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Modelling the seismic response of an unreinforced masonry structure

THESIS

In order to obtain the degree Master of Science

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Normal crack strain in one of the last steps of the cyclic pushover analysis
PREFA CE

This thesis is written in order to obtain my Master’s degree in Civil Engineering at Delft University of Technology. The subject of this report is the modelling the seismic response of an unreinforced masonry structure.

The research is carried out under the guidance of Delft University of Technology and Witteveen+Bos from October 2014 to June 2015. The office of Witteveen+Bos in Amsterdam has provided me a workplace and resources to perform this research. Delft University of Technology provided me the license for the DIANA software.

I would like to thank my assessment committee; Jan, Max, Valentina, Eliam and Roel, for guiding me during this process. Furthermore, I would like to thank my family, friends and Witteveen+Bos colleagues for their support.

Wilco van der Mersch
In the last years, increasingly more earthquakes have occurred in north-east Groningen. These earthquakes are induced by gas extraction in the Groningen gas field. One of the problems that is caused by these earthquakes is the damage to masonry houses. The effect of these earthquakes on peoples lives makes research to them urgent.

Finite element analyses can be used to understand and predict the complex behaviour of these kind of structures. Validations against experimental tests have to be carried out in order to build a model that represents reality. There are already quite some benchmarks available in literature that contain such validations.

One of them is the large scale earthquake test on a building in the Enhanced Safety and Efficient Construction of Masonry Structures in Europe (ESECMaSE) project [5–7]. The structure that is considered there, is half of a two floor terraced house. It is composed of calcium silicate masonry walls and reinforced concrete floors. The finite element model in this study is based on this structure.

There are roughly two modelling approaches described in literature; micro and macro modelling. The main difference is the scale on which masonry is modelled. In the latter, that is also used in this report, the interaction between units and bricks is smeared out across all the elements.

The main question that will be answered in this report is: How can the behaviour of an unreinforced masonry terraced house under an earthquake load be modelled with a smeared-crack model?

This question is approached from the following perspectives: to which extent can the behaviour be modelled and which approach gives the closest approximation?

A finite element DIANA model is build from solid elements and a total strain fixed crack model. The seismic load is modelled to fit the experiments. It is quasi-static and applied with the use of an auxiliary frame attached to each floor. The constraints of the frame enable it to rotate, but also to keep the ratio of the forces on the floors constant. This ratio is the first mode shape that follows from an eigenvalue analysis. The imposed displacement of the frame can either be monotonic or cyclic.

Roughly three different analyses were performed; an eigenvalue analysis, monotonic pushover analyses and a cyclic pushover analysis. The mode shape, eigenfrequency, shear capacity and crack patterns were used to compare the results with the ESECMaSE project.

The first analysis that was carried out was the eigenvalue analysis. Only the fourth of the first six mode shapes from this analysis was not equal to the estimated mode shapes in ESECMaSE. The corresponding eigenfrequencies of the numerical model approximated the frequencies from the hammer impact test up to 8%. The sensitivity of the first four frequencies with respect to the Young’s modulus of masonry showed that there is not a single value for which this error is zero.

The next analyses were seismic and with increasing complexity. The first one was a Monotonic Pushover Analysis (MPOA). Different stages in the behaviour of the model were obtained from the shear-drift curves of this analysis. These stages are:

1. Initiation stage
2. Pseudo-linear stage
3. First severe crack stage
4. Crack propagation stage
5. Collapse stage
The initiation stage was different from the others, because in that stage, only the gravity and live load are applied. The shear-capacity of floor 1 and 2 followed from the crack propagation stage. They were respectively 45% and 60% higher than in the Pseudo-dynamic (PsD) test. The crack patterns in the MPOA were similar to the observations in the test.

The evolution of modal parameters was researched through stopping the MPOA at several points and subsequently performing an eigenvalue analysis. The mode shape and fundamental frequency that were obtained showed how the structure’s dynamic properties changed during the MPOA. The formation of cracks lead to a lower eigenfrequency and a larger displacement of the first floor with respect to the second floor. However, these changes appeared to be small and were therefore neglected.

The model in the MPOA followed from a sensitivity study to modelling properties, such as the type of smeared-crack model, Young’s modulus, tensile strength, fracture energy, tensile softening curve, damage based shear retention and load application method.

It became clear that the type of smeared-crack model (fixed single, rotating single or multi-directional fixed) has only a small influence on the shear capacity. The effect of the Young’s modulus on the shear-drift curve is even negligible. Higher tensile strengths resulted in more brittle behaviour. Models with a lower mode I fracture energy also had a lower deformation capacity. The effect of the type of softening was negligible.

Damage based shear retention and damage based Poisson’s ratio reduction have a more significant influence. Both phenomenons are present in real masonry. A constant shear retention factor shows an almost linearly increasing capacity as the structure deforms. This capacity developed far beyond the one from the experimental model and the corresponding crack patterns can be called unrealistic.

The modal pushover analysis is a DIANA built-in seismic analysis. The load vector is based on the contribution of each node to the mode shape. This analysis is force controlled and unable to overcome the formation of large cracks. This makes it unable to compute a shear capacity that is based on a horizontal plateau in the shear-drift curve. This is therefore not suitable to model the behaviour from the PsD test in the ESECMaSE project.

The most complex analysis that was performed was the Cyclic Pushover Analysis (CPOA). The shear-drift curve from that analysis followed a similar path as in the PsD test. The response was different from the MPOA in the sense that the shear force after the formation of large cracks was lower. The difference in shear capacity between CPOA and PsD test in the weakest direction of the structure is 17% for floor 1 and 29% for floor 2. In the exact opposite direction, the difference was respectively 40% and 49%. This analysis is considered to be the best approximation to the behaviour in the PsD test.

It is recommended that further research focusses on time-dependent behaviour. Such behaviour was not included here because it was also not taken into account in the ESECMaSE project. The parameter viscous damping is related to this behaviour. A Time History Analysis (THA) includes this type of behaviour and is therefore more equal to that of a real earthquake. In the case of Groningen, research should also be carried out to circumstances typical for that area, such as clay units, cavity walls, soil-structure interaction and induced-earthquake loading.
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A few years ago, the amount of research to earthquakes in the Netherlands was very small. Because there were almost no earthquakes, there was no need to research this phenomenon. This was also the reason why buildings were generally not designed on earthquake loads. Many structures in the area are masonry houses. The earthquakes have lead to large cracks in these houses. Temporary strengthening measures, such as in Figure 1.1, are not uncommon. The number of houses that should be strengthened in this area according to new earthquake regulations, which are currently in the design phase, is possibly more than 150,000.

Figure 1.1: 26 June 2014, a house in Fraamklap Groningen is severely damaged due to the earthquakes. Picture by Kees van de Veen, Hollandse Hoogte

It effects peoples lives so much, that solving this problem is now a priority to the government. There is a technological challenge in evaluating the behaviour of masonry structures under seismic loading. Despite the fact that these earthquakes are induced, the effect they have on structures is difficult to predict.

Finite element models can be an aid in understanding the effect of an earthquake on a certain structure. A lot of research has already been performed to model earthquakes in a finite element model. Such numerical models are able to predict the behaviour of a structure to some extent and to assess the response to a theoretical earthquake. Building these models, especially for masonry structures, can be very difficult. The difficulty lies in the approximations that have to be made and how the results are interpreted.

1.1 OBJECTIVE AND SCOPE

The validity of finite element models can only be assessed if the results are compared with the response of real-life structures. They can be either obtained from experimental tests or measurements during an actual earthquake. One of the projects in which such tests were carried out is the Enhanced Safety and Efficient Construction of Masonry Structures in Europe (ESECMaSE) project. The large scale earthquake test on a building that was performed in that project is the basis for this research. The main question that will be answered is:
How can the behaviour of an unreinforced masonry terraced house under an earthquake load be modelled with a smeared-crack model?

This questions is approached from two perspectives. They are closely related to each other and formulated in the following sub-questions:

A. Which approach gives the closest approximation?

B. To which extent can the behaviour be modelled?

The approaches that are used are variations in material-model parameters and modelling aspects. The quantities that are used to describe the behaviour are modal parameters, such as mode shape and eigenfrequency, shear capacities and crack patterns.

In order to compare the results of finite element analyses with those of the ESECMaSE project, the research is focussed on simulating the test set-up. The software package that is used to build a three-dimensional model is Displacement Analyzer (DIANA) 9.5. Furthermore, the model is smeared-cracked and composed of solid elements. The type of seismic analyses that are performed are non-linear pushover analyses.

1.2 OUTLINE OF THE CONTENTS

The first step that was carried out is a literature study to the existing research with regard to masonry numerical modelling. The wide range of masonry types requires to put research in perspective through different typologies (Chapter 2). There are various modelling techniques for masonry (Chapter 3) and its seismic behaviour (Chapter 4). Similar studies, under which the ESECMaSE project, can be regarded as benchmarks and are discussed in Chapter 5.

In the second part of this report, the properties of the numerical model are outlined (Chapter 6). The modal parameters are identified with the aid of an eigenvalue analysis (Chapter 7).

The last part comprises seismic numerical analyses. One of the most common analyses is the monotonic pushover analysis, which is discussed in Chapter 8. A sensitivity study to certain aspects is carried out with respect to this analysis (Chapter 9). Subsequently, a cyclic pushover analysis is discussed in Chapter 10.
Part I

LITERATURE STUDY

The goal of this study is to summarize existing literature on the seismic behaviour of masonry. This part starts with a discussion on different types of masonry (Chapter 2). There are several ways to model masonry and its seismic loading. These are discussed respectively in Chapter 3 and Chapter 4. The correlation between these chapters is taken into account by predominantly discussing the literature on both seismic modelling and masonry structures. Several benchmarks, in which such models are analysed, are outlined in Chapter 5.
Masonry is one of the oldest building materials that is still being used today. There are many different types of masonry. Before one starts with the modelling of masonry it is important to understand which type one is dealing with. The typology of a masonry structure can almost always be determined at first sight. The properties of certain typologies can be substantially different from each another.

The most important classifications and properties are outlined in this chapter. The available literature on masonry structures can be distinguished on the basis of these descriptions.

2.1 CONSTRUCTION METHODS

There are several construction methods. In (regular) Unreinforced masonry (URM), the behaviour is determined by the properties of the units\(^1\), mortar and their interaction. In other construction methods, such as reinforced masonry or confined masonry\(^2\), the properties of steel and concrete also play a role (see Figure 2.1).

![Figure 2.1](image-url) Classification of different masonry construction methods [10]

2.2 BONDS

Different types of bond are used in each part of the world (see Figure 2.2). The most common bond in the Netherlands is running bond. Within the different types of bonds distinctions can be made between whether the head joints\(^3\) are filled with mortar or not.

2.3 MATERIAL CLASSIFICATIONS

Other classifications can be made on the basis of masonry material. Important aspects are the age of masonry, the type of units (e.g., hollow or solid), the type of brick-material and the type of mortar. Historic masonry buildings are often composed of a great variety of materials and are often already cracked because of the loading history. Materials for bricks can, for example, be clay and calcium-silicate. Clay bricks are very common in the Netherlands because of the presence of large amounts of clay.

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1 Synonyms for this word are bricks or blocks. In the rest of this report they are generally referred to as units
2 Confined masonry is also referred to as reinforced concrete infilled frames
3 Vertical masonry joint between units
The properties of masonry’s constituents determine its mechanical behaviour. There are elastic properties, i.e. Young’s modulus and Poisson’s ratio, and inelastic properties. The structure of units and mortar generally makes masonry an orthotropic material.

The properties are obtained with the aid of experimental tests. One should realize that if the constituents are tested separately, their interaction is neglected. Especially with regard to mortar, this can be a bad approximation. The properties of mortar are considerably effected by their interaction with units during the hardening of mortar.

### Masonry unit behaviour

Masonry units are quasi-brittle materials with a disordered internal structure that contains a ‘large number of randomly oriented zones of potential failure in the form of grain boundaries’ [9]. Quasi-brittle means that after the peak load, the force gradually reverts to zero. This type of softening is characterised by the development of micro-cracks into macro-cracks. The strength and stiffness parameters of masonry units can be determined by experimental tests.

**Stiffness** Traditionally, masonry compressive behaviour is regarded as the most important kind of behaviour. The modulus of elasticity is often determined from the compressive part of the $\sigma,\varepsilon$-diagram. It is difficult to obtain the E-modulus from just the linear-elastic part of the diagram, because the development of micro-cracks, which is non-linear behaviour, already takes place under a relatively small load.

There are several ways to obtain the modulus of elasticity. In CUR171 [20], the modulus of elasticity is computed from 35% of the peak load in the $\sigma,\varepsilon$-diagram. Kaushik et al. [38] suggest a range for the elastic modulus of clay units of $150f_b \leq E_b \leq 500f_b$ in which $f_b$ is the compression strength of the unit. These values are obtained from 33% of the peak load in the $\sigma,\varepsilon$-diagram. It seems that up to that load, the behaviour of units is primarily linear elastic. The non-linear behaviour becomes significant further on in the loading process. According to Barraza [10], the modulus of elasticity for calcium silicate units can be estimated as $E_b = 355f_b$.

**Tensile behaviour of units** This behaviour can be disaggregated into two different stages:
1. **Pre-peak stage**: An elasto-plastic process in which micro-cracks develop in a stable way. At the end of this stage the peak strength $f_t$ is reached.

2. **Post-peak stage**: This stage is characterised by softening behaviour around the fracture zone. Micro-cracks develop into macro-cracks and the cracking process becomes unstable. This bridging effect is responsible for the long tail of the curve in Figure 2.3. Characteristic values of this curve are the tensile strength $f_t$ and the (mode I) fracture energy $G_f$.

![Diagram of tensile stress-strain behavior](image)

**Figure 2.3**: Typical behaviour of quasi-brittle materials under tension [9]

The tensile strength of masonry units can be obtained through several experimental tests. The most commonly used are the uniaxial, splitting, flexural and bone-shaped uniaxial tensile tests [10, 33]. Different strength parameters are obtained from the tests which can be used to describe the entire tensile behaviour.

There can be substantial differences in strength parameters because of the wide range of materials from which the units are made and the great dependence on the manufacturing method. Masonry units can behave as both heterogeneous and anisotropic materials which means that the tensile and compressive behaviour is not the same. Bakeer [9] lists several authors who have done experiments to establish the characteristic values of masonry units.

### Compressive behaviour of units

The compressive behaviour for quasi-brittle materials is characterised by the diagram in Figure 2.4. The compressive strength of masonry units is usually found through a compression test. Similar to concrete, masonry can bear compressive stresses more than tensile stresses. Tests are performed parallel and perpendicular to the bed joint\(^4\) to determine the whole behaviour of a unit [10, 33].

The strength that is obtained through tests is an artificial compressive strength. In NEN-EN1996-1-1+A1 [56, §3.6.1], this strength is normalized with respect to a cubic specimen in order to account for the direction of loading. The normalized strength is not the same as the true strength [44]. It is almost impossible to compute the real compressive fracture energy $G_c$, because the area under the curve is not finite. After crushing, the residual stress approaches a constant value in the stress-strain diagram.

### Biaxial behaviour of units

Masonry units respond differently under loading in perpendicular directions. In order to account for the total behaviour of units, biaxial, or even triaxial, tests have to be carried out [9]. Unfortunately, these kinds of tests are often omitted. In the case of special types of units, such as units with perforations, the orthotropy of the material increases the importance of knowing their biaxial behaviour [44].

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\(^4\) Horizontal masonry joints between units
2.4.2 Mortar behaviour

One of the most important influences on the mechanical properties of mortar is the proportion of its components, viz. cement, sand, lime and gypsum. Different types of mortar can be distinguished [10]:

**General Purpose Mortar** is a traditional mortar which is used in joints with a thickness larger than 3 millimetre;

**Thin Layer Mortar** is used for thinner joints, i.e. with a thickness between 1 and 3 millimetre;

**Lightweight Mortar** is made using special materials and is applied when specific requirements have to be met.

**Stiffness** Similar to masonry units, Kaushik et al. [38] recommend a range for the elastic modulus of strong mortar of $100f_m \leq E_m \leq 400f_m$. This is based on the compression strength $f_m$ and also obtained from 33% of the peak load in the $\sigma$, $\epsilon$-diagram. For weaker mortar, i.e. mortar with a higher proportion of sand, the post peak behaviour is unknown because of the 'brittle and explosive crushing failure' [38] of the specimens. This supports the presumption that the modulus of elasticity is a very sensitive parameter in masonry with small joints thicknesses [50].

**Strength** There are two ways to determine the mortar strength properties through experimental tests. One is to use bulk mortar prisms or cylinders and the other is to take disks from masonry joints. The big difference is that in the first one, the effect of water adsorption by the units is ignored. As mentioned before, the properties of mortar are highly dependent on the interaction with the units [50].

In the latter method, the behaviour of mortar can be fully characterized. It is clear that the properties from tests on bulk mortar do not represent the mortar inside the masonry composite [44]. The most simple test to carry out is a compression test. Similar to masonry units, also tests in the tensile region, e.g. an uniaxial, splitting or flexural tensile test, can be carried out [10]. Mortar strengths are around 4 to 8 MPa [38].

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**Figure 2.4:** Typical behaviour of quasi-brittle materials under uniaxial compression [9]
2.4.3 **Unit-mortar interface behaviour**

The unit-mortar interface is often the weakest link in masonry composite. Cracking is usually concentrated in these locations. The deformation capacity of masonry is therefore largely dependent on the non-linear behaviour in this interface. The interface-strength depends predominantly on the absorbency of the units, water retention capacity, porosity of mortar, amount of binder and curing conditions [9]. Detailed research was carried out by Lourenço [44] in order to determine the properties of this interface. Two separate phenomena can be distinguished, one related to tensile failure (mode I) and one to shear failure (mode II).

**Mode I behaviour** There are different methods to determine the tensile strength behaviour of the unit-mortar interface [1]. Deformation controlled tests have resulted in an exponential tension softening curve (Figure 2.5). The observed behaviour was development of micro-cracks into macro-cracks. The mode I fracture energy $G_I$ ranges from 0.005 to 0.015 $J/mm^2$. This fracture energy represents the amount of energy that is required to form a complete crack along the interface. It can be observed from the scatter of the curve that the relationship between the bond strength and the fracture energy is present but not in a very clear way [9].

![Figure 2.5: Envelope of typical stress-crack displacement results for tensile bond behaviour for clay units [44]](image)

The specimens also showed that the bond area is smaller than the cross-sectional area (Figure 2.6). The irregular bond surface is localized in the inner part of the cross-section, which could be a result from the shrinkage of the mortar and the process of laying units in the mortar [44]. Van der Pluijm [76] postulated that this localization effect is responsible for the scatter in the results.

![Figure 2.6: Tensile bond surface [44, 66]](image)

**Mode II behaviour** Just as with the tensile behaviour, the shear behaviour can be obtained through several tests [9]. The shear behaviour is characterized by a gradual decrease in strength up to a constant non-zero stress level (Figure 2.7). This property is different from mode I behaviour. Furthermore, there is a clear relationship between the peak shear strength and the compression strength normal to the bed joint. It is a progressive linear relationship between the confining stress and the mode II fracture energy $G_{II}$. This type of response is a Coulomb type of friction [50].
2.4.4 Composite behaviour

It is difficult to obtain the behaviour of masonry as a composite material, because the interaction between the constituents is complicated. Therefore, stiffness and strength parameters have to be determined from experimental tests on masonry specimens. Separate tests on units and mortar are not sufficient to capture this behaviour.

When considering the in-plane behaviour of masonry, the angle with the bed joint $\theta$ is a parameter that reflects the loading-angle. The effect of the loading-direction in masonry can be found in tests that were performed by Anthoine [3], Dhanasekar et al. [25], Grabowski [33], Page [59] (also see Figure 2.8).

![Figure 2.8: Modes of failure for biaxial loading [25]](image)

**Stiffness** The short term secant elastic modulus of masonry $E_m$ is defined as $K_E \cdot f_k$, in which $K_E$ is a factor (700 in the Dutch National Annex)[56, §3.7.2] and $f_k$ the characteristic compressive strength. Unfortunately, there is not much literature on the experimental determination of masonry stiffness properties. Kaushik et al. [38] are one of the few that paid...
attention to this topic. The proposed elastic modulus $E_m \approx 500f'_{m}$ (where $f'_{m}$ is similar to $f_k$).

The effect of anisotropy and non-linearity on stiffness is discussed in Section 3.3. It should be noted that stiffness also changes during the fracture process. The difference in initial and induced stiffness is an important characteristic that has to be taken into account when the yield function is defined [50].

**Tensile behaviour** Failure in tension is generally caused by failure of the joints. There are different test set-ups with respect to the direction of loading (Figure 2.8). The overall tensile strength of masonry is determined by the lowest value of the bond strength between unit and joint and the tensile strength of the unit [45]. The failure in tension can be divided into two types: a zigzag crack through head and bed joints (see top left sketch in Figure 2.8) and a vertical crack through unit and mortar (see middle top sketch in Figure 2.8). The post-peak behaviour is characterised by the fracture energy of the involved constituents [44].

**Compressive behaviour** Compression tests are traditionally regarded as the only relevant material test for masonry. The relatively expensive RILEM test is used the most to determine the uniaxial compressive strength normal to the bed joints [10, 33]. It is widely accepted that the difference between the elastic properties of unit and mortar influences failure most [45].

**Biaxial behaviour** The complete behaviour of masonry cannot be described by uniaxial tests only. Especially in a seismic response, the combination of the response in different directions is important to include in the model. The biaxial strength envelope has to be determined from either the full stress vector or the combination of principal (in-plane) stresses and rotation angle $\theta$ [44](also see Figure 2.9).

![Figure 2.9: Failure surface for masonry projected on $\sigma_1, \sigma_2$-plane [59]](image-url)
Several ways to model masonry are explained in the literature, each with its own difficulties. It should be noted that there is not just one correct way of modelling masonry. Each method has its advantages and disadvantages. The objective of any analysis should be determinative for the usage of an approach. There are roughly two types of approaches; micro and macro. Their difference is outlined in Section 3.1. What they entail is explained respectively in Section 3.2 and Section 3.3.

3.1 Micro versus macro approach

There are generally two kinds of models, micro-models and macro-models (see Figure 3.1). The prefixes micro and macro stand for the scale of modelling; microscopic or macroscopic. The macroscopic scale is considered to be at least 10-100 times larger than the microscopic scale [50]. Modelling on a scale that is in between these two (mesoscopic) is considered in most literature to be a special form of micro-modelling, also called simplified micro-modelling.

In micro-models, every element of masonry (i.e. units, mortar, and their interface) is modelled separately. The disadvantages of this method are that all the properties of the constituents have to be known, there is a great number of degrees of freedom and it takes a lot of effort to build the model. Micro-models are used when studying the behaviour of a single structural component, such as a wall or a floor.

Macro-models, on the other hand, tend to be more descriptive. They usually require less input data than micro-models and can be constructed more easily. Computationally, they also have the advantage of taking less time and requiring less memory. The downside to these kind of models is that the constitutive equations can become complicated if every failure mechanism is taken into account.

The step from micro to macro-models is called homogenisation. This step can only be justified when the ‘stresses across or along a macro-length [are] essentially uniform’ [44]. If a macro-model is applied, then one is more interested in finding the global behaviour of the structure instead of obtaining local effects.

Since masonry failure can basically be described as the development of micro-cracks into macro-cracks, it can be questioned whether macro-models are able to model this behaviour at all. A compromise has to be made between accuracy (micro) and efficiency (macro) [47]. Since most engineers like to have a little bit of both, nowadays, a lot of effort is put into finding intermediate types of models or simple macro-models that capture the behaviour of micro-models.
3.2 MICRO-MODELLING APPROACH

Ever since the beginning of masonry-modelling, micro-models are used to describe the behaviour of masonry. Micro-modelling is essentially the modelling of individual components of masonry, viz. unit and mortar. There are two ways to use a micro-modelling approach [9, 44, 50]:

- **Detailed Micro-modelling**: Units and mortar are represented by continuum elements and the unit-mortar interface is modelled using interface elements. The non-linear behaviour is predominantly lumped in the interfaces. They serve as planes where cracking, slipping and crushing can occur. The interface is usually given initial dummy stiffness to avoid interpenetration of both continua.

- **Simplified Micro-modelling**: Units are represented by continuum elements. Mortar is scaled down to zero-volume interface elements and units are expanded bricks to maintain the geometrical continuity. The mortar is modelled in an averaged sense. That is the omission of Poisson’s ratio of mortar and part of the accuracy. Practically, this means that some types of failure cannot be modelled with this simplified approach [10]. The scale of the model is often called mesoscopic in order to avoid confusion with the detailed micro-model [50].

An accurate micro-model should be able to describe all failure mechanisms. Several mechanisms can be distinguished. Andreaus [2] has defined ten mechanisms, of which the most important can be categorized as unit, joint or combined unit-joint mechanisms (see Figure 3.2).

![Figure 3.2: Masonry failure mechanisms [44]](image)

In a simplified model, the damage is usually concentrated in the joints. This hypothesizes that the failure mechanisms from Figure 3.2 only have a non-linear effect on the mortar. If possible, an extra mechanism can be incorporated by pre-defining tensile cracks in the middle of each unit. Lourenço [44] has shown that this approach results in similar failure envelopes to those found in experiments and can therefore be seen as a good alternative to detailed micro-modelling.

The mechanical behaviour (from Sections 2.4.1 to 2.4.3) is described by a material model in finite element software. One type of interface model is widely described in literature to model the unit-mortar interface. This cap model originates from soil mechanics. The three yield functions, which are defined by Lourenço et al. [47] are (also see Figure 3.3):

1. Tension cut-off criterion;
2. Coulomb friction criterion;
3. Compressive cap criterion.

The difficulty of these composite yield criteria are the singular points in the corners. There are algorithms that are unconditionally stable, that enable this criterion to be used in a stable and robust manner [44, 46].
3.3 **MACRO-MODELLING APPROACH**

The difference between the macro-model approach and the micro-model approach is that all aspects of masonry behaviour are smeared out over the material. Therefore, it is also called a single-phase material [9]. The result of considering masonry as a homogeneous anisotropic material is that its constitutive equations are different from those of its constituents.

What makes this method so powerful is that it requires less computational power than the micro-modelling approach [22]. Disadvantages of this method are that it only reproduces general structural behaviour [10] and that plasticity models contain apexes and corners [44]. It can therefore only be applied to large structures, i.e. where the dimension of the structure is much larger than the unit size [3]. The accuracy of this approach is only good enough and the behaviour of separate units can only be neglected for these kind of structures.

It becomes clear from Figure 2.8 that the macroscopic damage is related to the internal structure of masonry. Localized damage, in the order of the thickness of the joints, should be taken into account when defining a material model [50]. The scale transition from local to global behaviour is called homogenisation. In the case of the macro-modelling approach, it can be seen as the relationship between the actual material behaviour and the behaviour of the material in the model.

In the homogenisation process of a composite material such as masonry, assumptions have to be made on which part of the behaviour can or cannot be included in the model. It is evident that the objective is to make as few assumptions as possible in order to most fully describe the actual behaviour. Over the years, many homogenisation techniques for masonry were developed, such as those in Berto et al. [11], Dhanasekar et al. [25], Gambarotta and Lagomarsino [31], van Zijl [78].

Each material model has its limitations. The post-peak softening behaviour and localized damage is often not included. It also is often tacitly assumed that a material model based on proportional loading is also valid for non-proportional loading. This could be questioned because of the huge effect of damage induced anisotropy. Important aspects in homogenisation techniques are periodic geometry, non-linearity, bond and/or damage-induced anisotropy. A couple examples are listed below.

**EXPERIMENTAL** In a certain way this is the easiest approach to gain an accurate constitutive material model. The tests to obtain material parameters in Section 2.4.4 are used to obtain a set of data. The material model, with a multi-yield or single-yield surface, is fitted to these parameters. Disadvantage of this method is that many experimental tests have to be performed. The resulting material model is also only valid for the range of material parameters from the tests. It therefore limits the use of such material models to other situations.

A well documented and widely discussed material model is the Rankine-Hill model developed by Lourenço et al. [47]. This is a phenomenological model and it is based on
experimental results. It comprises a Rankine-type tensile and Hill-type compression failure criterion (see Figure 3.4).

![Proposed composite Rankine-Hill yield surface with iso-shear stress lines](image)

**Figure 3.4:** Proposed composite Rankine-Hill yield surface with iso-shear stress lines [44]

**TWO-STEP** Masonry is considered as a double-layered material. The concept is that the two-phase composite of units and mortar has two sets of joints, which are head and bed joints. Within this homogenisation technique, different approaches exist such as superimposing both sets of joints or subsequently introducing them [43, 60, 61, 65].

**RVE** In the last decade, much attention was given to the use of a Representative Volume Element (RVE). In this approach, the head and bed joints are introduced together. It can also be regarded as a one-step technique. The bond pattern is taken into account by considering a certain repetitive shape within the masonry bond. This shape is called an RVE and is an intermediate step in the homogenisation progress [4, 11, 13, 14, 48, 51, 84].

**MULTI-SCALE** An underexposed homogenisation technique is the one where homogenisation is almost completely avoided. This seems rather contradictory. In essence, the homogenisation techniques mentioned above are all about scale-bridging. Luciano and Sacco [48] have tried to nest the mesoscopic scale into the macroscopic scale. It is very difficult to implement this homogenisation technique into a numerical model.

**OTHER MATERIALS** One can also ask the question if masonry should be modelled in a more special way than other geologically-based materials, such as concrete. There is a lot of literature available on concrete smeared-crack models. As long as masonry’s properties are taken into account, these models can be used as well. There are roughly three different types of smeared-crack models [49]. It is unnecessary and too comprehensive to discuss all their differences. A short overview is shown below.

**TOTAL STRAIN FIXED SINGLE CRACK MODEL** After the exceedance of a tension cut-off criterion, an element is considered to be cracked. The orientation of the crack-coordinate system is fixed upon the initiation of the crack per element.

**TOTAL STRAIN ROTATING SINGLE CRACK MODEL** After the exceedance of a tension cut-off criterion, an element is considered to be cracked. The orientation of the crack-coordinate system is thereafter continuously updated with the direction of the principal stress in the element.

**MULTI-DIRECTIONAL FIXED CRACK MODEL** After the exceedance of a tension cut-off criterion and a threshold angle, a crack occurs inside an element. If both criteria are met again, a new crack occurs. The direction of each crack is fixed upon initiation.
There are different methods to model the seismic behaviour of a structure. In order to model the behaviour of an earthquake, it is first discussed what an earthquake is and which mathematical aspects are involved (Section 4.1). Thereafter, two types of methods are discussed; testing methods (Section 4.2) and computational methods (Section 4.3).

Only the most used methods are outlined. Other methods are variations to those that are discussed here. It should be noted that analytical methods, which belongs to the group of computational methods, are left out. Codes and regulations often refer to analytical methods. The set-up of the seismic loads in analytical methods is similar to the computational methods discussed in Section 4.3.

4.1 SIMPLE EARTHQUAKE MODEL

An earthquake is ‘a sudden movement of the earth-crust caused by the release of stress accumulated along geologic faults or by volcanic activity’ \(^1\). Induced earthquakes are caused by the release of stresses due to human activity.

In structural engineering, the wave that is created in the soil is usually considered as a random vibration. Despite the randomness of the wave that is created, this vibration has also some characteristics. These are, for example, the Peak ground acceleration (PGA), the probability distribution of the PGA and the response spectrum. The behaviour of a structure under seismic loading, is therefore random but not unpredictable.

The seismic load on a two-floor structure can be simplified with the use of a damped two degree of freedom system (see Figure 4.1). Depending on the number of dominant masses (which are for example the number of floors), this model can be extended to less or more degrees of freedom. However, the approach stays the same.

![Figure 4.1: Damped two degree of freedom system](image)

The equations of motion that describe a one-directional load on this simple model are shown in Equations (4.1a) and (4.1b). The earthquake load is generally modelled as the acceleration of the ground \(a_g\). The forces \(f_1\) and \(f_2\) that are exerted on the masses due to

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this acceleration are defined in Equation 4.1c. Either the acceleration \( a_g \) or the forces \( f_1 \) and \( f_2 \) are applied. The effect on the model is the same.

\[
\begin{bmatrix}
    m_1 & 0 \\
    0 & m_2
\end{bmatrix}
\begin{bmatrix}
    \ddot{u}_1 \\
    \ddot{u}_2
\end{bmatrix}
+ \begin{bmatrix}
    c_1 + c_2 & -c_2 \\
    -c_2 & c_2
\end{bmatrix}
\begin{bmatrix}
    \dot{u}_1 \\
    \dot{u}_2
\end{bmatrix}
+ \begin{bmatrix}
    k_1 + k_2 & -k_2 \\
    -k_2 & k_2
\end{bmatrix}
\begin{bmatrix}
    u_1 \\
    u_2
\end{bmatrix}
= \begin{bmatrix}
    f_1 \\
    f_2
\end{bmatrix} = \begin{bmatrix}
    m_1 \\
    m_2
\end{bmatrix} a_g(t) \tag{4.1a}
\]

\[
M\ddot{u} + C\dot{u} + Ku = F \tag{4.1b}
\]

in which:

- \( a_g(t) \) ground acceleration
- \( c_1 \) damping between floor 1 and base
- \( c_2 \) damping between floor 1 and 2
- \( f_1 \) seismic force acting on floor 1
- \( f_2 \) seismic force acting on floor 1
- \( k_1 \) stiffness between floor 1 and base
- \( k_2 \) stiffness between floor 1 and 2
- \( m_1 \) mass floor 1
- \( m_2 \) mass floor 2
- \( u_1 \) displacement floor 1
- \( u_2 \) displacement floor 2

In the different seismic modelling methods, some parts of this equation are neglected or approximated. This equation is used to explain these assumptions.

### 4.2 Testing Methods

Four types of test set-ups to simulate an earthquake load are discussed (see Figure 4.2). That are the Shaking Table (ST) test, Pseudo-dynamic (PsD) test, Effective Force (EF) test and hybrid test.

**Figure 4.2: Seismic testing methods**

**Shaking Table Test** The ST test is the most natural way to simulate a ground motion on a structure. The structure is placed on a shaking table that is connected through actuators to a rigid surface. The dynamic motion is simulated in real-time with an artificial
accelerogram and with the aid of a computer [9]. This way, the mass-inertial effects of any mass in the structure is taken into account.

The total number of degrees of freedom from Figure 4.1 and Equation (4.1a) that are modelled in the ST test can be regarded as infinitely large. The size of the shaking table limits the size of the model. Therefore, models are often scaled. If this is done properly, the least amount of aspects are assumed to model the seismic response and the simulation is closest to real-life. Some examples of research that are carried out with a ST can be found in Clough et al. [18], Gambarotta and Lagomarsino [32], Gülkan et al. [35].

**Pseudo-dynamic test**  In this test, the seismic load is applied in a quasi-static manner. The structure is rigidly connected to the ground surface. The test is carried out with the aid of servo-hydraulic actuators attached to each dominant mass. The actuators are operated using a deformation controlled algorithm. The forces in the actuators are iteratively changed to reach a certain displacement. Thereafter, the forces are fed back into a computer that solves the equation of motion (see Equation 4.2). The time-dependent ground acceleration is discretised and applied in steps (see top-left of Figure 4.3).

The damping-term in Equation 4.1a is assumed constant or left out altogether. Together with the discretisation in time, this is the reason why the PsD test is quasi-static. The stiffness-term from Equation 4.1a is replaced by the measured force \( r(u) \) [26, 27, 63]. The result is Equation 4.2:

\[
M\ddot{u} + C\dot{u} + r(u) = f
\]  

(4.2)

Results from a PsD test can be sensitive to experimental errors. Therefore, some control parameters are defined. They were taken from equivalent linear models and identify the quality of tests [53, 71, 73]. In the classical method, the equation of motion is step-wise integrated (see Figure 4.3) in the following steps:

- Stabilisation hold period \( \Delta T_{b1} \)
- Measurement hold period \( \Delta T_{b2} \)
- Computation hold period \( \Delta T_{b3} \)
- Ramp period \( \Delta T_{ram} \)

**Continuous pseudo-dynamic**  A variation to this method is applied in the ESECMASE project (see Section 5.3). The difference with the classic one is that within every displacement step a second iteration is performed. This reduces the stabilisation period and ramp period to zero. The accuracy depends on the time scale factor \( \lambda \), which is used to scale from the discrete steps in the ground accelerogram to the actual specimen displacement [5, 63].

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![Figure 4.3: The classical (left) and continuous (right) PsD method [63]](image-url)
**Effective Force Test** The test set-up of an EF test is similar to the PsD. The computational part is carried out a priori. Equation 4.1 is solved beforehand for a given accelerogram. This means that there is no computational time for taking into account the response of the structure. The seismic action is therefore in real-time [26, 28, 72, 80]. This type of test is called an open-loop test, because there is no feedback to the input of the system [67].

**Hybrid Test** This is a combination between the ST and PsD test. An advantage of this technique is that simple parts of the structure are modelled in a linear way. Therefore, they do not need to be constructed in the test set-up. The influence of these substructures is applied through a hydraulic actuator. The part of the structure that is difficult to model computationally is experimentally tested on a shaking-table [21, 67, 68].

### 4.3 Computational Methods

The computational methods that are discussed here can be disaggregated based on two aspects; linear or non-linear material behaviour and quasi-static or dynamic loading. The result is four different combinations of aspects to analyse the seismic loading. The focus in this section will lie on the numerical methods, because they are used later on in this research.

In quasi-static computational methods, the seismic load is applied through forces (see right red box in Figure 4.1). Dynamic methods, on the contrary, apply the load through a ground acceleration (see left red box in Figure 4.1). The type of method that is considered to be allowable by the NEN-EN1998-3 [58] follows from the Knowledge Level (KL). This level is based on the amount of structural properties that are known. The Confident Factor (CF) is directly related to these levels and provides, together with safety factors, a safe approximation in unity checks.

**Lateral Force Analysis** The Lateral Force Analysis (LFA) is mostly used in codes and regulations. It can be applied to 'buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode' [57, §4.3.3.2]. The loading is applied through a distribution of concentrated forces at each storey (which are in fact the dominant masses). This distribution depends on the relative product of the mass either with the height or the fundamental mode shape. The capacity of the structure is based on its linear-elastic behaviour.

**Pushover Analysis** The Pushover Analysis (POA) is the non-linear (quasi-static) variant of the LFA. The important difference is that the entire material behaviour is considered. The force distribution is also either according to the height or modal shape. According to NEN-EN1998-1 [57, §4.3.4.2], both distributions have to be checked. Different earthquake magnitudes can be obtained by scaling the forces [22]. Examples of a POA can be found in Galasco et al. [29], Griffith et al. [34], Mendes and Lourenço [52], Pelè et al. [64], Yi et al. [81].

**Sequentially Linear Analysis** To overcome convergence problems that occur in the softening of large-scale brittle structures in a POA, it can be combined with a Sequentially Linear Analysis (SLA) to improve robustness and predict damage in a greater extent. The softening part is discretised with a saw-tooth curve (see Figure 4.4). The teeth of the curve are formed by a series of linear analyses. After each linear analysis, a damage increment is applied which creates the saw-tooth shape [22–24, 40, 74].

**Response Spectrum Analysis** The Response Spectrum Analysis (RSA) is a linear dynamic analysis to model seismic behaviour. The accelerogram, provided from an earthquake record or generated by a computer according to a code, is input for this type of analysis. There are two different types: direct-RSA and modal-RSA. The modal method is usually preferred because it gives insight into the response to the natural frequencies. Sev-
eral approaches exist to find the maximum modal seismic response, viz. Absolute, Square Root of the Sum of the Squares (SRSS) and Complete Quadratic Combination (CQC) [19, 49].

**Time History Analysis** An accelerogram is also used as input in a THA. The difference with a RSA is that the response is evaluated in terms of displacements (and its derivatives) instead of frequencies. It is also different from the POA in the sense that time-dependent behaviour is included. It is thought that the results show similar starting points for yielding but ‘diverge at highly non-linear responses. Non-linear static procedures are thought to produce story overturning moments that are overconservative at the base and story shears that are unconservative over the height of a structure’ [36]. From all the analyses that are mentioned in this section, the THA is the most extensive and most complex.

In this non-linear dynamic analysis, the response of the structure is integrated over space and time. The loading history during each interval and the initial conditions of each interval are input for the equation of motion [19](see Equation 4.1). The total number of degrees of freedom in a finite element model determine the size of the equation of motion that is solved. It is most complex, partly because every element can be given a different mass, damping and stiffness.

A type of damping that is often applied in a THA is Rayleigh damping. It requires the input of two constants, which are $\mu$ and $\lambda$. The evolution of damping is taken into account through these constants and their multiplication with the mass and stiffness matrices (see Equation 4.3). Examples of this type of analysis in masonry structures can be found in Galasco et al. [29], Lam et al. [41], Mendes and Lourenço [52], Rota et al. [69], Zhuge et al. [83].

$$C = \mu M + \lambda K$$  \hspace{1cm} (4.3)

in which:
- $M$ mass matrix
- $C$ damping matrix
- $K$ stiffness matrix
- $\mu$ mass proportional damping coefficient
- $\lambda$ stiffness proportional damping coefficient

![Stress-strain relationship for the consistent saw-tooth diagram][22]
BENCHMARKS

The last part of this literature study shows a small overview of other studies that are similar to the problem that is approached in the next part of this report. These studies are referred to as benchmarks. The definition of a benchmark that is used here is: a study which

- Involves a masonry structure,
- Is seismic loaded (real, experimental, analytical or numerical),
- Is a frame-type of structure (opposed to arch-like structures such as masonry vaults),
- Is three-dimensional and
- Consists of more than one floor (and thus multiple structural elements)

An overview of the benchmarks in this chapter is chronologically shown in Table 5.1 and Figure 5.1. The acronyms in the column LOAD are classified using the loading types mentioned in Chapter 4. The words micro or macro refer to the simplified micro-model and macro-model approach discussed in Chapter 3. Further specifications or elaborations on the used methods can be found in the corresponding sections.

<table>
<thead>
<tr>
<th>BENCHMARK</th>
<th>MODEL</th>
<th>LOAD</th>
<th>SOFTWARE</th>
<th>ELEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 5.1</td>
<td>Experimental</td>
<td>PsD</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section 5.2</td>
<td>Experimental</td>
<td>EF (cyclic)</td>
<td>ABAQUS</td>
<td>Continuum (macro)</td>
</tr>
<tr>
<td></td>
<td>Numerical</td>
<td>POA (cyclic)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section 5.3</td>
<td>Experimental</td>
<td>PsD</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section 5.4</td>
<td>Numerical</td>
<td>THA</td>
<td>LS-DYNA</td>
<td>CFDEM (micro)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section 5.5</td>
<td>Experimental</td>
<td>ST</td>
<td>DIANA</td>
<td>Shell (macro)</td>
</tr>
<tr>
<td></td>
<td>Numerical</td>
<td>POA, THA</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section 5.6</td>
<td>Real</td>
<td>-</td>
<td>DIANA,</td>
<td>Continuum (macro)</td>
</tr>
<tr>
<td></td>
<td>Numerical</td>
<td>POA</td>
<td>SAP2000</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.1: Benchmarks overview

5.1 PSEUDO-DYNAMIC TESTING OF UNREINFORCED MASONRY BUILDING WITH FLEXIBLE DIAPHRAGM

The model in this benchmark is both tested experimentally and analytically. The considered URM building has a flexible floor and roof. The goal of this benchmark is to obtain the impact of wall continuity at the building corners. Two corners were built discontinuous, i.e. with vertical gaps. The properties of the two wythes1 of solid brick walls were determined through tests on half bricks, mortar cubes and five-brick prisms.

1 ‘A wythe of masonry refers to a thickness of wall equal to the thickness of the individual units’ (Whole Building Design Guide)
The (classical) $P_sD$-test with a synthetic ground motion showed a stable combined rocking and sliding mechanism. The large deformations that occurred during the tests did not cause significant strength degradation. The analytically computed lateral shear capacity showed results that were similar to the observed behaviour. Despite the fact that the wall-continuity is not mentioned in the codes, it was observed that this effect had a negligible impact [62].

5.2 ANALYSES OF A TWO-STORY UNREINFORCED MASONRY BUILDING

The goal of the experimental part of this benchmark is to validate a method in which global behaviour was extrapolated from component behaviour. The model consists of weak timber floor and roof diaphragms and both two-wythe and three-wythe masonry walls. The experimental model is loaded with a first-vibration-mode displacement. The distribution is updated by using the stiffness of a two degree of freedom system [81].

This benchmarks consists of four different numerical analyses: a 3D linear elastic, a rigid body, a 3D non-linear and a 2D non-linear analysis. The goal of the linear elastic analysis was to determine how the behaviour of the structural elements is coupled in different load case. The flange effects were determined with the aid of a rigid body analysis. These lead to a 3D non-linear model in which the hysteretic behaviour was investigated with a cyclic displacement load. The brick elements in this model were given an E-modulus that was 60% of the original masonry in order to account for softening. The 2D non-linear analysis was a FEMA-356 modification to that model [82].
The discrepancies between experimental and numerical results are caused by the large degree of uncertainty of masonry material. The rigid body analysis is the easiest and quickest to use while the 3D non-linear analysis required many computational parameters which had to be chosen carefully to stabilize the analysis procedure. The 2D non-linear analysis was a good balance between both. It is therefore best for seismic evaluation and retrofit.

5.3 ESECMaSE LARGE SCALE EARTHQUAKE TESTS ON A BUILDING

The Enhanced Safety and Efficient Construction of Masonry Structures in Europe (ESECMaSE) project is a large project from which the large scale earthquake tests on a building are only a small part. Because the results of these tests are used in the next parts of this report, they are discussed in more detail. It is important to understand that in the other parts smaller tests were performed. Some of these tests are also used in this report to determine, for instance, the material parameters (see Chapter 6).

The experimental model in the ESECMaSE project is based on a terraced house. Only half of the original house is constructed because of quasi-symmetry. Two versions of this house were tested; one made of clay bricks and one made of calcium-silicate bricks. Only the latter is discussed in this report.

The structure is comprised of different structural elements, viz. two floors, four long walls, four short exterior shear walls and two longer interior shear walls (see Figure 5.1d). There is an opening in the floors between the plane of symmetry and the interior wall to account for the presence of a staircase. The floors are made of reinforced concrete C20/25 and the walls of running bond masonry with thin mortar bed joints. The head joints of the walls are unfilled and the interior shear wall is connected to the long wall through a continuous vertical mortar joint with metal strips as connectors. Furthermore the joints between the exterior shear walls and the long walls are parallel to the long wall [5].

5.3.1 Hammer impact test

The modal parameters; mode shapes, frequencies and damping ratios, are determined from an hammer impact test. They can be used to 'calibrate the elastic characteristics of a numerical model' [7]. An advantage of such a test is that it is non-destructive, i.e. it does not cause damage to the structure and change the structure’s behaviour. The principle of this test is to hit the structure with a relatively low mass (5 kg in this case) and measure its response. The Frequency Response Function (FRF) is obtained by computing the ratio between a Fast Fourier Transform (FFT) of the output signal and a FFT from the input. The frequency and damping ratios are thereafter extracted from the FRF with the Peak Picking Method (PPM) and averaged over eight tests [7].

The mode shapes are estimated (!) with a finite element model (see Figure 5.2). Their relationship is highly questioned because nothing is reported about the set-up of this model and a direct relationship is placed without any substantiation. Especially the correspondence between the frequency and these shapes is not certain.

![Figure 5.2: Modal shapes from the dynamic identification](image-url)
5.3.2 Pseudo-dynamic test

The continuous pseudo-dynamic testing method (also see Section 4.2) is applied to model the seismic load. A type 1 response spectrum with a PGA of 0.04g and ground type B [57] is used to generate an artificial acceleration time history (see Figure 5.3).

![Reference accelerogram for intensity 0.04g](image)

Figure 5.3: Reference accelerogram for intensity 0.04g [6]

The load is applied with the aid of two pairs of hydraulic actuators. Actuators on the same floor impose equal displacements as to 'prevent rotation around the vertical axis' [5]. Although the specimen is asymmetric, this approach is valid because of the plane of symmetry with respect to the whole terraced house.

The seismic equivalent force $f(t)$ is computed from the product of the ground acceleration and a theoretical mass matrix (see Equation 4.1). Figure 5.3 is scaled for each intensity. Viscous damping is neglected. The damping that was measured therefore only originates from the 'hysteretic behaviour of the specimens' [6].

A couple of types of results that are documented and discussed in Sections 5.3.2.1 and 5.3.2.2 are [6]:

- Crack locations (only between each test)
- Envelope of the shear-drift curves for level 1 and level 2 (see Figures 5.5a and 5.5b)²

5.3.2.1 Crack patterns

A three-dimensional failure mechanism of a structure due to a seismic load can be very complex. In order to simplify the comparison between the cracks in the experimental and in the numerical model three different crack patterns, denoted by R, F and S, are distinguished (see Figure 5.4). They are based on the type of cracks and the locations after each test. The formation and occurrence of the three crack patterns takes places in different alternations and combinations (see Table 5.2).

Note that the crack patterns in Figures 5.4a to 5.4c are purely theoretical and only show the failure mechanism due to the first mode. They are related to the failure mechanism of the whole structure and observations of the structure’s response in Anthoine and Capéran [6]. It was only possible to determine these patterns from the results of the PsD test and they are therefore only applicable to this structure. The crack patterns are a guidance in checking the behaviour of the model.

² Note that level 1 is used for the ground floor and level 2 for the first floor
(b) Flexural (horizontal) cracks in the long walls (F)

(c) Stepwise cracks in the shear walls (S)

Figure 5.4: Crack patterns

<table>
<thead>
<tr>
<th>INTENSITY</th>
<th>TYPE AND LOCATION</th>
<th>PATTERN</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02g &amp; 0.04g</td>
<td>no damage could be detected</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>non-linear behaviour due to rocking</td>
<td>F</td>
</tr>
<tr>
<td>0.06g</td>
<td>first noises</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>some (flexural) horizontal cracks at the base of the long walls in both storeys</td>
<td></td>
</tr>
<tr>
<td>0.08g &amp; 0.1og</td>
<td>more noises</td>
<td></td>
</tr>
<tr>
<td></td>
<td>not any significant new cracks</td>
<td></td>
</tr>
<tr>
<td>0.12g</td>
<td>first significant damages</td>
<td></td>
</tr>
<tr>
<td></td>
<td>horizontal cracks opened in the outside of the left (west) long wall</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>at mid-height of level 1, near the shear wall</td>
<td></td>
</tr>
<tr>
<td></td>
<td>horizontal cracks opened in the outside of the right (east) long wall</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>at mid-height of level 1, near the interior shear wall</td>
<td></td>
</tr>
<tr>
<td></td>
<td>large stepwise cracks in the left shear wall of level 1</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>stepwise cracks in the interior shear wall of level 1</td>
<td>x</td>
</tr>
<tr>
<td>0.14g</td>
<td>horizontal cracks along the bottom slab-wall joint in level 2</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>horizontal cracks below the top layer of bricks in level 2</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>horizontal cracks in the long walls below the top layer of bricks at each level</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>stepwise cracks in the interior shear wall of level 1</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>stepwise cracks in the right (east) shear wall of level 1</td>
<td>x</td>
</tr>
<tr>
<td>0.16g</td>
<td>stepwise cracks in the interior shear wall and left (west) shear wall of level 1</td>
<td>x</td>
</tr>
<tr>
<td>0.18g</td>
<td>very few new cracks</td>
<td></td>
</tr>
<tr>
<td>0.20g</td>
<td>severy cracking at the back-left (north-west) top corner at level 2</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.2: Crack development during PsD tests

5.3.2.2 *Shear-drift curves*

In almost all literature, the behaviour of a structure under a static seismic equivalent load (experimental or numerical) is evaluated using a *top-displacement, base-shear curve*. That is
often the case because forces and displacements are not measured halfway the height of a structure and they can be used directly for designing purposes. In the ESECMaSE project, it was chosen to report separate envelope shear-drift curves for each level (see Figures 5.5a and 5.5b).

The advantage of this approach is that more detailed conclusions about the behaviour of the structure can be drawn. There are four quantities available for comparison instead of two. Disadvantage is that the correlation in the response between both levels is hard to determine. The envelope of the results is used, because the PsD tests are loaded in a cyclic manner. The reader is referred to Appendix A for a detailed explanation of the definition of these quantities.

It can be seen from the shear-drift curves that the behaviour of the experimental model is highly non-linear. There is almost no distinguishable linear elastic response. One can conclude that cracks occur from the start. This is exactly what is discussed in Chapter 3 about the development of micro into macro-cracks. Right from the start of any load on a masonry structure one should expect that the structure is (micro) cracked. The reason that (macro) cracks are only visible in a later stage (also see Table 5.2) is because of that difference between micro and macro-cracks.

Something that is quantifiable from the shear-drift curves is the shear capacity. The definition of shear capacity that is used here is the horizontal line that is being approached
in the curve. The shear capacity for level 1 and 2 is estimated to 125 kN and 80 kN. The higher value for level 1 can be explained because of the dead load from the walls and floors above. Due to this load, the walls of level 1 are under a higher compressive stress and it will take relatively more load to create the tensile strain under which a crack forms.

The shear capacity can only be determined from a horizontal plateau. If the results lead to an differently shaped shear-drift curve or if the shear force keeps on increasing under higher displacements, this is not possible.

5.4 COLLAPSE ANALYSIS OF MASONRY STRUCTURES UNDER EARTHQUAKE ACTIONS

On the basis of the ESECMaSE project, Bakeer [9] performed a numerical analysis. The Combined Finite-Discrete Element Method (CFDEM) and a simplified micro approach are used to model the structure. The dynamic characteristics were obtained from a 0.04g PGA. The results showed that the biggest variation in normal forces is caused by rocking of the interior wall and that the moment is proportional to the normal force in each shear wall. The cracks are similar to the ones observed in the PsD test.

In the Time History Analysis (THA), the ground was given a real-time acceleration. Roughly three types of earthquake intensity (weak, moderate and strong) were modelled. This enabled the structure to be loaded until collapse.

In the analyses with weak earthquake intensities, i.e. up to 0.18g, similar cracks as in ESECMaSE were found. For moderate intensities, i.e. 0.20g, 0.24g and 0.26g, tensile and diagonal cracks were found respectively along the bed joints in the long walls and in the interior shear wall. The higher the intensities, the more the existing cracks opened. The long wall completely buckled at stronger intensities and it even ‘lost stability and underwent progressive collapse’ [9] at an intensity of 0.40g.

5.5 SEISMIC ASSESSMENT OF MASONRY GAIOLEIRO BUILDINGS IN LISBON

The experimental model is based on a poor quality masonry structure. It was build on a 1:3 reduced scale. In fact, two different models were constructed; one with a weak and one with a strong wall-floor connection. The models are loaded with an artificial accelerogram for stiff soil according to the Portuguese code [52].

Before the numerical model was tested with a seismic load, the modes and dynamic characteristic were identified. A modal identification test was used to calibrate the Young’s moduli and a dynamic identification test was used to find both Rayleigh damping coefficients. A total strain crack model is adopted with fracture energies that were adjusted to obtain the crack pattern from the experiment.

The seismic load represented by a response spectrum for ground type A [57] was applied in two orthogonal directions in the subsequent THA. Among the results is seismic coefficient $\alpha_h$ which has a maximum in each direction of 0.2 and 0.65.

The POA is carried out with load distributions proportional to mass, to modes and adaptive to the damaged first modal shape. In the first analysis, the damage is concentrated at the lower zones of the structure and did not simulate the expected performance correctly. The second analysis led to a similar seismic coefficient as in the THA, but showed only in-plane damage. The last analysis demonstrated a decrease of the seismic coefficient in transverse direction and almost no damage in longitudinal direction.

5.6 ASSESSMENT OF SEISMIC VULNERABILITY OF A HISTORICAL MASONRY BUILDING

This benchmark involved many different masonry types. Because the structure that is considered is 800 years old, there have been a lot of renovations, several building techniques were used and there is a huge variety in bricks. The type of test can be regarded as a dynamic identification test. The measurements are carried out with the aid of accelerometers
on different points over the height of the building and the load was exerted through the fall of a concrete block on a nearby truck [15].

Different analyses are performed afterwards to model the seismic behaviour. A distinction is made between a model with (model A) and without (model B) basement. In the linear dynamic analysis the effect of the Young’s modulus and unit weight on the vibration shaped and (first two) frequencies was assessed. From the results, it can be concluded that in model B the structure has a larger deformation capacity. A variation of 50% in the Young’s modulus lead to a variation of 25% in the frequencies.

The aim of the non-linear POA was to determine the sensitivity of the structure with respect to the shear retention factor and the tension softening relationship that fits the results best. It is concluded that a shear retention factor of 0.01 is the safest prediction. The tensile behaviour had a big influence on the response of the structure. Most of the panels reached the ultimate tensile strain and in the case of an ideal plastic relationship, the same strain was exceeded up to three times. The behaviour factor $q$ [57] was used to compare the response of the structure according to several design codes. It showed that numerical factors are higher than those in the Italian code and significantly higher than those in the Eurocode.
Part II

MODEL OF THE ESECMASE BUILDING

This part focusses on the properties of the finite element model that is constructed on the basis of the model in the ESECMASE project. A substantiation of properties, that are in some cases also varied later on, is given in Chapter 6. In order to construct the model, it was also necessary to determine dynamic properties of the model, such as the mode shape and eigenfrequency (Chapter 7).
Finite element models can be set up in different ways. The assumptions that are made in the early phase of constructing a model can have a large influence on the results. The properties of the model, that are essentially the result of these assumptions, are discussed in this chapter. They are based on literature, experimental tests from the ESECMaSE project and eigenvalue analyses.

The order in which the model is constructed is also the order in which errors due to wrong design choices can build up. This is therefore the order in which the properties are discussed. That means that first the geometry of the models is discussed (Section 6.1) and then the mesh (Section 6.2). The smeared-crack material model is outlined in Section 6.3 and the load application method in Section 6.4. The solution procedure and processing of the results is discussed in Section 6.5.

### 6.1 Geometry of the model

The geometry of the structure is the same as the experimental model in ESECMaSE. It is a simplified half terraced house of which the plane of symmetry is located at the front of Figure 6.1. The main direction under which the structure was tested is its weakest direction, the x-direction. This direction is the weakest, because the long walls provide a larger stiffness in the z-direction. Another weak direction is the y-direction, but due to gravity it is probably not dominant.

![Geometry of the numerical model](image)

Figure 6.1: Geometry of the numerical model

The model consists of two concrete slabs (one for each floor) and several masonry walls. The opening in the floors, which resembles the presence of a staircase, is accompanied by an interior (or long) shear wall. The other two shear walls are located at the façade of the structure and can also be called piers. The piers that are usually in between the windows are regarded as weak elements and are therefore left out. The protrusions at the back and front of the structure are designed for the attachment of the hydraulic actuators.

A first step was to approximate the structure with a two-dimensional model (see Appendix B). However, the interaction between the structural elements could not be realistically modelled because of the locally high stress concentrations. The resulting equivalent stiffnesses are therefore not mesh independent, which is a requirement for any finite element model. A three-dimensional model seemed to be the only one able to take into account these stress concentrations.
6.1.1 Connection stiffness

In a discrete-crack model, the crack locations, for instance between structural elements such as walls and floors, is fixed beforehand. The response of such a model in an eigenvalue analysis gives more information about the interaction between these elements and helps in understanding the behaviour of the structure, which is also useful in a smeared-crack model.

The choice of locations should also be related to connections in the experimental model. The building method discloses some properties that cannot be seen at first sight. For instance, during the construction of the ESECMaSE specimen, the ‘concrete slabs were [sic] poured directly on the top layer of units without any mortar joint’ [5]. Furthermore, all the shear walls are connected to the long walls ‘through a continuous vertical joint’ [5] parallel to the long walls. In the calcium-silicate specimen it is also the case that the “corners” belong to the long walls’ [5].

The first mode shape has generally the largest contribution to the total behaviour of the structure. The locations of the interfaces that are listed below are based on this shape (also see Figures 6.2a to 6.2c). The interfaces were given a certain thickness, because of the complexity of the geometry (see Appendix C).

1. Between the inner shear wall and long wall
2. Between the inner shear wall and slab
3. Between all the shear walls and slabs

The interfaces are modelled as pre-defined cracks with a high initial dummy stiffness. This dummy stiffness enables the stresses to build up locally and the interface to crack before other elements are subjected to large strains. The crack occurs in the direction of the interface and in the interface only, which results in locally large discrepancies. The dummy stiffness is computed according to Equation 6.1.

\[
D_{mn} = \lambda_{\text{normal}} \frac{E}{l} \quad D_{ss} = \lambda_{\text{shear}} \frac{G}{l}
\]

in which:

- \(D_{mn}\) linear normal stiffness (N/mm³)
- \(D_{ss}\) linear tangential stiffness (N/mm³)
- \(\lambda\) dummy factor, usually 1000
- \(E\) (lowest) Young’s modulus of neighbouring material (MPa)
- \(G\) (lowest) Shear modulus of neighbouring material (MPa), in this case 0.4E [56]
- \(l\) length of adjacent element (mm)

The value of the dummy factor \(\lambda\) is chosen arbitrarily high. The aim is that this results in the exact same behaviour as a model without interfaces. The influence of non-linear interface properties on the behaviour of a structure can be estimated from an eigenvalue
analysis with lower interface stiffnesses. These lower stiffnesses create local weak spots. It is expected that weak spots results in a weaker structure with a lower eigenfrequency. This can also be seen in Figures 6.3a and 6.3b.

![Graphs showing influence of normal and shear stiffness on first eigenfrequency](image)

(a) Influence of normal stiffness on the first eigenfrequency

(b) Influence of shear stiffness on the first eigenfrequency

Figure 6.3: Influence of stiffness on the first eigenfrequency

It can be concluded from Figures 6.2 and 6.3 that lowering the stiffnesses of the interfaces between all the shear wall and floor connections (model 3) results in the largest eigenfrequency reduction. A lower normal stiffness (or dummy factor) of these interfaces results in a lower eigenfrequency than the same dummy factor in the shear stiffness. This can be explained with the translational shape of the first mode. The shear walls are the only structural elements that provide in-plane stiffness to displacement of the first mode. A weak wall-slab connection results in rocking behaviour of the walls and a house-of-cards-like behaviour.

The normal stiffness of the interfaces in model 1 has no influence on the eigenfrequency. The connection between the shear wall and slabs prevents the occurrence of large discrepancies along these interfaces. The shear stiffness of the same connection does have an influence on the eigenfrequency.

This connection is in the experimental model created through masonry connectors (metal strips). They could be better able to transfer normal stresses than shear stresses. This means that the shear stiffness of this connection in the experimental model is lower than in a rigid
connection. It might be very realistic to incorporate this interaction, if the numerical model does not show similar behaviour as in the Pseudo-dynamic (PsD) tests.

The eigenvalue analyses give information about the interaction between the structural components. Many connections are rigid in the experimental model. Using interfaces in a model that has less rigid connections can be useful. Smeared and discrete crack models are usually not combined, because it is difficult to hypothesize the distribution of non-linearity between the discrete and smeared cracks. Interfaces will therefore not be used in the models presented hereafter.

6.2 Finite Element Discretisations

A common approach when generating a mesh is to set the size in such a way that the length of the elements is equal in all directions. In this case, that would be equal to the wall thickness (i.e. 175 mm). The accuracy of the elements is, among others, dependent on the integration scheme and the use of mid-side nodes. Two of the possibilities that are offered in DIANA are the HX24L and CHX60 elements (Figures 6.4a and 6.4b). Both elements are based on a Gauss integration scheme. The influence of both types on the first eigenfrequency is shown in Figure 6.5.

![Solid brick elements in DIANA](image)

Figure 6.4: Solid brick elements in DIANA

![Influence of mesh refinement on first eigenfrequency](image)

Figure 6.5: Influence of mesh refinement on first eigenfrequency

The smaller the elements and the higher the order, the more accurate the results. In an ideal situation, the size of mesh would be infinitely small. Since the relationship seems to be almost linear, one could draw a dotted line through zero which results in an eigen-frequency of 7.38 Hz for both types. Presumed that this is the actual numerical eigen-frequency, a mesh of 200-millimetre CHX60 elements results in an error that is less than 0.05 Hz. A similar mesh of lower order HX24L elements results in an error of 0.08 Hz. Although this error is probably constant for all the results, i.e. all the results are out of line with the same amount, the order is close to the size of the graphs in this chapter. The error
of the CHX60 elements is considered to be small enough to compare the numerical and experimental results.

6.3 MATERIAL MODEL

The material properties of more or less homogeneous materials, e.g. steel or concrete, are relatively easy to determine. As discussed extensively in Section 3.3, the macro-modelling approach for masonry requires to make generalisations due to its orthotropic nature. These generalisations bring along uncertainties about the correctness of the material properties with respect to reality. The material properties that are discussed in this section comprise both the assumed setting and the options or bandwidth within that setting, which is used later on in Chapter 9.

The type of masonry that is modelled is composed of calcium-silicate bricks and unfilled head joints. Different types of smeared-crack models are discussed in Section 6.3.1. Different aspects of the stress-strain relationship of masonry are discussed in Section 6.3.2. The effect of taking into account the orthotropic nature of masonry is shown in Section 6.3.3. This section concludes with different damage based reductions in masonry’s constitutive behaviour (Section 6.3.4).

CONCRETE PROPERTIES  The focus in this report is on the non-linear behaviour of masonry. The non-linear behaviour of the reinforced concrete slabs is neglected. This approximation can be regarded as valid, because the cracks in masonry are bigger and lead to more degradation of the structure’s stiffness and capacity. The slabs are probably stiff enough to distribute the stresses over the connections with the walls.

The designed concrete in the experimental model is C20/25. Because the concrete is cast inside the testing facility, the conditions are close to ideal. The Young’s modulus of concrete in the numerical model is assumed to be 30 GPa [55, table 3.1], the mass density is 24 kN/m$^3$ [54, table A.1] and its Poisson’s ratio is taken as 0.15.

6.3.1 SMEARED-CRACK MODEL

It is chosen to use the total strain fixed single crack model from Section 3.3, because the cracks in masonry are almost always along the joints. The influence of this choice on the results of a Monotonic Pushover Analysis (MPOA) is researched in Section 9.1. The crack locations are fixed with respect to the orientation of the joints, which is dependent on the type of bond (see Section 2.2 for examples). Because the joints are perpendicular to each other the threshold angle under which multiple cracks occur is very high. Taking into account such angle is therefore probably not necessary.

6.3.2 MASONRY’S STRESS-STRAIN RELATION

Stress-strain relations can be split up into two parts, the tensile and the compressive behaviour. The relationship that is used in this report is shown in Figure 6.6 and Table 6.1. In the subsections hereafter, it is explained how both diagrams are obtained. Because the tensile strength of masonry is relatively low, most of the attention is paid to model the tensile parameters, i.e. Young’s modulus (Section 6.3.2.1), tensile strength (Section 6.3.2.2), softening curve (Section 6.3.2.3) and mode I fracture energy (Section 6.3.2.4). The compressive behaviour is outlined in Section 6.3.2.5.

6.3.2.1 YOUNG’S MODULUS

Besides the fact that the Young’s modulus is very much dependent on the type of units, the bandwidth within one type is also large. Moreover, even if the amount of tests is large, the coefficient of variation can still be more than 40% [16]. This makes it very difficult to
determine one modulus of elasticity that represents all the masonry. It is meaningful to know what the effect is of a higher or lower E-modulus.

The effect of different E-moduli is researched in an eigenvalue analysis. In a smeared-crack model the Young’s modulus of a specimen should be used and not the value of the unit itself. The second is of course higher than the first. The starting point is an E-modulus that is computed according to NEN-EN1996-1-1+A1 [56] (see Equation 6.2). A bandwidth around this value and up to the Young’s modulus for the units (i.e. 9090 MPa [9]) is researched in an eigenvalue analysis.

\[
E = K_E \times f_k = K_E \times K \times f_b^{0.85} = 700 \times 0.55 \times 22.8^{0.85} = 5490 \text{MPa} \tag{6.2}
\]

in which:

- \(K_E\) 700 [56, §3.7.2]
- \(K\) 0.55 [56, Table 3.3]
- \(f_k\) characteristic compressive strength
- \(f_b\) normalized mean compressive strength of the units from Grabowski [33, Table 5]
Figure 6.7 shows that the E-modulus can be chosen in such a way that one of the eigenfrequencies is exactly the same as the experimentally obtained one. The (approximate) values are shown besides the intersection points. It can be questioned whether such an approach is right. A numerical model with an E-modulus of 6800 MPa results probably in the same eigenfrequencies for the first experimental mode, but in totally different eigenfrequencies for higher modes.

It can also be seen that the sensitivity of the E-modulus to the first eigenfrequency is relatively low. The second eigenfrequency has a relatively high sensitivity. The sensitivities of the first four eigenfrequencies are different from each other. That means that there is not a linear relationship between the Young’s modulus and the model’s dynamic response.

The relationship between mode shapes and eigenfrequencies, as was already questioned in Section 5.3.1, is difficult to quantify. The lines that are drawn in the diagram are generalisations of the relationship between mode shape and eigenfrequency. In other words, eigenmode x can have a completely different mode shape for an E-modulus of 3000 MPa than for 9000 MPa. This is clearly visible in the line for the fourth eigenfrequency in Figure 6.7. The sudden drop in the figure is accompanied by a drastic change in the mode shape. This phenomenon is discussed in more detail in Appendix D.

It can be concluded that the eigenfrequencies of the numerical model are most close to those obtained from the experimental model when they are based on a Young’s modulus between 4700 and 6800 MPa. This is valid under the assumption that lower modes have a bigger contribution in the structure’s response than higher modes.

If that is true, then the first couple of frequencies give a bandwidth that is accurate enough. Remarkably, the E-modulus from Equation 6.2 is approximately in the middle of this bandwidth. This is why an E-modulus of 5490 MPa is used in the other models in this report. Since the influence on the eigenfrequencies is significant, its sensitivity is checked in a MPOA in Section 9.2.1.
6.3.2.2 Tensile strength

The tensile strength of masonry can be obtained through tests on units (see Section 2.4.4). The mean of a sufficient number of tests is commonly used as the tensile strength for the whole model. Twelve specimens were tested in the ESECMaSE project to determine the tensile strength [77, §5.2]. The resulting mean tensile strength of 0.28 MPa is used throughout this study.

An error that can easily be made is the usage of the tensile strength of the units. In the ESECMaSE project this value was also determined. The type of units that are used in this structure have a tensile strength of 1.67 MPa [33, Table 12]. The sensitivity of the model with respect to the range of these values is researched in Section 9.2.2.

6.3.2.3 Tensile softening curves

A direct result from the approximation of a smeared-crack model is the difficulty to pick a single tensile softening curve that fits all the possible failure mechanisms of masonry (also see Figure 2.8). If it is assumed that masonry mostly fails in its joints, then a decaying curve fits its behaviour best (see Figures 2.5 and 2.7). DIANA offers a couple of possibilities to model such behaviour in a total strain single crack model:

- Linear
- Exponential
- Hordijk

One of the things these tensile softening curves have in common is that the area under the curve is based on the mode I fracture energy. Because the cracks are smeared out over each element and elements have different sizes, the crack bandwidth \( h \) is also taken into account. This crack bandwidth is related to the element volume. The elements in the numerical model that is considered here are ideally cubes of 0.2 metre long, but in reality they all have a slightly different crack bandwidth (see Figure 6.8b).

The sensitivity of the model with respect to the different types of softening curves is researched in Section 9.2.4. The same fracture energy is used in order to compare them properly. It can be seen from Figure 6.8a that this leads to different ultimate strains. The other analyses were performed with a linear tensile softening curve.

6.3.2.4 Mode I fracture energy

The mode I fracture energy can also be determined with the aid of experimental tests. This is very difficult in the case of calcium silicate masonry, because this type of masonry collapses in an ‘uncontrolled manner’ [20]. The fracture energy therefore has a coefficient of variation up to 50% [75]. Although this is not mentioned specifically, it is probably the reason why this parameter was not determined in the ESECMaSE project.

A value of 70 N/m is used\(^2\) as mode I fracture energy. Because the coefficient of variation of this parameter can be large, the sensitivity of this parameter between 10 and 100 N/m is researched in Section 9.2.3.

6.3.2.5 Compressive strength

Masonry can bear more compressive stresses than tensile stresses. That means that the material will sooner collapse under tension than under compression. The compression behaviour of masonry can therefore be modelled in a simpler way than the tensile behaviour. Although a linear elastic compressive stress-strain relationship would be the most simple way, an ideal plastic relationship is used here instead (see Figure 6.6b). This relationship was chosen in order to include some part of masonry’s plastic compressive behaviour.

---

1. Note that only the non-linear material, i.e. masonry, is plotted here.
2. rounded off from 67 J/m\(^2\) [20, Table 12]
The material is fully plastic from the tensile strength 22.7 MPa [33, Table 5] onwards. Appendix E shows that the size of the compressive strength has a marginal influence on the results.

6.3.3 Elastic orthotropic masonry

It was already mentioned in Section 3.3 that the disposition of bricks and mortar enable masonry to have different properties in orthogonal directions. The type of masonry that is considered here is unique because of its square bricks (250 x 250 x 175 mm), unfilled head joints and thin mortar bed joints. The masonry is modelled using a macro-modelling approach which means that a homogenisation technique should be applied to smear out the properties over the elements.

The usage of homogenisation techniques can be difficult. One of the biggest disadvantages is that only the elastic properties can be determined this way. Before such an approach is used, one should be certain whether the advantages weigh up against the disadvantages. The masonry in the ESECMaSE project is homogenised with the two-step homogenization technique from Section 3.3. This results in Young’s moduli and Poisson’s ratio that are not more than 2% different from the isotropic value. Masonry is therefore modelled as an isotropic material.

6.3.4 Poisson’s ratio reduction and shear retention

DIANA 9.5 (released in June 2014) offers the possibility to take into account stiffness degradation in different ways. The smeared-crack models that are outlined in Section 6.3.1 have
a number of options that can be used. They are also relatively new in the development of the software package. Up to DIANA 9.3 (released in October 2008), they were not included and could not be chosen. In the default mode, the non-linear behaviour is implemented through the decrease of Young’s modulus and a constant shear retention factor $\beta$.

Taking into account the degradation of stiffness seems logical from a theoretical point of view. As the material cracks, its the amount of loads that can be carried decreases. This is not only the case in the direction normal to the crack, but also tangential to the crack. Therefore, not only the E-modulus, but also Poisson’s ratio and the shear modulus change. This has to be applied specifically in the model.

The most advanced option enables both quantities to decrease in the same rate as the Young’s modulus. The result is an orthotropic constitutive relation, which is explained detailed in Appendix F. The effect of the different options on the results of a MPOA is explained in Section 9.3. The degradation of Poisson’s ratio and shear modulus is taken into account in all other non-linear analyses.

### 6.4 Load Application Method

There are numerous ways to apply a quasi-static earthquake load. Because the structure globally consists of two dominant masses the load is applied on each separately. This means that the ratio of the loads on each floor can be changed.

A choice that is made in the early stage of construction of the model is to use displacement or load control (Section 6.4.2). Two load distributions are discussed in Section 6.4.1 and the difference between cyclic and monotonic loading is discussed in Section 6.4.3.

#### 6.4.1 Proportional to mode or mass

The load bearing capacity of a structure should be determined on the basis of the following two load distributions according to NEN-EN1998-1 [57, §4.3.3.4.2]:

- **uniform**: proportional to the elevation and mass of each floor
- **mode**: proportional to the first-mode displacement and mass of each floor

A different distribution was used in the ESECMaSE project. The loads on the floors are only the product of the mass of the floors and the input ground motion (see Equation 4.1c). The loads are not proportional to each other, which is the case in the distributions above.

The first-mode shape is a structure’s property. This displacement was applied in Section 9.4.1 to research its effect. Two different distributions are used, one with respect to the floors and one with respect to each element. The following definitions are used for these distributions:

- **mass and mode**: Proportional to the first-mode displacement and mass of each element
- **mode**: Proportional to the first-mode displacement of each floor

The modal pushover analysis in DIANA applies the load according to the first distribution. It should be noted that this analysis is load-controlled and in order to obtain the same base shear its size has to be calculated separately (see Appendix G).

#### 6.4.2 Fixed load proportional or displacement proportional

The load distribution is either applied through displacements or forces. The disadvantage of applying the load through forces is that the post-peak behaviour cannot be obtained. More important is that the peak itself is difficult to obtain, since one does not know whether or not the peak is the maximum peak or not. A way to overcome these problems is to use a displacement controlled analysis.

---

3 Note that some options are not be available for certain models
The difficulty that arises in a pushover analysis is that the load distribution is applied through seismic equivalent forces. A way had to be sought in which the forces were kept in a fixed ratio while the analysis was displacement-controlled. The result was a very stiff auxiliary frame that is able to rotate around a line of control-nodes (i.e. the blue line in Figure 6.9, also see Appendix H). The displacement is applied on the control-node-line and the corners of the frame are tied to the structure.

![Image](image.png)

(a) The auxiliary frame in the finite element model

![Image](image.png)

(b) Sketch of the connection between auxiliary frame and structure

Figure 6.9: Definition of the auxiliary frame

One can ask what the effect is of applying the forces in a fixed ratio or the displacements in a fixed ratio. This is researched in Section 9.4.2. The auxiliary frame is used in the other analyses.

6.4.3 Cyclic and monotonic loading

The cyclic nature of earthquakes is almost never taken into account in quasi-static analyses. In NEN-EN1998-1 [57], the cyclic load is only taken into account for local failure mechanisms, e.g. failure of connections. The hysteretic behaviour is neglected in these cases. The loading history of masonry can play a large role in a structure’s behaviour. Cracks that were formed due to previous loads and that are possibly closed, would open up sooner than a new crack (in another direction) is formed.

This hypothesis is researched in more detail in Chapter 10. An envelope of the shear-drift curve is used to compare the results between monotonic and cyclic pushover analyses. This concept is based on the ESECMaSE project. All the other analyses were carried out with monotonic loading.

4 In the model, the connection is set-up with very stiff spring elements in order to extract the internal forces and displacements, which is not possible from a tied connection
6.5 ANALYSIS PROCEDURE

The non-linear analyses that are executed are both physical and geometrical non-linear. Iterations are performed through a $3 \times 3 \times 3$ Gauss integration scheme and a Regular Newton-Raphson method. It was chosen to use an energy convergence criterion of 0.001. The Cyclic Pushover Analysis (CPOA) is the longest analysis that was carried out. Its calculation time was around 7 hours.

The time to pre-process the model and post-process the results has to be added up. Obtaining the sensitivity of a model can therefore be very time consuming. In order to overcome this drawback, several macros were written with Visual Basic for Applications (VBA) (see Appendix I).
IDENTIFICATION OF MODAL PARAMETERS

The dynamic properties of the model cannot be determined beforehand and have to be identified with the aid of an eigenvalue analysis. They are also called modal parameters. The modal parameters that are identified in this chapter are the mode shape (Section 7.1) and the eigenfrequency (Section 7.2).

Both quantities are compared with the results from the hammer impact test from the ESECMASe project (also see Section 5.3.1). The live load is not included in order to make a valid comparison. The mass density of certain parts of the floors are changed in order to take into account the mass-inertia of the extra loads on the floors in the experimental model (see Figures 7.1a to 7.1d).

![Figure 7.1: Load on the experimental model during hammer impact tests (top) and mass density refinement of the numerical model during eigenvalue analysis (bottom)](image)

7.1 MODE SHAPES

Only a visual comparison can be made between the several mode shapes. The mesh and pushover load in Chapters 8 to 10 are dependent on the first mode shape. The first mode
shape is therefore quantified as the ratio between the displacement of the first floor and second floor (see Equation 7.1).

\[ \alpha = \frac{\bar{u}_1}{\bar{u}_2} \]  

(7.1)

in which:
\( \alpha \) the first-mode-shape (also used in Equation H.1)
\( \bar{u}_1 \) first mode displacement floor 1
\( \bar{u}_2 \) first mode displacement floor 2

The mode shapes from the ESECMaSE tests and eigenvalue analysis are shown in Figures 7.2a to 7.2n. Each mode shape can be compared to its counterpart. There is much similarity between the estimated mode shapes from the ESECMaSE project and the computed mode shapes from the numerical model.

Mode shapes 4 and 7 are slightly different. Since shape 4 is a local vibration mode and shape 7 a second order bending mode, this difference is not important. It should be underlined that the mode shapes from the hammer impact test are only estimations. The value of \( \alpha \) is 0.41 according to the fundamental mode shape in Figure 7.2h. This value is used in the set-up of the seismic analyses (see Appendix H) and later on referred to as the force-ratio\(^1\).

### 7.2 Eigenfrequencies

The eigenfrequencies can be obtained directly from the output of the eigenvalue analysis. The equation that is solved in a free vibration eigenvalue analysis is shown in Equation 7.2a and the result of that equation is given in the form of the eigenfrequency \( f \). The first seven eigenfrequencies are shown in Table 7.1. The largest difference between the eigenfrequencies from DIANA and ESECMaSE is 8%.

\[ K_{el}\phi = \lambda M_{cons}\phi = \omega^2 M_{cons}\phi \]  

\[ f = \frac{\omega}{2\pi} \]  

(7.2a)

(7.2b)

(7.2c)

in which:
\( K_{el} \) linear elastic stiffness matrix (N/m)
\( M_{cons} \) consistent mass matrix (kg)
\( \phi \) eigenvector (m)
\( \lambda \) eigenvalue
\( \omega \) natural circular frequency (rad/s)
\( f \) frequency of motion (Hz)

---

\(^1\) Note the difference in definition with Equation 7.1.
Figure 7.2: Modal shapes from ESECMASE (top) and DIANA (bottom)

<table>
<thead>
<tr>
<th>MODE</th>
<th>VIBRATION TYPE</th>
<th>ESECMASE</th>
<th>DIANA</th>
<th>DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1st longitudinal bending</td>
<td>8.03</td>
<td>7.42</td>
<td>8%</td>
</tr>
<tr>
<td>2</td>
<td>1st transversal bending</td>
<td>16.63</td>
<td>17.49</td>
<td>5%</td>
</tr>
<tr>
<td>3</td>
<td>1st bending of the 2nd floor</td>
<td>19.02</td>
<td>19.59</td>
<td>3%</td>
</tr>
<tr>
<td>4</td>
<td>1st bending of the 1st floor</td>
<td>21.33</td>
<td>21.37</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>5</td>
<td>1st torsion</td>
<td>21.46</td>
<td>22.54</td>
<td>5%</td>
</tr>
<tr>
<td>6</td>
<td>1st bending of the free wall</td>
<td>24.11</td>
<td>25.58</td>
<td>6%</td>
</tr>
<tr>
<td>7</td>
<td>2nd longitudinal bending</td>
<td>28.92</td>
<td>26.80</td>
<td>7%</td>
</tr>
</tbody>
</table>

Table 7.1: Dynamic identification
Part III

NUMERICAL SEISMIC ANALYSES OF THE ESECMASE BUILDING

The numerical model from the previous part is set out against the experimental model in the ESECMASE project. The pseudo-dynamic test and pushover analysis are both quasi-static methods. That means that mass-inertia and viscous damping are neglected. Quasi-static analyses can therefore only be used to research ‘slow or creeping failure’ [9].

The results of a non-linear monotonic pushover analysis are presented (Chapter 8). The assumptions in material properties and modelling methods that were done in an earlier phase of the research are established by a sensitivity study (Chapter 9). This study can be regarded as substantiation of the monotonic pushover analysis and the modelling aspects presented in Chapter 6. The results of a cyclic pushover analysis are discussed in Chapter 10.
The Monotonic Pushover Analysis (MPOA) is a non-linear quasi-static seismic analysis. The results of this analysis are a starting point for the sensitivity study in Chapter 9. Different stages in the behaviour are disaggregated:

1. Initiation stage
2. Pseudo-linear stage
3. First severe crack stage
4. Crack propagation stage
5. Collapse stage

The stages are discussed in order of appearance in Sections 8.1 to 8.5. The main results are the shear-drift curves (see Figure 8.2) and the locations of the cracks. It was chosen to present only these results because they can be compared with the results from the ESECMaSE project (Section 5.3.2). The reader is referred to Appendix J for a broader overview of the results, i.e. stress and strain distributions at the end of each stage.

The Adaptive Pushover Analysis (APOA) is an advanced variant and can be performed in order to model the structure’s behaviour more realistically and take into account the evolution of modal parameters. This evolution is discussed in Section 8.6.

**Live load** The load on the structure is different from the eigenvalue analyses in Chapter 7. The main difference is the presence of water tanks to account for live loads (see Figure 8.1). The load from the water tanks (red circles), safety frames (blue triangles) and hydraulic actuators (purple circles) is taken into account through distributed loads.

![Figure 8.1: Load on the experimental model during PsD tests [6]](image)

(a) Loading conditions calcium-silicate specimen floor 1  
(b) Loading conditions calcium-silicate specimen floor 2

**8.1 Initiation stage**

The gravity is the only force that is applied in the initiation stage of the analysis. As a result, the shear force in the shear-drift curve of both levels\(^1\) is negative. This force is a direct result from the connection to the auxiliary structure, which is a horizontal translational constraint. It prevents the model to slightly sag rightwards. This sag would normally occur because of the asymmetrical geometry. Because the left side of the structure in Figure 8.3a

\(^{1}\) Note that level 1 is used for the ground floor and level 2 for the first floor
is stiffer, it deforms less than the other side. As a result some small cracks in level 2 appear (see Figure 8.3b).

The application of gravity is not a step that was separately applied in the PsD test. During the construction of the experimental model, gravity was of course always present, which caused a gradual increase in (compression) stresses while the construction work was progressing. The casting of the concrete slabs was carried out with the use of form-
work. Measurements to the structure’s response were done during the removal of this formwork.

These displacements were in the order of 0.01 mm [7]. It can be assumed that this also lead to some small (micro) cracks. These cracks are not visible and are therefore not reported. The displacements in the numerical model are of the same order. It is assumed that the fact that the frame slightly hangs on the auxiliary structure can be neglected. The off-set of the shear-drift curve with respect to the stages of loading is small.

8.2 PSEUDO-LINEAR STAGE

The second stage is characterized by an almost linearly increasing shear-drift curve. The stiffness of the numerical model is clearly larger than the shear-drifts of the PsD envelope. The formation of new cracks during this stage show that the behaviour is not linear.

The tests that can be compared with this stage are the 0.02g, 0.04g and 0.06g tests. The drifts in those tests are about the same as the drifts in the model. The crack patterns that were observed during and after these tests are used to compare the behaviour (see Table 5.2). The horizontal flexural cracks in the long walls (pattern F) are clearly visible in Figure 8.4b.

Altogether, the model responds similarly as the test set-up, but not exactly the same. The properties that can be quantified coincide (to some extent) and the properties that cannot be quantified from the test (such as the stiffness) do not coincide.

8.3 FIRST SEVERE CRACK STAGE

The beginning and end of the next stage is defined by the formation of a crack in the base of the left long wall (Figure 8.5b). This process indicates the start of a snap-through
response in the shear-drift curve. The formation of the crack at the base is part of a slightly overturning deformation of the structure. It can also be formulated as rocking of the individual components and opening of the cracks between the shear walls and slabs.

The displacements coincide with the 0.08g PsD test. Anthoine and Capéran [6] reported the behaviour as ‘more noises’ and ‘not any significant new cracks’. This is different from the behaviour of the numerical model. The latter type of cracks can be placed among the response of the 0.06g test. The numerical model catches up with the experimental model.

This is also resembled in the shear-drift curve (Figure 8.2). The relatively sudden formation of a crack over the whole length of the long wall is accompanied by an energy release. This energy release is shown in the decay of the shear force and it cannot be found in the results of the PsD test. The fact that this crack occurs suddenly can be caused by the approximative nature of smeared-crack models in general.

The local formation of cracks can simply not be modelled in a smeared manner. The behaviour and deformation of the structure requires the formation of this crack and lead to a build up of energy that is released in a more sudden way than in the experimental test. The size and type of the behaviour is a local effect that can be attributed to the smeared-crack model. It marks the transition from a linear shear-drift curve to a non-linear curve.

8.4 Crack Propagation Stage

Existing cracks open and new cracks form. The opening of the cracks in patterns 1 and 2 follows from the large deformations of the elements in these locations (see Figure 8.6b). The cracks at the base and top deform extensively. The left-hand walls of level 1 crack along their base and the right-hand walls of level 2 crack along their top. This behaviour matches part of the behaviour in the 0.14g test (also see Table 5.2).
The diagonal (or stepwise) cracks that occur in the PsD test only occur partially in the model. Figure 8.6b shows that crack pattern S has not formed completely. All three crack patterns that were discussed in Section 5.3.2.1 are present. It is not possible to quantify them more, but the occurrence of the different patterns is considered to be sufficient to conclude that the behaviour of the cracks is the same.

The shear-drift curve has a more or less horizontal plateau. The curve after the linear part and before the peak, i.e. the curve in this stage, is also the part from which the (shear) capacity is usually determined. The shear capacity is obtained by averaging the shear forces of the levels within this part of the curve only. This results in a capacity around 180 kN for level 1 and 130 kN for level 2. A clear difference with the ESECMASE envelope is that the shear capacity is respectively 45% and 60% higher for level 1 and 2.

8.5 collapse stage

The last stage is characterized by the formation of a stepwise crack in the right shear wall on level 1 (also see Figure 8.7b). This crack significantly reduces the stability of the structure, because there are no structural elements that are able to carry the shear load in the right-hand side on level 1. This crack is also present in the experimental model at the end of the 0.16g test.

![Figure 8.7: Results after collapse stage](image)

There is only one other crack that forms afterwards in the PsD test, which is a severe crack at the right-front corner of the top of the structure. This crack appears in the numerical model in an earlier stage of the structure’s response (see Figure 8.6b). The shear-drift curve shows a gradual decay of about the same rate as its increase in the previous stage. It should be noted that this decay in shear-force did not occur in the PsD tests.

The type of crack and deformation show that the structure has collapsed. The large displacements and local distortions of the elements are unrealistically enough to support this. In reality, the edge of cracks in the shear wall of level 1 would not deform back to their original state as they do here. Fact is also that convergence of the last steps took place in an unstable manner. Small differences in step sizes and solution procedure lead to a slightly different shear-drift curve and lower amount of converged steps.

8.6 evolution of modal parameters

It is clear from Figure 8.7 that the properties of the structure have changed. The cracks have lead to a permanent degradation of the structure’s stiffness. It is not possible that cracked elements change back to their original state because of the energy that is dissipated throughout the analysis. The structure after the last converged step is therefore different from the structure before the analysis.

---

2 Take in mind that the displacements are enlarged a hundred times
Since the pushover load (on the auxiliary frame) is based on the modal properties of the structure, one can presume that this load should also change in the same manner. The response of a real structure to an earthquake is related to its current state and not its original state, which could be decades ago.

The Adaptive Pushover Analysis (APOA) takes into account changes in stiffness and the subsequent effect of higher modes [8, 17, 30, 36, 64]. Such an analysis is conceptually complicated and requires a lot of computational effort. It is difficult to apply.

In order to determine the effect of an APOA, the analysis is stopped at several points and followed by an eigenvalue analysis. The results are used in a dynamic identification of the modal parameters (see Figures 8.8a and 8.8b). The modal shape, which is used to determine how the auxiliary structure is constraint, is then re-computed. This results in Figure 8.9 where one can see how the modal parameters change during the monotonic pushover analysis. Note that $\beta$ originates from the last step in the pushover analysis and $\alpha$ from the eigenvalue analysis.

It is a logical result that during the cracking process the stiffness and the first eigenfrequency of the structure decrease. Only the application of gravity and the second last load step slow a slightly different behaviour. The self weight results in a compressive stress in most of the structure. It is generally known that rock-like materials such as concrete and
masonry can bear more load under compression and therefore respond stiffer under such a load.

The increase in eigenfrequency in one of the last steps could be the results of a redistribution of the stresses. It is possible that the load is carried in a different way and allowing a slightly stiffer response of the structure. On the other hand, both of the increases in eigenfrequency are small.

It is interesting to see that the ratio of first-mode displacements $\alpha$ increases. This means that over time, the stiffness of level 1 approaches the stiffness of level 2. Suppose that $\alpha$ is 0.5, then the displacement of floor 1 is exactly half the displacement of floor 2 in the first mode. The displacement of floor 1 is not merely dependent on the stiffness of level 1 (see Equation 4.1b). The rotation of level 2 also causes a rotation of the walls in level 1.

There is also the intersection between $\alpha$ and $\beta$ in Figure 8.9. This intersection coincides with the end of the first severe crack stage. Beyond this intersection, the shear-force ratio of the floors is higher than the displacement ratio. A higher relative force is required on level 1 with respect to a relative displacement of floor 1.

It is very difficult to draw firm conclusions, because both modal parameters in Figure 8.9 are relative. Therefore, only conclusions with respect to the dependency of the levels are discussed. They do not have a dimension, but it can be concluded that there are relatively more cracks on level 1 than on level 2. Cracking is the only quantity that is discussed here that is part of the memory of the model and the reason why the stiffness decreases. This is clearly visible from the crack patterns in Appendix J.

Another conclusion can be drawn based on the relationship between the ratio of the forces $\beta$ and the corresponding modal shape $\alpha$. The auxiliary frame is able to rotate depending on the stiffness of each level. Since the modal shape also depends on the stiffness, both are indirectly related to each other.

The change in $\alpha$ is only small and the effect of different $\alpha$’s on the shear-drift curve is negligible (see Figure 8.10). The amount of effort required to do an APOA does therefore not outweigh the effect it has on the results. $\alpha$ increases only 8%.
Figure 8.10: Shear-drift curves for different force ratios
The sensitivity to the modelling aspects that were assumed in Chapter 6 is researched. The parameters that were eventually used are based on this study. It can be seen as an iterative process in which the results of the sensitivity to one parameter lead to changes in the model which is thereafter used to research the sensitivity of another parameter. The Monotonic Pushover Analysis (MPOA) in Chapter 8 is used as starting point for this sensitivity study. The reason to perform this research was the high coefficient of variation and uncertainty of certain parameters.

The results are presented in the order of increasing sophistication of the model. The different smeared-crack models are discussed in Section 9.1. The sensitivity of some of the options in masonry’s stress-strain relationship are researched in Section 9.2 and the effect of different damage based models is outlined in Section 9.3. The sensitivity to different load application methods is discussed in Section 9.4.

9.1 FIXED, ROTATING AND MULTI-DIRECTIONAL SMEARED-CRACK MATERIAL MODELS

Monotonic pushover analyses with three different smeared-crack models were carried out. The smeared-cracks models that are discussed in Section 3.3 are used with the properties that are shown in Table 9.1. The same properties as in the total strain fixed single crack model are used as far as possible. Not all the properties could be chosen the same, because this is inherently the difference that is researched here.

<table>
<thead>
<tr>
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<td>constant</td>
</tr>
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<tr>
<td>Compression softening diagram</td>
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</tr>
<tr>
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<td>70</td>
</tr>
<tr>
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</tr>
<tr>
<td>Poisson’s ratio reduction</td>
<td>damage</td>
<td>damage</td>
<td>none*</td>
</tr>
</tbody>
</table>

* this is default and no other option can be chosen
† this is taken into account via rotation of the constitutive matrix

Table 9.1: Non-linear smeared-crack material properties

Regarding the shear-drift curves of the analyses, the differences are only small (see Figure 9.1). The differences can be found in the shear capacity and ability to converge in the collapse stage. It can even be said that only the fixed single crack model shows post-peak behaviour. One can conclude from that, that a shear capacity that is determined on the basis of the results of the other two models is not correct, simply because the peak is less certain to be the maximum peak.

The difference in the crack patterns is also small (see Figure 9.2). The connection between the left exterior shear walls and the first floor slab is slightly more cracked in Figures 9.2b
Figure 9.1: Shear-drift curves of different smeared-crack models

and 9.2c. The stepwise crack in the right shear wall of level 1 is not visible in the rotating single crack model. The fixed single crack model is eventually used for all other non-linear analyses, because there are only small differences.

Figure 9.2: Normal crack strains in the step with the maximum shear in different smeared-crack models
The relationship between the stresses and strains in masonry is commonly presented in the from of a stress-strain diagram. There are various parameters that influence the shape of this diagram (see Sections 2.4 and 6.3.2). The correlation between some of these parameters is quite high. If, for instance, only the tensile strength is increased then the ultimate strain changes as well. If this results in a more brittle behaviour, then that is not solely the result of a higher tensile strength but also because of the lower ultimate strain.

The material parameters have an influence and this correlation only makes it more interesting to discuss it. The relationship between the parameters can be researched in many ways. The approach that is used here is that only one aspect of the stress-strain relationship is varied. In some cases that meant that two parameters had to be changed in equal rate. The aspects of which the sensitivity is researched are shown in Figure 9.3a. The influence of these aspects are discussed from left to right in Sections 9.2.1 to 9.2.4.

![Diagram](image)

(a) Subjects of the sensitivity study to masonry’s stress-strain relationship

![Diagram](image)

(b) Bandwidth of research to tensile strength

![Diagram](image)

(c) Bandwidth of research to mode I fracture energy

Figure 9.3: Research to the sensitivity of masonry’s stress-strain relationship

### 9.2.1 Young’s modulus

As discussed in Section 6.3.2.1, the obtained Young’s modulus from a masonry panel can be very different per test. The coefficient of variation is in fact too high to determine a single value that fully represents every interaction between unit and joint. In order to assess the consequence of using an E-modulus that is not representative for the complete material, two analyses were carried out. One with the Young’s modulus computed from NEN-EN1996-1-1+A1 [56] (see Equation 6.2) and one with the intersection of the result from the hammer impact test in Figure 6.7.

The shear-drift curves of the analyses are shown in Figure 9.4. Changing the Young’s modulus has most effect on the pseudo-linear stage. This makes sense because the stiffness of the structure is higher in case of a higher E-modulus. It even seems that the curves for both levels are horizontally scaled. Apparently, there is no influence on the shear capacity and the structure cracks in the same manner. This is a logical result, because, from the first severe crack stage onwards, the structure cracks and behaves completely non-linear. The change
in the non-linear region of the stress-strain relationship is only small (see Figure 9.3a). It is therefore concluded that the influence of a slightly higher or lower Young’s modulus is negligible.

### 9.2.2 Tensile strength (and constant ultimate strain)

The influence of permitting a higher stress level in the material before it cracks is researched. Both the tensile strength and mode I fracture energy are modified to keep the ultimate strain constant between the analyses (see Figure 9.3b).

It is shown in Figures 9.5a and 9.5b that using the tensile strength of units (i.e. 1.67 MPa) leads to considerable different results. The behaviour of the structure in terms of crack-locations and stages of failure is still similar, but the peak load changes drastically. The difference is visible from the first severe crack stage onwards. This is logical because the effect of the higher tensile strength is only measurable after the peak in the stress-strain curve of the material model and thus after the formation of cracks. The fact that the peaks in shear are higher is a direct result from the higher strength.

An unexpected result is the difference in snap-back behaviour. The drift after the formation of the first severe crack even decreases. Not only the peak is higher, but also the decrease in shear. The shear in the structure is higher after the formation of this first se-
9.2 Masonry Stress-Strain Relations

Figure 9.5: Results of analyses with different tensile strengths

vere crack. This type of response can be attributed to the more brittle material model. The difference between the ultimate strain and the elastic strain of masonry becomes smaller as the tensile strength increases.
The snap-back behaviour occurs when the base of the left long wall changes from uncracked to fully cracked in only a small change in deformation. The more brittle response has an effect on the suddenness of the crack (see Figure 9.5c). Because of the crack, stresses are redistributed, the auxiliary frame rotates and the ratio between the displacements $\beta$ increases.

It is remarkable that this process is more gradually in the 0.28 MPa analysis than in the others. In those analyses it takes only one step to go from a displacement ratio of 0.40 to 0.46. It was not possible to perform an analysis with smaller step sizes that converge beyond this peak. The response of $\beta$ in the last three analyses is about the same up to half of the crack propagation stage.

![Normal crack strains in 0.28 MPa model](image)

![Normal crack strains in 0.75 MPa model](image)

![Normal crack strains in 1.20 MPa model](image)

![Normal crack strains in 1.67 MPa model](image)

**Figure 9.6:** First severe crack with respect to different tensile strengths

The crack patterns in Figures 9.6a to 9.6d support this explanation. The higher the tensile strength, the more sudden the crack at the base. There are also almost no neighbouring cracks that indicate an imminent collapse. The three analyses with the highest tensile strength share the property that there are no stepwise cracks in the shear walls (crack pattern S).

It can be concluded that a higher tensile strength leads to a higher shear capacity. The local effect of the first severe crack in the shear-drift curve increases, which is not as much present in the experimental model. This means that not all crack patterns occur in the model’s behaviour. It underlines the statement that the tensile strength in a smeared-crack masonry model should be based on the tensile strength of a masonry specimen (see Section 2.4.4). Higher tensile strengths result in a response that is not only more brittle, but also unrealistic with respect to the $P_sD$ test from ESECMaSE.
9.2.3 Mode I fracture energies (and constant tensile strength)

Another parameter of which the influence is researched is the mode I fracture energy. This parameter is directly related to the ultimate strain, but also to the slope of the softening curve (see Figure 9.3c). Because the coefficient of variation for this parameter can be 50% \([75]\), it is varied between 10 and 100 N/m, which is -85% to +40% with respect to 70 N/m.

![Shear-drift curve of level 1](image)

(a) Shear-drift curve of level 1

![Shear-drift curve of level 2](image)

(b) Shear-drift curve of level 2

Figure 9.7: Shear-drift curves for different Mode I fracture energies

The shape of the shear-drift curve of the different analyses is roughly the same (see Figure 9.7). The difference seems merely to be a scale factor. It does not make a difference to compare their crack patterns, because there are almost the same.

The lower the fracture energy, the lower the peak during the first severe crack stage and the lower the shear capacity. This seems to be related to the fact that cracks were fully opened in an earlier step. The ultimate strain is reached earlier and the deformation capacity is lower.

It is not only more difficult to obtain the shear-capacity from this smaller and less horizontal plateau, but the structure also collapses due to a lower displacement. In earthquakes, displacements increase and decrease cyclically. Structures that are able to resist more drift can also resist larger earthquakes, because it is less likely that they will collapse. A large earthquake will cause damage, but people will be able to leave the building before it collapses.
A structure with a smaller horizontal plateau is probably less safe than one with a large plateau. The fracture energy of 70 N/m that is used in Chapter 8 can be regarded as a good value. The horizontal plateau of the shear-drift curve is sufficiently large to determine the shear capacity. One can postulate that calcium silicate masonry composed of weaker mortar or with a more brittle composition (for example because of existing cracks) has a lower mode I fracture energy and will collapse relatively quicker.

9.2.4 Linear, exponential and Hordijk tensile softening curves

The shape of the softening curve can also have an influence on the structure’s behaviour. In most cases, linear softening is an adequate approximation. The behaviour of masonry’s constituents separately is more of an exponential type of shape (see Figures 2.3 and 2.7). Several analyses with different tensile softening curves are carried out. The mode I fracture energy is kept the same.

![Shear-drift curve of level 1](image1)

Figure 9.8a: Shear-drift curve of level 1

![Shear-drift curve of level 2](image2)

Figure 9.8b: Shear-drift curve of level 2

The shear-drift curves that are obtained from analyses with different tensile softening curves are shown in Figures 9.8a and 9.8b. Some differences can be found in the first severe crack stage and the collapse stage. The behaviour of the structure is the same. This means

Note that the masonry in ESECMA is already quite weak because of the thin mortar joints and unfilled head joints.
9.3 Damage based shear retention and Poisson’s ratio reduction

that the shape of the softening curve, and inherently the ultimate strain, only have a minor influence on the shear capacity. The long tail of the diagrams in Figure 9.9 show that the energy release after the formation of the crack is less sudden. The snap-through in the shear-drift curve is smaller and the stress during the formation of the first severe crack decreases slower.

![Stress-strain relations of different tensile softening curves](image)

Figure 9.9: Stress-strain relations of different tensile softening curves

The fact that the Hordijk and exponential softening model are able to resist higher strains before an element is fully cracked is clearly shown in the collapse stage. The size of the stepwise crack in the last converged step is substantially larger for these models (see Figure 9.10).

This phenomenon comes from aggregate interlock in concrete. It enables concrete to transfer some tensile stresses after it is already cracked. It can be questioned whether masonry also has this property. Units in masonry can also be regarded as (large) aggregates.

It makes sense that masonry is capable to transfer some post-peak stresses. However, the additional strains that are brought along with it are somewhat unrealistic. When masonry is deformed up to such an extent, the stress could very well be overestimated in the Hordijk and exponential softening curve. The extra capacity of the material in the higher strain regions can therefore be regarded as not appropriate for masonry.

9.3 Damage based shear retention and Poisson’s ratio reduction

There are four different options in DIANA regarding the stiffness degradation due to cracking (see Section 6.3.4). The reduction of Poisson’s ratio and the damage based shear retention can both be included and excluded. This leads to four material models with which monotonic pushover analyses are performed to investigate their effect.

The shear-drift curves in Figure 9.11 show that the effect of taking into account the changing stiffness during cracking logically takes place from the first severe crack stage onwards. This is also the case with the crack patterns (see Figures 9.12a, 9.12d and 9.12g). One can also notice that shear retention plays an important role in the behaviour of the model. Both models with a damage based shear retention show a considerably lower ca-
capacity and, more important, not a gradually increasing stiff behaviour. The behaviour of the other two models is almost linear in the crack propagation stage.

There is also a difference between the two models with a constant shear retention factor. It is remarkable that the same difference is not present in the two models with a damage based shear retention. This difference is not noticeable until after the first severe crack stage (see Figures 9.12b and 9.12e). There is no stepwise crack in the interior shear wall on level 1 in the model with no Poisson’s ratio reduction (see Figure 9.12b), which is the case in Figure 9.12c.

This effect is enlarged as the cracking continues. In one of the later stages, the behaviour of the cracks due to rocking and the horizontal flexural cracks goes further and further apart (see Figures 9.12c and 9.12f). Predominantly the connections between the walls and slabs respond differently in models with and without a damage based Poisson’s ratio reduction (respectively model B and A). The model with this reduction shows the least expected behaviour. The exterior shear wall on level 1 seems to displace almost independently of the slab and base. The other differences are only a matter of larger normal crack strains.

An explanation can be found in the difference between the constitutive relationship of the models (see Equations (F.4a) and (F.5a)). The lateral reduction of stiffness enables that once an element is cracked, they are sooner fully cracked. This results in larger displace-
ments, because the fully cracked element has less stiffness. This is clearly shown in the wall to slab connections in Figures 9.12c and 9.12f.

Damage based shear retention is important for the behaviour of the model. A damage based Poisson’s ratio reduction combined with a constant shear retention factor of 0.2 can lead to results that are the most different from the PsD tests. Although both damage based shear retention and Poisson’s ratio reduction are not that much different from damage based shear retention itself, both are used. That conclusion is based on the possible effect of a too high Poisson’s ratio and the theory that this effect is also present in real masonry.

9.4 Load application methods

There are numerous possibilities to apply a quasi-static seismic equivalent load (see Section 6.4). The effect of applying the load in a certain way is researched in this section. The influence of the size of the force vector is discussed in Section 9.4.1 and the influence of the quantity with which the load is applied is discussed in Section 9.4.2.

9.4.1 Mass proportional or not

The ability to perform a MPOA directly from the results of a dynamic analysis is possible since this version of DIANA, version 9.5. In contrast with the regular mode-proportional pushover analysis, the load is applied through an equivalent modal force vector. There are mainly three differences:
1. The analysis is force controlled;
2. The forces are applied to all the nodes;
3. The mass of the nodes is taken into account.

Because there are no restoring forces on the first floor, only one force-displacement curve can be drawn (see Figure 9.13). This curve is based on the top displacement and the base shear (see Figure 9.13).

Figure 9.13: Base shear-top displacement curves for different load vector pushover analyses

There are two differences between the shear-drift curves. One is that the mass proportional analysis does not converge beyond the formation of the first severe crack and two is
that the base shear is slightly higher. The first is a direct consequence of using a load controlled analysis. Such an analysis cannot overcome the snap-back behaviour in the mass and mode proportional curve. It is also unable to converge in the higher-displacement steps.

The second can only be a result from the difference between the force vectors in Figures 9.14b and 9.14d. The force, as is shown in the first figure, is only a rough approximation of the first mode shape. The distribution in Figure 9.14d is based on the exact mode shape (and the product with the nodal equivalent mass). Because the mode proportional analysis is unable to overcome the first severe crack stage, it is considered to be unsuitable to research the seismic behaviour of this model.

9.4.2 Fixed force and fixed displacement proportional

The difference between a load controlled analysis and displacement controlled analysis becomes clear from the shear-drift in Figure 9.15. The distributions in both analyses are proportional to the first mode. Both analyses are displacement controlled in order to overcome the first severe crack stage. The difference is that either the ratio between the forces or the ratio between the displacements is fixed. This makes the analyses similar to force and displacement controlled analyses. The auxiliary frame is only used in the case where the ratio between the forces is fixed. In the other analysis, the displacements are directly applied on the protrusions of the slabs.

The difference between the shear-drift curves becomes visible from the first severe crack stage onwards. Right after the cracks at the base are formed, the fixed-displacement ratio model behaves differently. While the shapes of the shear drift curve of both levels are the same in the fixed-force ratio analysis they are completely different in the other. Level 1 in the fixed-displacement analysis has a lower shear capacity. In fact, there is not even the slightest sign of snap-back behaviour. The shear force decreases more after the peak and reaches a horizontal plateau almost immediately.

The second level shows the exact opposite behaviour. After the first severe crack, the shear force keeps on increasing. The curve does not reach a horizontal plateau. The last converged load step is also the step with the highest shear. This shear force is higher than the shear capacity in the fixed-force ratio analysis.

This difference is less clear from the crack patterns. Figure 9.16a shows the locations of the cracks in one of the last steps of the fixed-displacement ratio analysis. The main differences are the crack between the top floor and left exterior shear wall and the stepwise crack in the interior shear wall on level 2.

All the connections between the second floor and the walls are fully cracked in Figure 9.16a. The second floor displaces almost independently of the other parts of the structure. Unlike in the other analysis, this wall is exposed to high strains, which lead to a full crack at the top of the walls. It almost seems as if the top floor is pulled off from the structure.

These results can be explained by the fact that the (rotation of the) auxiliary frame in the fixed-force ratio analysis is able to find a new kind of equilibrium that is based on the stiffness of both levels. One can say that this is a kind of Adaptive Pushover Analysis (APOA), because the changes in the properties of the structure are taken into account by the load on the structure. This adaptation follows from the rotation of the auxiliary frame.

In the fixed-displacement ratio analysis, this is not the case. There, the structure is forced to follow a certain displacement field. As soon as the cracks, and the way the forces are transferred prohibit this displacement, the analysis fails to converge. The lack of convergence is possibly caused by the application of the seismic load and not the stiffness degradation. The observations during the PsD tests do not mention such a large discrepancy between both levels. It is concluded that a fixed-force ratio analysis better simulates the behaviour of this model during a PsD test than a fixed-displacement ratio analysis.
Figure 9.15: Shear-drift curves of different load applications

(a) Shear-drift curve of level 1

(b) Shear-drift curve of level 2

Figure 9.16: Normal crack strains in a fixed-displacement ratio (left) and fixed-force ratio analysis (right)
In most pushover analyses, the word ‘monotonic’ is omitted. The Monotonic Pushover Analysis (MPOA) is generally considered as standard. The cyclic nature of earthquake loads can have an impact on the crack patterns and hence the structure’s behaviour. It implies a load in positive and negative direction, which causes cracks to (partly) open, close and re-open.

In this chapter, the model that was extensively discussed in the previous chapters is loaded cyclically. This is considered to be the last step in the research to the sensitivity of the model. It is explained how this load is applied and what the results are in terms of shear-drift curves (Section 10.1) and crack patterns (Section 10.2).

**Cyclic Seismic Load** In order to perform an analysis that is similar to the Pseudo-dynamic (PsD) test, the *pseudo-dynamic equivalent control-node displacement* is computed. This equivalent displacement is computed from the forces and displacements in every step of the PsD test.

The selection of the total number of steps is based on the shear-drift envelope of the PsD test (see Figure 5.5). The minimum/maximum shear/drift of level 1/level 2, resulted in eight equivalent displacements per intensity. The maxima and minima of these eight values are used to impose a displacement on the control node (see Figure 10.1). That the equivalent displacements are based on the ratio of forces that was used in the PsD test, but they are applied in the Cyclic Pushover Analysis (CPOA) in a fixed ratio that is determined from the dynamic identification in Chapter 7. This is a generalization that has to be made because of the auxiliary frame. It is not possible to change the mesh of the frame during the analysis.

---

1 Note that the first load case is the self weight
10.1 SHEAR-DRIFT CURVES

The envelope shear-drift curve of the PsD test and the full shear-drift curve of the MPOA and CPOA are shown in Figures 10.2a and 10.2b. The pushover-envelope has a similar shape as the pseudo-dynamic-envelope, but is different from the one in the Monotonic Pushover Analysis (MPOA). None of the points of equal intensity coincide and the envelope does not follow the same path around the origin. The similarity between both envelopes is less for
level 2 than for level 1. The shear-drift envelope of level 1 in the positive direction is most close to the envelope of the PsD test.

The definition of the different stages from Chapter 8 cannot be used here, because there are not as distinctive. The snap-back in the first severe crack stage is visible in both curves. This supports the statement that this crack is a local effect once more (also see Section 8.3). The first severe crack can be regarded as a phenomenon that is typical for this numerical model but not for this experimental model. It is likely that this crack is caused by the use of a macro modelling approach. The interaction between units and bricks is a lot more simplified.

The shear-drift curves can be quantified with the shear capacity. In contrast to the monotonic shear-drift curve, the shear capacity is determined in the positive and negative direction. It is computed from the average of the envelope-drifts that are more than 3 millimetre (see Table 10.1). This value is chosen because the shear-drift curves are almost horizontal beyond that point.

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<tr>
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</table>

* from Section 8.4

Table 10.1: Shear capacity of the PsD test, the CPOA and the MPOA

10.2 CRACK PATTERNS

The different stages in which the formation of the cracks is reported (see Table 5.2) are discussed separately. The load cycles of equal intensity as in the PsD test are discussed from beginning to end in Sections 10.2.1 to 10.2.5. The results of each separate cycle can be found in Appendix K.

(a) Rocking of the shear walls (R)
(b) Flexural (horizontal) cracks in the long walls (F)
(c) Stepwise cracks in the shear walls (S)

Figure 10.3: Crack patterns

10.2.1 0.02g and 0.04g equivalent cycles

No damage was detected after the PsD test with intensities 0.02g and 0.04g. It was assumed that the non-linear behaviour was caused by rocking of the shear walls. This coincides with the results in Figure 10.4. The cracks between the slabs and shear walls (crack pattern R) are clearly visible. The values of the normal crack strain also show that the cracks are small.
If this would be the exact same behaviour as in the test, then that explains why there no damage was detected. The cracks were probably too small for visual detection.

A possible difference in behaviour with the tests is that there are no horizontal cracks (crack pattern F) along the base of the left long wall on level 2. This could be a result of the fixed force ratio that is applied via the auxiliary frame. As discussed in Section 8.1, this could be a result of the application of the gravity, which is different from the PsD test. The gravity always exerted a force during the construction of the experimental model and the geometry of the numerical model is idealized. The numerical model does not deform due to gravity prior to the attachment of the auxiliary frame (or hydraulic actuators in the experimental model).

The existence of these cracks indicate that the non-linear behaviour that was detected, but not observed, originates not only from rocking of the shear walls but also from the flexural cracks along the long walls. This cannot be proven because the cracks in both models are small. There is no evidence that the horizontal cracks were also present in the PsD test.

10.2.2 0.06g equivalent cycle

At the end of the PsD tests, flexural horizontal cracks were observed at the base of all the long walls. There were also noises heard during the test. The horizontal cracks are also found in the numerical model, but have not yet fully developed in the left-hand of Figure 10.5. It is not clear if this was also the case in the PsD test, since only ‘some’ horizontal cracks were reported.
Something that was not reported in the PsD test, is the crack at the bottom of the connection between the shear wall and long wall on level 1 (lower right on Figure 10.5a). This crack is a result of the displacement that is imposed during this load step. According to Anthoine [5], this connection ‘belongs [sic] to the long walls’ and it is considered to be a rigid connection. Although this crack is not mentioned in the observations that were done during the PsD tests, it can be an explanation for the noises that were heard. The formation of a crack in such a rigid connection could be accompanied with a large energy dissipation, that was might heard through noises.

10.2.3 0.08g and 0.10g equivalent cycles

The tests that are equivalent to the fourth and fifth cycles in the pushover analyses revealed no significant new cracks. The biggest difference between the previous cycle (Figure 10.5) and these cycles (Figure 10.6) are that there are new stepwise cracks and flexural cracks. One can dispute whether these are significant cracks or not, because the number of new cracks is large but the normal-crack strain is small.

One thing that is clear, is that the strain in the existing cracks increased and the cracks opened up. It can be assumed that has also happened in the PsD test. A relatively large displacement, without the formation of new cracks, hypothetically leads to a development of the existing cracks.

![Figure 10.6: Normal crack strains after 0.10g equivalent cycle](image)

One of the new cracks that develop in these cycles is the stepwise crack in the right shear wall on level 2. The elements show a diagonal cracking pattern from the left toe to the upper corner. Another new crack is the flexural crack at the base of both long walls on level 1. In the PsD tests, this crack already occurred during the 0.06g test.

![Figure 10.7: Normal crack strains at the end of the first severe crack stage in the MPOA](image)
The suddenness in which this first severe crack is formed (it takes only two cycles for the crack to fully open) is also discussed in Section 8.3 and Chapter 9. It is remarkable that the relatively sudden crack is also present in the MPOA (see Figure 10.7). Although cyclic and monotonic analyses are different from each other in terms of load application, this crack occurs in both analyses in a short change of displacement. The fact that the load is cyclically applied does not prohibit the suddenness with which this crack occurs.

10.2.4 0.12g equivalent cycle

A whole series of new cracks developed in the 0.12g PsD test. Horizontal cracks opened at the outside and mid-height of the long walls of level 1 and large stepwise cracks formed in the right exterior shear wall and interior shear wall. It can be questioned if these cracks would occur in the same manner in a smeared-crack model.

![Figure 10.8: Normal crack strains after 0.12g equivalent cycle](image)

The latter cracks, at mid-height, seem to be the result of a higher mode out-of-plane displacement of the wall (see Figure 10.9). The intrinsic property of this model, that cracks are smeared out over elements, prevents the local formation of such higher-mode crack patterns. It is suggested to regard the behaviour of structural elements as a whole (see the left sketch in Figure 10.9). The discretized plastic hinges in a rocker member are similar to the flexural out-of-plane bending mechanism. It would be wrong to expect a crack exactly at the row of elements at mid-height of the wall.

This approach is substantiated by the crack patterns in Figure 10.8. Both long walls of level 1 are cracked over (almost) the entire height. If this mechanism would be discretized in its 1st mode (see middle sketch in Figure 10.9), then only the connections at the top and bottom would crack. In this case they are completely cracked, which indicates that a higher-mode out-of-plane mechanism is present. The two (or more) plastic hinges in a rigid rocker member are smeared out over all the elements. It is the behaviour of the whole long walls on level 1 from which can be concluded that the same crack pattern as in the PsD test is present.

The large stepwise cracks in the eastern exterior shear wall on level 1 and the cracks on the outside of the long walls on level 1 are also present in the model (see Figure 10.8). These cracks are either smeared out over the whole structural element or they develop in the connections. All the crack-patterns that were observed in the PsD test also developed in the CPOA.

A crack that is not reported in the PsD test but that is visible in the analysis results, is the crack between the interior shear wall and the long wall. In the experimental model, this connection is made through metal strips. That makes it hard to draw any conclusions from the fact that this crack is not mentioned in Anthoine and Capéran [6].
10.2 Crack Patterns

10.2.5 Last converged cycle

Horizontal cracks along the bottom and the top of the long walls in level 2 were detected at the end of the 0.14g PsD test. Stepwise cracks in the left exterior shear wall on level 1 occurred. Although the numerical model could not converge up to the last step of the imposed equivalent displacement, there are cracks from the last converged step that can be compared.

Both types of cracks that are mentioned are in fact already slightly visible in the previous analysis (see Figure 10.8). There are not any significant new cracks in this cycle. The stepwise crack in level 1 closes and the cracks along the base of the right walls open up again, because of the cyclically applied load.

Figure 10.10: Collapse of the structure in last converged step of cyclic analysis

The reason why the step fails to converge can be determined from the cracks at the base. If one takes a closer look at Figures 10.8a and 10.8b, one can see that the only element at
the base that is not cracked, is the element in the connection between the right long wall and the shear wall. Because the load is directed leftwards in the figure, an overturning mechanism is formed as soon as that one last element is fully cracked. This mechanism is exaggeratedly sketched in Figure 10.10b.

It should be mentioned that the experimental model behaves differently. The last PsD test had an intensity of 0.20g. Although the structure did not collapse, there was a severe crack in the top right corner. Despite the fact that this is not the mechanism under which the CPOA fails to converge, that type of crack is present in Figure 10.8b. It can be concluded that most of the cracks from the PsD tests are also present in the results of the CPOA.
The finite element model that is used is based on the experimental model in the *large scale earthquake test on a building* in the ESECMaSE project. It has several masonry walls and two concrete floors. The structure resembles half of a terraced house and was pseudo-dynamically tested.

The approach that gives the closest approximation to these tests, is a model that is composed of solid quadrilateral quadratic elements. The size of the elements is equal to the size of masonry units. The elements that represent masonry are modelled with a total strain fixed crack material model. This model includes a reduction of Poisson’s ratio and a shear retention that is based on the amount of damage. The tensile and compressive strengths are taken from experimental tests on masonry specimens that were performed in the ESECMaSE project. The Young’s modulus and the mode I fracture energy are obtained from other literature. The pushover load is cyclically applied in a fixed force ratio with the aid of an auxiliary frame.

A sensitivity study to several modelling aspects was carried out to ensure that this approach is the closest approximation. This study is performed with the aid of a Monotonic Pushover Analysis (MPOA). One of the results is that the Young’s modulus has a negligible influence on the shear capacity and crack-patterns. A higher tensile strength or lower fracture energy leads to more brittle behaviour. They also effect the shear capacity. A higher tensile strength or higher fracture energy leads to a higher shear capacity.

The influence of the shape of the softening curve on the shear capacity and crack patterns is almost non-existent. A model with damage based shear retention shows a more realistic behaviour than a model with constant shear retention. Taking into account a reduction of Poisson’s ratio has almost no influence if the shear retention is based on damage.

The sensitivity study also focussed on load application methods. The MPOA resulted in higher shear capacities than the Cyclic Pushover Analysis (CPOA). The force controlled MPOA with a mass and mode proportional load was unable to converge after the formation of several large cracks. The study also showed that the shear-drift curve of each floor (or level) was very different in the case of a MPOA with a fixed displacement ratio. In a MPOA with a fixed force ratio, this curve was similar for both floors.

The extent to which the behaviour can be modelled is discussed through four quantities; mode shape, eigenfrequency, shear-capacity and crack patterns. The first two follow from an eigenvalue analysis and the last two from pushover analyses. These values can be compared with the eigenfrequencies from the hammer impact test and the shear-capacities and crack patterns that are determined from the PsD test.

The mode shapes in the ESECMaSE project are an estimation. The mode shapes that follow from the eigenvalue analysis are in good agreement with this estimation. Only the fourth mode shape is different. The other first six mode shapes are similar.

The eigenfrequencies from the hammer impact test were approximated up to 8%. It is possible to reduce the fundamental frequency with interfaces. The locations of these interfaces were chosen at the position of wall-to-wall and wall-to-slab connections and on the basis of the first mode shape. Lower interface stiffnesses lead to a lower fundamental frequency. The interfaces were eventually not applied, because the connections in the ESECMaSE project seemed to be rigid.

The differences between the shear-drift curves from a monotonic and a cyclic pushover analysis are visible after the formation of a severe crack at the base of the structure. The shear capacity that is obtained from the CPOA approximates the shear capacity from the Pseudo-dynamic (PsD) test up to 17% and 29% for respectively floor 1 and floor 2 in the
Conclusions

Weakest direction. It was able to approximate this value up to respectively 45% and 60% with a MPOA. The shear capacity in the exact opposite direction is approximated up to respectively 40% and 49% in the CPOA.

In order to make a comparison between the crack patterns, three patterns were distinguished; rocking of the shear walls, flexural cracks in the long walls and stepwise cracks in the shear walls. The sensitivity study has shown that occurrence of all of these patterns is a minimal requirement to obtain the same behaviour as in the PsD test. All three patterns were visible in both the MPOA and CPOA. The cracks in the CPOA are more smeared out over the structural elements and therefore represent the cracks in the PsD test better.

The difference in approach between MPOA and CPOA, that enables the latter to approximate the PsD test better, is that it includes the load-history of previous cycles. This is important, because the decrease in shear capacity is significant. The hysteretic behaviour, which is the dependence of the output on the current input and past input, cannot be neglected. A parameter that takes such behaviour into account is damping. Despite the fact that damping is generally not present in a quasi-static analysis, the cyclic load is able to include a part of it.

The behaviour of an unreinforced masonry terraced house under an earthquake load can be modelled with a smeared-crack model as follows.

The properties of the structure have to be determined. This can be carried out through experimental tests on specimens, with the aid of literature and a preliminary eigenvalue analysis. Thereafter, the model can be subjected to seismic analyses.

Some properties have a larger coefficient of variation than others. Different monotonic pushover analyses are able reveal the effect of this uncertainty on the shear-capacity and crack patterns. If a coefficient of variation is large but its effect is negligible, for instance in the case of the Young’s modulus, then that consolidates the outcome of the analysis. If the effect is large, for instance in the case of shear retention, then more effort should be put into making the model more realistic.

The behaviour of the building in the ESECMaSE project can be modelled if the first eigenfrequency and fundamental mode shape are determined, a total strain crack model is used, the stress-strain relationship is based on tests, damage based shear retention is taken into account and the load is cyclically applied in a fixed-force-ratio displacement-controlled analysis.

Discussion

The approach that is outlined above was only proven to be correct for this specific structure and this specific load. The geometry of the structure in the ESECMaSE project is, for instance, different than a typical Dutch house. Clay bricks are more common in the Netherlands than calcium silicate bricks. The behaviour of buildings on weaker soils, such as clay or peat, will probably depend more on the soil-structure interaction. This was not taken into account in this finite element model, because the goal was to approximate the model from the PsD test.

The applied earthquake load is also hypothetical, specific to a certain case and only in one orthogonal direction. Its intensity and reference response spectrum determine the size of the cycles and the outcome of the CPOA. In the case of Groningen, where earthquakes are induced, the load is probably different. Induced earthquakes generally have a low magnitude and a high Peak ground acceleration (PGA). A higher PGA leads to higher values in the elastic response spectrum, which means that the load in the PsD test would also be higher.

It was chosen to only compare eigenfrequencies, mode shapes, shear capacities and crack patterns with the results of the ESECMaSE project. There are also other quantities that can be used to assess the behaviour of a structure. For example the behaviour factor q, unity checks on the load and capacity of cross sections, evolution of frequencies and damping,
and so on, can be used. The lack of availability of these quantities from measurements and the large generalizations they make of the structure’s behaviour were reasons why these were not applied in this study.

RECOMMENDATIONS

This study has shown that some modelling properties affect the results more than others. It is important that in future research the sensitivity to uncertain parameters is investigated and taken into account. The results of a single analysis cannot be the basis of solving a complex problem that involves masonry and seismicity.

Aspects such as modal parameters, damage based material model, type of load and application of the load can significantly influence the results. It is shown that using the outcome of one analysis as input for the other, results in a better understanding of the structure’s behaviour. The interaction between the different types of analyses, which are the eigenvalue analysis, Monotonic Pushover Analysis (MPOA) and Cyclic Pushover Analysis (CPOA), is considered to be crucial in obtaining a model with a response that is similar to reality.

Something that was not included in this research is the time-dependent behaviour. It could be neglected because it was not included in the ESECMaSE project. It can play a role in a real-life structure under an earthquake load. The effect of mass-inertia and damping on the structure’s behaviour was not completely taken into account in this study. The mass-inertia was only used in the eigenvalue analysis and hysteretic damping only in the cyclic load. Other types of damping, such as viscous damping were not taken into account at all. A Time History Analysis (THA) should be performed to fully include all these aspects.

In order to assess the response of a terraced house due to an induced earthquake, future research should focus on the aspects that are typical for the situation in Groningen. For instance, clay units, the soil-structure interaction and the induced earthquake load should be taken into account. The influence of some typically Dutch characteristics, such as large windows/openings and cavity walls should be a requirement for this research.
| **ACRONYMS** |
|-------------|------------------|
| APOA        | Adaptive Pushover Analysis |
| CF          | Confident Factor   |
| CFDEM       | Combined Finite-Discrete Element Method |
| CPOA        | Cyclic Pushover Analysis |
| CQC         | Complete Quadratic Combination |
| DIANA       | Displacement Analyzer |
| EF          | Effective Force    |
| ESECMaSE    | Enhanced Safety and Efficient Construction of Masonry Structures in Europe |
| FART        | Find And Replace Text |
| FEMA        | Federal Emergency Management Agency |
| FFT         | Fast Fourier Transform |
| FRF         | Frequency Response Function |
| KL          | Knowledge Level    |
| LFA         | Lateral Force Analysis |
| MPOA        | Monotonic Pushover Analysis |
| PGA         | Peak ground acceleration |
| POA         | Pushover Analysis  |
| PPM         | Peak Picking Method |
| PdD         | Pseudo-dynamic    |
| RSA         | Response Spectrum Analysis |
| RVE         | Representative Volume Element |
| SLA         | Sequentially Linear Analysis |
| SRSS        | Square Root of the Sum of the Squares |
| ST          | Shaking Table      |
| THA         | Time History Analysis |
| URM         | Unreinforced masonry |
| VBA         | Visual Basic for Applications |


[33] Grabowski, S. ESECMaSE D5.5 Material properties for the tests in WP 7 and 8 and the verification of the design model of WP 4. Technical report, Department of civil engineering and geodesy, Technical University of Munich, Munich, Germany, 2004.


Part IV

APPENDICES
DEFINITIONS OF SHEAR, DRIFT AND ENVELOPE

The shear and drift are defined by Equation (A.1) and Figure A.1a

\[ d_1 = u_1 \]  \hspace{1cm} (A.1a)
\[ d_2 = u_2 - u_1 \]  \hspace{1cm} (A.1b)
\[ s_1 = F_1 + F_2 \]  \hspace{1cm} (A.1c)
\[ s_2 = F_2 \]  \hspace{1cm} (A.1d)

in which:
\[ d_1 \] inter-storey drift of level 1
\[ d_1 \] inter-storey drift of level 2
\[ s_1 \] shear force in level 1
\[ s_2 \] shear force in level 2
\[ u_1 \] average displacement of floor 1
\[ u_2 \] average displacement of floor 2
\[ F_1 \] summation of the restoring forces on floor 1
\[ F_2 \] summation of the restoring forces on floor 2

Figure A.1: Definition of the envelope shear-drift curve

The envelope of the shear-drift curves is used to explain the behaviour due to a cyclic load. These curves are obtained by extracting the following four values from the full shear-drift curve per intensity (also see Figure A.1b).

- Maximum shear force
- Maximum inter-storey drift
- Minimum shear force
- Minimum inter-storey drift
A logical start to model a structure is to diminish the amount of uncertainties. One of the biggest uncertainties one can have in a model is the distribution of quantities (stress and strain) in each of the three orthogonal directions. The approximation that is often made is to neglect the direction in which this distribution is most constant.

![3D structure](image1)

![Equivalent frame](image2)

Figure B.1: The structure

The building that is being discussed here, can, when one ignores the presence of the shear walls, be simplified to a simple two-storey frame (see Figure B.1). It is assumed that the distribution of stresses and strains in the out-of-plane direction can be neglected. Because the shear walls are not present in the whole out-of-plane direction, they are left out. Of course, such an assumption is very rigorous. The shear walls will have a contribution to the stiffness of the frame in the in-plane directions.

Therefore, the two-dimensional approximation should only be regarded as an equivalent frame in which the properties of the original structure are taken into account (see Figure B.1b). To be more specific, the equivalent bending stiffness of the walls in the frame should be equal to the bending stiffness of combined shear and long wall in the original structure. The advantages of such an assumption are numerous. From a static point of view, the force distributions can easily be determined and checked with hand calculations.

Nevertheless, these advantages can also be regarded as limitations. It is important that the two-dimension assumption is verified. In a linear elastic model, this can be done through checking whether the determination of the equivalent bending stiffness by means of the original bending stiffness is mesh independent.

The relationship between the equivalent and original stiffness can be computed via a numerical model of the left and right wall (Figures B.2a and B.2b) and the equation for the bending stiffness of a beam that is clamped at both ends (Equation B.1). The beam is clamped at both sides because the first modal shape is translational horizontal in-plane. The bending stiffness $EI$ in Equation B.1 is the equivalent bending stiffness, which means that the second moment of inertia $I$ is taken from the wall in the equivalent frame.

$$u = \frac{FL^3}{12EI}$$ (B.1)
in which:

- $u$ displacement of the top of the beam
- $F$ reaction force at the top of the beam
- $L$ length
- $E$ equivalent stiffness
- $I$ second moment of area of the cross section $= \frac{1}{12}dt^3$
- $d$ depth of the wall in the equivalent frame
- $t$ thickness of the wall in the equivalent frame

The reaction force is determined through a displacement controlled analysis. Thereafter, Equation B.1 is solved for $E$. The results are plotted for meshes with different element sizes in Figure B.3. It is clearly visible that the equivalent stiffness is not mesh independent. The mesh dependency can be explained by a local effect in the relationship between displacement and force. The reaction force has a high peak in the corner where the shear wall is connected to the long wall (see Figure B.4 for the left shear wall and long wall connection). This is a logical result. The shear wall provides most of the stiffness in the out-of-plane direction of the long wall. The finer the mesh, the closer the high force in the upper part of the corner is approximated. The force will consequently be higher and thus the equivalent stiffness lower.

An infinitely fine mesh should approach the local effect of the connection. If it is assumed that the relationship between element size and equivalent stiffness is in the form of a second order polynomial, then this relationship can be quantified with the formulas that are shown in Figure B.3. Hypothetically, the equivalent stiffnesses for a very fine mesh should be equal to 4200 MPa for the left and 11400 MPa for the right wall. A two-dimensional eigenvalue analysis with these hypothetical equivalent stiffnesses results in an first eigenfrequency of 11.8 Hz.

Besides the fact that an infinitely fine mesh is required, this eigenfrequency is 47% higher than the value that is obtained in the ESECMaSE hammer impact test. Therefore, the two-dimensional that is suggested here cannot be used to model this structure.
Polynomial trendline order 2

(a) Left shear wall and long wall combination

(b) Right shear wall and long wall combination

Figure B.3: Mesh dependency of the equivalent stiffness

Figure B.4: Reaction force in the left shear wall to long wall connection
It is common practice to model interfaces with a zero thickness. Each pair of nodes opposite to each other then has the same coordinates, but the material on both sides is connected to only one of the two nodes (see Figure C.2). However, the geometry of the structure in the ESECmaSE project and therefore also the mesh makes this impossible.

The combination of the different interface-locations in Figures C.1a to C.1c require to give the interfaces a certain thickness. This becomes clear from Figure C.3b. A zero thickness interface element would enforce the highlighted white nodes to overlap. In the case of the interface-elements (red and green) this is not a problem, but in the case of the solid brick CHX60 elements it is. In fact, because of the mid-side nodes, there are three nodes on top of each other in the dark-blue element. This results in the error-message Coinciding nodes for element.

The only solution to this problem is to construct a mesh with a thickness that is small enough to create a negligible error in the model’s first eigenfrequency. Therefore, the influence of the interface thickness is researched by means of eigenvalue analyses with different models (see Figure C.3c). It makes sense that, because of the dummy stiffness of the interfaces, a thicker interface results in a stiffer behaviour of the structure and hence a higher first eigenfrequency. Furthermore, the variation in the first eigenfrequency in Section 6.1.1 is smaller than 0.05 Hz. The interface thickness that is small enough not to notice this difference is 0.0001. The eigenvalue analyses that were performed with interfaces are based on this thickness.
(a) Complete model with interfaces

(b) Floor to long wall to shear wall connection

(c) Influence of the interface thickness

Figure C.3: Difficulty with zero thickness interfaces
The relationship between eigenvalues (or eigenfrequencies) and eigenvectors (or eigenmodes) is analytically very clear. Each eigenvalue $\lambda$ is accompanied by an eigenvector $\phi$ that can be computed through the characteristic equation, which is noted here as $(K_{el} - \lambda M_{cons})\phi = 0$. Therefore, the relationship between the eigenvalues and eigenvectors is linearly dependent of the consistent mass matrix $M_{cons}$ and the stiffness matrix $K_{el}(E_m, E_c)$.

However, a variation of masonry’s E-modulus $E_m$ does not lead to the same variation between $\lambda$ and $\phi$. That is true, because only a part of the stiffness matrix is changed. The entries of the stiffness matrix that are only dependent of the stiffness of concrete $E_c$ are not changed.

This means that, besides the fact that a lower E-modulus of masonry leads to a lower eigenfrequency, the modal shape that corresponds to that eigenfrequency can also change. The mode shape that corresponds to the second eigenfrequency in one analysis is therefore different than the mode shape in another analysis. The more the mode shapes change during the variation of masonry’s Young’s modulus, the more the lines that are drawn in Figure D.1 become invalid.

If one takes a closer look at the modes shapes in Figure D.2, it is clear that more than five shapes can be distinguished. From a logical point of view one can say that the fourth and fifth mode shape in the 3000-MPa analysis should be switched (see Figure D.1). But in reality, the fifth mode from that analysis does not only correspond to the fourth mode in the 5490-MPa analysis but also to the third mode in the 9000-MPa analysis (see Figures D.2i, D.2k and D.2m). A similar comparison can be made for the mode shapes in Figures D.2f and D.2h. Only the first mode shape stays the same when masonry’s stiffness is changed. The fact that the other mode shapes change will probably not have a significant influence on the results, because only the first mode is used here.
Figure D.2: The change in mode shapes for different E-moduli
The impact of non-linear compressional behaviour on the results of a MPOA can be found at the location where the compressive principal stresses are the highest. In this structure, that are the shear walls (or piers). Their in-plane stiffness prohibits the structure from collapsing like a house of cards. It is logical that the compressive stresses in these piers are the highest in their toes (see Figure E.1).

Figure E.1: Minimum principal compressive stresses ($\sigma_3$) in the piers at the last converged load step

The ideal plastic compression softening model enables the stresses to increase infinitely when the compressive strength is reached. This is the case in the steps during the collapse stage. In other, earlier, steps this is definitely not the case. There, the stresses are below the compressive strength. Because the strength is reached and the value is used in the material model, the effect of this choice is researched through several pushover analyses. The compressive strength is varied between 10 and 25 MPa and the corresponding shear-drift curves are plotted in Figure E.2.
Despite the fact that the model is ideally plastic, the effect of the compressive strength is only minor. In most of the analyses, the behaviour is even exactly the same. There is not a clear relationship between the compressive strengths and the behaviour of the model. Moreover, there is no effect on the shear-capacity and the crack-patterns. It would therefore be superfluous to use a more comprehensive softening model.
There are four possibilities to include damage in a total strain fixed crack model in DIANA 9.5 [49, §20.2, Vol. Material Library]:

- No damage reduction;
- Damage based Poisson’s ratio reduction;
- Damage based shear retention;
- Damage based Poisson’s ratio reduction and shear retention.

The relative coarse explanation in Manie [49] makes it unclear what the difference is between the analyses with these type of models. An extended interpretation is given below. It is also based on DeJong et al. [24].

In general, a structural non-linear analysis starts in an uncracked isotropic elastic state. The nodal coordinates are defined in a global x,y,z-coordinate system (see Figure F.1a). Secondly, the elements are loaded. A crack is initiated when the cut-off stress is exceeded in the principal stress direction. The direction of these stresses is the direction in which they are maximum and minimum. It is noted as the local 1,2,3-coordinate system (see Figure F.1c). However, the directions of the crack are denoted in another coordinate system, which is the n,t,s-coordinate system. This direction is fixed with respect to the global coordinate system as soon as a crack is formed (see Figure F.1c).

Furthermore, the constitutive relation in any of these coordinate systems can be written down as:

\[ \sigma = D \star \varepsilon \]  

(F.1)

The constitutive matrix D is dependent of the initial Young’s modulus \( E_0 \) and the initial Poisson’s ratio \( \nu_0 \). Both of these values must be specified mandatory in any structural analysis. They are the starting point. The stress vector \( \sigma \) is defined as

\[ \begin{bmatrix} \sigma_{xx} & \sigma_{yy} & \sigma_{zz} & \sigma_{yz} & \sigma_{zx} & \sigma_{xy} \end{bmatrix}^T \]
and the strain vector $\epsilon$ is defined as $[\epsilon_{xx} \; \epsilon_{yy} \; \epsilon_{zz} \; \gamma_{yz} \; \gamma_{zx} \; \gamma_{xy}]^T$. As a result, the constitutive matrix in the global coordinate system is written as follows:

$$
D = \begin{bmatrix}
D_{nn} & D_{n0} \\
D_{0n} & D_{00}
\end{bmatrix}
$$

(F.2a)

$$
= \frac{E_0}{(1 + \nu_0)(1 - 2\nu_0)}
\begin{bmatrix}
1 - \nu_0 & \nu_0 & \nu_0 & 0 & 0 & 0 \\
\nu_0 & 1 - \nu_0 & \nu_0 & 0 & 0 & 0 \\
\nu_0 & \nu_0 & 1 - \nu_0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 - 2\nu_0 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 - 2\nu_0 & 0 \\
0 & 0 & 0 & 0 & 0 & 1 - 2\nu_0
\end{bmatrix}
$$

(F.2b)

The subscripts $n$ and $\theta$ stand for the normal and shear direction. They might confuse a bit, because $n$ is also used in the n,t,s-coordinate system. These notations are used to maintain consistency with the DIANA-manual \[49\]. Generally, one can replace the entries on the diagonal in $D_{00}$ by $G$ with the following definition:

$$
G = \frac{E_0}{2(1 - \nu_0)}
$$

(F.3)

Furthermore, it is assumed that no damage recovery takes place. That means that elements remember the state in which they were cracked before. Although cracks can close, the stiffness of the element is still the one that belongs to this (previous) cracked state.

The constitutive relation in Equation F.2b can be transformed with a transformation matrix $T$ in order to obtain the relationship in a different coordinate system. After the initiation of a crack in a total strain fixed crack model, the transformation matrix is fixed. Thereafter, the constitutive relation is not isotropic linear elastic. The relationship depends on which of the four options above is modelled.

### F.1 No Damage Reduction

$$
D_{nn} = \frac{E_0}{(1 + \nu_0)(1 - 2\nu_0)}
\begin{bmatrix}
\frac{\partial \sigma_{nn}}{\partial \epsilon_{xx}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{yy}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{zz}} \\
\frac{\partial \sigma_{nn}}{\partial \gamma_{yz}} & \frac{\partial \sigma_{nn}}{\partial \gamma_{zx}} & \frac{\partial \sigma_{nn}}{\partial \gamma_{xy}}
\end{bmatrix}
\begin{bmatrix}
1 - \nu_0 & \nu_0 & \nu_0 \\
\nu_0 & 1 - \nu_0 & \nu_0 \\
\nu_0 & \nu_0 & 1 - \nu_0 \\
0 & 0 & 0 \\
0 & 0 & 0 \\
0 & 0 & 0
\end{bmatrix}
$$

(F.4a)

$$
D_{\theta\theta} =
\begin{bmatrix}
\frac{\partial \sigma_{ns}}{\partial \gamma_{ns}} & 0 & 0 \\
0 & \frac{\partial \sigma_{st}}{\partial \gamma_{st}} & 0 \\
0 & 0 & \frac{\partial \sigma_{ts}}{\partial \gamma_{ts}}
\end{bmatrix}
= \begin{bmatrix}
G & 0 & 0 \\
0 & G & 0 \\
0 & 0 & G
\end{bmatrix}
$$

(F.4b)

in which:

$$
G = \frac{\beta E_0}{2(1 + \nu_0)}
$$

(F.4c)

$\frac{\partial \sigma_{ij}}{\partial \gamma_{kl}}$ is the modified tangent stiffness matrix according to the equivalent uniaxial strain concept.
\[ D_{\text{nn}} = \begin{bmatrix} \frac{\partial \sigma_{nn}}{\partial \epsilon_{nn}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ns}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} \\ \frac{\partial \sigma_{nn}}{\partial \epsilon_{ns}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ns}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} \\ \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} \end{bmatrix} \begin{bmatrix} 1 - \nu_x^2 + e_x^2 + 2\nu_{xy}\nu_{yx} \\ \nu_{xy}^2 + \nu_{yz}^2 + \nu_{zx}^2 \\ \nu_{xy}^2 + \nu_{yz}^2 + \nu_{zx}^2 \end{bmatrix} \]

\[ D_{\theta\theta} = \begin{bmatrix} G_{ns} & 0 & 0 \\ 0 & G_{st} & 0 \\ 0 & 0 & G_{tt} \end{bmatrix} \]

in which:

\[ \Delta' = 1 - \nu_{xy}^2 - \nu_{yz}^2 - \nu_{zx}^2 - 2\nu_{xy}\nu_{yz}\nu_{zx}^2 \]

\[ \nu_{ij} = \frac{\nu_i E_i}{E_0} \]

\[ G_{ij} = \frac{\beta E_0}{2(1 + \nu_0)} \]

\[ D_{\text{nn}} = \frac{E_0}{(1 + \nu_0)(1 - 2\nu_0)} \begin{bmatrix} \frac{\partial \sigma_{nn}}{\partial \epsilon_{nn}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ns}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} \\ \frac{\partial \sigma_{nn}}{\partial \epsilon_{ns}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ns}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} \\ \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} & \frac{\partial \sigma_{nn}}{\partial \epsilon_{ss}} \end{bmatrix} \begin{bmatrix} 1 - \nu_0 & \nu_0 & \nu_0 \\ \nu_0 & 1 - \nu_0 & \nu_0 \\ \nu_0 & \nu_0 & 1 - \nu_0 \end{bmatrix} \]

\[ D_{\theta\theta} = \begin{bmatrix} G_{ns} & 0 & 0 \\ 0 & G_{st} & 0 \\ 0 & 0 & G_{tt} \end{bmatrix} \]

in which:

\[ G_{ij} = \frac{E_i}{2(1 + \nu_{ij})} \]
The modal pushover analysis in DIANA is a load controlled analysis that is based on one eigenmode. That makes it different from the monotonic pushover analysis in Chapters 8 and 9, which is displacement controlled and only based on the displacement of the floors in the first eigenmode.

In order to make a valid comparison between the modal analysis and the PsD test, their sizes should somehow be the same. Because the size of the load vector is equal to the number of nodes in the analysis and equal to two in the tests, a value is chosen that characterises both vectors. The only load-related value that fits this condition, and also happens to be used in the shear-drift curve, is the base shear. The base shear is, in fact, the summation of the entries in the (externally applied) load vector.

The modal load vector is determined with the following equation [49, §6.4, Vol. Analysis Procedures]. Note that, because only one earthquake direction is considered, the summation over different directions is omitted.

\[ f_{\text{modal}} = a_{\text{eq}} \cdot M \cdot \hat{\phi}_n \cdot x \]  

in which:
- \(a_{\text{eq}}\) (equivalent) specified acceleration
- \(a_g\) maximum ground acceleration (from PsD test)
- \(f_{\text{modal}}\) modal (pushover analysis) load vector
- \(f_{\text{psd}}\) pseudo-dynamic load vector
- \(F_{\text{modal}}\) modal base shear
- \(F_{\text{psd}}\) pseudo-dynamic base shear
- \(i\) identity vector
- \(M\) (consistent) nodal mass matrix (size NxN)
- \(M_{\text{psd}}\) pseudo-dynamic mass matrix (size 2x2)
- \(N\) total number of nodes
- \(x\) earthquake direction \(j\)
- \(\alpha^n\) normalizing factor of the \(n^{th}\) eigenvector
- \(\gamma^n\) participation factor of the \(n^{th}\) eigenmode
- \(\phi^n\) \(n^{th}\) eigenvector
- \(\hat{\phi}^n\) \(n^{th}\) eigenvector, normalized with respect to the mass

The eigenvector is normalised with respect to the mass as follows.

\[
(\hat{\phi}^n)^T M \hat{\phi}^n = m^{nn} \quad (G.2a)
\]

\[
(\phi^n \alpha^n)^T M (\phi^n \alpha^n) = 1 
\quad (G.2b)
\]

The corresponding participation vector \(\gamma^n\) is:

\[
\gamma^n = \frac{(\hat{\phi}^n)^T M i}{m^{nn}} 
\quad (G.3a)
\]

\[
= \sum_{p=1}^{N} \sum_{q=1}^{N} \frac{\phi^n_p}{\alpha^n} m_{p,q} 
\quad (G.3b)
\]
However, it is not necessary to solve Equations (G.2) and (G.3) yourself. The output of an eigenvalue analysis already contains the participation vector for each mode. The total base shear force is then:

$$F_{\text{modal}} = \sum_{p=1}^{N} f_{\text{modal},p} \quad (G.4a)$$

$$= \sum_{p=1}^{N} \sum_{q=1}^{N} a_{pq} \cdot m_{p,q} \cdot \left( \frac{\phi_q^n}{\alpha^n} \right) \quad (G.4b)$$

$$= a_{\text{eq}} \cdot y^n \quad (G.4c)$$

A similar procedure can be carried out for the PsD test. The load vector is obtained from the mass-matrix that is chosen and used in the master controller of the test [5, 63]. The corresponding base shear can be obtained from the summation over the entries in the vector.

$$f_{\text{psd}} = M_{\text{psd}} \cdot i \cdot a^g \quad (G.5a)$$

$$= \begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \cdot \begin{bmatrix} 1 \\ 1 \end{bmatrix} \cdot a^g \quad (G.5b)$$

$$F_{\text{psd}} = \sum_{p=1}^{2} m_p a^g \quad (G.5c)$$

$$= (m_1 + m_2) a^g \quad (G.5d)$$

The equivalent acceleration $a_{\text{eq}}$ is defined here in such a way that the base shear of the modal pushover analysis is equal to the base shear in the PsD test. Because the base shear in the test alternates, the maximum applied ground acceleration $a^g$ from the accelerogram is used (see Figure 5.3).

$$F_{\text{modal}} = F_{\text{psd}} \quad (G.6a)$$

$$a_{\text{eq}} = \frac{(m_1 + m_2) a^g}{\gamma^1} \quad (G.6b)$$

The masses of the floors are respectively 29.000 and 26.200 kg [6] and the participation factor for the first mode from the dynamic identification in Chapter 7 is 186.1 kg. This results in the equivalent accelerations in Table G.1.

<table>
<thead>
<tr>
<th>INTENSITY</th>
<th>$A^G (\text{m/s}^2)$</th>
<th>$A^{\text{eq}} (\text{m/s}^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02g</td>
<td>0.234</td>
<td>69</td>
</tr>
<tr>
<td>0.04g</td>
<td>0.468</td>
<td>139</td>
</tr>
<tr>
<td>0.06g</td>
<td>0.702</td>
<td>208</td>
</tr>
<tr>
<td>0.08g</td>
<td>0.936</td>
<td>278</td>
</tr>
<tr>
<td>0.10g</td>
<td>1.17</td>
<td>347</td>
</tr>
<tr>
<td>0.12g</td>
<td>1.40</td>
<td>416</td>
</tr>
<tr>
<td>0.14g</td>
<td>1.64</td>
<td>486</td>
</tr>
<tr>
<td>0.16g</td>
<td>1.87</td>
<td>555</td>
</tr>
<tr>
<td>0.18g</td>
<td>2.11</td>
<td>625</td>
</tr>
<tr>
<td>0.20g</td>
<td>2.34</td>
<td>694</td>
</tr>
</tbody>
</table>

Table G.1: Load steps modal pushover analysis
SET-UP OF THE AUXILIARY FRAME

In order to perform a displacement controlled analysis with a fixed force ratio, an external (auxiliary) frame is connected to the floors (see Figures 6.9 and H.1). Despite the fact that the connection is meant to be rigid, it is applied through spring-elements. It was chosen to use these type of elements because they enable to extract the internal forces. The connection is made rigid by applying a very high spring-stiffness.

The ratios between the forces and displacements are defined in Equations (H.1a) and (H.1b). The frame is able to rotate around a control node, denoted as point 3. Therefore, the displacements of the corners of the frame can be expressed in the displacement of this node and the frame’s rotation (see Equations (H.1c) and (H.1d)).

\[
\alpha = \frac{F_1}{F_2} \quad \text{(H.1a)}
\]

\[
\beta = \frac{u_1}{u_2} \quad \text{(H.1b)}
\]

\[
u_1 = u_3 - \sin \theta \ast (L - x) \quad \text{(H.1c)}
\]

\[
u_2 = u_3 + \sin \theta \ast x \quad \text{(H.1d)}
\]

Figure H.1: Sketch of the connection between auxiliary frame and structure
in which:

\[ F_1 \] internal force acting on floor 1
\[ F_2 \] internal force acting on floor 2
\[ F_3 \] external force acting on control node
\[ L \] height of the frame
\[ u_1 \] displacement of floor 1
\[ u_2 \] displacement of floor 2
\[ u_3 \] displacement of control node
\[ x \] distance of control node from top of the frame
\[ \alpha \] shape of the force-vector
\[ \beta \] shape of the displacement-vector
\[ \theta \] rotation of the frame

The force distribution, that is obtained from the modal shape in the dynamic identification in Section 7.1, is applied through \( \alpha \). The internal forces and displacements can be eliminated from Equations (H.1a) to (H.1d) by taking the moment equilibrium around the control node, \( T_3 = 0 \).

The result is Equation H.2, which has only \( \alpha, \theta \) and \( u_3 \) as unknowns or variables. Since \( \alpha \) is constant during the whole analysis and used to create the mesh, the only variables left are \( \theta \) and \( u_3 \). This enables the plate to rotate around its constraints, depending on the stiffness ratio between level 1 and level 2.

\[
x = \frac{\alpha \cdot L}{1 + \alpha} \quad \text{(H.2a)}
\]
\[
\alpha = \frac{x}{L - x} \quad \text{(H.2b)}
\]
\[
\beta = -\frac{\sin \theta \cdot l - \alpha \cdot u_3 - u_3}{\sin \theta \cdot \alpha \cdot L + \alpha \cdot u_3 + u_3} \quad \text{(H.2c)}
\]

Equation H.2c can be plot for the case in which \( \alpha = 0.41 \) (this is the first modal shape in Section 7.1). The range for which this variable is plotted in Figure H.2 comes forth from the range in which \( u_3 \) and \( \theta \) change during the MPOA in Chapter 8.

![Figure H.2: Plot of \( \beta \) in the boundaries of the MPOA](image-url)
A finite element analysis is roughly composed of three phases:

1. Pre-processing
2. Solution
3. Post-processing

The whole process of constructing a finite element model and obtaining plots with stresses and shear-drift curves is shown in the flow chart in Figure I.1. The yellow boxes denote the phases mentioned above. Each of the phases is discussed separately in Appendices I.1 to I.3

Figure I.1: Flow chart of the file management and the solution procedure

### I.1 PRE-PROCESSING

The pre-processing of the model is carried out in iDIANA. It is possible to type the commands by hand or make a batch file in which a whole series of commands is written. The latter method is used here (also see Manie [49, §24.8.1, Vol. iDIANA]). Listing I.1 is the corresponding batch file to create the model for the MPOA in Chapter 8.

Listing I.1: FileName.BAT

```
0  FEMGEN H2
  PROPERTY FE-PROG DIANA STRUCT.3D ; YES
10  !!
  !!
  GEOMETRY POINT COORD P1 0
  GEOMETRY SWEEP P1 P2 TRANSLATE TR1 .175 0 0
  GEOMETRY SWEEP P2 P3 TRANSLATE TR2 1 0 0
  GEOMETRY SWEEP P3 P4 TRANSLATE TR3 .725 0 0
  GEOMETRY SWEEP P4 P5 TRANSLATE TR4 1.5 0 0
```
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GEOMETRY SWEEP P5 P6 TR3
GEOMETRY SWEEP P6 P7 TR2
GEOMETRY SWEEP P7 P8 TR1

!!

GEOMETRY POINT COORD P9 2.15 0 .15
GEOMETRY SWEEP P9 P10 TRANSLATE TR5 0.5 0 0
GEOMETRY SWEEP P10 P11 TR5
GEOMETRY LINE STRAIGHT P4 P9

20

GEOMETRY LINE STRAIGHT P5 P11

!!

CONSTRUCT SET SET1 APPEND LINES L2 L3 L5 L6
GEOMETRY SWEEP SET1 SET2 TRANSLATE TR6 0 0 .175
GEOMETRY SWEEP P14 P18 TRANSLATE TR7 0 0 .775

25

GEOMETRY SWEEP P18 P19 TRANSLATE TR8 .25 0 0
GEOMETRY SWEEP P19 P20 TRANSLATE TR9 .26 0 0
GEOMETRY SWEEP P20 P21 TRANSLATE TR10 .48 0 0
GEOMETRY SWEEP P21 P22 TR9
GEOMETRY SWEEP P22 P23 TR8

30

GEOMETRY LINE STRAIGHT P15 P23

!!

GEOMETRY SWEEP L12 L29 TRANSLATE TR11 0 0 1
GEOMETRY SWEEP L29 L32 TRANSLATE TR12 0 0 .7875
GEOMETRY SWEEP L32 L35 TR11

35

GEOMETRY SWEEP L15 L38 TR11
GEOMETRY SWEEP L38 L41 TR11
GEOMETRY SWEEP L41 L44 TR11
GEOMETRY SPLIT L44 .8
GEOMETRY SWEEP L35 L49 TR12

40

GEOMETRY SWEEP L49 L52 TR11

!!

GEOMETRY SWEEP P1 P41 TRANSLATE TR13 0 0 4.75
GEOMETRY LINE STRAIGHT P39 P41

45

CONSTRUCT SET SET3 APPEND LINES L24 L25 L26
GEOMETRY SWEEP SET3 SET4 TRANSLATE TR14 0 0 1.0125
GEOMETRY SWEEP SET4 SET5 TR11
GEOMETRY SWEEP SET5 SET6 TRANSLATE TR15 0 0 .8375
GEOMETRY SWEEP P18 P54 TRANSLATE TR16 0 0 2.85
GEOMETRY LINE STRAIGHT P50 P54

50

GEOMETRY SWEEP P40 P55 TR3
GEOMETRY LINE STRAIGHT P54 P55
GEOMETRY SWEEP P23 P56 TRANSLATE TR17 0 0 2.225
GEOMETRY SWEEP P56 P57 TR6
GEOMETRY SWEEP P57 P58 TRANSLATE TR18 0 0 .45

55

GEOMETRY LINE STRAIGHT P53 P58
GEOMETRY SWEEP P55 P59 TR4
GEOMETRY LINE STRAIGHT P58 P59
CONSTRUCT SET SET7 APPEND LINES L8 L9
GEOMETRY COPY SET7 SET8 TRANSLATE TR19 0 0 5.05

60

GEOMETRY LINE STRAIGHT P55 P60
GEOMETRY LINE STRAIGHT P59 P62

!!

GEOMETRY SWEEP L83 L92 TRANSLATE TR20 .225 0 0
GEOMETRY SWEEP L92 L95 TRANSLATE TR21 .3 0 0

65

GEOMETRY LINE STRAIGHT P34 P65
GEOMETRY SWEEP L48 L99 TR6
GEOMETRY LINE STRAIGHT P66 P69
GEOMETRY SWEEP P66 P69 TRANSLATE TR22 0 0 1.4
GEOMETRY LINE STRAIGHT P59 P69

70

GEOMETRY SWEEP P68 P70 TR22
GEOMETRY SWEEP P8 P71 TR13
GEOMETRY LINE STRAIGHT P70 P71

!!

GEOMETRY SURFACE AUTOMATIC

75

GEOMETRY SURFACE 4SIDES L1 L55 L56 +L53 +L59 +L36 +L33 +L30 L16
GEOMETRY SURFACE 4SIDES L13 +L31 +L34 +L37 +L51 L54 L80 L81 L78 L22
GEOMETRY SURFACE 4SIDES +L9 L8 L10 L4 L11
GEOMETRY SURFACE 4SIDES L4 +L18 L22 +L23 +L24 +L25 +L26 L27 +L28 L19
GEOMETRY SURFACE 4SIDES L23 L78 L79 +L74 +L67 L68

80

GEOMETRY SURFACE 4SIDES L27 +L63 +L70 L77 L85 +L84 +L83 L82
GEOMETRY SURFACE 4SIDES L48 +L73 +L72 +L71 L79 L81 L86 L87
GEOMETRY SURFACE 4SIDES L86 L90 +L88 L89 L91
GEOMETRY SURFACE 4SIDES L14 +L28 L82 +L93 +L96 L98 +L45 +L42 L39
GEOMETRY SURFACE 4SIDES +L47 L90 L95 L102 L100

85

GEOMETRY SURFACE 4SIDES +L97 L94 +L84 L87 L104 L103
GEOMETRY SURFACE 4SIDES L7 +L21 +L40 +L43 +L46 +L101 L105 L107 L106

!!

CONSTRUCT SET LEVEL0 APPEND SURFACES ALL

90

GEOMETRY SWEEP LEVEL0 LEVEL1 TRANSLATE TR23 0 2.3 0
GEOMETRY SWEEP LEVEL1 LEVEL2 TRANSLATE TR24 0 .2 0
GEOMETRY SWEEP LEVEL2 LEVEL3 TRANSLATE TR25 0 .2 0
GEOMETRY SWEEP LEVEL3 LEVEL4 TR24
GEOMETRY SWEEP LEVEL4 LEVEL5 TRANSLATE TR26 0 2.1 0
I.1 Pre-processing

GEOMETRY SWEEP LEVEL5 LEVEL6 TR24

GEOMETRY SWEEP LEVEL6 LEVEL7 TR25

CONSTRUCT SET OPEN FRAME

GEOMETRY SWEEP LEVEL5 LEVEL6 TR24

GEOMETRY SWEEP LEVEL6 LEVEL7 TR25

CONSTRUCT SET CLOSE FRAME

GEOMETRY LINE STRAIGHT P575 P283

GEOMETRY LINE STRAIGHT P578 P284

GEOMETRY LINE STRAIGHT P577 P567

GEOMETRY LINE STRAIGHT P580 P568

CONSTRUCT SET SPRING1 APPEND LINES L1512 L1513

CONSTRUCT SET SPRING2 APPEND LINES L1514 L1515

CONSTRUCT SET BOD1 APPEND BODIES B1 B25 B145 B169

CONSTRUCT SET BOD2 APPEND BODIES B4 B36 B148 B180

CONSTRUCT SET BOD3 APPEND BODIES B23 B167 B178

CONSTRUCT SET BOD41 APPEND BODIES B99 B100 B103 B104

CONSTRUCT SET BOD42 APPEND BODIES B85 B86 B89 B92 B94 B95 B96 B97 B98

CONSTRUCT SET BOD43 APPEND BODIES B23 B167 B178

CONSTRUCT SET BOD4 APPEND BODIES BOD41 BOD42 BOD43

CONSTRUCT SET BOD51 APPEND BODIES B243 B244 B247 B248

CONSTRUCT SET BOD52 APPEND BODIES B239 B231 B232 B234 B235 B237

CONSTRUCT SET BOD53 APPEND BODIES B217 B218 B219 B220 B221 B222 B223 B224 B225 B226

CONSTRUCT SET BOD54 APPEND BODIES B227 B228 B229 B230 B231 B232 B233 B234 B235 B236 B237 B238 B239 B240 B241 B242

CONSTRUCT SET BOD55 APPEND BODIES B245 B246 B249 B250 B251 B252

CONSTRUCT SET BOD56 APPEND BODIES B501 B502 B503

CONSTRUCT SET BOD6 APPEND BODIES B243 B244 B247 B248

CONSTRUCT SET BOD7 APPEND BODIES B59 B70 B131 B142 B203 B214

CONSTRUCT SET BOD8 APPEND BODIES B60 B132 B204

CONSTRUCT SET BOD92 APPEND BODIES B81 B145 B84 B148

CONSTRUCT SET BOD10 APPEND BODIES BOD7O BOD7I BOD7L BOD8

CONSTRUCT SET WALLS APPEND BODIES BOD1 BOD2 BOD3 BOD6 INTERS

CONSTRUCT SET FLOORS APPEND BODIES BOD4 BOD5

CONSTRUCT SET BRICK APPEND BODIES WALLS FLOORS

CONSTRUCT SET BODALL APPEND BRICK FRAME

MESHING TYPES ALL HE20 CHX60

MESHING TYPES SPRING IP11 SP2TR

MESHING DIVISION ELSIZE ALL .2

MESHING DIVISION ELSIZE FRAME 1

CONSTRUCT SET DIV1 APPEND LINES L96 L97 L1192 L1193 L387 L388 L582 L583 L777 L778

CONSTRUCT SET DIV2 APPEND LINES L972 L973 L1167 L1168 L1362 L1363

CONSTRUCT SET DIV3 APPEND LINES L6 L15 L38 L41 L338 L440 L333 L335 L347 L350

CONSTRUCT SET DIV4 APPEND LINES L528 L530 L542 L545 L723 L725 L198 L200 L1113 L1115

CONSTRUCT SET DIV5 APPEND LINES L1127 L1130 L310 L312 L312 L312

CONSTRUCT SET DIV6 APPEND LINES L399 L407 L394 L402 L3379 L3379 L3379 L3379

CONSTRUCT SET DIV7 APPEND LINES L419 L420 L424 L425 L614 L615 L619 L620 L1199 L1200

CONSTRUCT SET DIV8 APPEND LINES L1284 L1205 L1394 L1395 L1399 L1400

CONSTRUCT SET DIV9 APPEND LINES L398 L400 L409 L410 L595 L604 L605 L1178 L1180

CONSTRUCT SET DIV10 APPEND LINES L1189 L1190 L1193 L1193 L1384 L1385

CONSTRUCT SET DIV11 APPEND LINES L47 L222 L417 L612 L807 L1002 L1197 L1200

CONSTRUCT SET DIV12 APPEND LINES L54 L216 L41 L606 L601 L996 L1191 L1386

CONSTRUCT SET DIV13 APPEND LINES L98 L231 L426 L616 L816 L1001 L1206 L1401

CONSTRUCT SET DIV14 APPEND LINES L423 L619 L820 L1398

CONSTRUCT SET DIV15 APPEND LINES L420 L597 L1192 L1377

CONSTRUCT SET DIV16 APPEND LINES L303 L332 L525 L527 L110 L1112 L1385 L1387

CONSTRUCT SET DIV17 APPEND LINES L376 L377 L379 L381 L371 L372 L374 L376 L1156 L1157

CONSTRUCT SET DIV18 APPEND LINES L1159 L1161 L1191 L1392 L1394 L1395

CONSTRUCT SET DIV19 APPEND LINES L1160 L216 L416 L611 L806 L1001 L1196 L1391

CONSTRUCT SET DIV20 APPEND LINES L43 L154 L348 L349 L543 L544 L739 L934 L1128 L1129

CONSTRUCT SET DIV21 APPEND LINES L55 L208 L395 L390 L785 L980 L1175 L1370

CONSTRUCT SET DIV22 APPEND LINES L412 L607 L1192 L1387

CONSTRUCT SET DIV23 APPEND LINES L406 L601 L1186 L1381

CONSTRUCT SET DIV24 APPEND LINES L1123 L1324

CONSTRUCT SET DIV25 APPEND LINES L55 L208 L395 L390 L785 L980 L1175 L1370

CONSTRUCT SET DIV26 APPEND LINES L412 L607 L1192 L1387

CONSTRUCT SET DIV27 APPEND LINES L406 L601 L1186 L1381

MESHING DIVISION LINE DIV1 2

MESHING DIVISION LINE DIV2 5

MESHING DIVISION LINE DIV3 6

MESHING DIVISION LINE DIV4 3

MESHING DIVISION LINE DIV5 1

MESHING DIVISION LINE DIV6 4
The advantage of this method is that relative small adjustments can be done easily without starting all over again. Nevertheless, it should always be checked whether the model
is constructed the right way. The mesh is, for instance, difficult to generate completely automatic because of the geometry that is used here. If only the material parameters are changed, then it is even easier to change them directly in the .DAT-file (see Listing I.2).

I.2 SOLUTION

The second phase is the main phase of the finite element analysis. If, for instance, one wants to research the effect of a different mode I fracture energy, then the materials-table in the .DAT-file can be altered as in Listing I.2. The mode I fracture energy is parametrized as $VAR_1$.

Listing I.2: Table Materials in FileName.DAT

```
'MATERIALS'
  1 NAME MASONRY
  YOUNG 5.49000E+09
  POISON 1.20000E-01
  DENSIT 1.80000E+03
  TOTCRK FIXED
  TENCRT LINEAR
  TENSTR 2.80000E+05
  GF1 $VAR1
  COMCRV CONSTA
  COMSTR 2.27000E+07
  SHRSCRV DAMAGE
  POIRED DAMAGE
```

A sequence of analyses in which this parameter is varied from 10 to 100 in 7 steps is performed with the batch file from Listing I.3 (see Section 9.2.3). The files that are required for each analysis are stored in the folder Original. Among these files is the open source program called Find And Replace Text (FART)\(^1\).

Firstly, all the files in this folder are duplicated and secondly, $VAR_1$ is replaced by a certain number. Thirdly, the finite element analysis is carried out and after that the whole process starts all over again. The whole batch is started by typing FileName.BAT in the Diana Command Box.

Listing I.3: FileName.BAT

```
@echo off
::20150420a
::Written by Jos van Dam
::Edited by Wilco van der Mersch
::Set main variables
set /a $VAR1_START="10"
set /a $VAR1_END="100"
set /a $STEPS="7"
set /a $NR="1"
set /a $VAR1_NOW=$VAR1_START
::go to main folder
cd N
::create analysis folder if it doesn't exist
if not exist N%$NR% mkdir N%$NR%
::copy original files into analysis folder
ROBOCOPY Original N%$NR% *.* /E
::go to analysis folder
cd N%$NR%
::replace variables with given quantities
FART N.dat $VAR1 %$VAR1_NOW%
::rename analysis-files
REN N.dat N%$NR%.dat
```

\(^1\) http://fart-it.sourceforge.net/
I.3 POST-PROCESSING

The post-processing of the analysis results is performed in two steps. First, all the information about the analysis is acquired to generate a post-processing command file (.FVC) (see Listing I.4). The VBA macro Post-process.xlsm is used to process this data. Among this data is the number of executed steps, iterations, cracks, etc (see Figure I.2). The .FVC-file is thereafter used to generate tabulated output and plots of certain quantities.

The table at the bottom of Figure I.2 contains as many rows as executed steps. The table and command file are constructed with the following Visual Basic code in Excel. Note that the file also contains the worksheet ‘Input’ which contains some general commands that are used in the Results environment of iDIANA.

Listing I.4: Worksheet ‘Loading’ in the VBA macro Post-processing.xlsm

Sub OutputFile()
    'Written by W.A. van der Mersch on 16-02-2015
Required file structure is:
A sub-folder with the name of the analysis, in which the model (.DAT) is placed and an output-file (.OUT) is placed.

The loop in which file #3 is printed should be modified using the iDIANA commands from the sheet "Input". The parameters on top of the sheet "Loading" should be modified to enable a consistent output with iDIANA. The output of this script (.FVC) can be used to generate tabulated output and plotfiles in iDIANA.

```
ThisWorkbook.Activate
Dim CrackedStep As Boolean
Dim ThisIsMasonry As Boolean
Dim ImageEnd As Integer
Dim ImageStart As Integer
Dim LastConvergedStep As Integer 'From this step no convergence at all
Dim Loading() As Integer
Dim lr As Integer 'Loadstep results
Dim NoConvergenceStep As Integer 'No convergence in this step
Dim p As Integer 'General count number
Dim StartCrack As Integer
Dim ThisLoadCase As Integer
Dim ThisPhase As Integer
Dim ThisStep As Integer
Dim TotalSteps(3) As Integer
Dim TableCorner As Variant
Dim CB As Variant 'table with checkboxes
Dim Analysis As String
Dim EmptyLine As String
Dim FilePathData As String
Dim FilePathOutput As String
Dim FilePathPostprocess As String
Dim LineFromFile As String
Dim ShapeC As String
Dim ShapeT As String
Dim sp As String 'Save plot output
Dim st As String 'Save tabulated output
Dim STRNT1 As String 'First strain level
Dim STRNT2 As String 'Second strain level
Dim STRNC1 As String 'First negative strain level
Dim STRNC2 As String 'Second negative strain level
Dim STRST1 As String 'First stress level
Dim STRSC1 As String 'First negative stress level
Dim WarningCrack As String
Dim BandWidth As Double
Dim Emodulus As Double
Dim Factor As Double
Dim Fc As Double
Dim Ft As Double
Dim Gc As Double
Dim Gt As Double
Dim Levels As Double

' Starting values
TotalSteps(0) = 2
TotalSteps(1) = 2
CrackedStep = False
p = 0
StartCrack = 0
ThisIsMasonry = False
ThisPhase = 1
BandWidth = 0.2

' Starting strings
Set CB = Worksheets("Loading")
Analysis = CB.Range("D3")
EmptyLine = "!!"
ImageEnd = CB.Range("D8")
ImageStart = CB.Range("D7")
Factor = CB.Range("D4")
Levels = CB.Range("D5")
sp = " " & Worksheets("Input").Range("A52")
st = Worksheets("Input").Range("A51")
Set TableCorner = CB.Range("A25")
WarningCrack = "Specified output STRAIN CRACK"

' File paths
FilePathData = ThisWorkbook.Path & "\" & Analysis & "\" & LCase(Analysis) & ".DAT"
```
File Path Output = ThisWorkbook.Path & "\" & Analysis & "\" & UCase(Analysis) & ".OUT"
File Path Postprocess = ThisWorkbook.Path & "\" & Analysis & "\" & Analysis & ".FVC"

'Clear the cells where information is placed
TableCorner.Resize(TableCorner.Cells.SpecialCells(xlCellTypeLastCell).Row, 10).Clear

'Get information from the output
Open File Path Output For Input As #1
Do Until EOF(1)
    Line Input #1, LineFromFile
    'The initiation of a phase
    If InStr(LineFromFile, "PHASE") <> 0 And InStr(LineFromFile, "INITIALIZED") <> 0 Then
        ThisPhase = Mid(LineFromFile, 8, 1)
    End If
    'The initiation of a step
    If InStr(LineFromFile, "INITIATED") <> 0 Then
        p = p + 1
        TableCorner.Offset(p - 1).Value = p
        ThisStep = Mid(LineFromFile, 11, 3)
    End If
    'Store this step in Loading-array
    ReDim Preserve Loading(2, p - 1)
    Loading(0, p - 1) = ThisPhase
    Loading(2, p - 1) = ThisStep
    TableCorner.Offset(p - 1, 1).Value = ThisPhase
    TableCorner.Offset(p - 1, 3).Value = ThisStep
    TableCorner.Offset(p - 1, 6).Value = Iterations
    TableCorner.Offset(p - 1, 4).Value = "Divergence"
    LastConvergedStep = NoConvergenceStep
    TotalSteps(ThisPhase - 1) = ThisStep - 1
End If
'Number of cracks
If InStr(LineFromFile, "TOTAL MODEL") > 0 And Trim(Mid(LineFromFile, 30, 4)) > 0 Then
    TableCorner.Offset(p - 1, 5).Value = Trim(Mid(LineFromFile, 27, 7))
End If
If p <= 1 Then
GoTo WriteLoading
ElseIf Loading(1, p - 1) - Loading(1, p - 2) <> 0 Then
    TableCorner.Offset(p - 1, 8).Value = TableCorner.Offset(p - 1, 7).Value
Else
    TableCorner.Offset(p - 1, 8).Value = TableCorner.Offset(p - 1, 7).Value - TableCorner.Offset(p - 2, 7).Value
End If
End If

'Type of convergence
If InStr(LineFromFile, "RELATIVE") > 0 And InStr(LineFromFile, "TRUE") Then
    If TableCorner.Offset(p - 1, 9) <> 0 Then
        TableCorner.Offset(p - 1, 9) = TableCorner.Offset(p - 1, 9) & " and " & StrConv(Trim(Mid(LineFromFile, 3, 32)), vbProperCase)
    Else
        TableCorner.Offset(p - 1, 9) = StrConv(Trim(Mid(LineFromFile, 3, 32)), vbProperCase)
    End If
End If

Loop
Close #1

'Compute ranges for elastic/plastic behaviour, based on material model
Open FilePathData For Input As #2
Do Until EOF(2)
    LineInput #2, LineFromFile
    If InStr(LineFromFile, "NAME") > 0 And InStr(LineFromFile, "MASONRY") > 0 Then
        ThisIsMasonry = True
    ElseIf InStr(LineFromFile, "NAME") > 0 And InStr(LineFromFile, "MASONRY") = 0 Then
        ThisIsMasonry = False
    End If
    If ThisIsMasonry = True Then
        LineFromFile = Replace(LineFromFile, ".", ",")
        If InStr(LineFromFile, "CRACKB") > 0 Then
            BandWidth = CDbl(Mid(LineFromFile, 15, 11))
        ElseIf InStr(LineFromFile, "YOUNG") > 0 Then
            Emodulus = CDbl(Mid(LineFromFile, 15, 11))
        ElseIf InStr(LineFromFile, "COMSTR") > 0 Then
            Fc = CDbl(Mid(LineFromFile, 15, 11))
        ElseIf InStr(LineFromFile, "TENSTR") > 0 Then
            Ft = CDbl(Mid(LineFromFile, 15, 11))
        ElseIf InStr(LineFromFile, "GC") > 0 Then
            Gc = CDbl(Mid(LineFromFile, 15, 11))
        ElseIf InStr(LineFromFile, "GF1") > 0 Then
            Gt = CDbl(Mid(LineFromFile, 15, 11))
        ElseIf InStr(LineFromFile, "COMCRV") > 0 Then
            ShapeC = Trim(Mid(LineFromFile, 15, 11))
        ElseIf InStr(LineFromFile, "TENCRV") > 0 Then
            ShapeT = Trim(Mid(LineFromFile, 15, 11))
        End If
    End If
Loop
Close #2

'Compressive behaviour
If ShapeC = "ELASTIC" Or ShapeC = "CONSTA" Or ShapeC = "THOREN" Then
    STRNT2 = Replace(CStr(Ft / Emodulus), ",", ".")
ElseIf ShapeT = "LINEAR" Then
    STRNT2 = Replace(CStr(2 * Gt / (BandWidth * Ft)), ",", ".")
 ElseIf ShapeT = "HORDYK" Then
    STRNT2 = Replace(CStr(5.14 * Gt / (BandWidth * Ft)), ",", ".")
End If

'Compressive behaviour
If ShapeC = "ELASTIC" Or ShapeC = "CONSTA" Or ShapeC = "THOREN" Then
ElseIf ShapeC = "PARABO" Then
    STRNC2 = "-" & Replace(CStr(-5 / 3 * Fc / Emodulus), ",", ".")
End If

'Print post-processing file (.FVC) for input in iDIANA
Open FilePathPostprocess For Output As #3
For lr = 0 To WorksheetFunction.Sum(TotalSteps) - 3
    'For each phase execute these commands
    If lr = 0 Then
        Print #3, Worksheets("Input").Range("A2") & " Analysis"
        Print #3, Worksheets("Input").Range("A4")
        Print #3, Worksheets("Input").Range("A5")
        Print #3, Worksheets("Input").Range("A13")
        Print #3, Worksheets("Input").Range("A15")
        Print #3, Worksheets("Input").Range("A18")
        Print #3, Worksheets("Input").Range("A22")
        Print #3, EmptyLine
        Print #3, EmptyLine
    End If
    'Set the load-step for this serie of commands
    Print #3, Worksheets("Input").Range("A29") & Loading(1, lr) & " & Loading(2, lr)
    'Only print images from a certain step onwards
    If lr >= ImageStart And lr <= ImageEnd Then
        'Results from front right (displacement)
        Print #3, Worksheets("Input").Range("A13")
        Print #3, Worksheets("Input").Range("A15")
        Print #3, Worksheets("Input").Range("A18")
        Print #3, Worksheets("Input").Range("A22")
        Print #3, EmptyLine
        Print #3, EmptyLine
    End If
    'Contour plots for the strains
    If CB.CBStressPiFr.Value = True Or CB.CBStressPiBa.Value = True Then
        Print #3, Worksheets("Input").Range("A19")
        Print #3, Worksheets("Input").Range("A46") & " STRAIN_ALL_FRONT_" & lr + 1 & sp
    End If
    If CB.CBStressPiBa.Value = True Then
        Print #3, Worksheets("Input").Range("A19")
        Print #3, Worksheets("Input").Range("A46") & " STRAIN_ALL_BACK_" & lr + 1 & sp
    End If
End If

'Contour plots for the stresses
If CB.CBStressAllFr.Value = True Or CB.CBStressAllBa.Value = True Then
    Print #3, Worksheets("Input").Range("A56")
    Print #3, Worksheets("Input").Range("A23") & " TO " & STRNT2 & " LEVELS " & Levels
    If CB.CBStressAllFr.Value = True Then
        Print #3, Worksheets("Input").Range("A20")
        Print #3, Worksheets("Input").Range("A46") & " STRAIN_ALL_FRONT_" & lr + 1 & sp
    End If
    If CB.CBStressAllBa.Value = True Then
        Print #3, Worksheets("Input").Range("A19")
        Print #3, Worksheets("Input").Range("A46") & " STRAIN_ALL_BACK_" & lr + 1 & sp
    End If
End If

...
Subsequently, the command file (.FVC) that is generated, is executed in iDIANA. This can be done in the same manner as the batch file in Section I.1. Among the results that are being generated is the tabulated output of the forces and displacements of the floors
per step. These files are used by the VBA code Results.xlsm to plot shear-drift curves (see Listing I.5).

Listing I.5: VBA macro Results.xlsm

```
Option Explicit
Function e(n As String) As Boolean
    e = False
    Dim ws As Variant
    For Each ws In ThisWorkbook.Sheets
        If n = ws.Name Then
            e = True
            If ws.Type = xlWorksheet Then
                ws.Cells.Clear
            End If
        End If
    Next ws
    Exit Function
End Function

Sub Postprocessing()
    ThisWorkbook.Activate
    Dim at As Integer 'Analysis type
    Dim EmptyFile(1, 1) As Integer
    Dim fl As Integer 'Floor
    Dim li As Integer 'Line
    Dim ls As Integer 'Loadstep
    Dim MaxRowNr As Integer
    Dim n As Integer 'General count number
    Dim NumberSheets As Integer
    Dim q As Integer 'Quantity
    Dim ResultFileMaxLines As Integer
    Dim RowNr As Integer
    Dim RowNrOutput As Integer
    Dim RowNrOutputAve As Integer
    Dim RowNrResult As Integer
    Dim StartingLoadStep As Integer
    Dim TotalFloors As Integer
    Dim TotalLoadStepsMax As Integer
    Dim TotalLoadStepsReal As Integer
    Dim AnalysisName As String
    Dim BenchmarkFile As String
    Dim FileName As String
    Dim FilePath As String
    Dim GraphsheetName As String
    Dim LineFromFile As String
    Dim ResultName(2, 2) As String
    Dim ResultQuantity(1) As String
    Dim ResultStep As String
    Dim SheetName As String
    Dim RatiosheetName As String
    Dim r As Variant
    Dim s As Variant
    Dim StartCell As Variant
    Dim Accuracy(1) As Double
    Dim MaxX(1) As Double
    Dim MinX(1) As Double
    Dim MinY(1) As Double
    Dim QuantityValue(1, 1) As Double
    Dim ResultValue() As Double
    ReDim Preserve ResultValue(1, 1, 500)
    Dim Ratio() As Double
    Dim L As Double
    Dim BenchmarkFile = "Results_benchmark_directionOLD_v6.xlsx"
    RatiosheetName = "RATIO"
    Workbooks.Open (ThisWorkbook.Path & "\" & BenchmarkFile)
    For n = 1 To Workbooks(BenchmarkFile).Worksheets.Count
        Dim NumberSheets = Workbooks(ThisWorkbook.Name).Worksheets.Count
        If e(Workbooks(BenchmarkFile).Worksheets(n).Name) = False Then
            Workbooks(BenchmarkFile).Worksheets(n).Copy After:=Workbooks(ThisWorkbook.Name).Worksheets(NumberSheets)
        Else
            Workbooks(BenchmarkFile).Worksheets(n).Cells.Copy Destination:=ThisWorkbook.Worksheets(n).Cells
        End If
    Next n
```

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Workbooks(BenchmarkFile).Close
Worksheets(RatiosheetName).Range("A1") = "Nr"
Worksheets(RatiosheetName).Range("B1") = ChrW(945) 'alpha
Worksheets(RatiosheetName).Range("D1") = ChrW(952) 'theta
Worksheets(RatiosheetName).Range("E1") = "u3"
Worksheets(RatiosheetName).Range("E2") = "mm"

'Import results from output of iDIANA
AnalysisName = UCase(InputBox("Which analysis(-folder) do you want to post-proces?"))

'Starting values
Accuracy(0) = 0.001
Accuracy(1) = 0.025
ResultFileMaxLines = 40
RowNrOutput = 5
RowNrResult = 7
StartingLoadStep = 0
TotalFloors = 2
TotalLoadStepsMax = 500
L = 2700 'height of the frame

'Some text
ResultQuantity(0) = "DISP"
ResultQuantity(1) = "NODALF"
For fl = 1 To TotalFloors
For q = 0 To UBound(ResultQuantity)
    EmptyFile(fl - 1, q) = 0
    SheetName = ResultQuantity(q) & "_FL" & fl
    'Create new sheets for results if sheets don’t already exist
    If e(SheetName) = False Then
        Sheets.Add.Name = SheetName
        Sheets(SheetName).Move After:=Sheets(ThisWorkbook.Worksheets.Count)
    End If
    Set StartCell = Worksheets(SheetName).Range("A1")
    'Import result values for each load step
    For ls = 1 To TotalLoadStepsMax + 1
        FilePath = ThisWorkbook.Path & "\" & AnalysisName & "\RESULTS_" & ResultQuantity(q) & "_FLOOR" & fl & "_" & ls & ".FVI"
        If Dir(FilePath) = "" Then
            TotalLoadStepsReal = ls - 1
            GoTo LastLoadStep
        End If
        Open FilePath For Input As #1
        li = 1
        RowNr = RowNrOutput
        Do Until EOF(1)
            Line Input #1, LineFromFile
            ' For first load step print the node number in left column
            If ls - EmptyFile(fl - 1, q) = 1 Then
                ResultStep = Mid(LineFromFile, 8, 6)
                StartCell.Offset(RowNr).Value = ResultStep
                ResultName(fl - 1, q) = SheetName
            End If
            ' Store the values of forces for each load step
            If (Mid(LineFromFile, 2, 2) = "-1" And q = 0) Or (Mid(LineFromFile, 2, 2) = "-2" And q = 1) Then
                ResultStep = Mid(LineFromFile, 14, 12)
                StartCell.Offset(RowNr, 14).Value = ResultStep
                RowNr = RowNr + 1
'Check if the results-file actually contained information
ElseIf li = RowNrOutput + 3 And Mid(LineFromFile, 2, 2) <> "-1" Then
    EmptyFile(fl - 1, q) = EmptyFile(fl - 1, q) + 1
    ResultStep = 0
    StartCell.Offset(RowWr, ls).Value = ResultStep
End If

'Check if load step existed at all
ElseIf li = 1 And Mid(LineFromFile, 2, 4) = "9999" Then
    Close #1
    TotalLoadStepsReal = ls - 1
    GoTo LastLoadStep
End If

li = li + 1 'Next line
Loop

'Print the text Nodes in the upper left corner
StartCell.Offset(RowWrOutput - 1).Value = "Nodes"

'Print and format the load step as headers
With StartCell.Offset(RowWrOutput - 1, ls)
    .Value = ls
    .Font.Bold = True
End With

'Print titles for result values
With StartCell.Offset(RowWrOutputAve)
    If q = 0 Then
        .Value = "Average Displacement"
    ElseIf q = 1 Then
        .Offset(-1).Value = "Nodes"
        .Value = "<19950"
    End If
End With

'Compute the average displacement/force per load step
If q = 0 Then
    ResultValue(fl - 1, q, ls - 1) = WorksheetFunction.Average(StartCell.Offset(RowWrOutput, ls)
        .Resize(RowNr - RowWrOutput + EmptyFile(fl - 1, q)))
ElseIf q = 1 Then
    ResultValue(fl - 1, q, ls - 1) = Application.DSum(StartCell.Offset(RowWrOutput - 1).Resize(
        RowNr - RowWrOutput + EmptyFile(fl - 1, q) + 1, ls + 1), ls + 1, StartCell.Offset(
        RowWrOutputAve - 1).Resize(2))
End If

'Print average displacement/force per load step
StartCell.Offset(RowWrOutputAve).Offset(, ls).Value = ResultValue(fl - 1, q, ls - 1)

MaxRowNr = RowNr

EmptyResultFile: Next ls 'Loadstep

LastLoadStep: 'Go to this location when last load step is reached
    Next q 'Quantity (displacement or force)
    Next fl 'Floor

''The number of loadsteps that should be skipped
For r = 0 To 1
    For s = 0 To 1
        If EmptyFile(r, s) > StartingLoadStep Then
            StartingLoadStep = EmptyFile(r, s)
        End If
    Next s
    Next r

''Write PsD results to the data-sheet
For fl = 0 To TotalFloors - 1
    GraphsheetName = "GRAPH_LVL" & fl + 1
    SheetName = "DATA_LVL" & fl + 1
    MaxX(fl) = 0
    MinX(fl) = 0
    MinY(fl) = 0
    ReDim Preserve ResultValue(1, 1, TotalLoadStepsReal - 1)
    ReDim Preserve Ratio(3, TotalLoadStepsReal - 1)
    For q = 0 To UBound(ResultQuantity)
        For ls = 0 To TotalLoadStepsReal - 1
"For ls = StartingLoadStep To TotalLoadStepsReal - 1

' Compute drift, shear force, alpha and beta
If fl = 0 And q = 0 Then 'drift level1
    QuantityValue(fl, q) = 1000 * -ResultValue(fl, q, ls)
ElseIf fl = 0 And q = 1 Then 'shear level1
    QuantityValue(fl, q) = 0.001 * (ResultValue(fl, q, ls) + ResultValue(fl + 1, q, ls))
ElseIf fl = 1 And q = 0 Then 'drift level2
    QuantityValue(fl, q) = 1000 * (-ResultValue(fl, q, ls) + ResultValue(fl - 1, q, ls))
End If

Ratio(0, ls) = ResultValue(fl - 1, q + 1, ls) / ResultValue(fl, q + 1, ls)
Ratio(1, ls) = ResultValue(fl - 1, q, ls) / ResultValue(fl, q, ls)
Ratio(2, ls) = QuantityValue(fl, q) / L
Ratio(3, ls) = -1000 * ResultValue(fl, q, ls) * tan(Ratio(2, ls)) + Ratio(0, ls) * L / (1 + Ratio(0, ls))

Worksheets(RatiosheetName).Range("A1").Offset(ls + 2).Value = ls + 1
Worksheets(RatiosheetName).Range("A1").Offset(ls + 2, 1).Value = Ratio(0, ls) 'alpha
Worksheets(RatiosheetName).Range("A1").Offset(ls + 2, 2).Value = Ratio(1, ls) 'beta
Worksheets(RatiosheetName).Range("A1").Offset(ls + 2, 3).Value = Ratio(2, ls) 'theta
Worksheets(RatiosheetName).Range("A1").Offset(ls + 2, 4).Value = Ratio(3, ls) 'u3
ElseIf fl = 1 And q = 1 Then 'shear level2
    QuantityValue(fl, q) = 0.001 * ResultValue(fl, q, ls)
End If

Worksheets(SheetName).Range("I4").Offset(ls, q).Value = QuantityValue(fl, q)

' Find minimum and maximum drift
If QuantityValue(fl, 0) - MaxX(fl) > Accuracy(0) Then
    MaxX(fl) = WorksheetFunction.MRound(QuantityValue(fl, 0) + 1000 * Accuracy(0), 1000 * Accuracy(0))
ElseIf QuantityValue(fl, 0) - MinX(fl) < -Accuracy(0) Then
    MinX(fl) = WorksheetFunction.MRound(QuantityValue(fl, 0) - 1000 * Accuracy(0), -1000 * Accuracy(0))
ElseIf QuantityValue(fl, 1) - MinY(fl) < -Accuracy(1) Then
    MinY(fl) = WorksheetFunction.MRound(QuantityValue(fl, 1) - 1000 * Accuracy(1), -1000 * Accuracy(1))
End If

Next ls 'Loadstep
Next q 'Quantity

' Make graphs
'Make chart-sheet
If e(GraphsheetName) = True Then
    Application.DisplayAlerts = False
    Sheets(GraphsheetName).Delete
    Application.DisplayAlerts = True
End If
ThisWorkbook.Charts.Add.Name = GraphsheetName
Sheets(GraphsheetName).Move After:=Sheets(SheetName)

'Add data to the chart-sheet
With Sheets(GraphsheetName)
    .ChartType = xlXYScatter
    If .SeriesCollection.Count <> 1 Then
        'Add data-serie when none available
        .SeriesCollection.Add Source:=Worksheets(SheetName).Range("F4")
    End If
    'Add data and format color and size
    For at = 1 To UBound(ResultQuantity) + 1
        With .SeriesCollection(at)
            'Some style options
            .Name = Sheets(SheetName).Range("B1").Offset(, 2 * (at - 1)).Resize(1)
            .MarkerSize = 7
            .Format.Line.Weight = 1.5
            .Border.Weight = 2.25
        End With
    Next at
End With
End If
ElseIf at = 2 Then
  .XValues = Sheets(SheetName).Range("F4").Offset(, 3 * (at - 1)).Resize(TotalLoadStepsReal)
  .Values = Sheets(SheetName).Range("G4").Offset(, 3 * (at - 1)).Resize(TotalLoadStepsReal)
  .Format.Line.ForeColor.RGB = RGB(192, 0, 0)
  .MarkerForegroundColor = RGB(192, 0, 0)
  .MarkerStyle = xlMarkerStyleX
End If
End With

If at = 1 Then
  'Add data-serie for next analysis-type
  .SeriesCollection.Add Source:=Worksheets(SheetName).Range("I4:I43")
End If
Next at 'Analysis type

'Configure lay-out
 .SetElement (msoElementChartTitleAboveChart)
 .SetElement (msoElementPrimaryCategoryAxisTitleAdjacentToAxis)
 .SetElement (msoElementPrimaryValueAxisTitleRotated)
 .SetElement (msoElementPrimaryCategoryGridLinesMinorMajor)
 .SetElement (msoElementPrimaryValueGridLinesMinorMajor)

'Set position of legend
 .Legend.Left = 555
 .Legend.Top = 385

'Label and format subsequently x and y axis
 With .Axes(xlCategory)
   .MinimumScale = MinX(fl) 'Minimum value x-axis
   .MaximumScale = MaxX(fl) 'Maximum value x-axis
   .MajorGridlines.Border.Color = RGB(0, 0, 0)
   .Format.Line.Weight = 1.75
   .AxisTitle.Text = "Inter-storey drift (mm) Level " & fl + 1
 End With
 With .Axes(xlValue)
   If Abs(MinX(fl)) < Accuracy(fl) Then
     .MinimumScale = 0 'Minimum value y-axis
   End If
   .MinimumScale = MinY(fl) 'Minimum value y-axis
   .MajorGridlines.Border.Color = RGB(0, 0, 0)
   .Border.Color = RGB(0, 0, 0)
   .Format.Line.Weight = 1.75
   .AxisTitle.Text = "Shear force (kN) Level " & fl + 1
 End With

'Label and format chart area
 .ChartTitle.Text = "Shear drift curves Level " & fl + 1
 .ChartArea.Border.LineStyle = xlNone

'Export graph to PDF
 .ExportAsFixedFormat Type:=xlTypePDF, FileName:=ThisWorkbook.Path & "RESULTS_" & AnalysisName & ".pdf", Quality:=xlQualityStandard, IncludeDocProperties:=True, IgnorePrintAreas:=False, OpenAfterPublish:=False
End With
Next fl 'Floor

'Save this workbook in results-folder
 ThisWorkbook.SaveCopyAs FileName:=ThisWorkbook.Path & "RESULTS_" & AnalysisName & ".xlsm"
ThisWorkbook.Save
End Sub
In this chapter, the results of the analysis that is discussed in Chapter 8 are shown more detailed. The results are grouped with respect to the five stages in Figures J.1 to J.5.

(a) Principal compressive strains
(b) Principal strains in the elastic region
(c) Normal crack strains
(d) Principal tensile stresses

Figure J.1: Step 1, results after the initiation stage
results monotonic pushover analysis

(a) Principal compressive strains
(b) Principal strains in the elastic region
(c) Normal crack strains
(d) Principal tensile stresses

Figure J.2: Step 11, results after the pseudo-linear stage

(a) Principal compressive strains
(b) Principal strains in the elastic region
(c) Normal crack strains
(d) Principal tensile stresses

Figure J.3: Step 23, results after the first severe cracking stage
(a) Principal compressive strains
(b) Principal strains in the elastic region

(c) Normal crack strains
(d) Principal tensile stresses

Figure J.4: Step 36, results after the crack propagation stage

(a) Principal compressive strains
(b) Principal strains in the elastic region

(c) Normal crack strains
(d) Principal tensile stresses

Figure J.5: Step 43, results after the collapse stage
In this chapter, the results of the analysis that is discussed in Chapter 10 are shown more detailed. The results are shown separately for each cycle in Figures K.1 to K.14.

(a) Displacement
(b) Principal elastic tensile strain
(c) Normal crack strain

Figure K.1: Results after load case 1 (self weight and live load)

(a) Displacement
(b) Principal elastic tensile strain
(c) Normal crack strain

Figure K.2: Results after load case 2

(a) Displacement
(b) Principal elastic tensile strain
(c) Normal crack strain

Figure K.3: Results after load case 3
Figure K.4: Results after load case 4

Figure K.5: Results after load case 5

Figure K.6: Results after load case 6

Figure K.7: Results after load case 7

Figure K.8: Results after load case 8
Figure K.9: Results after load case 9

Figure K.10: Results after load case 10

Figure K.11: Results after load case 11

Figure K.12: Results after load case 12

Figure K.13: Results after load case 13
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Figure K.14: Results after the last converged step