of the different parts of the models is the same as in the reinforced concrete model, as it is based on the real dimensions of the structure and not on the material properties used. The wall is a solid rectangular with dimensions 1m in y axis and 0.25m in x axis. The two floors are modeled in $y = 3$m for reasons of easier understanding of the reinforcement from DIANA and are solid rectangular with dimensions 1m in y axis and 0.2 in z axis.

**Reinforcement**

Two types of reinforcement are used: $\varnothing 12/150$ for the upper part of the floors and $\varnothing 10/150$ for the bottom part of the floors. The masonry wall is unreinforced. The model used for the material properties of the reinforcement steel is *Von Mises Plasticity* with Young modulus $2*10^{11}$ N/m² and hardening hypothesis *Work Hardening*. The Hardening Diagram, which is shown in Figure 63, has maximum values of stress and strain $594*10^6$ N/m² and 4.703% respectively.
8.5.1 ANALYSIS PROCEDURE

As described in Chapter 8.4.1, first the eigenvalue analysis will be executed and afterwards the non-linear pushover analysis based on the uniform lateral load distribution.

8.5.1.1 EIGENVALUE ANALYSIS

The modal analysis was executed for the beam model using DIANA FX+ (version 9.6) and the result gave one dominant mode of vibration. The first five modes are presented in Figure 64-Figure 67. The difference from the concrete model lays on the fact that masonry has smaller density that the concrete which results is lower mass and lower frequency in the masonry model. However, the modal shapes are the same.

FIGURE 63: HARDENING DIAGRAM OF REINFORCEMENT STEEL

![Hardening Diagram of Reinforcement Steel](image)

FIGURE 64: MODE 1: $f_0=1.26$ HZ  
FIGURE 65: MODE 2: $f_0=5.58$ HZ  
FIGURE 66: MODE 3: $f_0=20.88$ HZ

![Mode Shapes](image)
The modal analysis results are displayed in Table 13:

### TABLE 13: RESULTS OF THE MODAL ANALYSIS

<table>
<thead>
<tr>
<th></th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>Mode 4</th>
<th>Mode 5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Frequency f (Hz)</strong></td>
<td>1.26</td>
<td>5.58</td>
<td>20.88</td>
<td>25.44</td>
<td>27.11</td>
</tr>
<tr>
<td><strong>Period T (sec)</strong></td>
<td>0.79</td>
<td>0.18</td>
<td>0.05</td>
<td>0.04</td>
<td>0.036</td>
</tr>
<tr>
<td><strong>Modal mass (kg)</strong></td>
<td>6337.7</td>
<td>6462.8</td>
<td>2040.6</td>
<td>1562.2</td>
<td>747.6</td>
</tr>
<tr>
<td><strong>Participation factor Γ_i (%)</strong></td>
<td>0.78</td>
<td>0.79</td>
<td>0.25</td>
<td>0.19</td>
<td>0.09</td>
</tr>
</tbody>
</table>

### 1.2.1.2 Pushover Analysis

Following, the lateral load pattern should be determined. The present pushover analysis is based on a uniform distribution along the height of the structure.

The three lateral load distributions are presented in Table 14:

### TABLE 14: LATERAL LOAD DISTRIBUTIONS

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Modal Distribution</th>
<th>Linear Distribution</th>
<th>Uniform Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second Floor</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>First Floor</td>
<td>0.8531</td>
<td>0.5</td>
<td>0.870</td>
</tr>
<tr>
<td>Ground</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The load application consists of initially applying the vertical loads on the structure (the dead load followed by 30% of the live load), applied in 10 equal steps, 10% of the total vertical load each. Once this load has been applied, the seismic action is modeled, represented by a monotonically increasing pattern of lateral loads corresponding to one of the aforementioned distributions (uniform).

The lateral load in this case was applied in 1000 steps, with a load of 1kN and 1.15kN per step on the first and second floor respectively. The input given in DIANA are resumed in the following table:
TABLE 15: INPUT FOR NON-LINEAR ANALYSIS IN DIANA

<table>
<thead>
<tr>
<th><strong>Non-linear Effects</strong></th>
<th>Physical + Geometrical</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Load in Steps</strong></td>
<td>4) Dead load + 0.3* live load: 10 steps, max no. iterations: 100</td>
</tr>
<tr>
<td></td>
<td>5) Horizontal load: 1kN per step, 1000 steps, max no. iterations: 300</td>
</tr>
<tr>
<td></td>
<td>Arc-length: node 117, translation x, alpha=1</td>
</tr>
<tr>
<td><strong>Method</strong></td>
<td>Newton-Raphson, Regular</td>
</tr>
<tr>
<td><strong>Convergence Norm</strong></td>
<td>✓ Satisfy all specified norms</td>
</tr>
<tr>
<td></td>
<td>✓ Energy</td>
</tr>
<tr>
<td></td>
<td>✓ Displacement</td>
</tr>
<tr>
<td></td>
<td>✓ Force</td>
</tr>
<tr>
<td><strong>Convergence tolerance norm</strong></td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>0.01</td>
</tr>
</tbody>
</table>

8.5.2 RESULTS

The result of the non-linear analysis is the pushover curve given in Figure 69:

![Pushover Curve](image)

**FIGURE 69: PUSHOVER CURVE FOR MASONRY UNTIL THE POINT WHERE DIVERGENCE OCCURS**

The first part of the curve, namely the elastic and softening branch, is in agreement with the expected curve for masonry as derived from literature (Doherty et al., 2002), with the structure failing out of plane due to rocking mechanism: the wall is firstly uncracked and the point around which the rocking is activated has finite dimensions. Thus, the investigation of what is happening in the structure at the part where the base shear force becomes negative becomes questionable.

At that part of the curve, what is worth mentioning is that the shear base force and the top displacement have different signs: although the forces applied on the structure are negative, the displacement continues to increase in the positive direction. The explanation of this phenomenon lies on the geometrical nonlinearities taken into account in the analysis. Indeed, when executing the same analysis without geometrical nonlinearities the resulting pushover curve shows that at the same displacement where the shear force changed sign is the original analysis, the analysis stops since no equilibrium was reached. The moments caused by the vertical loads acting on top of the bearing wall in combination with the deformed shape of the structure, are larger than the ones resulting from the lateral loads and thus the structure continues to deform in the positive direction until no more equilibrium is found. This result can be seen in Figure 70.
FIGURE 70: COMPARISON BETWEEN ANALYSES WITH AND WITHOUT GEOMETRICAL NONLINEARITIES

However, the limit state examined in the present study is not taken as the point where divergence occurred in the FEM analysis, since the stage where the cracks are big enough that cannot be considered acceptable is observed much earlier than the actual collapse of the building, as also mentioned in previous chapters. Since the Near Collapse limit state is under examination, the point where shear base force reaches 80% of the maximum force in the descending part of the graph will be the ultimate point of the pushover curve. The value of the corresponding displacement is defined as the displacement capacity of the structure with the given material properties. Further in this study this value will be used for the comparison with the displacement demand for different earthquake scenarios. For the structure with material properties the mean values of each parameter, the pushover curve up to the Near Collapse limit state, is presented in Figure 71, and the ultimate displacement capacity is found 0.03192 m.
A comparison between the masonry and concrete structures can be made by comparing the two different pushover curves (Figure 72). In Figure 72 both pushover curves have as an ultimate point the point where 80% of the maximum shear force is achieved.

In the non-linear analysis executed, the number of load steps were made sufficient to ensure that the analysis would stop due to non-convergence and not due to lack of load steps. The arc length control was used to ensure that the analysis would continue regardless of the unloading.

Following, some interesting results coming from the analyses will be presented, which can contribute to the understanding of the behavior of the structure up to failure. The presented results are the crack pattern along
the frame of the structure for some specific points of the pushover curve and the characteristic result at the step examined \( k \) and the step \( k+1 \), in order to see the difference in the results.

**FIGURE 73: CHARACTERISTIC POINTS WHERE RESULTS ARE PRESENTED**

The points of interest are selected as follows:

- **A**: masonry reaches its tensile strength (step 21): the linear part of the pushover curve does not show a linear behavior further from point A which means a reduction in the stiffness of the structure. The reason for such a behavior is the appearance of fully open cracks in masonry, which only happens when the applied tensile stresses exceed the tensile strength of concrete (0.20 MPa).
B: masonry reaches maximum strain (step 49): During the second phase (area between points A and B) the tensile stresses are carried by the uncracked part of masonry only, since reinforcement does not exist while cracks in masonry continue to develop. At step 49 concrete reaches its maximum base shear force, 3.7154 kN.

C: descending part begins (step 123): The stress of the masonry during the stage B-C is almost constant around the yield stress 0.2 MPa. After that point (C) the pushover curve is descending.
The crack status for each of the aforementioned states, is presented in Figure 59:
FIGURE 78: CRACK STATUS FOR STAGES A-F AND LEGEND AT LAYER 33 → OUTER FIBER RIGHT SIDE OF THE WALL

Additionally, the following results will be presented in Appendix B for the aforementioned characteristic states of the analysis as indicated in Figure 53 (points A-E):

- The deflected shape of the structure
- Distribution of moments along the frame
- Distribution of normal forces along the frame
- Distribution of shear forces along the frame
- Strains for the masonry and the reinforcement
- Stresses for the masonry and reinforcement
- Element status
9 Sensitivity Analysis

9.1 Introduction

In the majority of real cases building properties are not well known due to the variability in building materials and building techniques. Even when the same material is used the properties may vary. Since the masonry structure examined in the present study is supposed to be representative of a class of buildings with similar structural characteristics, mechanical parameters have been considered as random variables. The material variability can be regarded as the most important source of uncertainty in the determination of structural response, with all other sources either deriving directly from its effects or being insignificant in size compared to it.

Based on realistic ranges of the parameters with variation, which are derived from experimental tests and literature, a probability density function (typically Gaussian) is assigned to the random variables. In the present study is the tensile strength of masonry, with mean value 0.2 MPa and coefficient of variation 0.2. The rest of the material properties of masonry (Compressive strength, fracture energy for tension and compression and modulus of elasticity) are calculated as functions of the random variable \( f_t \) through relations proposed by Eurocode 6 and Lourenco (STRUCTURAL MASONRY ANALYSIS: RECENT DEVELOPMENTS AND PROSPECTS) - Model Code 90, CEB-FIP (1993), which are described in the following paragraph. Values of the chosen parameter are then extracted from the distribution using appropriate sampling techniques. Finally, the sampled values are combined to define a series of structures with different characteristics, all nominally representing the same building.

The geometry of the building is treated as deterministic, which means than the plan and elevation configuration of the structure, as well as the wall size and position are supposed to be constant (not variables), fact that is acceptable since the examined structure is real with given location, plan and dimensions.

9.2 Material Properties Treated as Random Variables

The material properties needed to fully describe the tensile and compressive behavior of masonry are the tensile strength, compressive strength, modulus of elasticity and fracture energy for tension and compression. Given these values, the graphs presented in Chapter 2 are fully determined and the seismic behavior of the structure consisting of this material can be investigated. However, since the uncertainty in the material properties of masonry is taken into account, the aforementioned values are not constant values but should be treated as random variables, each described by a specific distribution and the mean and standard deviation derived from relevant studies-experiments. However, not all of these parameters are given distributions in literature, and even if they did, the exact way of their interdependency is not well known. The fully described variables found in literature are the tensile strength of masonry \( f_t \) described by a normal distribution with mean value 0.2 MPa and coefficient of variation 20%, which gives a standard deviation of 0.04 and the modulus of elasticity \( E \) which is also represented by a normal distribution with mean value 3500 MPa and coefficient of variation 10%, which gives a standard deviation of 350. In addition, given is the distribution of the maximum displacement capacity of the structure, which has a coefficient of variation 30%.

In the present study, the variability in the material properties of masonry will be based on the assumption that the tensile strength of masonry is the dominant random variable and the rest of the properties will still be variables, but dependent on the tensile strength through given relationships derived from literature as described further in paragraph 9.3. The accuracy of the assumption is questionable, and thus some sensitivity studies will be done in order to check the applicability of it: the first check is made in paragraph 9.3, where the original analysis based on the mean values of the material properties of masonry will be compared to the one based on the tensile strength equal to the mean value 0.2 MPa and the equations which connect the other properties to the tensile strength. The second sensitivity check is given in chapter 13.3 and is taking into account the
information for the modulus of elasticity $E = \sqrt{3500, 350^2}$, which is not followed based on the equations used and are described in 9.3.

### 9.3 Relationships between the Material Properties and the Dominant Random Variable

The relationships which connect the material properties of masonry to the random variable $f_t$, are given in equations (1)-(4):

$f_t$: Chosen variable. Mean value 0.2 MPa, $CoV = 20$

$$f_c = \frac{f_t}{0.05}$$  (1)

$$E = 1000 \cdot f_c$$  (2)

$$G_f = 25 \cdot (2f_t)^{0.7}$$  (3)

$$G_{fc} = 0.16 \cdot f_c$$  (4)

In order to assess the applicability of the proposed relationships in the present study, a comparison is made between the capacity of the structure with the mean values of the material properties and a structure with tensile strength equal to the mean, 0.2 MPa, and the other values determined through the proposed relationships (1)-(4). The input for the two different cases are presented in Table 16.

#### Table 16: Mean Values of Material Properties vs Values of Material Properties Based on $f_t=0.2$ MPa and Equations (1)-(4)

<table>
<thead>
<tr>
<th></th>
<th>Mean Values given</th>
<th>Values based on Eq 1-4 for $f_t=0.2$</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_t$ [MPa]</td>
<td>0.2</td>
<td>0.2</td>
<td>-</td>
</tr>
<tr>
<td>$f_c$ [MPa]</td>
<td>6</td>
<td>4</td>
<td>33</td>
</tr>
<tr>
<td>$E$ [MPa]</td>
<td>3500</td>
<td>4000</td>
<td>-14.3</td>
</tr>
<tr>
<td>$G_f$ [J/m²]</td>
<td>15</td>
<td>13,16</td>
<td>12.2</td>
</tr>
<tr>
<td>$G_{fc}$ [J/m²]</td>
<td>5000</td>
<td>6400</td>
<td>-28</td>
</tr>
<tr>
<td>Max $u$</td>
<td>0.03192</td>
<td>0.02778</td>
<td>12.97 (acceptable)</td>
</tr>
</tbody>
</table>
FIGURE 79: COMPARISON BETWEEN PUSHOVER CURVES OF THE 2 MASONRY BUILDINGS WITH $f_t=0.2$ MPA

From Figure 79 information about the capacity of both structures can be obtained. The draft limits corresponding to the chosen ultimate limit state (80% of the maximum base shear force) are 0.03192 m and 0.02778 m for the structure with the mean values and the one base on the proposed equations. The difference is 12.97% which is considered acceptable based on the fact that the maximum drift limit has a mean value of 0.03192 m and coefficient of variation 30%, as mentioned in literature.

The above result reveals that the suggested equations can be used for a sensitivity study on the variability of the material properties of masonry.

**9.4 SENSITIVITY STUDY**

Based on the assumption that the random variable (tensile strength of masonry) is described by a normal distribution with mean 0.2 MPa and coefficient of variation 0.2, ten different values of tensile strength $f_t$ should be generated in order to perform the sensitivity study. A sample of random variables having a given type of Cumulative Distribution Function (CDF) can be generated from a sample of uniformly distributed random variable $U$, $0 < U < 1$ as long as the inverse CDF, $x = F_x^{-1}(P)$ can be computed. Thus, given $U$ with uniform distribution on $(0,1)$ and $X$ a continuous random variable with CDF $F_x(x)$, the random variable $F_x^{-1}(U)$ has the distribution $F_x$ as shown in.
FIGURE 80: EXAMPLE OF THE GENERATION OF RANDOM VARIABLES FROM THE INVERSE CDF METHOD. THE DENSITY OF THE VERTICAL ARROWS U IS UNIFORM, WHEREAS THE DENSITY OF THE HORIZONTAL ARROWS X= F⁻¹(U) IS PROPORTIONAL TO Fₓ(X) (HARVEY ET AL. (2015))

That means that, generating ten different values of the random variable U~ U(0,1) (which actually stands for the probability P[x≤X]), using the MATLAB command u=rand(1,N), with N=9, and then making the transformation x= F⁻¹(U) will provide the nine different tensile strength values desired to examine the variability in material properties of masonry. After the random number generator and the aforementioned transformation has been applied, the different values of fₜ are given in the 2nd row of Table 17. These different values describe the tensile strength of masonry. After applying equations (1)-(4) using the generated values of fₜ all the material characteristics will be calculated. The resulting values represent the nine different cases of structures that are going to be examined and can be seen in Table 17.

TABLE 17: MATERIAL PROPERTIES OF MASONRY BASED ON EQUATIONS 1-4

<table>
<thead>
<tr>
<th></th>
<th>μ-σ</th>
<th>μ</th>
<th>μ+σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>fₜ</td>
<td>0.16</td>
<td>0.1787</td>
<td>0.1795</td>
</tr>
<tr>
<td>fₑ</td>
<td>3.2</td>
<td>3.574</td>
<td>3.5897</td>
</tr>
<tr>
<td>E</td>
<td>3200</td>
<td>3574</td>
<td>3589.7</td>
</tr>
<tr>
<td>Gₑc</td>
<td>5120</td>
<td>5718.769</td>
<td>5744</td>
</tr>
</tbody>
</table>

The general outcome from the random variable fₜ and the combinations resulting from the equations (1)-(4) gives a realistic range of properties. However, it is obvious that the modulus of elasticity does not have the 10% coefficient of variation as stated in literature, having a big range from 3200 MPa to 4932.5 MPa instead of 3150 MPa to 4000 MPa. This, as well as the uncertainty regarding the exact correlation between the different variables and fₜ should be kept in mind since they increase the uncertainty of the capacity of the structure. A further investigation of this interdependence is recommended in future studies for a more accurate result, although, the ones used in the present study are acceptable for the level of accuracy expected.

The results from the static pushover analyses are presented in Figure 81:
FIGURE 81: PUSHOVER CURVES: VARIATION IN MATERIAL PROPERTIES WITH DOMINANT RANDOM VARIABLE THE TENSILE STRENGTH FT VARYING FROM 0.16 TO 0.2466 MPA

In order to make use of the results obtained from the analyses a bilinearisation of the pushover curves is needed, based on the N2 method as described in Chapter 2. The results are presented in Figure 82.
From the bilinear curves, the information needed in order to continue with the Lin & Miranda method to calculate the inelastic displacement demand for each earthquake are the yield strength $f_y$ and the maximum displacement. The two displacements will be compared for the derivation of the fragility curves.
10 APPLICATION OF THE PROBABILISTIC SEISMIC HAZARD ASSESSMENT

The purpose of the present chapter is to make an application of the seismic hazard assessment presented in Chapter 5 to the area of Groningen. The acceleration response spectra are going to be derived, using Jayaram’s method which is described in Chapter 5.6 in order to illustrate the seismic actions which will be used further to determine the displacement demand for all the given structures.

10.1 INPUT GIVEN FOR THE GROUND MOTION PREDICTION

The parameters needed in order to derive the ground motion for the specific area of Groningen, based on Chapter 5.6, are determined following:

- **Magnitude of the earthquake M<sub>W</sub>:** the different magnitudes that are examined are all the values between 4 and 7.5 with an incremental step of 0.025 giving a number of 140 values.
- **Source-to-site distance R (km):** the distances that can be used are the epicentral (R<sub>epi</sub>), hypocentral (R<sub>hyp</sub>) and Joyner-Boore (R<sub>JB</sub>). Here, the hypocentral distance is used with R<sub>hyp</sub> = 10 km and is treated as constant since the specific location, where the masonry structure is built, is examined.
- **ε:** The value of ε is determined such, that in conjunction with the values of magnitude M<sub>W</sub> and distance R, to give the maximum likelihood of the severest earthquake to occur. Thus, after a full probabilistic seismic hazard analysis in the area of Groningen, done by NAM, three values of ε should be used depending on the value of the peak ground acceleration (PGA). The three areas are as follows:
  - ε=0.8 for 0 < PGA < 0.15g
  - ε=1.25 for 0.15g < PGA < 0.4g
  - ε=1.75 for PGA > 0.4g
- **Style-of-fault dummy variables F<sub>N</sub>, F<sub>R</sub>:**
  - F<sub>N</sub>=1 and F<sub>R</sub>=0 for normal faults: used in the present study
  - F<sub>N</sub>=0 and F<sub>R</sub>=1 for reverse faults
- **V<sub>S30</sub> = 185 m/s**
- **V<sub>REF</sub>=750 m/s : the reference of V<sub>S30</sub>**
- **V<sub>CON</sub>=1000 m/s : used for limiting V<sub>S30</sub> after which the site amplification is constant**

10.2 RESULTS

Following, the results after applying Jayaram’s method are presented, both in linear and logarithmic scale.

![Acceleration Response Spectra Based on Jayaram's Method: Linear Scale](image)
FIGURE B4: ACCELERATION-DISPLACEMENT SPECTRA BASED ON JAYARAM’S METHOD: LINEAR SCALE

FIGURE B5: ACCELERATION RESPONSE SPECTRA BASED ON JAYARAM’S METHOD: LOGARITHMIC SCALE
From the Monte Carlo simulations, 1000 correlated conditional spectra were extracted based on the conditional mean spectrum. Thus results is 1001 different curves for each earthquake (each magnitude), which makes 140 graphs of 1001 curves each, thus 140140 spectra.
11 CALCULATION OF THE INELASTIC DISPLACEMENT DEMAND: APPLICATION OF LIN & MIRANDA METHOD

The proposed equivalent linear systems is useful for the evaluation of existing buildings using a nonlinear static procedure. In the present chapter an application of the noniterative procedure of Lin & Miranda will be done, in order to determine the maximum inelastic displacement demand of the masonry building. As a reference, the application will be done only for the case described in Chapter 8.5 where the mean values of the material properties are used. The same procedure is followed for the rest of the material properties (given in Chapter 9.4) and the results are directly provided in Chapter 12 as fragility curves.

1. Execute pushover analysis and derive the pushover curve (base shear-roof displacement) (Chapter 8.5.2):

![Figure 87: Pushover curve of the masonry structure with the mean values of the material properties](image)

2. Bilinearise the obtained pushover curve and derive the yield strength $f_y$, the postyield stiffness $\alpha$ and the strength ratio $R$, where $R = \frac{mS_2}{f_y}C_m$:

![Figure 88: Bilinearization pushover curve of the masonry structure with the mean values of the material properties](image)
The postyield stiffness $\alpha$ is 0 since in the bilinear curve the second part is horizontal based on the N2 method described in Chapter 3.2.3.3.

The yield strength is taken from the bilinear curve and is equal to $f_y=3.37574$ kN. However, the normalized value of the yield strength will be used, since in the formula of $R$ it is divided by the mass of the system. Thus, it will be:

$\frac{f_y}{mg} = \frac{3.37574 \times 1000}{8150 \times 9.81} = 0.0422 \text{ W}$, where $W$ is the total weight of the frame

$C_n=0.77716$

The eigenperiod of the structure is taken form the eigenvalue analysis given in chapter 8.5.1.1 (first eigenperiod)

$T_0 = 0.79 \text{ sec} \Rightarrow S_a = 1.48 \text{ m/s}^2$

The strength ratio can now be calculated:

$R = \frac{mS_a C_m}{f_y} = \frac{W(1.48)(0.77716)}{(0.0422 \text{ Wg})} \Rightarrow R=27.62$

3. Compute the equivalent period $T_{eq}$ and damping $\xi_{eq}$ using the equations (8) and (9) of Chapter 4.1.

$T_{eq}=10.67 \text{ sec}$

$\xi_{eq} = 55.9 \%$

4. Construct the $\xi_{eq}$-damped elastic displacement spectrum $S_a(T,\xi_{eq})$ which can be obtained from the 5%-damped elastic design spectrum $S_a(T,\xi=5\%)$ and the damping modification factor $B$, which is calculated from Table 15.6-1 of NEHRP-2003 and is given in Table 5.

$S_a(T_{eq}, \xi = 5\%) = 0.04 \text{ m/s}^2$: ordinate of the 5%-damped elastic design spectrum at the period $T_{eq}=10.67 \text{ sec}$.

From Table 5 the damping modification factor results: $B=2.58$

5. The maximum displacement demand ($u_1$) for the 1st mode is the displacement corresponding to the period $T_{eq}$ in the $\xi_{eq}$-damped elastic displacement spectrum:

$u_1 = S_a(T_{eq}, \xi_{eq}) = \frac{T_{eq}^2}{4\pi^2} S_a(T_{eq}, \xi=5\%) = 0.045 \text{ m}$

6. Convert the displacement obtained from step 5 to global (roof) displacement ($u$) based on the square-root-of-the-sum-of-the-square rule as

$u = \sqrt{\sum_{j=1}^{N} (u_j \cdot \Gamma_j)^2} = 0.777 \times 0.045 \times 0.03497 \text{ m} > 0.03192 \text{ m}$

Where: $\Gamma_1 = 0.777$: modal participation factor for the 1st mode

$N=1$: total number of modes considered

The result form this procedure was that for the given earthquake, the reference masonry structure is collapsing.

The above result can also be observed from Figure 89, where the Capacity spectrum method that is actually being applied (and is described in Chapter 3.2.3.2) is applied.
From Figure 89 it is obvious that the inelastic displacement demand is larger than the displacement capacity of the structure, which leads to failure. It is worth mentioning, that the spectral accelerations are computed for $\varepsilon=1.75$. 
12 DERIVATION OF THE FRAGILITY CURVES

After the application of the capacity spectrum method for a masonry structure with variability in the material properties (ten different cases), and variability in the response spectra coming from the earthquake (1001 different spectra for each earthquake magnitude), the fragility curves can be derived.

The procedure followed in Chapter 11 is applied for all the spectra 1001 of each earthquake and then for all the different cases (ten in total) derived from the variation in the masonry material properties. As described in Chapter 10.1 the procedure will be followed for three different values of $\varepsilon$ (0.8, 1.25, 1.75), as follows:

- $M_w$: 4-7.5 with a step of 0.025, $R=10$ km, $\varepsilon=0.8$, $V_{s30}=185$ m/s$^2$ for $0 < PGA < 0.15g$
- $M_w$: 4-7.5 with a step of 0.025, $R=10$ km, $\varepsilon=1.25$, $V_{s30}=185$ m/s$^2$ for $0.15g < PGA < 0.4g$
- $M_w$: 4-7.5 with a step of 0.025, $R=10$ km, $\varepsilon=1.75$, $V_{s30}=185$ m/s$^2$ for $0.4g < PGA$

When this procedure is completed the probability of failure is calculated for each earthquake, which can be translated to each Peak Ground Acceleration (PGA), for all different values of $\varepsilon$. The result can be plotted in a graph PGA in x-axis and the probability of failure in y-axis. The outcome is three different fragility curves for each of the ten different combinations of material properties, presented in Figure 90.

The procedure that was followed and the detailed scripts of MATLAB can be found in Appendix A and are described in Flowcharts A and B.
FLOWCHART B: Capacity Spectrum method application procedure

Apply the Capacity Spectrum Method for 1 structure and 1 response spectrum of 1 magnitude for 1 ε

- **U\(_{\text{demand}}\) > U\(_{\text{capacity}}\)?**

- **NO**
  - Repeat for all the capacity curves (10)
  - Combine the 3 areas of the fragility Curve
  - Calculate the Mean fragility curve from the 10 capacity curves

- **YES**
  - Failure
  - Calculate the Probability of Failure for the given magnitude
  - Calculate the Probability of Failure for the 140 magnitudes
  - Plot the 3 fragility curves: x-axis: 140 values of PGA, y-axis: Probability of failure
  - Repeat for all the values of ε
  - Repeat for 1001 response spectra of the given magnitude
  - Repeat for 140 magnitudes

**Note:**
- Repeat for a ll the values of ε
- Plot the 3 fragility curves: x-axis: 140 values of PGA, y-axis: Probability of failure
- Combine the 3 areas of the fragility Curve
- Calculate the Mean fragility curve from the 10 capacity curves
The three fragility curves that are obtained, are now combined to one fragility curve as described in the former paragraph. The combined fragility curves for all the (ten) combinations of the material properties are presented in Figure 91. Finally, the mean fragility curve can be calculated from the ten curves representing the different material properties, and the result is a fragility curve which includes the variability of both the material properties of masonry and the ground motion and is presented in Figure 92.

After the points of the fragility curves are derived, the obtained curve is fitted by a lognormal distribution. The type of the distribution to fit was chosen such that agrees with the distributions fitting of fragility curves in literature. The procedure followed is described further:

\[
\text{fragility curve } \equiv F_x(x) = \Phi \left( \frac{\ln(x) - \mu_y}{\sigma_y} \right), \quad F_x(x) \sim LN(\mu_y, \sigma_y)
\]

The values \( \mu_y \) and \( \sigma_y \) represent the mean and standard deviation, which are needed in order to describe the lognormal distribution and they are calculated using the least-squares solution of the inconsistent system \( Ax=b \), which coincides with the nonempty set of solutions of the normal equations \( A^T A \mathbf{x} = A^T \mathbf{b} \), as follows:

\[
\Phi^{-1} (F_x(x)) = \frac{\ln(x) - \mu_y}{\sigma_y} = \ln(x) \cdot \frac{1}{\sigma_y} - \frac{\mu_y}{\sigma_y}
\]

\[
y = A \cdot x
\]

Where:
\[ y = \begin{bmatrix} y_1 = \Phi^{-1}(F_X(x_1)) \\ y_2 = \Phi^{-1}(F_X(x_2)) \\ \vdots \\ y_n = \Phi^{-1}(F_X(x_n)) \end{bmatrix}, \quad x = \begin{bmatrix} x_1 = 1/\sigma_y \\ x_2 = \mu_y/\sigma_y \end{bmatrix}, \quad \text{and} \quad A = \begin{bmatrix} \ln(x_1) & -1 \\ \ln(x_2) & -1 \\ \vdots & \vdots \\ \ln(x_n) & -1 \end{bmatrix} \]

Then:

\[ x = (A^T A)^{-1} A^T \cdot y \]

\[ \mu_y = x_2/x_1 \]

\[ \sigma_y = 1/x_1 \]
FIGURE 91: FRAGILITY CURVES FOR THE TEN CASES OF MATERIAL PROPERTIES
The lognormal fit for the mean fragility curve can be found in Figure 93. With parameters $\mu_y = -0.3159$ and $\sigma_y = 0.362$. The extreme values of the lognormal fits are $\mu_y = -0.3694$ and $\sigma_y = 0.3617$ for $f_t = 0.16$ MPa and $\mu_y = -0.3836$ and $\sigma_y = 0.3558$ for $f_t = 0.2466$ MPa.
13 Further Sensitivity Study

During the execution of the present study several assumptions have been made, without knowing the exact effect that they have on the final result, which is the fragility curve. Thus, some further sensitivity studies have been made in order to get an image of the stability of the results after changing some assumptions. Three different cases are examined, which will be explained in the following paragraphs:

13.1 Influence of the Lateral Load Distribution Shape

The first sensitivity study is done for the distribution of the lateral loads in the pushover analysis. As described in Chapter 7 the analyses were done based on uniform lateral load distribution. However, in such analyses it is very common to use the shape of the first mode. Such an analysis was done and the comparison is made in Figure 94, where both pushover curves are presented.

The two analyses are very close, with maximum displacement capacity 0.03193 m for the new case, instead of 0.03192 m for the original case with uniform load distribution.

From this comparison, the conclusion can arise that the lateral load distribution does not have an effect on the result. Thus, the simplification is acceptable.

13.2 Influence of Choice of the Drift Limit that Corresponds to the Near Collapse Limit State

As described in Chapter 6.3 there are many different drift limits that can be used in order to describe the Near Collapse Limit State. The one chosen in the present study is when the building resistance deteriorates below an acceptable limit, 80% of the maximum resistance. The importance of this choice is examined by producing the fragility curve for a given structure for a new drift limit, which in this case will be the interstorey drift of 0.6%H. This value is calculated based on the relation between the drifts of each floor from the eigenvalue analysis and has been derived 0.0204 m, instead of 0.03192 m of the original case. The result can be seen in Figure 95.
Obviously, the two fragility curves (as well as their lognormal fits) are close enough, with a difference that could be adopted by the variability of the material properties of masonry.

13.3 INFLUENCE OF THE CHOICE OF MODULUS OF ELASTICITY IN THE SENSITIVITY ANALYSIS OF THE MATERIAL PROPERTIES

In Chapter 9, where the sensitivity study of the masonry material properties is done, some equations are presented that relate each material property to the random variable that is in this case the tensile strength. However, it is known from literature that the modulus of elasticity has a coefficient of variation equal to 10%. This value does not agree with the equation used $E=1000f_c$, which is taken from Eurocode 6, since it is known that $f_t$ and thus $f_c$, has a coefficient of variation equal to 20%. The importance of this assumption (the use of this equation for $E$) is examined in the present paragraph, where the fragility curve will be obtained for the case that $E$ has CoV=10% and will be compared to the original one.

In order to do so, the case where the tensile strength is taking the value $\mu+\sigma=0.24$ MPa will be examined. In this case, $E$ will be 3850 MPa instead of 4800 MPa of the original case. The result of the analyses can be seen in Figure 96, where the pushover curves (and the bilinear curve) for each case is presented. Following, the capacity spectrum method will be applied again for the two cases and the results will be compared by plotting both fragility curves in the same graph (Figure 97).
It can be observed that the two fragility curves (as well as their lognormal fits) are very close, with a difference that could be adopted by the variability of the material properties of masonry. Compared to the sensitivity studies done in Chapters 13.1 and 13.2, this one has smaller effect than the choice of the drift limit but larger that the lateral load distribution.
14 CONCLUSIONS-RECOMMENDATIONS

The objective of this thesis was to produce the fragility curves for the masonry series of Rijtjeshuis, which are affected by earthquakes in the area of Groningen taking into account the variability in the material properties of masonry as well as the variability in ground motions. The results obtained have been presented in the previous chapters and result in the following conclusions and recommendations for further studies:

14.1 DISCUSSION OF THE ASSUMPTIONS AND CONCLUSIONS

The derivation of the fragility curves is based on different methods and assumption, which are not all proved to be accurate. Each of them will be discussed following:

1. Modelling of the structure: a real case of masonry structure is used for the creation of FEM model. However, not the whole structure was modelled but part of it, and some boundary conditions were adopted in order to make use of the continuity of the structure in the two directions. These boundary conditions reveal continuous reinforced concrete floors through the houses, which is not the case in reality, since hollow core slabs are used in reality, with only in 4 cores over the length of the wall a connection with reinforcement bars. This rises the uncertainties of the created model, making it non-conservative. The fact that the shear walls are ignored, though, makes the model more conservative. The combination of the two assumptions reveals an acceptable model comparable to models that represent whole structures.

2. The connections between floors and wall in reality are not fixed. This study makes the assumption that they are fixed but due to the big difference in the strength of concrete and masonry the cracks will develop in the masonry wall and thus the real behaviour of the structure will arise. A more detailed model however, where the connection of the floors and walls follow the sequence of their construction, would give more reliable regards, although the one used can be considered acceptable.

3. The present model is making a macroscopic modelling of the masonry structure. A microscopic one would give more accurate results about the crack formation time and location.

4. In the present model a hinge was adopted as foundation. The real one though, may not be complete hinge but can resist some moments. The use of a rotational spring at the base of the structure would have an influence in the pushover curve and thus in the capacity of the structure.

5. Conditional nonlinear static pushover analyses were executed for the determination of the capacity of the structure. Although this is very simplified analysis, the accuracy of the results is acceptable if considered with the small computation time that it needs compares to other methods. A more accurate result could have been obtained if the adaptive pushover analysis was used, where the material properties of the structure would change in every load step. Full dynamic analyses also provide the most accurate results, but are not needed for the scope of the present study.

6. In the concrete model the structure fails when a plastic hinge is formed (step 536): A plastic hinge is created on a rigid connection when the number and width of the cracks are such that do not allow for the connection to transmit any moments. Thus, the connection becomes a hinge and consequently, the moment distribution along the height of the first floor (wall between two hinges – foundation and plastic hinge) is zero.

7. The masonry structure is failing out of plane due to rocking mechanism: the wall is firstly uncracked and the point around which the rocking is activated has finite dimensions.
8. The masonry models that have been used for the description of the tensile and compressive behaviour are the exponential and the parabolic ones respectively. Other models are also present in literature, something that can be examined further.

9. The tensile strength is considered the dominant random variable. However, this is not validated and should be examined further.

10. Full correlation is assumed between the different material properties and the tensile strength. The validity of this assumption, although questionable, gives reasonable results.

11. The equations used to relate the different material properties are taken from literature. Other equations are also available but only for some of the material properties. The sensitivity study done in Chapter 13.3 revealed that the influence of the choice of the modulus of elasticity is not affecting a lot fragility curves.

12. Only ten different cases of masonry structures are used. The more the cases are, the more reliable the final mean fragility curve becomes. The ten cases are enough for the present study, since they cover the whole range of the possible material properties based on the coefficient of variation of the tensile strength of masonry.

13. Only one direction of the earthquake was examined, the one considered weakest. The other direction should also be examined and the two obtained results should be combined using the SRSS method (Square Root of the Sum of the Squares).

14. Lin and Miranda method was used for the determination of the inelastic displacement demand of each earthquake. The method clearly states that can be used for all ranges of periods, but with lower accuracy when the eigenperiod is lower than 0.8 sec. However, even in these case the error is still very small and acceptable.

15. Probabilistic Seismic Hazard Assessment: as described in previous chapters the highest likelihood of appearance of an earthquake is represented by three values: \([M, r, \varepsilon]\). In the present study, a PSHA was used, were the distance \(R\) is constant and the value of \(\varepsilon\) takes three values depending on the level of peak ground acceleration. In a more detailed study full probabilistic seismic hazard assessment should have been done. Thus, the drop in the fragility curves observed at the point where the two areas are met could have been avoided.

16. The fragility curves are fitted by lognormal distributions as found in literature. The examination of other distributions to fit the fragility curves is an option for future studies, although not being the scope of this thesis.

14.2 RECOMMENDATIONS FOR FUTURE STUDIES

It is recommended that further studies on the present topic will take into consideration the following aspects:

1. The shear walls are totally ignored, something that makes the model conservative. However, the reinforced concrete floors are assumed to be continuous, which is not true in the real structures and increases the capacity of the model. Thus, a more detailed representation of the structure would give a more realistic behaviour of the structure.

2. Full scale experiments of a masonry house can be done, with known material properties under given ground motions. Thus the correctness of the obtained fragility curves can be derived.

3. Lin & Miranda method can examined further in order to reduce the error at the cases where the period is lower than 0.8 sec.

4. More explicit research on the relationships between the material properties: experiments should be done to validate their applicability for the range of their values and not only for the mean values.

5. Closer look to the correlation between the material properties and their distribution functions: research should be done on how each material property is dependent, or not, on each of the other ones.

6. The tensile strength is considered the dominant random variable: other material properties can also be assumed as dominant and the difference in the results can reveal the relationships among them.
7. The equations used to relate the different material properties should be examined and validated through experiments.
15 LITERATURE RESEARCH


NAM, N. A. (November 2012). *Study and Data Acquisition Plan for Induced Seismicity in Groningen*.


APPENDIX A

%----------------------------------
% Script to compute the response spectrum accelerations (displacements)
% based on the Akkar 2014 GMPE model.
% Correlation-data from Akkar Bommer 2013.
%----------------------------------

% Required input parameters
M = (4:0.025:7.5);
R = 10;
%epsilon = 1.75;
Vs30 = 185;
FN = 1; FR = 0; % (FN,FR)=(1,0) or (FN,FR)=(0,1) or (FN,FR)=(0,0).
N(ormal) and R(everse) faults and SS.
Tstar = 0;
used_model = 'hyp'; % used_model = {'JB','epi','hyp'}

% Fixed parameters
V_ref = 750; V_con = 1000;
a2 = 0.0029; a5 = 0.2529; a6 = 7.5;
a7 = -0.5096; c1 = 6.75; c = 2.5; n = 3.2;

% Read model parameters.xlsx into Matlab.
if ~exist('GMPE_Models','var')
    GMPE_Models = read_model_parameters('model_parameters.xlsx');
end

% Use the parameters corresponding to the used model
eval(['model = GMPE_Models.' used_model ' ;']);
fields = fieldnames(model);
for i=1:length(fields)
    eval([fields(i) ' = model.' fields(i) ' ;']);
end

% The Ground-Motion Predictive Equation (Akkar 2014)
%GMPE;
EPSILON = [0.8,1.25,1.75];
P(numel(EPSILON),length(M)) = 0;
for e = 1:numel(EPSILON)
    epsilon = EPSILON(e);
    % Two cases for lnY_ref
    for j = 1:length(M);
        if M(j) <= c1
            a_27(j) = a2;
        else
            a_27(j) = a7;
        end
    end

    S=0;
    numberofSamples = length(a1);
    lnY_ref = zeros(numberofSamples,length(M));
    lnS = zeros(numberofSamples,length(M));
    for j = 1:length(M);
        for i=1:numberofSamples

```
\[
\ln Y_{\text{ref}}(i,j) = a_1(i) + a_{27}(j) * (M(j) - c_1) + a_3(i) * (8.5 - M(j))^2 + (a_4(i) + a_5 * (M(j) - c_1)) * \log(\sqrt{R^2 + a_6^2}) + a_8(i) * FN + a_9(i) * FR + S;
\]
end

PGA_{\text{ref}} = \exp(\ln Y_{\text{ref}});

% Two cases for \( \ln S \)
for j = 1:length(M);
    if Vs30 <= V_{\text{ref}}
        for i = 1:numberofSamples
            \ln S(i,j) = b_1(i) * \log(Vs30 / V_{\text{ref}})
        end
    else
        for i = 1:numberofSamples
            \ln S(i,j) = b_1(i) * \log(\min(Vs30, V_{\text{con}}) / V_{\text{ref}});
        end
    end
end

for j = 1:length(M);
    sigma_{\text{2}}(:,j) = sigma(:,);
    sigma_{\text{check}}(:,j) = (\tau(:,).^2 + \phi(:,).^2).^0.5;
end

for j = 1:length(M);
    if norm(sigma_{\text{2}}(:,j) - sigma_{\text{check}}(:,j))/norm(sigma_{\text{2}}(:,j)) > 1E-3
        warning('Something may be wrong');
        sigma_{\text{eff}}(:,j) = sigma_{\text{2}}(:,j);
    else
        sigma_{\text{eff}}(:,j) = sigma_{\text{check}}(:,j);
    end
end

\ln Y = \ln Y_{\text{ref}} + \ln S + \varepsilon * \sigma_{\text{eff}};
\ln Y_{\text{e}}(:, :, e) = \ln Y;

% Monte carlo simulation and results
% statistical_sa_analysis;

close all;

% Statistical parameters
for j = 1:length(M);
    mu_{\text{T}_j}( :, j) = lnY(2:end-1, j);
    mu_{\text{T}_\star}( :, j) = lnY(1, j);
    sigma_{\text{T}_j}( :, j) = sigma_{\text{eff}}(2:end-1, j);
    sigma_{\text{T}_\star}( :, j) = sigma_{\text{eff}}(1, j);
end

for j = 1:length(M);
    covari = sigma_{\text{eff}}(1:end-1, j) * sigma_{\text{eff}}(1:end-1, j)'; * GMPE_Models.Correlations;
    Sigma11( :, :, j) = covari(2:end, 2:end);
    Sigma12( :, :, j) = covari(2:end, 1:end);
    Sigma21( :, :, j) = covari(1:end, 2:end);
    Sigma22( :, :, j) = covari(1:end, 1:end);
end
for j=1:length(M);
    Mu = mu_Tj(:,j) + Sigma2(:,1,j) .* epsilon.* sigma_Tj(:,j);
    Sigma(:,j) = Sigma11(:,j) - (1/Sigma22(1,1,j)) * Sigma2(:,1,j)*Sigma21(1,:,j);
    sigmas(:,j) = sqrt(diag(Sigma(:,:,j)));
end

for j=1:length(M);
    lnX(:,:,j) = lhsnorm(mu_Tj(:,j),Sigma(:,:,j),1000);
end

X(:,:,j)=exp(lnX(:,:,j)');

% Scaling wrt 5%
xi = 5;
eta = max(0.55,sqrt(10/(5+xi)));
X=eta*X;

SD1 = X;
g = 9.81;

for i=1:size(SD1,1)
    SD1(i,:) = SD1(i,:) * g * (Periods(i+1)/(2*pi))^2;
end

% Find interpolated sa values for given T
T0 = 0.692473;
% T1 = 0.75;
% T2 = 1;

% sa(100)=0;
for i1=1:length(M)
    for j=1:1000
        saj(i1,j)=interp1(Periods(2:end-1),X(:,j,i1),T0);
    end
end

for i2=1:length(M)
    saj(i2,j)=interp1(Periods(2:end-1),X(:,j,i2),Teq(i2,j)); % for Teq>4 saj gives NaN
end
clear  j  i2

u1 = (Teq/(2*pi)).^2 .* sajeq ./ Bj;  % also includes NaN values

U = Cm * u1;  % also includes NaN values

% Probability of exceedance
Ud = 0.023855;
P(length(M)) = 0;
for  i= 1:length(M);
  for  j=1:1000;
    if  (U(i,j) >= Ud) || (isnan(U(i,j)))  % it is assumed that for
      U(i,j)==NaN value will be higher
        P(e,i) = P(e,i) + 1;
    end
  end
end

%%
Y = exp(lnY_e);
for  e=1:numel(EPSILON)
  for  i=1:length(M)
    Y_axis(e,i) = Y(1,i,e);
  end
end

%Combination of the 3 areas
Y_new=Y_axis(1,1:23) Y_axis(2,8:56) Y_axis(3,38:end));
P_new=[P(1,1:23) P(2,8:56) P(3,38:end)]/1000;

figure;
plot(Y_new(1,:),P_new(1,:),'-o','LineWidth',2,'MarkerSize',5);
figure; hold all;
plot(Y_axis(1,:),P(1,:),'-o','LineWidth',2,'MarkerSize',5);
plot(Y_axis(2,:),P(2,:),'-o','LineWidth',2,'MarkerSize',5);
plot(Y_axis(3,:),P(3,:),'-o','LineWidth',2,'MarkerSize',5);
xlabel('PGA [g]');
ylabel('Probability of exceedance [%]');
title('Fragility curve for T=0,79 sec');

%% fit lognormal ditribution
y = norminv(P_new(43:176));
for  k=1:134
  A(k,1)=log(Y_new(k+42));
  A(k,2)=-1;
end
x=inv(A'*A)*A'*y;
m=x(2)/x(1);
sigma__=1/x(1);
z=(log(Y_new)-m)/sigma__;
Fx=normcdf(z);
figure; hold all
plot(Y_new,Fx);
plot(Y_new,P_new);
APPENDIX B

CONCRETE MODEL

THE DEFLECTED SHAPE OF THE STRUCTURE AT POINTS A, B, C, D AND E
DISTRIBUTION OF MOMENTS ALONG THE FRAME AT POINTS A, B, C, D AND E
DISTRIBUTION OF NORMAL FORCES ALONG THE FRAME AT POINTS A, B, C, D AND E
Distribution of shear forces along the frame at points A, B, C, D and E
Maximum principal strains for the concrete at points A, B, C, D and E.
Maximum principal strains for the reinforcement at points A, B, C, D and E
ELEMENT STATUS AT POINTS A, B, C, D AND E

- P Partially open
  - loading
- PI Partially open
  - unloading
- O Fully open
  - loading
- OI Fully open
  - unloading
- C Closed
- No crack yet
MASONRY MODEL

THE DEFLECTED SHAPE OF THE STRUCTURE AT POINTS A, B, C, D AND E
DISTRIBUTION OF MOMENTS ALONG THE FRAME AT POINTS A, B, C, D AND E
DISTRIBUTION OF NORMAL FORCES ALONG THE FRAME AT POINTS A, B, C, D AND E
DISTRIBUTION OF SHEAR FORCES ALONG THE FRAME AT POINTS A, B, C, D AND E
Strains for the masonry at points A, B, C, D and E
ELEMENT STATUS AT POINTS A, B, C, D AND E

- Partially open
- loading
- unloading
- Fully open
- loading
- unloading
- Closed
- No crack yet!
SIDE VIEW
GROUND FLOOR PLAN
FIRST FLOOR PLAN
SECOND FLOOR PLAN

- Dimensions:
  - 6400
  - 120
  - 60
  - 5100
  - 120
  - 50
  - 120

- Annotations:
  - Krisp vl d=230mm

- Scale:
  - 1:100

- Details:
  - Structural elements
  - Annotations for dimensions and references
SECTION B-B