Robustness of Modular Timber Buildings

An investigation into alternative load paths in volumetric timber post and beam modules

Master Thesis

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Robustness of Modular Timber Buildings

An investigation into alternative load paths in volumetric timber post and beam modules

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”Successful engineering is all about understanding how things break or fail.”

HENRY PETROSKI

Preface

Before I came to Delft to pursue my master's, I already had the idea to do a thesis on the topic of timber. My interest in timber originates from moments in my youth when I worked with my dad on various construction projects while working simultaneously for a construction materials supplier and also a contractor. For some reason, wooden products were always in my hands, which, funny enough, must have significantly influenced me. My interest in timber grew even stronger when I learned about the serious environmental benefits the material offers compared to traditional construction materials and discovered how little of it is still applied nowadays. It became my goal to contribute to the body of knowledge, hoping it would accelerate the adoption of timber in the construction industry and create a better, more sustainable world.

So when I was looking for a more specific topic in 2021, I came into contact with the engineering firm Pieters Bouwtechniek. I told them about my desire to do something with timber. I also told them my wish to do something with modular construction because I foresee it has great potential. Rob Doomen and Chris Bosveld then introduced me to a project they were working on with a new kind of post and beam module. They explained the difficulties they face considering the robustness of it. It turned out to be the perfect topic for me and made me highly motivated to research.

For now, this thesis marks the final part of my education. It has been a long and unforgettable journey, starting with an HBO bachelor in Civil Engineering at Inholland University of Applied Sciences, continuing to Delft, where I started with a bridging programme, to finally graduating with a master's degree in Civil Engineering. However, research into the robustness of timber buildings is not finished. It is an awesome topic which will require a lot more studies, and I hope this thesis will inspire others to do so.

Joep Knuppe
Delft, December 2022

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My committee from TU Delft Thank you, Geert Ravenshorst, for being the chair of my committee. Your guidance and enthusiasm for the topic were of great value. Francesco Messali, thank you for the support on the theory of finite element analysis methods. Your knowledge and way of explaining were invaluable to me. Finally, thank you, Maria Felicita! I remember I came to you in the first place to talk about a thesis topic. You sent me the first papers, helped me specify my interest and contributed a lot to incorporating the right aspects to improve the scientific relevance of the work.

My fellow students from U-BASE A big thanks to all my fellow students and friends from U-BASE. Spending time together at study tours, activities, parties, lunch and coffee breaks was fantastic. My time at university was such a pleasure because of you.

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Abstract

Increasing housing demand in Europe and the need to be more sustainable are asking the construction industry to leave the traditional pathways and innovate. An emerging construction method with volumetric timber modules potentially offers the solution. As a construction material, timber offers significant environmental benefits over traditional materials such as concrete and steel, and modular construction can potentially reduce the construction time and costs of a building. Innovations, however, introduce uncertainties that impose threats to structural safety. To ensure structural safety, building regulations require a building, depending on its use and occupancy, to resist a disproportionate collapse resulting from an unexpected event. A property that improves collapse resistance is structural robustness; it describes the global survival tolerance of a structure to local damages independent of the cause. To accommodate robustness, a structure requires alternative load paths; they halt the propagation of failure after the initial damage. Nevertheless, guidelines for designing for robustness are limited, and research on volumetric modular timber construction is scarce. In fact, there are no established guidelines for ensuring alternative load paths in volumetric modular timber structures specifically. Therefore, this thesis aims to advance the current research regarding collapse resistance in volumetric modular timber buildings.

To achieve that goal, first, in this thesis, a literature study into robustness and modular buildings was performed. The study identified the importance of redundancy, proper connection design, ductility, and tying in modular structures for robustness. It was found that to determine the structural robustness of a modular building, an alternative load path analysis incorporating the behaviour of the connections should be performed since they characterize the global behaviour. Because the failure cause is not relevant, such an analysis is performed with a scenario-independent approach, thus based on notional element removal, i.e. hypothetical sudden removal events of load-supporting members which cause dynamic effects due to the sudden loss of internal forces. Common procedures for analysis were found and reviewed, and it was concluded that either a nonlinear static or nonlinear dynamic analysis could best be used. Of the two, the dynamic analysis is the most complete one, which inherently accounts

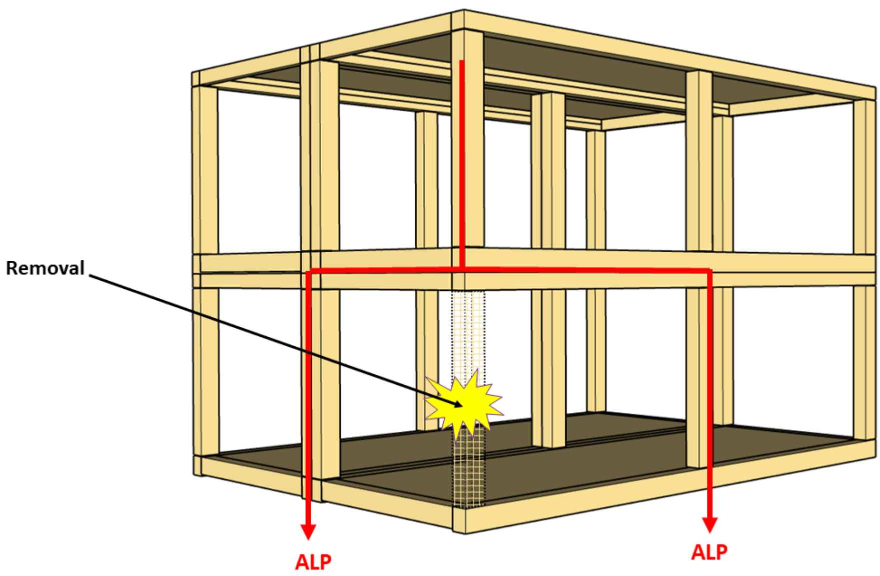

Figure 1: Case study example of alternative load path analysis in volumetric timber post and beam module

for dynamic effects. The static one, on the other hand, is less complex and more often used in practice but requires a dynamic amplification factor (DAF). The DAF is a load factor which converts a transient dynamic load into a static load which is equivalent in displacement demand. It should be applied to the effects of actions on the structure that comprise the alternative load path, not the entire structure. Commonly for timber structures, a DAF of 2.0 is used. Which is also currently the proposed prescription in the new working draft of Eurocode 5. Yet is that seen as the conservative upper bound, it places high demands on a structure. In this thesis, it is explored whether a lower value than 2.0 is achievable.

Secondly, a case study was performed in this thesis. The case study provided the structural design concept of a newly developed volumetric timber post and beam module by Lister Buildings, structurally designed by Pieters Bouwtechniek. The modules consist of glued laminated beams and columns with cross-laminated floor and ceiling slabs. They are not explicitly designed for robustness. Therefore, this study explored whether the initial design is inherently robust. With the modules, a hypothetical residential building of 5 storeys was created, which consecutively was decomposed into equivalent two-dimensional frame structures suited for 2D finite element analysis. To account for the mechanical behaviour of the connections in the models, nonlinear springs were used. To establish the springs, the component method was adopted. Their behaviour was determined from the elastic, plastic and failure performance of the individual components of a connection in translational and rotational degrees of freedom. Then based on a notional element removal concept, multiple possible loss events of load-bearing elements were analysed in the finite element software of Abaqus to examine the existence and development of alternative load paths. To do that, both nonlinear static and nonlinear dynamic analyses were performed. Subsequently, parameter variations in stiffness and resistance of the connection properties were studied to investigate their influence on the system's robustness and particularly dynamic response.

The results of this study indicate that the new design concept performs quite well on robustness. Several alternative load paths were identified, among which cantilever action, bridging action, and catenary action, depending on the removal event and scenario. For most removal events, resistance against collapse was sufficiently provided by a flexural mechanism. However, such a resistance mechanism should be treated with caution since, generally, it is considered not the most ideal because it may ask for over-dimensioning. Of key importance to the other alternative load paths, is the behaviour of the connections. They provide resistance against collapse by allowing moments to develop in the frames and by transferring a tensile force to create a horizontal tie, enabling the possibility of developing a catenary. Finally, did the parameter study reveal that the dynamic amplification factor for a modular timber system is very close to the upper bound of 2.0, e.g. values of 1.90 were identified. It was found that the factor depends on the stiffness and resistance of the connections, the removal speed and the removal scenario. On the one hand, increasing a connection's stiffness and resistance increases the dynamic amplification. On the other hand, increasing only the stiffness, and stimulating a ductile failure mode, allowed energy to dissipate and reduced the dynamic amplification.

The results of this research imply that for alternative load path analysis of timber structures, a DAF of 2.0 should be applied in a (non)linear static analysis. More research will be necessary to examine whether a lower value than 2.0 can be used. The next step for a more detailed analysis and quantification of the structural robustness plus dynamic amplification is the consideration of a whole 3D geometry. In that way, currently missing load-distributing mechanisms can be accounted for. Additionally, the influence of the connection parameters needs to be studied more thoroughly with a larger variety of parameters. Finally, including appropriate structural and material damping will further improve the accuracy of the dynamic amplification estimates.

Nomenclature

Abbreviations

Abbreviation	Definition
ALP	Alternative Load Path
ALPA	Alternative Load Path Analysis
ALS	Accidental Limit State
CLT	Cross Laminated Timber
DAF	Dynamic Amplification Factor
DLF	Dynamic Load Factor
DoF	Degree of Freedom
FEA	Finite Element Analysis
FEM	Finite Element Method
GLT	Glued Laminated Timber
NLS	Non Linear Static
PBT	Pieters Bouwtechniek
SF	Section Forces
SM	Section Moments
SLS	Serviceability Limit State
ULS	Ultimate Limit State

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1 Introduction

1.1. Context

Multi-storey buildings made of timber have the last decade experienced massive growth in interest because of society's grown awareness of the environmental benefits of construction with biobased materials. In addition, also new construction methods have been introduced, such as modular construction, a way of construction in which buildings are erected with mostly prefabricated 2D or 3D elements.

Modular construction has been gaining popularity due to its significant contributions in improving construction productivity, quality control and increasing cost savings. A recent case study on completed projects revealed that construction time could be reduced by 50 % and cost by 20% when compared to traditional construction [2]. In recent years several low- and mid-rise modular timber buildings have been built around the world. A famous example in the Netherlands is Hotel Jakarta in Amsterdam, which was finished in 2018. However, because of its novelty, there is still a limited understanding of the structural performance of these types of buildings. Due to a lack of design rules and codes, many innovative structural systems have been invented. Each innovation has unique characteristics, which impose unique challenges to predict the structural behaviour. A result is that many uncertainties have been introduced in these types of buildings which impose threats to structural safety. To prevent the worst thing that could happen, i.e. a future collapse, a better understanding of the structural behaviour of these systems is desired. Specifically, the knowledge about collapse resistance and prevention of progressive collapse of these systems needs to be improved.

1.2. Problem

An essential structural property which prevents collapse propagation is robustness. Robustness describes the global survival tolerance of a structure to local damages. To accommodate robustness, a structure should have so-called alternative load paths, which halt the propagation of failure and thus prevent collapse. Unfortunately, there are no established guidelines for alternative load path analysis/design related to timber- and specifically modular timber buildings.

1.3. Goal

The meta goal of this thesis is to advance the current academic knowledge on the robustness of modular timber buildings. Because by increasing that knowledge, scientists and engineers will have a better understanding of the structural behaviour of those structures in case of abnormal events, which will decrease the possible occurrence of progressive collapse events in the future. Obviously, the last thing we would want is that a modular timber structure will fail and that we have to learn from our mistakes.

To achieve that main goal, some specific objectives related to this research are set out:

1. Describe the state of the art regarding robustness (prevention of disproportionate collapse) and its application to modular timber buildings.

2. Develop models to identify and quantify the alternative load paths (ALPs) in subsystems and components of modular timber buildings.
3. Investigate the influence of different design parameters on the development of ALPs in modular timber buildings.

1.4. Research Questions

Part I - Literature Review
The first part of the thesis consists of a literature study on robustness in general. Additionally, a focus is added on timber and modular construction.

- What are existing design methods to ensure alternative load paths in a modular building?
- What kind of alternative load paths can occur in a modular building in case of notional element removal?
- What are common procedures for alternative load path analysis of a (modular/timber) building?
- What kind of models provide reliable results for identifying and quantifying alternative load paths in a (modular/timber) building in case of notional element removal?
- What are promising design methods to develop alternative load paths in modular timber buildings to increase their structural robustness?

Part II - Modelling
The second and main part of this research will consist of the modelling of a modular timber structure within an FEA package to perform alternative load path analysis (ALPA) by notional element removal. ALPA will give the most insight into the true structural behaviour of timber modules and provides a viable solution to quantify robustness.

- What alternative load paths do occur in a *'post and beam timber module'* in case of notional element removal?
- What is the influence of different connection properties on the development of alternative load paths in a timber building constructed out of volumetric post and beam modules in case of notional element removal?
- What connection properties influence the dynamic amplification of a timber building constructed out of volumetric post and beam modules in case of notional element removal?

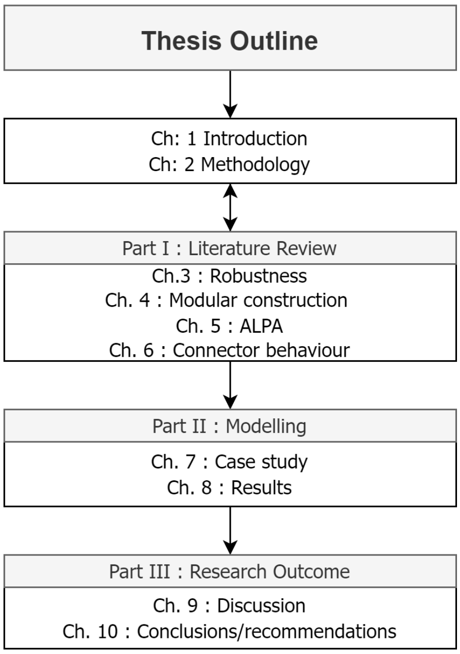
Part III - Research Outcome
The final part of the thesis will summarize the findings of the research and will discuss the results. Here, conclusions will be drawn, and recommendations for structural designers/engineers and building codes will be given.

- How does a timber post and beam volumetric modular building behave in case of notional element removal, and to what extent can that behaviour be improved by design?
- What are appropriate dynamic amplification factors to asses a timber building constructed out of volumetric post and beam modules in nonlinear static analysis?

Main research question
When the questions mentioned above are answered, it is deemed possible to answer the main research question:

- To what extent can a timber building constructed out of volumetric post and beam modules develop sufficient alternative load paths to prevent disproportionate collapse in case of notional element removal?

1.5. Structure



2
Methodology

2.1. Literature review
As mentioned before are modular timber structures still relatively new, and because of that, there is very little literature available on the topic, especially related to robustness. However, information regarding timber and modular construction individually is more readily available. In particular, concrete and steel modular structures have been studied more extensively on robustness. Therefore, those structures will be reviewed. Accordingly, does the literature review need to qualitatively incorporate information on robustness regarding both timber and modular construction. Besides, does the literature review need to provide insight into the modelling of a structure for alternative load path analysis (ALPA). Finally, necessary analytical design parameters need to be acquired.

2.2. Modelling
To perform an ALPA, a model needs to be created using an FEA package. The most common FEA package found in the literature for that kind of analysis is ABAQUS. Because of that, the author has decided to use that package for this thesis. The literature points out 4 different analysis procedures as described in chapter 3. Marjanishvili [11], recommends starting with a less complex calculation procedure and then increase towards the more complex procedures. That methodology was applied in this project.

Casestudy

This thesis is carried out in collaboration with engineering consultant Pieters Bouwtechniek Delft b.v. (Pieters). Currently, Pieters is working on several projects with volumetric timber modules. One of these projects is Koffiefabriek, a multi-story residential building in Amsterdam by Lister Buildings. Because of that, there is a lot of documentation available about the structural system, the connections, the loads etc. Consequently, it lends itself perfectly to this study.

2.3. Scope
Modular construction is a wide topic. In addition, is robustness a complex field in engineering. To make the research feasible for a master thesis the work will be bounded by scope limitations.

1. This research will investigate *'3D volumetric modules'*; they are the most widely used structural system for modular buildings and offer the most benefits.
2. This research will investigate *'post and beam timber modules'*. This is a type of module that allows large open spaces and is flexible in its application. In other literature, these kind of modules are also common under the name of *'corner supported modules'*, see chapter 4. However, the modules in this study have intermediate columns and thus is the term *'post and beam module'* proposed and used.
3. Robustness can be analysed with 3 different approaches, this research will only use one approach: a *deterministic analysis*.

Part I

Literature review

3 Robustness

3.1. Introduction

This chapter provides a review on structural robustness with an emphasis on timber buildings. Regulatory implications and recommendations are presented, and an answer is sought to the following literature research questions.

- *What are existing design methods to ensure alternative load paths in a (modular) building?*
- *What kind of alternative load paths can occur in a (modular) building in case of notional element removal?*
- *What are promising design methods to develop alternative load paths in (modular/timber) buildings to increase their structural robustness?*

3.1.1. Background

All kinds of buildings may be subjected to extreme events, take for example, the occurrence of earthquakes, explosions, vehicle impacts, fires, etc. Events like that usually cause damage to the structure of a building, which may propagate through the building and ultimately cause its partial or entire collapse. That phenomenon is referred to as ”progressive collapse” [7]. An important structural quality which prevents progressive collapse is robustness, the insensitivity of a structure to local failure[21]. Ever since the partial collapse of the Ronan Point building in 1968, progressive collapse and structural robustness have gained more attention in the world of building engineering. Due to the unexpected event of a gas explosion on the 18th floor, an entire corner of a building collapsed, resulting in several casualties. After that event, building codes changed and required to restrict the damage due the loss of a column or wall to the immediate surroundings. The Institution of Structural Engineers (IStructE) was afterwards the first one to propose prescriptive design methods to prevent progressive collapse by providing minimum tie forces in 1970 [21].

3.1.2. Definition

The term robustness is used ambiguously in different publications and often named together with progressive and disproportionate collapse. Generally, it's related to the ability of a structure to resist collapse and especially disproportionate collapse is mentioned frequently, as can be found in the definition of Robustness in the Eurocode (EN 1991-1-7). Disproportionate refers to something which is judged by some observations and measures.

1 *The ability of a structure to withstand events like fire, explosions, impact or the consequences of human error without being damaged to an extent disproportionate to the original cause.*

Qualitative model

Several authors presented a qualitative model to distinguish robustness from other properties. The model below [11], describes the probability of a disproportionate collapse. It is based on the occurrence of an abnormal event on a structure, *E*, which may lead to initial local damage, *D*. Robustness is the conditional probability of a disproportionate spreading of failure given the initial damage [11].

$$P(C) = \underbrace{P(C \mid D)}_{\text{Robustness}} \cdot \underbrace{P(D \mid E)}_{\text{Vulnerability}} \cdot \underbrace{P(E)}_{\text{Exposure}} \tag{3.1}$$

Bret and Lu (2013), explain that robustness and vulnerability together make up a building's collapse resistance and that they are the only properties which may be affected by structural engineering [11]. Whether or not damage is spread is described as an intrinsic property independent of any abnormal event. Likewise, do Huber et al. define robustness as an intrinsic property of the structure alone which enhances a building's global tolerance to local failure, independent of the failure cause [11].

3.1.3. Alternative Load Paths

To avoid progressive collapse in buildings, alternative load paths must be available to transfer the load supported by the damaged element (eg. a column) to neighbouring elements. Without alternative load paths, progressive collapse is inevitable unless certain design measures are introduced, see Section 3.2.

In the case of a column removal scenario in framed structures, five resisting mechanisms can provide alternative load paths by redistributing the loads to the healthy parts of a structure and minimise the risk of progressive collapse [1].

- Bending of the beam where the column has failed (generally ineffective mechanism since the beams have to be over-dimensioned, so this design criterion is seldom used).
- Vierendeel behaviour of the moment frame over the failed column.
- Arch effect of the beams where the column has failed (effective mechanism when horizontal displacement of the neighbouring columns is small).
- Catenary/membrane behaviour of beams/slabs, bridging the damaged column (Figure 3.1) by means of large rotations and displacements.
- Contribution of non-structural elements such as external walls and partitions

Additionally, an extra resisting mechanism exists called *hanging action*, which is rarely found in literature. According to Mpidi Bita, hanging action facilitates a new load path which allows floors to hang on the column above. This ALP results in vertical forces accumulating to the top of the building and requires effective vertical and horizontal ties [24].

Catenary/membrane action From the possible resisting mechanisms, caternary/membrane action is considered as the line of defence [1]. Nevertheless, it's also the ideal load distribution mechanism since it is able to maintain very high loads and deformation [35]. The term catenary action actually refers to the ability of beams to resist vertical loads through the formation of a catenary-like mechanism. Membrane action, on the other hand, refers to the three-dimensional situation of this case and develops within slabs, see Figure 3.1.

Catenary action implies the development of large enough deformations such that gravity and associated debris impact loads are mainly resisted by the vertical components of axial forces that develop in the beams, i.e. catenary forces. To develop a catenary, connections need to be ductile enough and possess sufficient rotational capacity to allow the system to achieve a new configuration. In addition, for the load transfer mechanism to work, sufficient integrity and high continuity should be maintained to support the developing catenary forces [30].

Ductility To achieve an alternative load path, a structure requires ductility. Ductility is the ability of a structure to deform plastically under loading without rupture. Through plastic straining, energy can be dissipated, which is beneficial in the scenario of sudden failure. In timber structures, ductility is predominantly supplied by steel connections, because timber itself fails in a brittle failure mode when in tension, bending or shear [11]. In compression, however, timber exhibits some ductile behaviour. Many international codes and other scientific publications mention the importance of ductility in enabling alternative load paths and increasing the capacity of a structure to withstand an instantaneously applied load [21], [11], [1]. Amongst those publication ductility is commonly defined as a ratio between the respective displacement (Δ_{Fmax}) and the yield displacement (Δ_{Fy}) or a variation of that. However, recently an extensive study into the ductility of timber connections was performed by Ottenhaus et al [26]. They suggest a performance-based absolute definition for ductility: *the ability to sustain a given load under increasing displacements*. Which is, according to the authors more suited than a ratio.

Furthermore, they recommend designing the connection on ductility performance. The connections in a timber structure need to be designed as "fuses", somewhat like a fuse in an electrical system. This to prevent brittle failure causing progressive collapse. Regardless the load case the connection should therefore be the weakest link in the capacity chain. In that way, it can develop its designed ductility without premature brittle failure [26].

Rotational capacity To allow catenary action to mobilise under large deformations, sufficient rotational deformation capacity is required. The rotational capacity is mostly provided by sufficient ductility (rotational ductility). With reference to timber structures, it's strongly related to steel connections. Despite the importance, most international codes don't give any requirements for rotational capacity, only some codes such as the US - Department of Defence (DoD) prescribe certain rotation of 11.4° (0.2 rad) for their tying force requirement. A recent experimental study by Lyu et al. showed that most commercially available connections can provide sufficient rotations to form a catenary mechanism [23], however, to fully take advantage of the mechanism and increase the robustness of a building, the connections with the larger capacities should be used i.e larger deformations result in lower catenary forces. Lyu et al. propose to use a double-plate connector, which has the capacity of 0.2 rad [23].

3.2. Robustness desing strategies

To provide robustness in structures, some different design methods can be chosen. Generally, they are classified as either indirect or direct design methods [10]. The former resembles prescriptive rules which aim to provide robustness without any explicit calculation of the damaged scenario. The latter however, is based on structural evaluations of damage scenarios. Recently Huber et al. [10], provided a very extensive overview of design methods to ensure robustness in timber buildings, the paragraphs here below just summarize their work.

3.2.1. Tie forces

A design method which is prescribed by many international codes to increase a structures robustness is tying. Tying is usually provided through the creation of continuous horizontal and vertical load paths in and around the perimeter of the structure [21]. Continuous ties increase the possibility of load transfer in the scenario of a local failure. Ties can be created by connecting several structural elements with mechanical connections or by additional elements. Within in the codes generally, a minimum tying force is prescribed to create a tie [1]. Alternative load paths are indirectly designed with this method because it does not require the analysis of the damaged scenario [11]. The method is mostly recommended for structures of low-consequence classes or buildings with low risk of progressive collapse. The efficiency

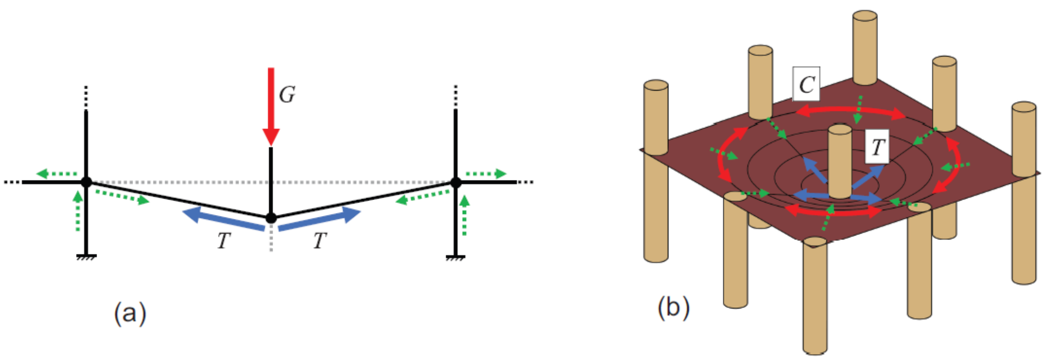


Figure 3.1: Catenary (a) and Membrane Action (b) [11].

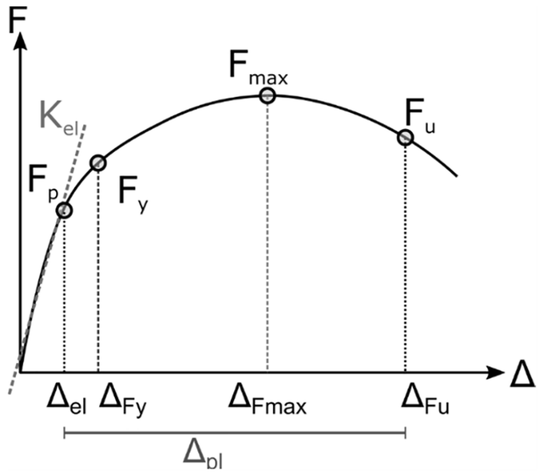


Figure 3.2: Example of ductile load-displacement curve with elastic and plastic displacements [26]

of tying is part of the debate, recent studies showed that the rotations required in the connections to form a tensile member, see catenary action, are unobtainable in many cases [1].

3.2.2. Redundancy

Redundancy can be seen as the existence of alternative load paths. A structure is redundant if after damage loads can be redistributed through other members. Redundant systems are statically indeterminate with several parallel members participating in the load transfer. Two types of redundancy are generally considered, active- and passive redundancy. The former is when parallel members share the load at low load levels. The latter refers to the case when other elements start taking up loads after a certain amount of damage [12]. Redundancy may have adverse effects on robustness because if additional members can not assist in the extra load transfer then they can promote progressive collapse [11].

3.2.3. Alternative Load Path Design

Alternative load path design provides load distribution paths in a structure after the removal of a load-carrying element. This design method explicitly considers an element removal scenario and evaluates the structure's ability to redistribute the loads [32]. Two of the main mechanisms of load distribution which can be utilised for this method are catenary action and membrane action [9], as described in 3.1.3. In catenary action, the beams above a failed element behave like a chain to support the remaining structure. In membrane action, the floor slabs or floor diaphragms above a failed element form a membrane and support the remaining structure [10].

3.2.4. Key Elements

A direct design method and scenario-dependent approach which prevents local failure is key element design. Key elements have over-strength which makes them less vulnerable to damage. They are designed to withstand a certain exposure, usually a blast load of 34kPa. According to Arup, should this method be used as a last resort, only when the removal of structural elements can not be tolerated [21]. In addition, does applying this method have a negative effect on material consumption.

3.2.5. Compartmentalisation

Damage and collapse propagation can be controlled by introducing a discontinuity in the structure. Through division, individual compartments are created which are in themselves robust but may collapse in case of an extreme event. Such a collapse will then not propagate to the rest of the structure. It is an effective solution for horizontally aligned structures with low height but less applicable for tall vertical buildings [11]. In general, it may be considered when a collapse resulting from a local failure has to limited to an acceptable extent [11].

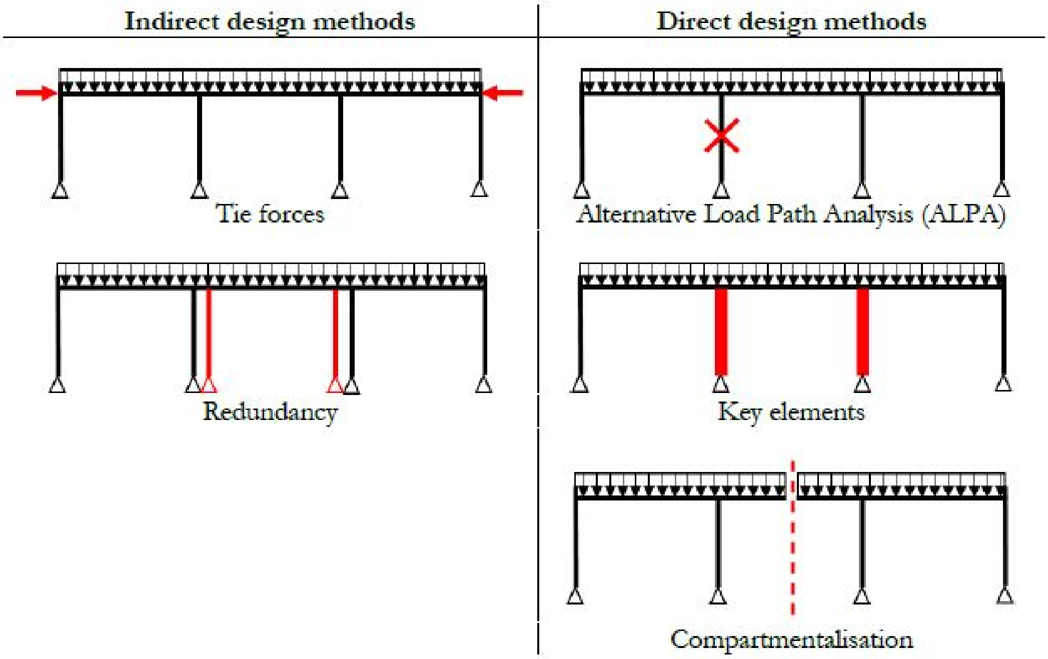


Figure 3.3: Direct and indirect design methods for robustness [35].

3.3. Analysis Methods

Robustness can be analysed in different ways. Various literature pointed out 3 different approaches which vary in complexity and all 3 yield measures to quantify robustness [35][11].

1. Reliability Analysis
2. Risk Analysis
3. Deterministic Analysis

Both risk and reliability analysis use calculation methods from probability theory, whereas deterministic analysis applies non-probabilistic mechanical methods. In case of structural design, do international codes mostly require performing a deterministic analysis, because that one has the ability to analyse how a structure subjected to local damage is capable to redistribute additional loads. As a result, a deterministic method captures the true behaviour of a structure after damage has occurred [1].

3.3.1. Reliability Analysis

Reliability analysis quantifies the probability of the performance of a structural system over a building's service live. The probability of performance is the complement to the probability of failure [10]. According to Voulpiotis reliabilities of undamaged and damaged scenarios of a structure can be compared after performing a structural analysis [35].

3.3.2. Risk Analysis

Risk analysis can be both carried out in both a qualitative or quantitative way. A risk assessment can indicate if hazards are likely to happen and if measures need to be taken to prevent them. Generally, the probabilities and effects of accidental and extreme actions need to be estimated, considering a suitable set of possible hazard scenarios. The consequences can for example be estimated in terms of the number of casualties and economic losses. Risk methods can be very conceptual and arduous, Eurocode 1991-1-7 provides an example of how risk analysis can be performed.

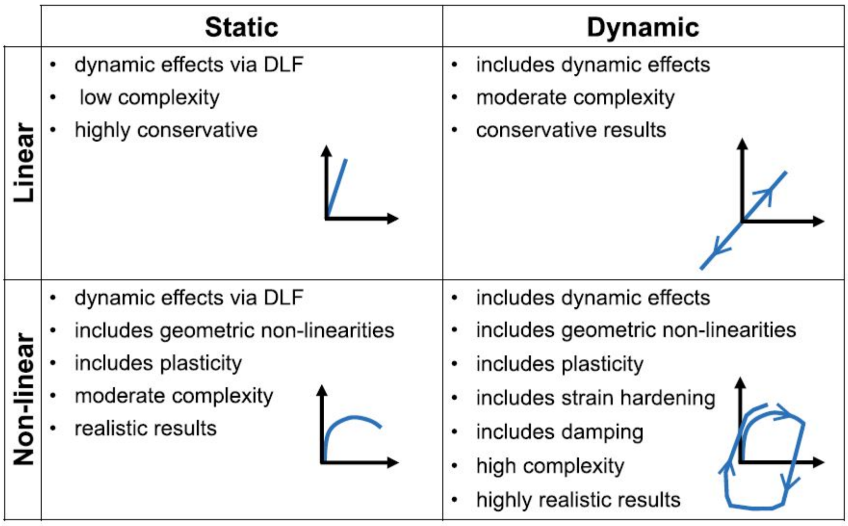


Figure 3.4: Analysis procedures for an ALPA [21].

3.3.3. Deterministic Analysis

Deterministic analysis calculates the detailed structural response to either a specific damage (i.e. a scenario-independent approach) or to a specific exposure (i.e. a scenario-dependent approach) to quantify robustness. Common terms found in literature are notional-damage and notional element removal. Notional refers to something that does not exist in reality but only as an idea or theory (hypothetical) . An analysis method that uses notional damage is a scenario-independent approach whereas approaches which consider specific exposure are scenario-dependent [11]. Given that the general philosophy of robustness is that a structure is insensitive to local failure, independent of the failure cause, the scenario-independent approach the most appropriate one.

Alternative load path analysis

A common way of practice to study the structural behaviour after notional element removal is *alternative load path analysis* (ALPA). The objective is to assess how loads are absorbed along alternative paths in the structure after the initial damage, and to quantify the extent of the collapse progression [21]. The analysis can be conducted on different levels of complexity. Usually linear static, non-linear static, a linear dynamic or a non-linear dynamic procedure is used [11] as can be seen in Figure 3.4.

Quantification of Robustness

In a deterministic robustness analysis, different measures can quantify robustness. Some specifically look at the changes in structural properties such as stiffness and strength, while others quantify the extent of collapse progression after an assumed initial damage. That last measure is the most simple measure and can be done by comparing the damaged masses or floor areas by the limit values in the building codes.

3.4. Building Codes

The Eurocode 1 (2006) [28], classifies buildings in 4 consequence classes, i.e. 1, 2a, 2b and 3, depending on size and use. For class 1 buildings, no requirements are specified regarding robustness, for the higher classes, however, the code demands horizontal tying. In addition, does the code demand vertical tying for class 2b, to create an alternative load path. Alternatively, for 2b, an ALPA may be performed as an alternative to tying. The code then demands that each load-bearing column and each beam supporting a column, or any nominal length of a load-bearing wall, be removed one by one on each storey. A nominal length should generally be 2.25H at most, where H is the storey height. For external walls of timber, the nominal length should be taken as the length between two vertical supports. The recommended limit for resulting damage is the smaller of 100m2 or 15% of the floor area in each of two adjacent storeys. However, Eurocode 1 (2006) gives no further guidance for the conduction of an

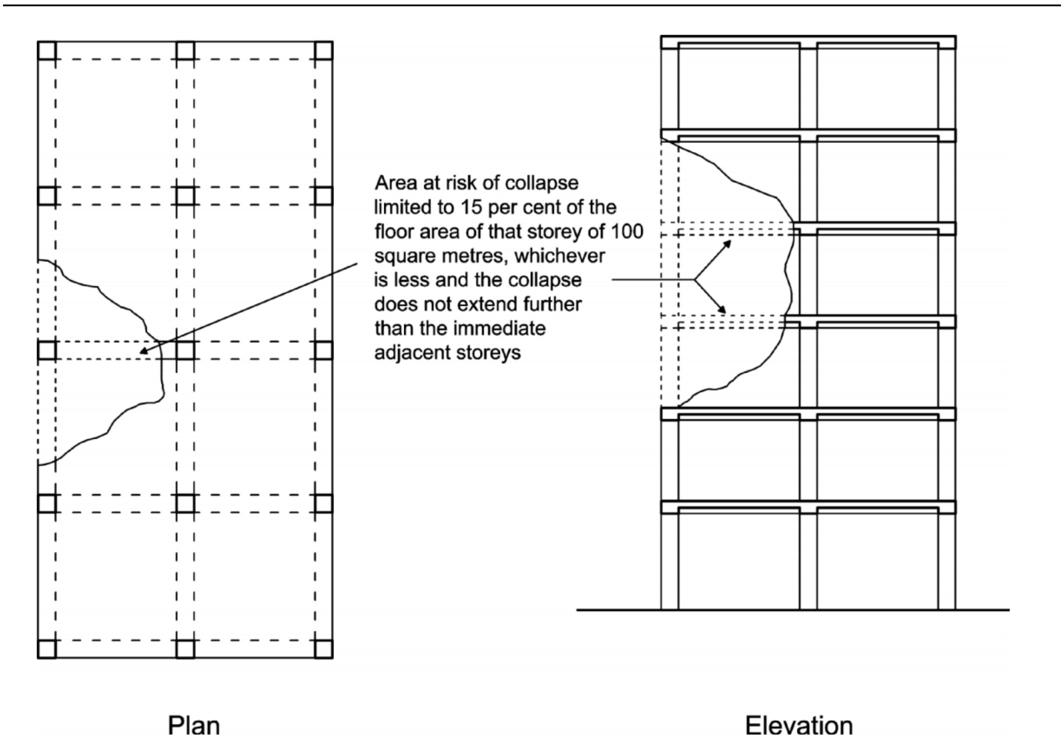


Figure 3.5: Recommended limit for resulting damage

ALPA, e.g. which procedures to use, which loads and dynamic load factors (DLFs) to assume, or how the connections should be treated. Lastly, for class 3, the code demands a systematic risk assessment. [11].

To check the design in an ALPA, U.S. DoD (2016) implements strength reduction and load increase factors to compare strength supply to strength demand for all structural elements. In contrast to Eurocode, the application of linear static, nonlinear static and nonlinear dynamic calculation procedures for an ALPA are described for different materials, with additional guidance on software implementation [11].

3.4.1. New working draft Eurocode 1995

The new draft of the Eurocode 5 [29] prescribes that design for resistance by removal of load-carrying elements should consider two consecutive design scenarios:

- failure of a load-carrying element, including dynamic effects and impact of falling debris
- remaining structure, without the failed element.

The performance of a structure under an element-removal scenario should be verified either by a dynamic analysis or a quasi-static analysis in which dynamic effects are represented by quasi-static loading using appropriate dynamic amplification factors.

Table 3.1: Load-duration classes, combinations of actions, and dynamic amplification factors when designing for removal of load-carrying elements

Design scenario	Load duration class	Combination of actions	DAF γ_{dyn}
Failure of load-carrying element	Instantaneous	Accidental design situation	γ_{dyn} = 2.0
Structure without removed element	Short-term	Accidental design situation	N/A

- The dynamic amplification factor should only be applied to the effects of actions in the structural components that comprise the alternative load path, not to the entire structure.
- The dynamic amplification factor γ_{dyn} only takes into account the sudden reduction of the internal forces transferred through the removed element or elements, it does not take into account the impact of debris.
- Some element-removal scenarios might not impose dynamic effects on the structure (e.g. gradual loss of an element during a fire, excessive settlement of a foundation). In such cases, only the second design scenario is relevant.
- The selection of an element or elements to be removed or the acceptable extent of failure should be specified by the relevant authority or agreed upon for a specific project by the relevant parties.

3.5. Failure Cases

Failures in the bracing system, especially during construction, are a main reason for the collapse of posts and beams (Frühwald Hansson 2011), and therefore their effects on stability need to be considered by notional element removal[21]. The institution of Structural Engineers (ISE) together with Arup recommends that for a design scenario with columns if several columns are located within a diameter of nominal length, they should be removed simultaneously [11] [12]. Additionally, the Unified Facilities Criteria (UFC 4-023-3) and the General Service Administration (GSA 2013) provide detailed and prescriptive guidelines, including removal scenarios for studying alternative load paths to limit the extent of damage.

3.6. Conclusion

Robustness is an intrinsic property of a structure alone which enhances the global survival tolerance of a building to local failure. The general philosophy of a robust building is that it's capable of preventing the disproportionate spreading of failure, independent of the failure cause. To prevent disproportionate collapse, a structure should be able to develop so-called alternative load paths, yet other methods exist too, though they are not always applicable. Robustness-related aspects should be taken into account in the early stages of design, with designing for ductility, redundancy etc. Designing for robustness, therefore, is related to the structural concept, choice of structural materials, and structural detailing. Building codes demand buildings to be designed for robustness based on their consequence class. A higher class means the more severe the risk for loss of lives. Therefore, do the higher consequence classes demand more robustness aspects. Eventually, most international codes require for the higher classes to prove a certain level of robustness by performing a deterministic analysis. With such an analysis, the exact behaviour of a structure after a damage has occurred can be studied. Since the failure cause is unimportant, a scenario-independent approach should be used. In conclusion, should notional element removal be done, it provides the best method to quantify robustness since it can answer the question of whether alternative load paths exist and whether a building is robust.

4 Modular Construction

4.1. Introduction

This chapter entails a review of modular buildings with an emphasis on structural aspects and especially robustness behaviour. Several studies into volumetric modular structures are presented with related relevant recommendations and conclusions. Together with the previous chapter about Robustness, the following literature research questions are answered.

- *What are existing design methods to ensure alternative load paths in a (modular) building?*
- *What kind of alternative load paths can occur in a (modular) building in case of notional element removal?*
- *What are promising design methods to develop alternative load paths in (modular/timber) buildings to increase their structural robustness?*
- *What are promising design methods to develop alternative load paths in modular timber buildings to increase their structural robustness?*

4.1.1. Background

Modular construction in broad terms involves a way of construction where buildings are assembled on-site with prefabricated elements. In its simplest form, single 1D elements are used, such as beams and columns which are connected with standard connections and interfaces[2]. Next, there are 2D elements, like panels and slabs which are common for walls and floors. Finally, in its most complex form, there are 3D volumetric units, usually fully fitted out-out offsite of which is known to be the most efficient class, since it allows up to 95 % of a building to be prefabricated [31]. Additionally, there also exits combinations of the above, also called hybrid systems. However, most authors speak of modular construction when only 3D room-sized volumetric units are used, also known as "building blocks, which is also the case in this thesis. [17] [22] [31].

In recent years, especially timber volumetric units are getting more popular amongst start-up companies and developers in the construction industry of Europe and especially the Netherlands due to the many sustainable and economical advantages they possess compared to traditional on-site construction. The modules particularly find their application in cellular-type buildings, such as educational buildings, hotels, student residences, social housing and more recently also medium-rise residential buildings [17] [22]. High-rise buildings are also possible to construct with modules but widespread adaption still requires some challenges to be solved. Notably, referring to components and connections, are modular systems very complex, which makes structural modelling and analysis complicated in comparison to traditional (non-modular) buildings.

4.1.2. Benefits

According to Lawson et al. [17], does modular construction offer both sustainable and economical benefits. Arguments for economical benefits are faster and safer manufacturing and improved quality control. Related to sustainability does modular construction reduce the environmental impact due to material efficiency and limited waste [9]. Furthermore, does the speed of installation on site, reduce the disruption to the locality during the construction process. In addition, modular buildings can potentially be dismantled, refurbished and reused, consequently beneficial for a circular economy.

4.1.3. Downsides

An inherent disadvantage of a 3D volumetric approach is the size limitation. Commonly the maximum width for road transport that does not require a police escort is around 3.5 meters. This either increases the cost of transporting larger units or limits the size of modules [2]. Next, there is the additional height of a modular building due to a double-structural floor. The modules are stacked on top of each other, having both a ceiling and floor slab. In areas where there are restrictions to the building height, developers need to maximize the number of storeys they can fit in order to compete with traditional construction, the increased storey height then is not beneficial. Additionally, there is the weight limitation, hoisting the modules on-site requires a mobile crane or a tower crane. Depending on the project this puts limitations on the maximum length of a module, to have a feasible weight [31].

4.2. History

Historically modular construction started in markets with a strong demand for housing, and low availability of labour. The method especially experienced a post-war boom in the United States and the UK due to a rapid need for social housing and fast construction methods. However, the tragedy of the Ronan Point apartment tower reverted interest due to concerns about the safety of prefabricated buildings. Additionally, social housing blocks experienced negative social reputations [2]. Nowadays prefabricated construction methods have become familiar and the trend of increased housing shortages around the world provokes a new interest. Though modular construction previously was mostly applied on low-rise buildings, particularly in the UK, Singapore, China and Australia, recently also high-rise buildings have been completed. There are still a lot of structural challenges for modular construction in high-rise, but leaping the gap can maximize the benefits of the modular construction method [31].

(a) Jakarta Hotel - Ursem

(b) The Natural Pavilion - Noordereng Groep

(c) Sara Kulture Centre - Derome

(d) Woodie - Kaufmann Bausysteme

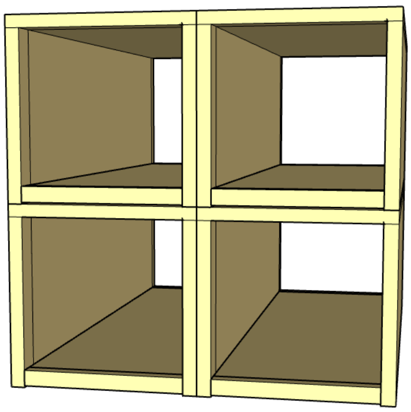
Figure 4.1: Overview of recent buildings with timber 3D volumetric modules and their manufacturers

4.2.1. Reference projects of timber modules

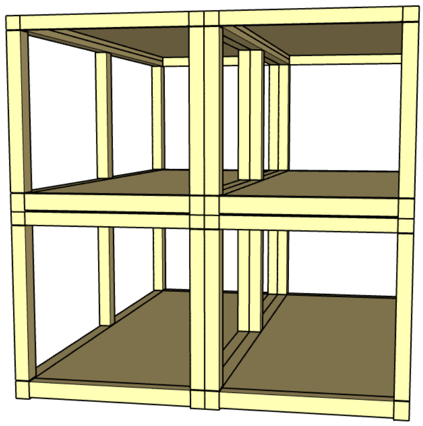
Usually, the volumetric modules are made of steel and timber, they are lightweight and have logistical advantages. The other option is concrete, they are heavy but have good performance on acoustics and fire safety. Steel modules are having an overhand and can be used for high-rise buildings as they offer significant advantages such as design flexibility due to their open frame configuration and high strength-to-weight ratio. Nevertheless, especially due to the developments around climate change, have timber modules experienced a growth in application in Europe since they can massively reduce a building's carbon footprint. They can be stacked up to create a 75-meter tall building as is done with the Sara Kulture Centre in Skellefteå, Sweden, see Figure 4.1. An example in the Netherlands is the Jakarta Hotel in Amsterdam and in Berlin, Germany you can find Woodie. All three buildings have a thing in common, they are constructed with CLT modules which are closed of around the sides, limiting their use to a single room. In contrast, there is also an open frame module possible, as is used for the Natural Pavilion in Almere, the Netherlands. More about their difference in the sections below.

4.3. Structural system

The primary load-bearing component of a modular building is the module itself. Generally, two structural forms for a module are considered, namely *load-bearing modules* and *corner-supported modules*. They differ in the way of load transfer and their application range for building designs. When stacked to create a building there are three main approaches to ensure a stable structure, this is done by either a core, a podium or a bracing truss. A podium is only feasible for low-rise buildings. On the contrary, when the building becomes higher and lateral loads due to wind are more governing, only a core or bracing truss are applicable. Such a bracing truss is sometimes referred to as a lateral infill system. It is usually made of steel and can be produced traditionally or in a modular way. In modern practice, modular high-rise buildings are usually constructed with a concrete core because unlike a bracing truss it can accommodate lifts, stairs and service risers [31].



(a) load-bearing



(b) corner-supported

Figure 4.2: 2 different module forms

4.3.1. Modular system

Load bearing module

Load bearing modules have stiff wall panels on their sides through which loads are transferred. The side walls should align vertically through the building and can have openings in some cases, depending on the loading situation. The axial load is transferred via direct wall-to-wall bearing. Diaphragm action in those walls can make the modules stiff in itself but can also provide lateral stiffness for the building in its whole up to a certain story height.

Corner-supported modules

In modules of this system, the walls are non-load bearing. Instead forces are transferred via edge beams towards the corner posts. The corner posts align vertically throughout the whole building and in some cases, especially for longer modules intermediate posts are present to prevent edge beams from becoming to deep. In some cases the modules have moment resisting connections to make them stiff in their self, in other cases the modules have bracings along their edges. The topology of this system provides more flexibility in composition of floor spaces, but for a multiple story building separated stabilizing elements are needed such as vertical bracings or a core [16] [31].

4.3.2. Connectors

Connections of modular systems can typically be categorised in three different categories: intra-module, inter-module, and foundation-connections. Intra-module connections assemble the module itself, inter-module-connections connect the modules to each other and foundation-connections connect usually to a foundation or podium structure in some cases.

Inter-module connection systems

Connections in modular buildings are very important, especially the inter-modular ones because they play an import role in ensuring structural integrity, overall stability and robustness of the entire building [31]. Timber modular systems are very novel and unfortunately not so well defined, because of that examples of inter-modular connections can not be found in literature. In contrast to timber modules, multiple studies regarding concrete and particularly steel modular buildings have been carried out. For volumetric steel modules generally three different inter-modular connection types are classified: tie rods, connectors and bolts. [32].

An example of a commonly used connection in a steel modular building can be seen in Figure 4.4. The gusset plate ensures the formation of a horizontal tie. In a normal load case it generally transfers horizontal forces to the stability system. The vertical tie is here made by a tie rod, but a stud or bolt are also common. Next, a shear key can be observed, other than providing shear resistance, shear keys helps in aligning the column of the upper module as well as the adjacent modules. A base plate, which is welded at the bottom of column from a upper module, helps in the alignment during assembly by fitting with the shear key. In addition does the tie plate help in positioning the adjacent modules by fitting with the shear keys [18].

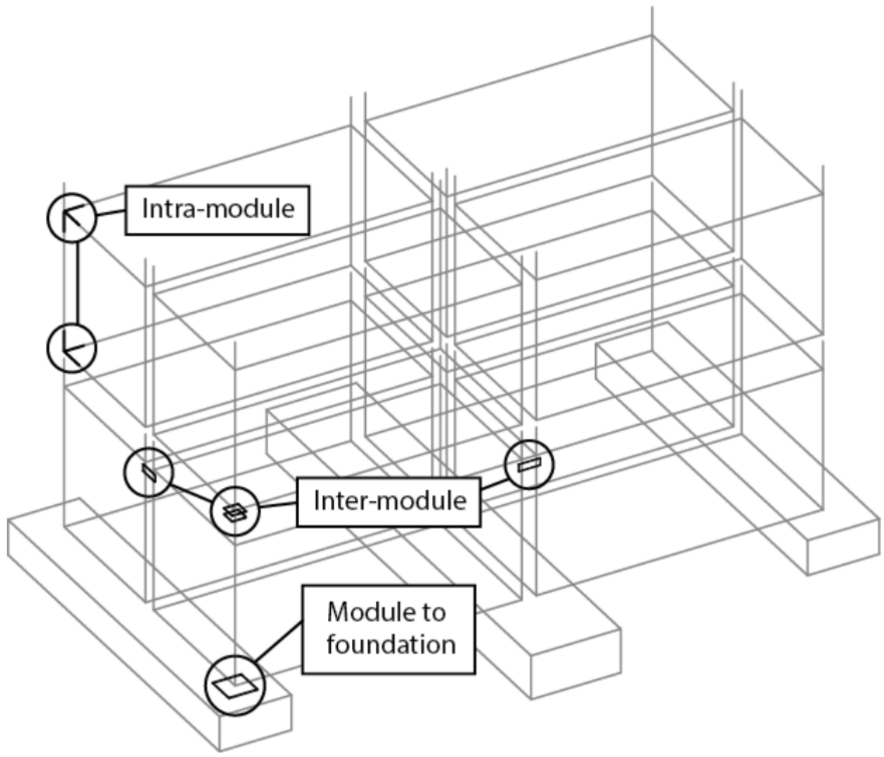


Figure 4.3: 3 connection categories in modular construction [16].

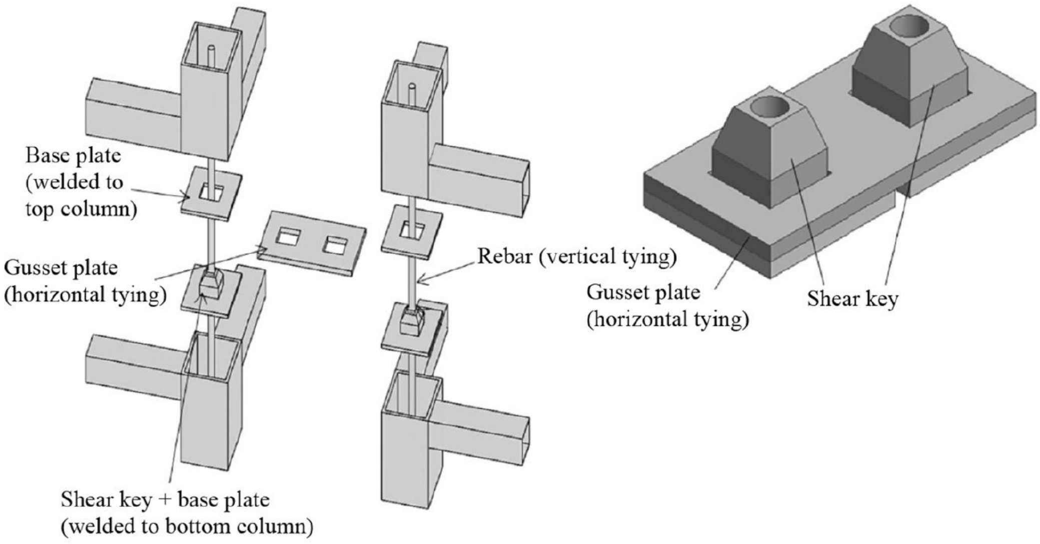


Figure 4.4: Common tie rod connection with gusset plate [31]

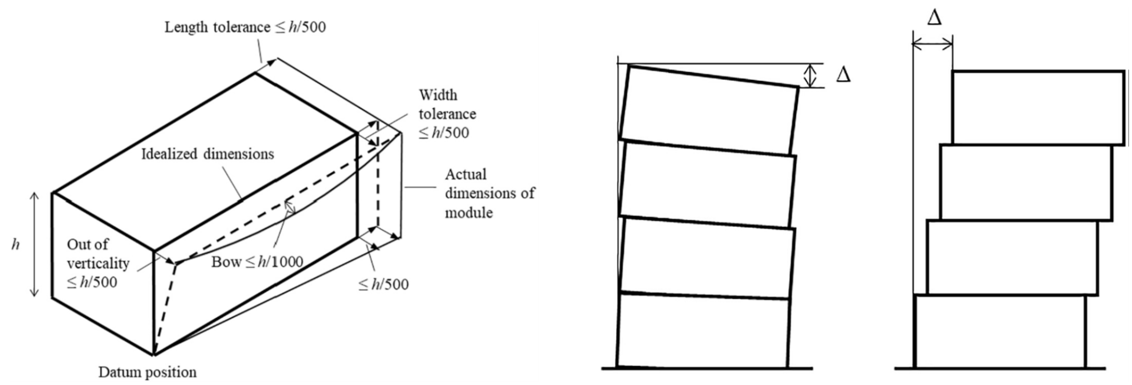
4.4. Structural Action and Behaviour

4.4.1. Stability

The overall performance of a modular building is different compared to a conventional one. Volumetric modules are in themselves usually stable and robust, because they need to be resistant to damage during transport and installation. During transport, they specifically need to withstand racking actions, which structurally can be more demanding. The modules are generally tied horizontally and vertically at their corners so that they can structurally act together to accommodate wind forces and provide alternative load paths in events of damage. Unlike traditional buildings with continuous floor-slabs are the floors in a modular building discrete. Because of that, load transfer between adjacent modules mainly occurs through the corner connectors. [22].

4.4.2. Tolerances

Important to consider is that eccentricities in manufacturing and installation of the modules can cause a build-up of additional moments and accentuate second-order effects. For modular buildings therefore tolerances are of utmost importance. Geometric and positioning tolerance can result in additional eccentric loads on the structural components in modules. Accurate positioning and installation of modules is therefore crucial to minimize the accumulative error amongst the modules as shown in Figure 4.5. Lawson et al. developed design guidelines to cope with the tolerances on the basis of the British and European codes [16].



(a) single module **(b)** stack of modules

Figure 4.5: Construction tolerances of modular buildings [31].

4.5. Structural analysis

Similar to conventional buildings, can structural analysis for modular buildings be performed by using; linear static (the most simple method ignoring both geometric and physical nonlinearities, (ii) nonlinear static (elastic or inelastic), and (iii) nonlinear dynamic (time-history analysis) as sometimes used for earthquake design [31]. As mentioned above are structural components of connected modules discontinuous and are the inter-modular connections of utmost importance. Therefore, to accurately capture the global behaviour of the entire building, the behaviour of the connections needs to be included. Various simplified techniques for modelling connections can be found in literature. A common method is to translate a connection into springs, an example can be seen in Figure 4.6, it can be utilised along with beam models available structural analysis software such as ETABS, SAP2000, RUAUMOKO, SeismoStruct, OpenSees, ABAQUS, LS-DYNA, ANSYS, etc [31].

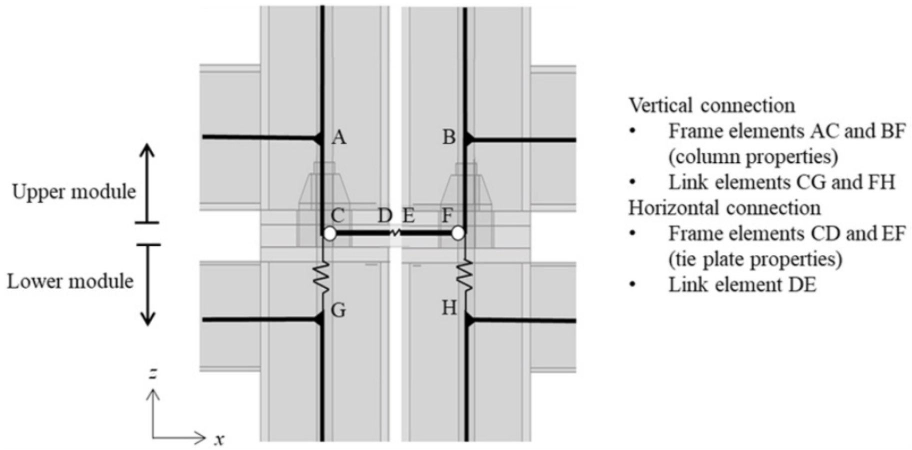


Figure 4.6: Translational spring model

With regards to the analysis of modular structures do Thai et al. [31] recommend taking into account the lateral stiffness of the floor diaphragms in each module, to accurately predict the global sway behaviour of the entire building

4.6. Robustness of modular buildings

Following the partial collapse of the Ronan Point building in 1968 comprehensive studies into the progressive collapse and robustness of conventional buildings were carried out. However, research on modular buildings is lacking. In recent years studies by Chua, Thai and Luo et al. did good efforts to improve that knowledge, and their work is noteworthy.

4.6.1. Vulnerability

Regarding damage to certain module, do different authors usually speak of the loss of a corner post or side of a module rather than the whole module [9]. Various studies reported that the removal of a corner module will cause the modules above to act as cantilevers [16],[5],[32]. On the contrary does the removal of an internal module cause the modules above it to span over it and form a bridge, see Figure 4.7. In those studies, it was reported that the worst-case scenario is the damage to or loss of a corner module. The ability of an assembly of modules to resist applied loads in the event of serious damage to a certain module is dependent on the development of tie forces at the corners of the modules, which need to be transferred through inter-module connections [17].

4.6.2. Studies

Recently three extensive studies have been carried out regarding the structural robustness of modular steel buildings. This chapter tries to summarise the researcher's findings and gives recommendations for the robust design of modular steel buildings. Since the structural concept of these buildings is similar to the timber one of this study, the recommendations are assumed to be applicable.

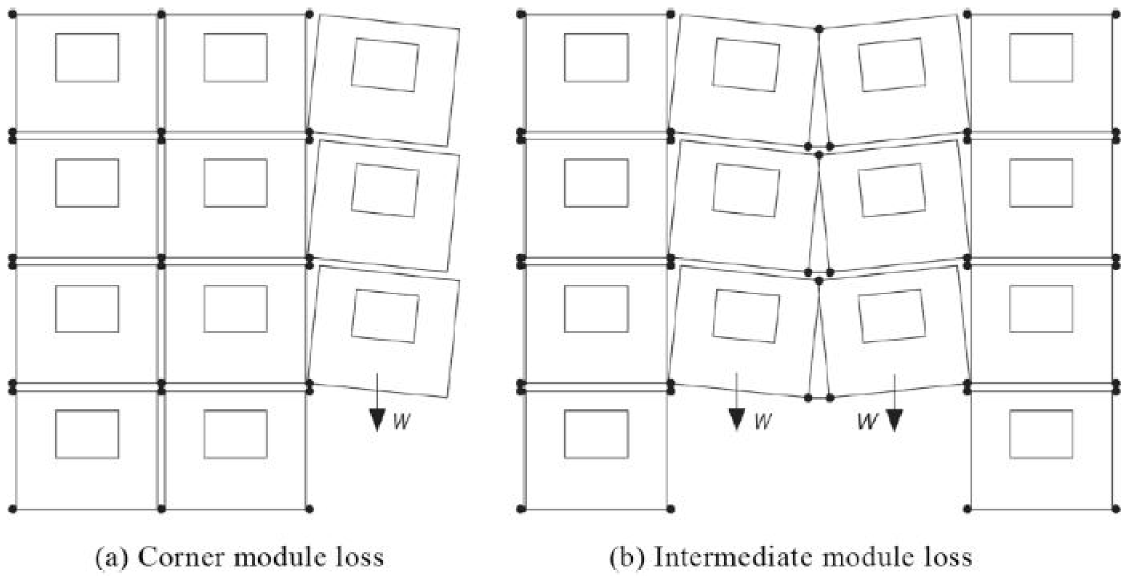


Figure 4.7: Module loss scenario [16].

Chua et al. In a study by Chua [5], it was discovered that alternative load path development under column removal scenario of a steel modular building highly depends on the horizontal tie plates which connect the modules at floor level. In addition they found that the axial forces in those tie plates seem to be higher than the ones that develop in beams of conventional buildings.

Regarding the resisting mechanisms, it was found that in the case of cantilever action, the tie plates are subjected to compression and that in case of intermediate column loss, the plates would be subjected to tension. Speaking of catenary action, Chua et al. concluded that the floor slabs help to some extent, reduce the deformations of the floor beam and, thus the development of a catenary. Furthermore, they say that membrane action is not able to develop due to the discontinuity of the floor diaphragm between the modules. The ability of floor slabs to provide progressive collapse resistance of modular buildings is therefore also limited [5].

In the same study, it was discovered that shorter beams in the modules play a major role in redistributing the load in a column removal scenario, the reason for that is, according to the authors the slightly higher stiffness of the shorter beams in comparison to the longer one.

In addition, was the sequence of plastic hinge formation studied and it was found that plastic hinges develop in the short ceiling and floor beams due to the higher bending moments. However, vertical displacement and hinge formation were observed to be rather localized to the damaged area, which suggests that catenary effects within the beams were not fully developed to redistribute the loads to adjacent modules.

Luo et al. In a study by Luo et al. it was discovered that modular buildings are most vulnerable for progressive collapse in case of the removal of corner modules or corner posts. In the study, corner posts were prone to buckling, and therefor it was recommended to ensure an adequate size of the corner posts, thus to increasing member capacity. The researcher furthermore performed analyses with pinned or rigid connections and based on the performance in nonlinear dynamic analyses, they concluded to enhance the collapse resistance of modular building by increasing the strength and rotational stiffness of inter-module connections, contributing restraint for building motion and providing improvement of global stability. Reducing the tensile force, which need to be transferred through the inter-module connection was also opted for by the researchers, and that can be done by adding more redundant load paths [22].

Thai et al. In the study by Thai et al. it was found that the progressive collapse of a modular building is more vulnerable to the removals of corner columns/modules than intermediate ones because of the different resisting mechanisms. In the scenario of intermediate column or modules loss, the load is shared to two adjacent modules through the development of catenary ‘bridging’ action. Whereas, in structures where a corner element is removed, loads can only be transferred to one adjacent module via ‘cantilever’ action. Therefore, regarding the structural robustness design of modular buildings, the authors recommend giving the corner modules and their corner posts special attention as they are the most vulnerable members [32].

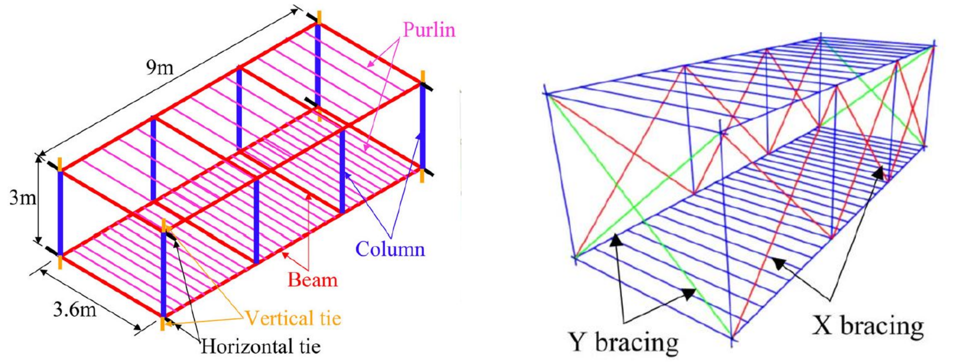
4.6.3. Applicability for timber modules

In the above described studies with steel modules, the inter- and intra-modular connections were modelled either as rigid or pinned, and most recommendations were based on the rigid systems. With steel modules it might be possible to design a rigid connection but with timber, this is hardly realisable. The applicability of the conclusions and recommendations of the studies is therefore questionable. To perform a proper and realistic analysis with a timber module, the connections need to be modelled correctly. Voulpiotis [35] showed elaborately how to do that with spring elements in different directions and rotations. Derivations for spring values can be done analytically with the Eurocodes, German code, Swiss code or research papers, however, the codes are very conservative. Nonetheless, experiments on modular timber connections are hardly done, so there is a big lag in information.

4.7. Robustness design methods

4.7.1. Common methods

Methods which are commonly used for the robust design of modular buildings are the tie force method and the ALP method, see 3.2. According to Lawson et al. [16], is the ALP method the most appropriate approach to asses the structural robustness of modular buildings, since it explicitly considers a damaged scenario.



(a) ties **(b)** bracings

Figure 4.8: Solutions to improve robustness [32].

4.7.2. Increasing robustness by design

Thai et al. carried out a parametric study in which they tested multiple different scenarios and their effect on the progressive collapse and structural robustness of modular buildings. The scenarios they considered were: column cross-section, inter-module connection, bracings system and configuration. It was discovered that the response of a modular building in non-linear time history analysis is very sensitive to the horizontal and vertical stiffnesses of the inter-module connection. Increasing the vertical stiffness of a module from 4 ties to 8 ties decreased the maximum displacement in a modular building significantly by 36.3% similarly the increase of horizontal stiffness led to significant decreases in displacements.

Related to the bracing systems were four different cases considered, including the base case (no bracing), bracing in the X-direction, bracing in the Y-direction, and bracing in both X- and Y-directions (i.e. exterior) as shown in Figure 4.8. It was observed, that especially the Y bracing in this case study, enhanced the structural robustness of modular high-rise buildings. Furthermore, note should be made that all models with bracing resisted progressive collapse without buckling of the corner posts. Specifically, the reduction of displacement due to the Y-direction bracing system being 88% less compared with the base case (no bracing).

In addition, it was found that increasing the amount of modules per floor (i.e more modules in the horizontal direction) has a positive influence on the robustness performance of the building. According to Luo et al., this is due to the higher lateral stiffness of such an assembly, which is beneficial to reduce the potential of excessive building motion that may cause instability and the resulting collapse [22].

Discussion Related to scenarios is according to Thai et al. [32] the probability of column loss is higher than the loss of an entire module. They refer to Hough and Lawson [9]. But actually, Hough and Lawson only say that realistically a loss of support would mean the loss or a corner post or side of a module, nothing about the probability of it.

4.8. Fire Safety

An inherent vulnerability of timber structures is their susceptibility to fire. Therefore, a possibility exists that a modular timber building suffers a module/compartment fire. The scenario that only one structural element is exposed is quite unlikely in a fire accidental case. Because in a volumetric module, travelling fires or full compartment fires are to be expected where significant combustible material is present. The collapse of the entire structure could be possible if the overall structure is not designed in a way to survive the failure or loss of a fire compartment.

However, there are substantial differences when comparing a fire-induced collapse and a damage-induced structural collapse. The largest difference in particular is that fire-induced collapse takes place over a longer timescale. Modelling of softening and heat-induced buckling would be necessary. Damage-induced structural collapse takes place over a fundamentally shorter time scale and the dynamic effects are much more present [21].

In contrast to what the definition of Eurocode 5 for robustness suggests, doesn't the code contain the possibility to consider a system effect due to a fire situation. Additionally, regarding fire safety are timber structures usually over-designed. The structural elements are larger than structurally necessary to account for 90 to 120 minutes of fire safety which for instance could be considered already as an approach to increase the robustness of a structural system, similar to key element design in which elements are made stronger [21].

4.9. Conclusion

- Modular buildings are most vulnerable for progressive collapse in case of the removal of corner modules or corner posts.
- Alternative load path development under column removal scenario of a steel modular building is highly dependent on the horizontal tie plates
- Floor slabs help to some extent, reduce the deformations of a floor beam and therefore reduce the development of a catenary mechanism.
- Membrane action is not able to develop in a modular building due to the discontinuity of the floor diaphragm between the modules.
- Beams on the short sides of modules play a major role in redistributing the load in a column removal scenario.
- Rigid inter-modular connections perform better in progressive collapse analysis.
- Strength and rotational stiffness of inter-modular connections should be increased to improve global stability and provide constraint for building motion.
- Corner modules and posts should be given more attention since they are the most vulnerable members.
- Structural solutions which can be considered to enhance the robustness of such vulnerable members are strengthening the column, use of bracing systems, and increasing the strength of the inter-module connections.

- Braces can create additional load paths, and the effectiveness of braces in structural robustness is highest when they are in the direction of building movement [32].

- The progressive collapse of modular buildings can be avoided by providing more inter-module connections. The connections between modular units of modular buildings play a significant role in developing alternative load paths during the initial load redistribution phase (more tie links between modules facilitate more load paths). However, the design of these ties must ensure the adequate transfer of load between modules such that undamaged parts fully receive the redistributed load.
- Multiple studies have shown that increasing the amount of modules per floor is an effective solution to enhance the structural robustness of a modular building, this would mean a modular building should become wider. This will ultimately increase the structural redundancy and in that case, a more effective distribution of the collapse load can be obtained [32].

5

Alternative Load Path Analysis

5.1. Introduction

In Chapter 3, it was introduced that a common way of practice in engineering is to use a deterministic analysis to quantify the robustness of a building. In this thesis, the decision is made to use a scenario-independent approach, which entails the performance of alternative load path analyses by notional element removal. The general philosophy of notional element removal entails that a structural element has failed instantaneously and that forces are to be distributed to other load paths. This chapter provides a review on how such an analysis can be performed and will introduce different methods and procedures found in the literature which can be used. That said, it provides an answer to the following two literature questions:

- *What are common procedures for alternative load path analysis of a (modular/timber) building?*
- *What kind of models provide reliable results for identifying and quantifying alternative load paths in a (modular/timber) building in case of notional element removal?*

5.2. Dynamic response

Alternative load path analysis is performed on the premise of notional element removal. When a structural element then is suddenly removed, a sudden loss of internal forces occurs, which results in a dynamic effect. The instantaneous additional load application to the remainder of the structure maximizes the amplification of load effects. To account for that effect in a static analysis, a dynamic amplification factor (DAF) is used.

Dynamic Amplification Factor (DAF)

A DAF, also known as dynamic load factor (DLF), dynamic magnification factor (DMF) or dynamic increase factor (DIF) is used to account for dynamic effects in a static analysis. The factor is applied to the gravity loads and converts a transient (dynamic) load into a static load which is equivalent in terms of the displacement of the structural system when statically applied [21]. DAFs, however do not take into account the impact of debris [29]. The DAF is conventionally defined as the ratio of the maximum dynamic displacement Δ_{dy} to the static displacement Δ_{st} for an elastic SDOF system under the same applied loading [4].

$$DAF = \frac{\Delta_{dy}}{\Delta_{st}} \tag{5.1}$$

However, for an inelastic SDOF system, the DAF may also be expressed with a force-based definition as the ratio of the static force P_{stat} to the dynamic force P_{dyn} under an equal displacement demand [34].

$$DAF = \frac{P_{stat}}{P_{dyn}} \tag{5.2}$$

Additionally, does another definition exist, which defines the DAF as the ratio between the maximum (peak) dynamic displacement Δ_{dy} to the final response in the new equilibrium state Δ_{fin} of the dynamic analysis [4].

$$DAF = \frac{\Delta_{dy}}{\Delta_{fin}} \tag{5.3}$$

In case of a single degree of freedom system (SDOF), assuming a structural response in the linear elastic range, the instantaneous loss of column corresponds to a DAF of 2, the upper bound. The factor reduces when the removal rate of the column is reduced, having a lower bound at 1.0 for quasi-static column loss, where inertia forces don't play a role. Commonly for timber structures, a conservative DAF value of 2.0 is applied. Yet does that place extremely high demands on a structure. Typically when structures possess sufficient ductility, and dissipation of energy due to plastic straining may occur, much lower values than 2.0 are observed [21]. Therefore, when conducting a nonlinear static procedure in which material and geometrical nonlinearity are taken into account (inelastic analysis), a lower DLF could be used [10]. However is also known that when structures develop stiffness in the post-elastic phase due to alternative load path mechanisms such as catenary action (see Chapter 3) the DAF will vary with ductility and will exhibit an increase [21]. Nevertheless, is available literature on the DLFs for a timber structure still limited, but values between 1.3 and 1.5 are proposed when enough ductility may be assumed [11]. That said, an appropriate value seems not to be established. In addition, is the estimation of an appropriate value difficult and subjective.

5.3. Analysis Procedures

As introduced in section 3.3, are generally four different procedures for ALPA considered: linear static, non-linear static, linear dynamic or non-linear dynamic. Most commonly in practice, a linear static procedure is used due to its simplicity. Only in some cases where safety is extra important, such as for the higher consequence classes, a dynamic analysis is used. Yet is known that that is highly complex. Whereas until recently, the Eurocode did not contain any guidance on the conduction of an ALPA, the new working draft of Eurocode 5 (EN 1995-1-1:20XX), now prescribes (informative) to perform one of those two analyses as a means for ALPA, see section 3.4 [29]. The procedures are presented in more detail in the subsections below, and the downsides and benefits are discussed.

5.3.1. Static Procedures

Linear static procedure with DLF

The most straightforward and simple approach is a linear static procedure with DAF. To include dynamic effects, dynamic load factors (DLFs) are used. The procedure is done under the assumption of a linear elastic material response with geometric linearity. Due to the geometric linearity, the method leaves it impossible to consider catenary and membrane action. Furthermore does the linear elastic material response limit the analysis to the elastic range, and if over-stressing of member is shown to occur, the analysis can become invalid [21].

Nonlinear static procedure with DLF

A nonlinear static procedure may take into account plastic material behaviour and large geometric changes (P-delta). We speak of elastic when only geometrical nonlinearity is included and of inelastic when also physical nonlinearity is incorporated. With inelastic analysis, the dissipation of energy in plastic strains can be considered, and because of that load distribution mechanisms which may enhance collapse resistance (see Section 3.1.3) can be taken into consideration [21].

Push-down method

A commonly used method for the static procedure is push-down analysis, similar to pushover analysis. In this case the load-displacement graph after element removal may be called a push-down curve [11]. Execution of this method is done by applying gravitational load in a predefined direction. Usually this is done with force control, where the gravity loads above the structure are monotonically increased. Additionally, this kind of analysis is often referred to as a *quasi-static analysis*, because load is applied with small increments. For examples, see the work of Ferraioli and Huber [8], [10].

Pull-down method
Another option is the pull-down method, a recently developed method by Liu et al [19]. Using the pull-down analysis, a single downward point force must be applied at the upper node of the removed column while the gravity loads within the directly affected bays are kept original instead of being amplified, an empirical DIF, however is necessary. Nevertheless, an energy-based analysis method also exists, see the work of Liu [20].

Nonlinear static pushover with simplified dynamic response
A procedure which is similar to the conventional nonlinear static analysis but which doesn't require a DLF exists. It was initially developed by Izzudin et al. [13] and it uses a simplified dynamic response to calculate the dynamic displacement. Izzudin et al. present the following framework:

1. nonlinear static response of the damaged structure under gravitational loading
2. simplified dynamic assessment to establish the maximum dynamic response under sudden column loss
3. ductility assessment of the connections

The dynamic response is obtained by the derivation of strain energy in the nonlinear static response of the system through pushover analysis and computation of external work through the statically applied load. The system will be in equilibrium if the external work performed by the gravitational load is equal to the internal strain energy, see figure 5.1. The level of suddenly applied gravity load P_{stat} that results in a specific maximum dynamic displacement is found in the energy balance of equation 5.4.

$$P_{dyn} = \frac{1}{u_{dyn}} * \int_0^{u_{dyn}} P(u)du \tag{5.4}$$

where $P(u)$ is the quasi-static push-down curve [13]. Accordingly, the found gravitational load leading to a maximum dynamic displacement is identical to the mean static resistance over the displacement range. As a follow-up, can the energy balance results be plotted on a separate graph in which the maximum dynamic displacement for the static load is presented. Izzudin terms that the *pseudo-static response*, see Figure 5.2. Consequently, the force-based DAF may be obtained by dividing the non-linear static response curve by its corresponding pseudo-static response curve up to the collapse resistance.

To finalize the procedure, is it necessary to compare the max dynamic displacement u_{dyn} following the applied gravity load P_0 (Pstat) to the ductility limit u_{lim}. The ductility limit can be established as the minimum displacement value u for which the deformation demand exceeds the supply in the connections [13]. The deformation u needs to be transformed in ductility demands in various components of a connection, both in the axial and rotational directions. Unfortunately, there is limited data available on the ductility limits of timber connections, especially of the ones used in the novel modular timber building systems.

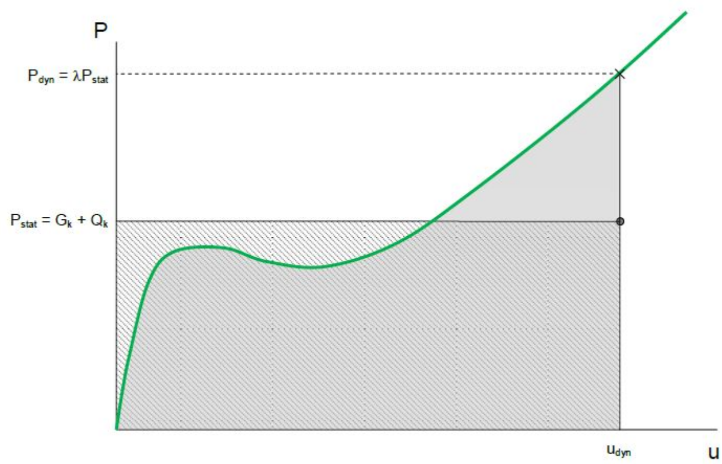


Figure 5.1: Push-down curve for dynamic response [21]

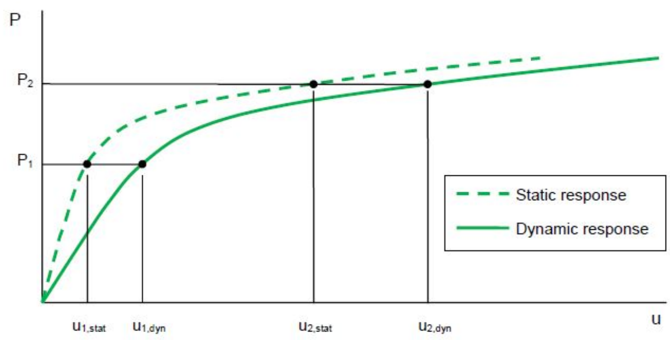


Figure 5.2: Pseudo static response [21]

5.3.2. Nonlinear Dynamic analysis procedure

The most accurate results for a progressive collapse analysis can be obtained by a nonlinear dynamic procedure. This procedure takes into account the structure's time-history response. It has the inherent advantage of taking into account the dynamic amplification and effects of internal forces without the need for a DAF. To get realistic results, accurate modelling of ductile and brittle failure is required. In contrast to other analysis procedures also, material strain rate effects and damping can be taken into account [10] [21].

Dynamic Analysis Method

A method to perform the dynamic procedure is to replace the to be removed element by its static equivalent reactions. By assigning the inverted reactions to the node where the element connects to the rest of the structure, the initial displacements of the loaded structure will be equivalent. To study the instantaneous loss of the element, the reactions need to be instantaneously reduced zero and the response of the system shall be studied over time. The speed at which those forces are reduced influences the dynamic amplification. Common removal times in literature are 0.01 or 0.02 seconds [24], [31].

5.4. Identifying Alternative Load Paths

Identification of ALP's is firstly possible by analysing the resulting force-displacement curves. It can be expected that the response starts with a linear phase followed by a phase of non-linearity. Next a plasticity phase will occur after which either hardening or softening occurs. With regards to an effective resistance mechanism, a larger force can be sustained with increasing displacements and therefore hardening in the response is ideally observed. Secondly, the deformed structure can be qualitatively observed, elements can be spotted that separate or make contact and together with the forces transferring through those locations, it is possible to identify a certain ALP.

5.5. Quantifying Robustness

Damage based

A relatively simple measure of robustness is obtained by comparing the extent of damage obtained in an analysis after assuming initial damage to a certain limit. Such limits are well-defined in building codes and are based on specific percentages or square meters of damaged floor area in the adjacent stories of the removed element. Commonly the recommended limit for resulting damage is the smaller of 100m2 or 15% of the floor area in each of two adjacent storeys [29].

Pseudo-static capacity

Another measure of robustness, initially proposed by Izzuddin et al. [13] yields a limit state which takes into account the relationship between the actual gravity load above the removed element and the pseudo-static capacity. This robustness measure embodies a ductility-based demand capacity ratio (DCR), and it quantifies the structural system's reserve ductility. The more reserves exist, the more robust is the structure.

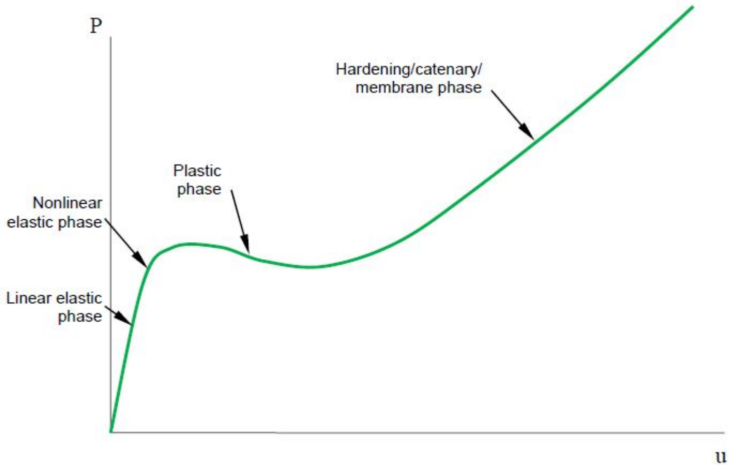


Figure 5.3: Phases of failure [21]

5.6. Finite element analysis implications

Solvers

For the FE models, both implicit (Abaqus/Standard) and explicit (Abaqus/Explicit) solvers can be used to do the analysis. Implicit solvers are excellent for smooth model responses, like static or (low-speed) nonlinear dynamic analyses without material failure (e.g., normal structural analysis). Explicit solvers however, are a suitable option when the response is non-smooth, like in dynamic analyses where the convergence of the implicit solver is difficult or impossible due to element- contacts and failures (e.g., in the collapse of a structure) [35].

Connections

To save computational costs can the mechanical behaviour of a complex connection in a model be replaced by nonlinear springs. The software package Abaqus has the ability to use custom connector elements for those springs. Such a connector element is a two-node 1D element that defines a constitutive behaviour between the degrees of freedom (DoF) of its nodes (node a and node b) (DS, 2014) [10]. The elastic, plastic, and failure properties of a custom connection can be applied in all degrees of freedom as wished. The connector elements can thus substitute screws, angle brackets or entire compartments of a building.

Connection properties can be estimated for all degrees of freedom by using the Eurocodes. A relatively standard approach is two use a bi-linear load-displacement curve as proposed by Voulpiotis [35], see Figure 5.5 with a backbone curve when necessary in compressive behaviour. For each degree of freedom, the curve can be characterised by its elastic stiffness (K_e) yield load (F_y) plastic deformation (δ_u) and ultimate load (F_u). For a more precise analysis also a formula could be used to model the spring if the exact behaviour is known from experimental testing.

5.7. Conclusion

For performing an alternative load path analysis, different analysis procedures are possible. Only some can study the true behaviour of a structure and others are very limited. To include the redistribution of forces and study alternative load path resisting mechanisms, a nonlinear procedure is necessary. That can be done with either a static procedure with dynamic load factors or a dynamic one. Many researchers use the nonlinear dynamic analysis to study the behaviour of a modular building in case of notional element removal. It is a complex and computationally expensive procedure but is most capable of capturing the true behaviour since it considers all relevant aspects and inherently includes dynamic effects. However, a less complex method, which also includes dynamic effects, and is beneficial for the use and identification of load distribution mechanisms with less computational effort required compared to the nonlinear dynamic analysis, is the nonlinear static pushover with simplified dynamic response [21], [11].

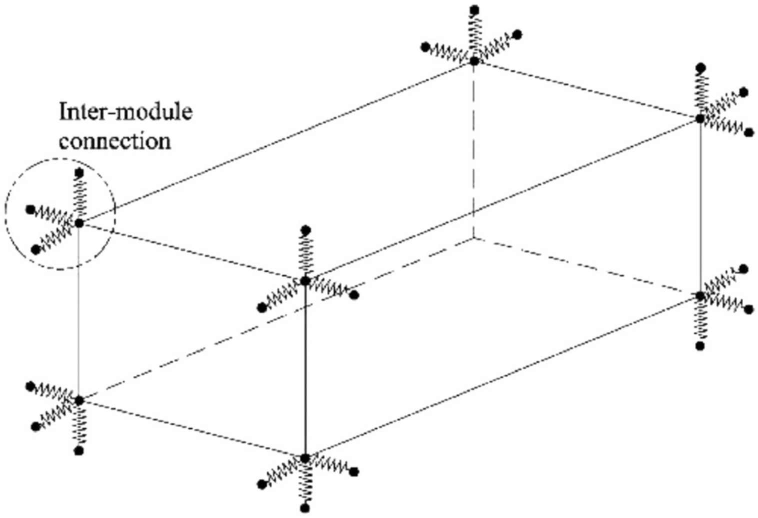


Figure 5.4: Inter-modular springs

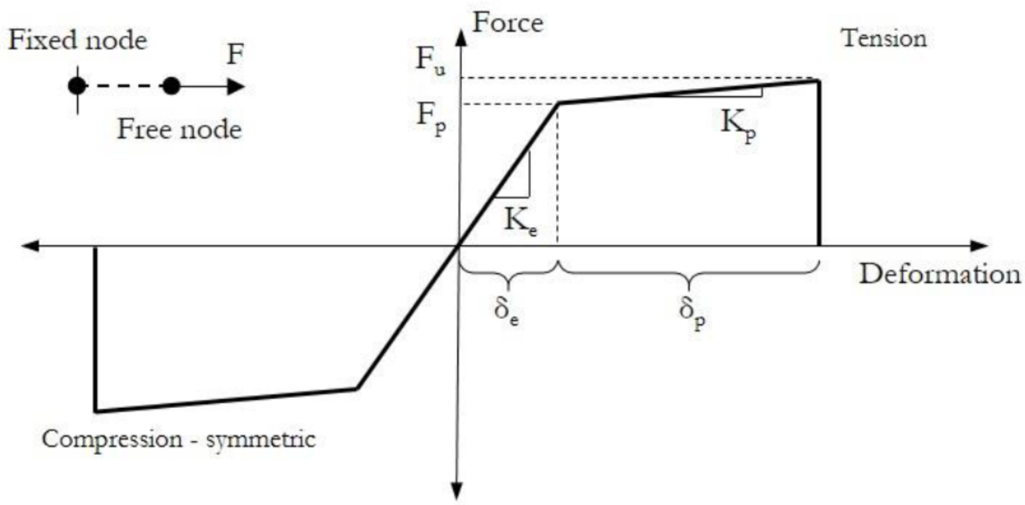


Figure 5.5: Idealised connector behaviour for axial degree of freedom [35].

6

Connector Behaviour

6.1. Introduction

This chapter provides an overview of the theory for timber/steel connector behaviour. Different theoretical models are presented which will be implemented in the models.

6.2. Definition of Load Slip behaviour fasteners

A proper determination of the load-slip curves of a timber connection is fundamental for the accurate modelling of the global behaviour. An example of an ideal load-slip curve is presented in Figure 6.1. In which Ke is the elastic stiffness, Fy the yield capacity and ΔFy the corresponding yield displacement, F_{max} the peak capacity and ΔF_{max} the respective displacement, Fu the ultimate capacity at the point of failure and ΔFu being the corresponding ultimate displacement. The plastic displacement Δpl = ΔFu - Δel

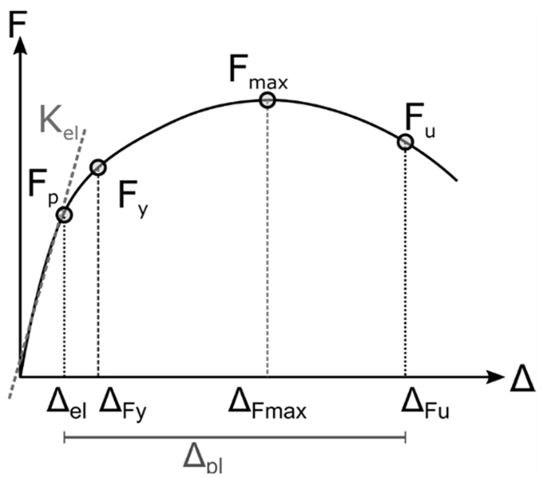


Figure 6.1: Example of ductile load-displacement curve with elastic and plastic displacements [26]

6.2.1. Lateral Resistance & slip

Using the European Yield Model (EYM) as prescribed by the Eurocode, both F_y (yield point) and F_{max} (peak resistance) can be predicted in the lateral direction of a connection. To compute F_y, the minimum of all EYM equations, it is necessary to compute the effective embedment strength and the fastener yield moment. For a connection with nails, dowels, screws, bolts and threaded rods, the EC5 prescribes to use of the following equation for the characteristic yield moment of the fastener:

$$M_{y.k} = 0.3 \cdot f_{u.k} \cdot d^{2.6} \tag{6.1}$$

The equation above is a regression equation and is established on the fact that the load-carrying capacity of a fastener F_u is reached before or at a slip of 15 mm, as EN 26891 defines. A recent study by Ottenhaus et al. [26] pointed out that the equation above, therefore, assumes that full plasticisation of fasteners is hardly achieved in timber connections, which is according to the authors, not the case, as by doing experiments on dowel type of fasteners they found larger yield moments. They recommend using the elastic section modulus to predict F_y and the plastic bending moment to predict F_{max}.

The ultimate load capacity F_u at point of failure is defined differently for static and cyclic loading. According to EN 26891, it is defined as the load reached before or at a slip of 15 mm. For cyclic loading on the other hand, usually, F_u is defined as 80% of the peak resistance (F_u = 0.8 F_{max}).

$$M_{el} = \pi \cdot f_{y.k} \cdot \frac{d^3}{32} \tag{6.2}$$

$$M_{pl} = f_{y.k} \cdot \frac{d^3}{6} \tag{6.3}$$

However, most manufacturers only determine f_u and not f_y since, in experimental testing, it can be very difficult to derive the yield strength of a fastener [26]. Nevertheless, the authors recommend using the formulas above to derive the capacities of the connection. If a value for f_y is not specified by the manufacturer, a conversion factor might be used. See equation 6.4 by Blass & Colling [3]. Additionally, the authors recommend predicting the yield capacity with an effective embedment strength of $f_{h,e}$ = 0.8 f_h.

$$f_{y,ef} = \begin{cases} \frac{0,9\cdot(f_y+f_u)}{2} & \text{for } f_u < 450\text{MPa} \\ 0,9 \cdot f_u & \text{for } f_u > 450\text{MPa} \end{cases} \tag{6.4}$$

Embedment strength

The embedment strength of the fastener based on its diameter is presented in Table 6.1. According to clause 11.12.6 of the new draft for EC5, may for laterally loaded bonded-in rods inserted perpendicular to the grain (ε = 90°), the embedment strength according to the table be increased by 25 %. For laterally loaded bonded-in rods inserted parallel to the grain (ε = 0°), the embedment strength should be taken as 10 % from the value for ε = 90°.

Table 6.1: Embedment strength $f_{h,k}$ for CLT and GLT

Fastener Type	Formula	Unit
Screws and rods with wood screw thread	$f_{\text{h},\varepsilon,\text{k}} = \frac{0{,}019\rho_{\text{k}}^{1{,}24}d^{-0{,}3}}{2{,}5\cos^2\varepsilon+\sin^2\varepsilon}$	N/mm²
Bolts and bonded in rods	$f_{\text{h},\alpha,\text{k}} = \frac{0{,}082(1-0{,}01d)\rho_k}{k_{\text{mat}}}$	N/mm²

Lateral slip

With the help of the lateral slip modules, displacement at yielding can be determined Fy / K_{sls} = ΔFy. The displacement at maximum resistance: ΔF_{max}, of the fastener can be determined according to the Johanssen theory. With Eq.6.6 the distance (b_1 + b_2) between the two plastic hinges can be determined. Then with the known angle α, which is the plastic bending angle of the fastener, the total slip can be estimated. Firstly for α the expression of EN 409 can be used to determine it, this expression is derived on the basis of experiments. Secondly alpha = 45°can be used, which corresponds to a full plastic development.

$$\alpha = \frac{110}{d_1} \tag{6.5}$$

$$b_1 = \sqrt{\frac{2\beta}{1+\beta}} \cdot \sqrt{\frac{2M}{f_{h,1}d}} \text{ and } b_2 = \frac{b1}{\beta} \tag{6.6}$$

$$\Delta F_{max} = (b_1 + b_2)tan(\alpha) \tag{6.7}$$

Table 6.2: Values for the lateral slip modulus, ksls.ax mean, per shear plane per fastener or shear connector

Fastener Type	K_{sls}	Unit
Dowels, Bolts, Screws	$\rho_{mean}^{1.5} d/23$	N/mm
Bonded-in rods perpendicular to grain (ϵ = 90)	$\rho_{mean}^{1.5} d/25$	N/mm
Bonded-in rods parallel to grain (ϵ = 0)	$\rho_{mean}^{1.5} d/125$	N/mm

6.2.2. Axial resistance & slip

Generally, the axial resistance of a timber connection depends on the metal fastener's withdrawal resistance or tensile resistance and is taken as the minimum of the two. It is known that in the axial direction, a timber connection usually exhibits a brittle failure mode since the metal fastener will either be pulled out of the timber or break due to the tensile force.

Specifically for GIR (glued in rods) in GLT, the failure modes observed under uniaxial pull-out are mainly characterised by (a) rod/adhesive interfacial bond failure and (b) adhesive/timber interfacial bond failure. Such failure modes can be regarded as brittle failures due to the failure along an interface [25].

Table 6.3: Values for the withdrawal resistance, F_{wk} for screws and bonded-in rods in timber connections

Fastener Type	$F_{w,k}$	Unit
Threaded part of screws	$\pi d l_w f_{w,k}$	N
Bonded-in rods perpendicular to grain	$\pi d l_{b,eff} f_{vr,k}$	N

Table 6.4: Values for the axial slip modulus, KSLS,ax,mean for screws and bonded-in rods in timber connections

Fastener type	$K_{sls.ax}$	Unit
Threaded part of screws Bonded-in rods perpendicular to grain	$2d^{0.6} {l_w}^{0.6} \rho_{mean}^{0.9}$	N

Non-glued or screwed fasteners

The tensile resistance of a threaded rod is determined by:

$$F_{t,d} = \text{min}\left(\frac{1}{\gamma_{M0}} \cdot A_s \cdot f_{y.k}, \frac{1}{\gamma_{M2}} \cdot 0.9 \cdot A_s \cdot f_{u.k}\right) \tag{6.8}$$

where: As is the effective tensile stress area of the rod, $f_{y.k}$ the yield strength, $f_{u.k}$ the ultimate strength of the rod.

The stiffness of bolts k_{bt} and non glued rods in tension k_{rt}:

$$k_{\text{bt/rt}} = \frac{E_{\text{s}} A_{\text{s}}}{l_{\text{s}}} \tag{6.9}$$

where: E_{s} is the young's modulus of the steel, A_{s} the effective tensile stress area of the rod/bolt, and l_{s} the length of the rod/bolt.

6.2.3. Moment resistance & rotation

In the situation of bending moments, the figure and definition, as presented in Figure 6.1, are still applicable. However, should in this case, the component on the x-axis be replaced by the rotation θ, and the vertical y-axis contains bending moment M. In order to define a $M-\theta$ curve (moment-rotation), the following three structural properties should be determined: (a) moment resistance $M_{j,Rd}$; (b) initial rotational stiffness $S_{j,ini}$ and (c) rotation capacity (maximum rotation at failure) θ_u. To determine properties (a) & (b), the component method is used. This method which is more commonly used in steel structures and can be found in EN 1993-1-8. The method replaces each component of a connection by an individual extensional spring, which is independent and can be connected to others [36]. With characterisation and identification of each constitutive component, the overall behaviour of the connection by means of assembly can be modelled.

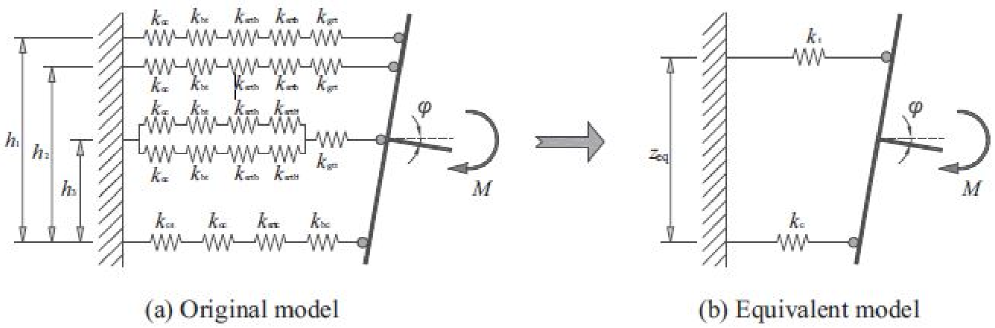


Figure 6.2: Mechanical model beam-column connection [36]

The initial rotational stiffness $S_{j,ini}$ will be determined according to EN 1993-1-8 as shown in the equation 6.10, where z_{eq} (m): the equivalent lever arm as shown in Figure 6.2; k_t (kN/m): the equivalent stiffness in tension zone as provided in Eq. 6.11 and shown in Figure 6.2; k_c (kN/m): the equivalent stiffness in compression/shear zone.

$$S_{j,ini} = \frac{{z_{eq}}^2}{\frac{1}{k_t} + \frac{1}{k_c}} \tag{6.10}$$

The equivalent stiffness in the tension zone k_t is determined according to Eq.6.11, where $k_{eff,r}$ (kN/m): the effective stiffness for component-row r taking into account the initial elastic stiffness k_i for the basic joint components and h_r is the distance between component-row r and the centre of compression. The effective stiffness $k_{eff,r}$ for component-row r should be determined from Eq. 6.12 where $k_{i,r}$ (kN/m): the initial elastic stiffness representing component i relative to it's row r.

$$k_{\mathrm{t}} = \frac{\sum_{\mathrm{r}} k_{\mathrm{eff,r}} h_{\mathrm{r}}}{z_{\mathrm{eq}}} \tag{6.11}$$

$$k_{\mathrm{eff,r}} = \frac{1}{\sum_i \frac{1}{k_{\mathrm{i,r}}}} \tag{6.12}$$

In case there is only one component row active in tension equations 6.11 & 6.12 can be simplified into Eq. 6.13.

$$k_{\mathrm{t}} = \frac{1}{\sum_i \frac{1}{k_{\mathrm{i,t}}}} \tag{6.13}$$

The equivalent stiffness in compression/shear zone k_c (kN/m) is determined by:

$$k_{\mathrm{c}} = \frac{1}{\sum_i \frac{1}{k_{\mathrm{i,c}}}} \tag{6.14}$$

The equivalent lever arm z_{eq} should be determined from the equation below:

$$z_{\text{eq}} = \frac{\sum_{\text{r}} k_{\text{eff,r}} h_{\text{r}}^2}{\sum_{\text{r}} k_{\text{eff,r}} h_{\text{r}}} \tag{6.15}$$

The moment resistance of the joint/connection is determined by Eq.6.16 where $F_{tr,Rd}$ (kN): the effective tension resistance of component-row r of the joint, which should be taken as the smallest value of the design tension resistance from the weakest component relative to it's row r, given that the compression resistance is larger then the weakest tensile resistance [36].

$$M_{\text{j,Rd}} = \sum_{\text{r}} F_{\text{tr,Rd}} h_{\text{r}} \tag{6.16}$$

With the above-mentioned formulas, the first two properties of an $M - \theta$ curve can be determined. Additionally, it is necessary to determine (c) the rotation capacity till failure θ_u. In a review on experimental research into ductility and moment resistance of glued-in rod timber connections by Reçoubas et al. [27], it was found that for beam-column connections with glued-in rods, a clear plateau occurs at yielding, after which maximum rotations can be reached of values close to 0.15 rad (monotonic) and 0.10 rad (cyclic) tests. Additionally, from the experimental data, it was observed that moment resistance at its ultimate point doesn't significantly differ from its value at yielding. Therefore, are in this research those values taken as input for the definition of the joint moment-rotation behaviour.

6.2.4. Ductility

Ductility ratio

To define the position of the peak force and ultimate force, in the load slip curve, it is necessary to determine the peak capacity displacement ΔF_{max} and the ultimate capacity displacement at the point of failure ΔFu. One possibility to do that is to use a ductility ratio since a ductility ratio generally gives a relation between the ultimate deformation and the deformation at yielding. The Swiss code for timber structures, SIA 265, gives the following definition, which is also cited by the European standard EN 12512:

$$\mu = \frac{\Delta F_u}{\Delta F_y} \tag{6.17}$$

However, more definitions for the ductility factor exist and a comprehensive overview is presented by Jorrisen and Fragiacomo [15]. For example, is a definition based on the peak force given. However, doesn't a definition based on peak force account for post-peak behaviour, and therefore doesn't it capture the entire displacement ability. For that reason, it's not useful for robustness analysis.

$$\mu = \frac{\Delta F_{max}}{\Delta F_y} \tag{6.18}$$

Another method for determining the slip at the peak force is possible by. Generally, for ultimate limit state design (ULS), the stiffness of the fastener is taken as 2/3 * Ks. See figure 6.3.

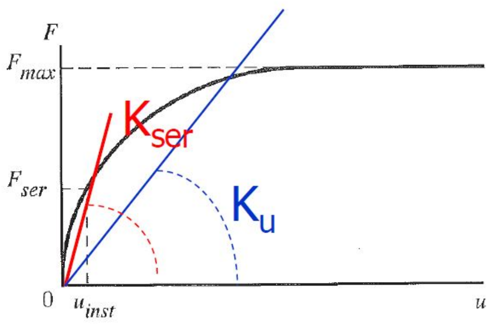


Figure 6.3: ULS stiffness

Levels of ductility

Besides a deformation-based definition for ductility, the Swiss code SIA 265, distinguishes two different levels of ductility for different types of connection as given in Table 6.5. The definition of ductility is based on the ultimate deformation according to Equation 6.17. The lower ductility levels with $\mu \approx 1$ correspond to connections that fail in a brittle way or with minimal plastic deformation when less than 2 plastic hinges per shear plane develop in the fastener.

In contrary to the definition of the Swiss code, Smith et al. (2006) proposed to use a definition based on deformation at peak (maximum) load for the ductility ratio according to Equation 6.18 with a broader distinction between different ductility levels, as can be seen in Table 6.6. However, he did not provide indications on how to achieve these levels [14].

Table 6.5: Ductility ratio for timber connection according to equation 6.17

Type of connection	Ductility factor μ
Shear connections with dowel-type satisfying less then 2 plastic hinges in the fastener per shear plane	μ =1...2
Shear connections with dowel-type satisfying 2 plastic hinges in the fastener per shear plane	μ > 3

Table 6.6: Ductility for timber connection according to equation 6.18

Classification	Average ductility factor
Brittle	μ<= 2
Low ductility	2 <= μ <= 4
Low ductility	4 <= μ<= 6
High Ductility	μ > 6

6.3. Defining the stiffness of glulam members

Shear stiffness

The stiffness k_{bs} of a glulam beam in shear:

$$k_{\text{bs}} = \frac{G_{\text{mean}} A_{\text{b}}}{z_{\text{eq}}} \tag{6.19}$$

where G_{mean} = shear modulus and A_{b} is cross-sectional area of the beam loaded in shear.

Compression stiffness

The stiffness k_{bc} of a glulam beam in compression perpendicular to the grain:

$$k_{\text{bc}} = \frac{E_{90,\text{mean}} b_{\text{b}} l_{\text{eff,bc}}}{h_{\text{b}}} \tag{6.20}$$

where b_{b} is the width glulam beam, and $l_{\text{eff,bc}}$ is the effective bearing length of the glulam beam in compression perpendicular to the grain.

The stiffness k_{cc} of a glulam column in compression parallel to the grain

$$k_{\text{cc}} = \frac{E_{0,\text{mean}} \sqrt{b_{\text{eff.cc}} l_{\text{eff,cc}}}}{\beta} \tag{6.21}$$

where $b_{\text{eff.cc}}$ and $l_{\text{eff,cc}}$ are the effective dimensions of the column in compression, and the factor β is a dimensionless coefficient which can be approximated to the value of 4 [33]

6.4. Design values

The design value of a strength property shall be calculated from Equation 6.22.

$$f_d = k_{mod} \cdot \Pi k_i \frac{f_k}{\gamma_M} \tag{6.22}$$

The design value of a strength property shall be calculated from Equation 6.23.

$$R_d = k_{mod} \frac{R_k}{\gamma_R} \tag{6.23}$$

where: f_k is the characteristic (5th-percentile) value of the strength property of the material, k_{mod} is the modification factor taking into account the effect of the duration of load and moisture content, R_k is the characteristic value of the resistance (load-carrying capacity), γ_M is the partial factor for the material property, γ_M is the partial factor for the resistance, and Πk_i is the product of the applicable modification factors, in addition to k_{mod}.

Part II

Modelling

7

Case Study

7.1. Introduction

This chapter presents the structural design of a unique modular structural building system that is taken as a case study in this research to study its robustness. The modular system as presented in this chapter is designed by engineering consultant Pieters Bouwtechniek Delft (PBT). PBT has designed the structural system with the Eurocodes and Dutch standards to comply with regular ultimate limit state (ULS) and serviceability limits state (SLS) criteria. However, the accidental limit state (ALS) was not designed for the structural system as it is presented here. Because of that, the structural system of this study is not specifically designed for robustness. However, PBT is interested to know whether they have to take design measures or if the current system is robust. Therefore, this thesis explores whether the designed structural system, though not specifically designed for robustness, still is robust. In Chapter 3 & 4 we learned that to analyse the robustness of the system on a global level, it is necessary to take into account the properties of the connections. Consequently, this chapter analyses the inherent properties of the connections in the design on a local component level, and it presents an idealisation of them together with the structure to perform analyses on a global level.

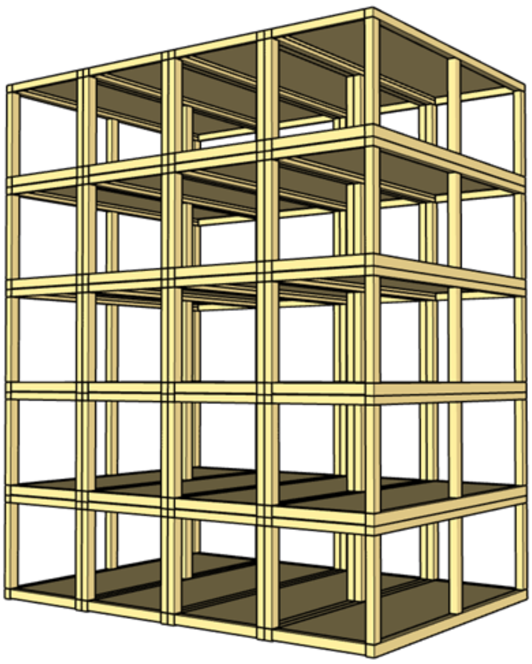


Figure 7.1: Fictitious building of 4x5 modules

Section 7.2 presents the building design and its unique characteristics. Section 7.3 goes into the connections of the system and presents the mechanical models to derive stiffness properties. Section 7.4 presents the analysis models used to study the alternative load path development in the case of notional element removal. In section 7.5, the removal events and scenarios are presented. Section 7.6, presents the procedures used to determine the alternative load path development in the building. Section 7.8 entails the used finite elements of the model. Finally section 7.9 presents a validation of the analysis methods.

7.2. Building design and structure

The modular building of this case study is constructed with 3D volumetric post and beam modules, i.e. the module type from the scope of this research. The structure consists of multiple horizontally aligned modules with other modules stacked on top. The longer sides of the modules face each other, and in practice, the modules are accessed from the short sides of the modules. Generally, a corridor is present on one side and the other side has a facade with windows. The stability of the building is achieved by steel lateral stability frames in both the lateral and longitudinal directions. Those frames are located on the outer edges of this building, where in reality, a staircase and elevator shaft are located. The modules have an open design, and in that way, multiple modules can be connected to create an apartment.

From the complete building, a specific part located in-between two stability frames is taken out and simplified by the author to make it feasible to study. That specific part consists of 6 modules placed next to each other in a horizontal direction, with 4 modules stacked on top, making a 6x5 grid.

7.2.1. Module

The volumetric modules are fabricated with glued laminated timber (GLT) for the columns and beams and with cross-laminated timber (CLT) for the floor and ceiling slab. A single volumetric module has 6 columns of the same size placed in between a floor and ceiling beam. Four columns are placed at the corners, and two columns are located halfway the span, see Figure 7.2. The CLT slabs forming the floor and ceiling are connected in between the beams.

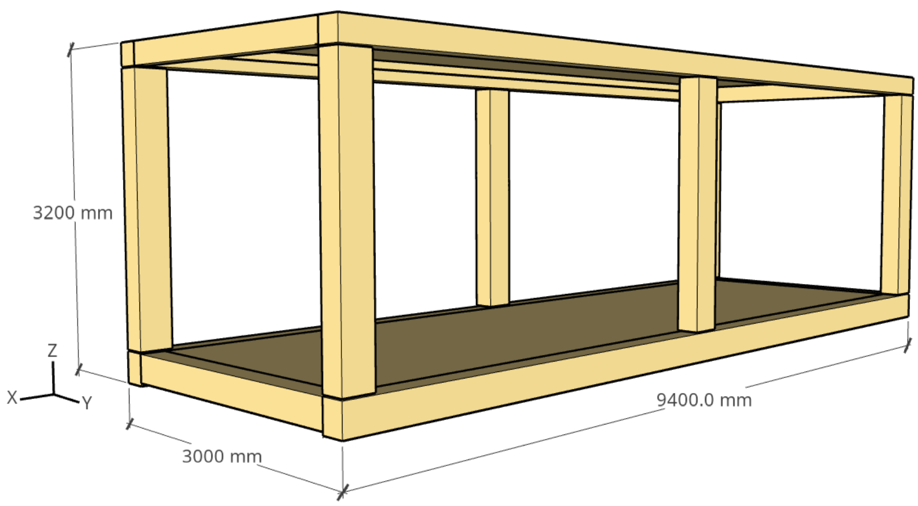


Figure 7.2: Single volumetric module

Table 7.1: Dimensions of Module

Properties	Values	Unit
Dimensions of module (l x w x h)	9400 x 3000 x x 3200	mm
Dimensions of floor beam GLT (h x w)	320 x 240	mm
Dimensions of ceiling beam GLT (h x w)	220 x 240	mm
Dimensions of columns GLT (h x w)	320 x 240	mm
Thickness of CLT slab	160	mm

7.2.2. Materials

The material properties of the glued laminated timber for the beams and columns are presented in Table 7.2.

Table 7.2: Material properties GLT24h

Property	Symbol	Value	Unit
Bending strength	$f_{\text{m;k}}$	24	N/mm^2
Density-char	ρ_{k}	380	kg/m^3
Density-mean	ρ_{rmean}	460	kg/m^3
Modulus of elasticity //	$E_{0;\text{ mean}}$	11600	N/mm^2
Modulus of elasticity //	$E_{0.05}$	9400	N/mm^2
Modulus of elasticity $\perp$	$E_{90;\text{ mean}}$	390	N/mm^2
Tension strength //	$f_{\text{t},0;\text{k}}$	16.5	N/mm^2
Tension strength $\perp$	$f_{\text{t},90;\text{k}}$	0.4	N/mm^2
Compression strength //	$f_{\text{c};0;\text{k}}$	24	N/mm^2
Compression strength $\perp$	$f_{\text{c};90;\text{k}}$	2.7	N/mm^2
Shear strength	$f_{\text{v,k}}$	2.7	N/mm^2
Shear modulus	G_{mean}	720	N/mm^2

7.2.3. Overview of Connections

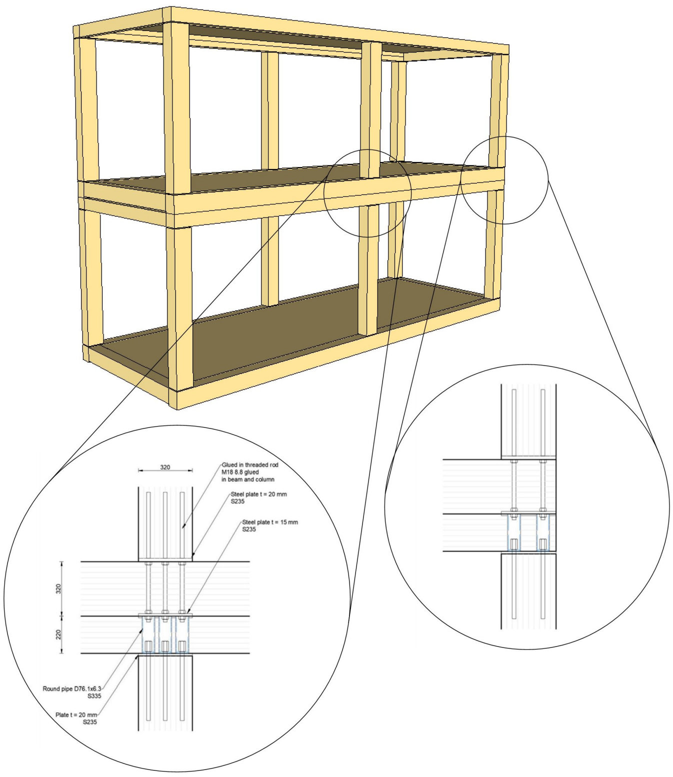
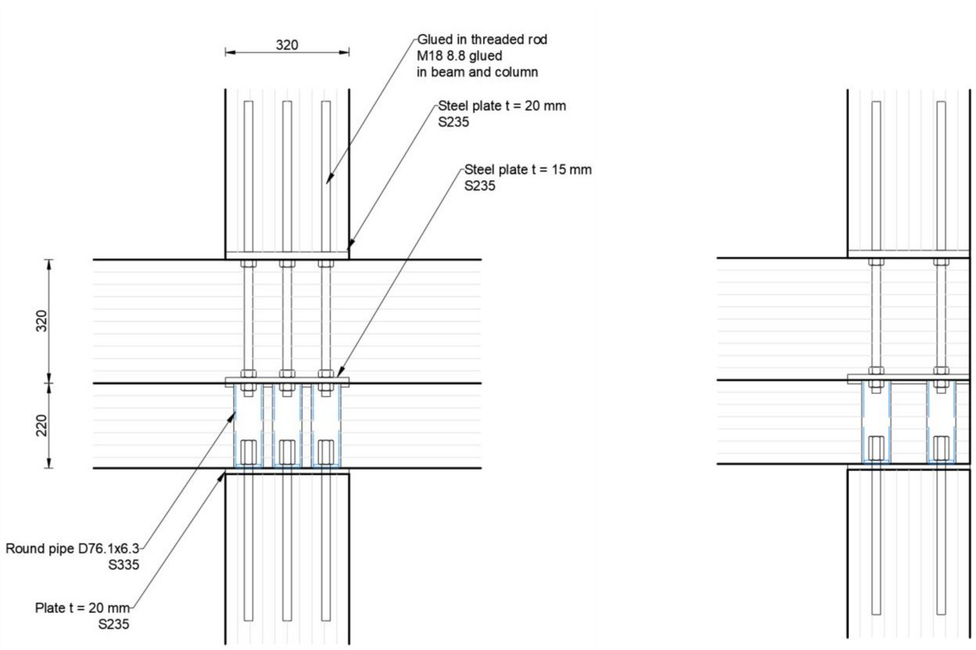


Figure 7.3: Intra-modular connection locations



(a) intra-module middle column

(b) intra-module edge column

Figure 7.4: Details of intra-modular connections

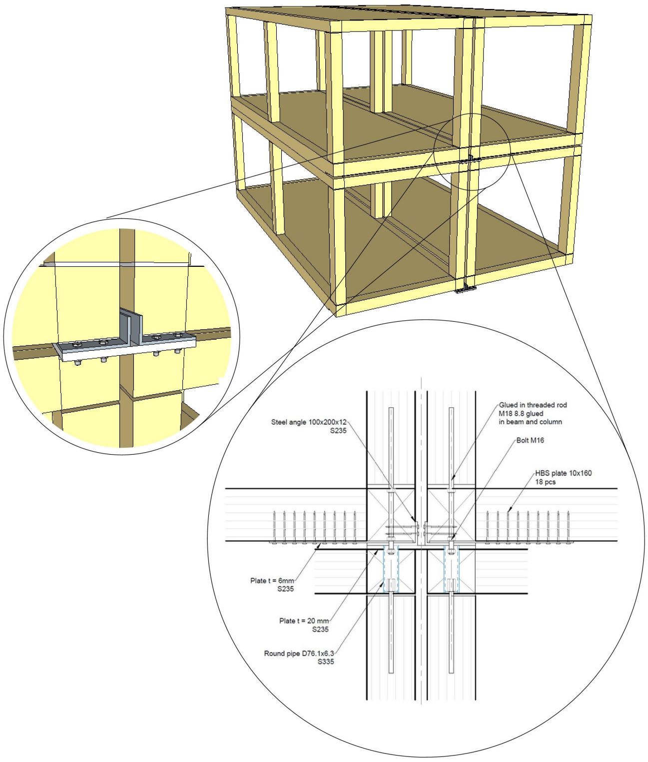


Figure 7.5: Inter-modular connection location

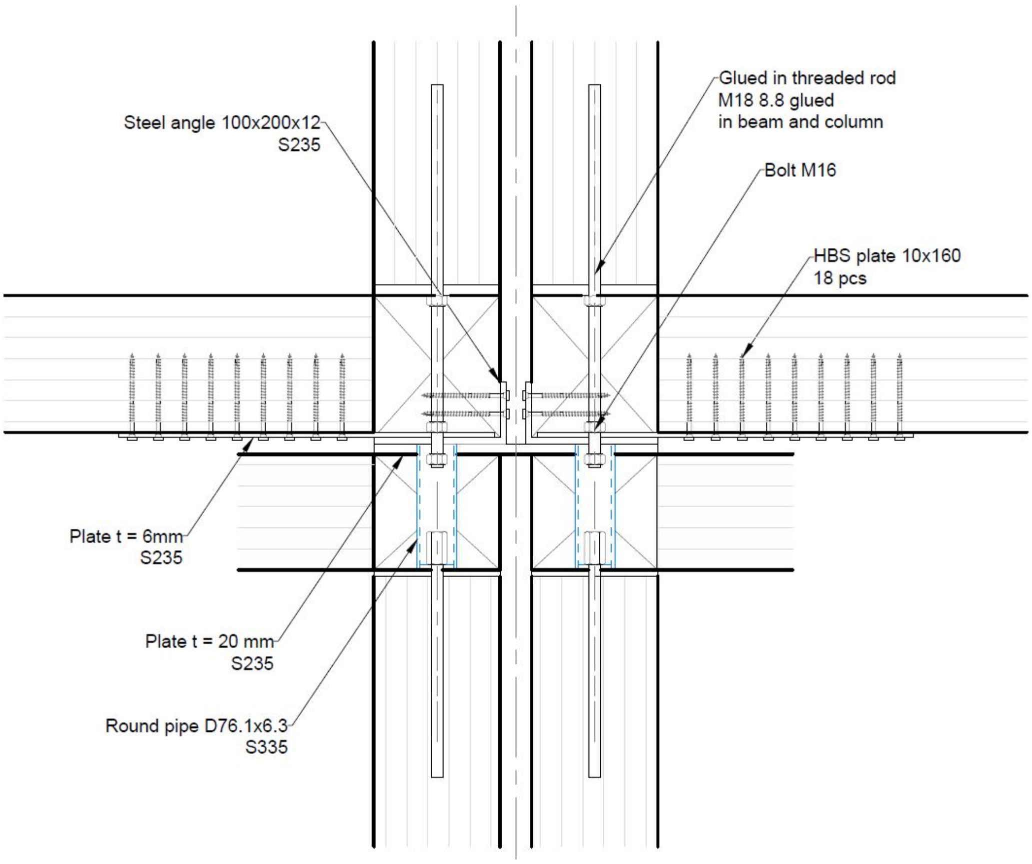


Figure 7.6: Inter-modular connection

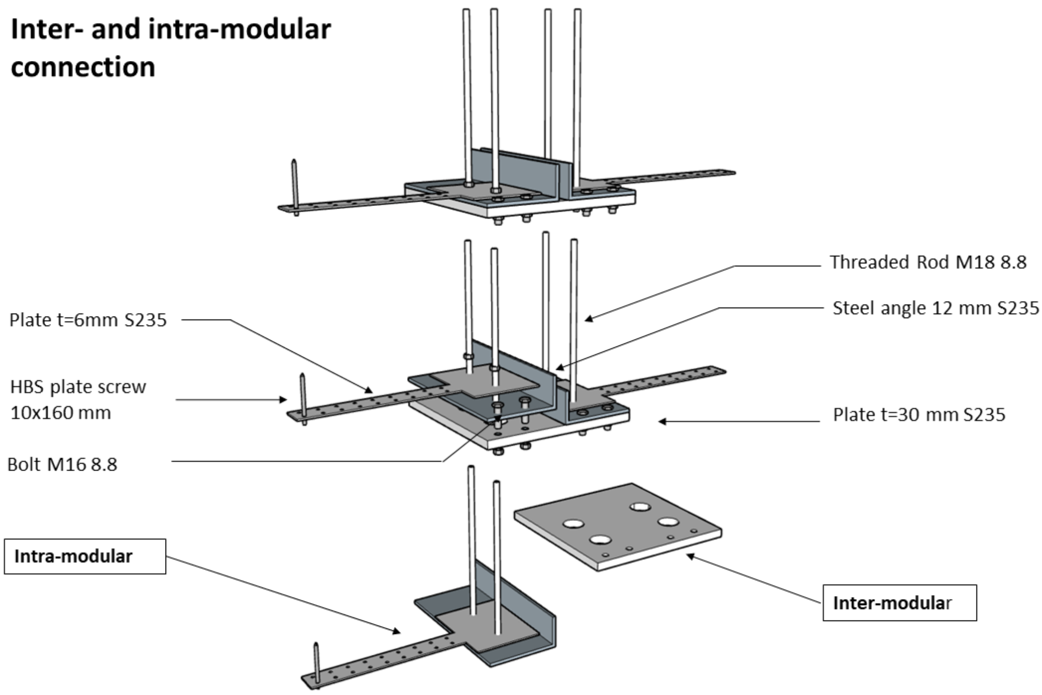


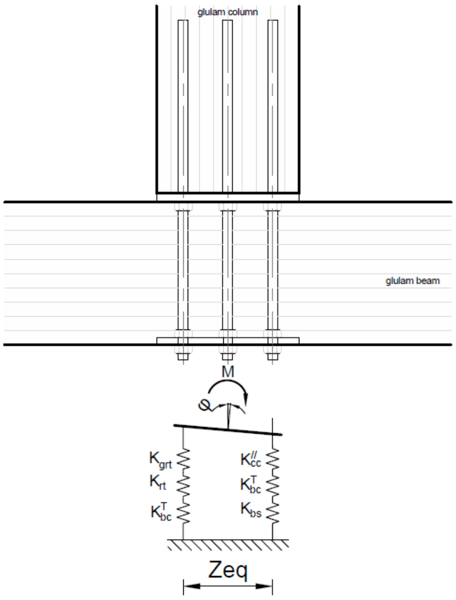
Figure 7.7: 3D overview of combined intra- and inter-modular connection their components

7.3. Connection Idealisation & Properties

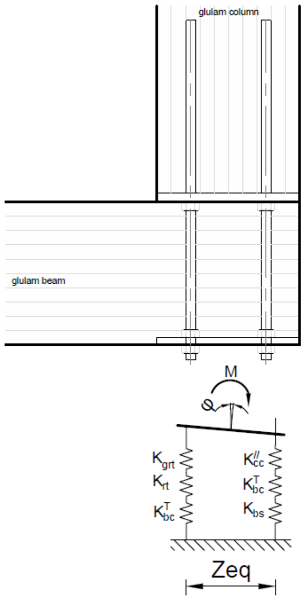
The connections in the modules consist of many different components. The independent behaviour of specific components is presented and explained in Chapter 6. In the current section, the overall rotational stiffness of a connection is determined on the basis of spring models with a serial addition of the different components and elements. The resistance of a connection is determined by the weakest link within the connection. In the spring models below, several abbreviations can be seen, they are presented in Table 7.3 below.

Table 7.3: Naming of the various components in the mechanical models

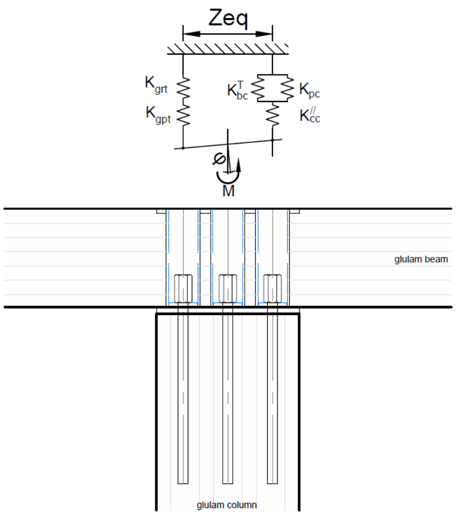
Component	Abbreviation
Beam in compression perpendicular to the grain	$bc^{\perp}$
Beam in compression parallel to the grain	$bc^{//}$
Column in compression perpendicular to the grain	$cc^{\perp}$
Column in compression parallel to the grain	$cc^{//}$
Beam in shear	bs
Plate in tension	pt
Rod in tension	rt
Glued in rod in tension	grt
Screws in shear	ss
Glued in rods in shear	$grts$
Pipe in compression	pc



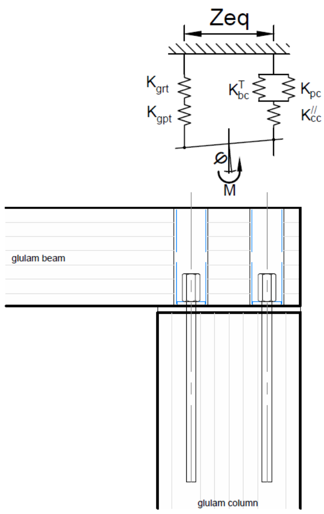
(a) floor beam - middle column



(b) floor beam - edge column



(c) ceiling beam - middle column



(d) ceiling beam - edge column

Figure 7.8: Equivalent spring models intra-modular connections

Table 7.4: Intra-modula floor beam - middle column properties

Property	Symbol	x-axis	y-axis	z-axis	Unit
Elastic stiffness	K_e	6912	6912	263700	kN/m
Yield resistance	F_y	24.30	24.30	21.28	kN
Plastic deformation	δ_p	20.59	21.59	0.00	mm
Ultimate resitance	F_u	35.37	35.37	21.28	kN
Rotational elastic stiffness	K_r	-	354.40	-	kNm/rad
Rotational resistance	M_y	-	18.50	-	kNm
Plastic rotation	θ_p	-	0.10	-	rad
Ultimate rotational resistance	M_u	-	18.50	-	kNm

Table 7.5: Intra-modula floor beam - edge column properties

Property	Symbol	x-axis	y-axis	z-axis	Unit
Elastic stiffness	K_e	4608	4608	149500	kN/m
Yield resistance	F_y	16.19	16.19	127.92	kN
Plastic deformation	δ_p	20.59	20.59	0	mm
Ultimate resitance	F_u	23.58	23.58	127.92	kN
Rotational elastic stiffness	K_r	115.20	280.30	-	kNm/rad
Rotational resistance	M_y	10.97	15.23	-	kNm
Plastic rotation	θ_p	0.042	0.046	-	rad
Ultimate rotational resistance	M_u	10.97	15.23	-	kNm

Table 7.6: Intra-modula ceiling beam - edge column properties

Property	Symbol	x-axis	y-axis	z-axis	Unit
Elastic stiffness	K_e	4608	4608	149500	kN/m
Yield resistance	F_y	16.19	16.19	127.92	kN
Plastic deformation	δ_p	20.59	20.59	0	mm
Ultimate resitance	F_u	23.58	23.58	127.92	kN
Rotational elastic stiffness	K_r	115.20	879.8	-	kNm/rad
Rotational resistance	M_y	10.97	15.23	-	kNm
Plastic rotation	θ_p	0.042	0.0827	-	rad
Ultimate rotational resistance	M_u	10.97	15.23	-	kNm

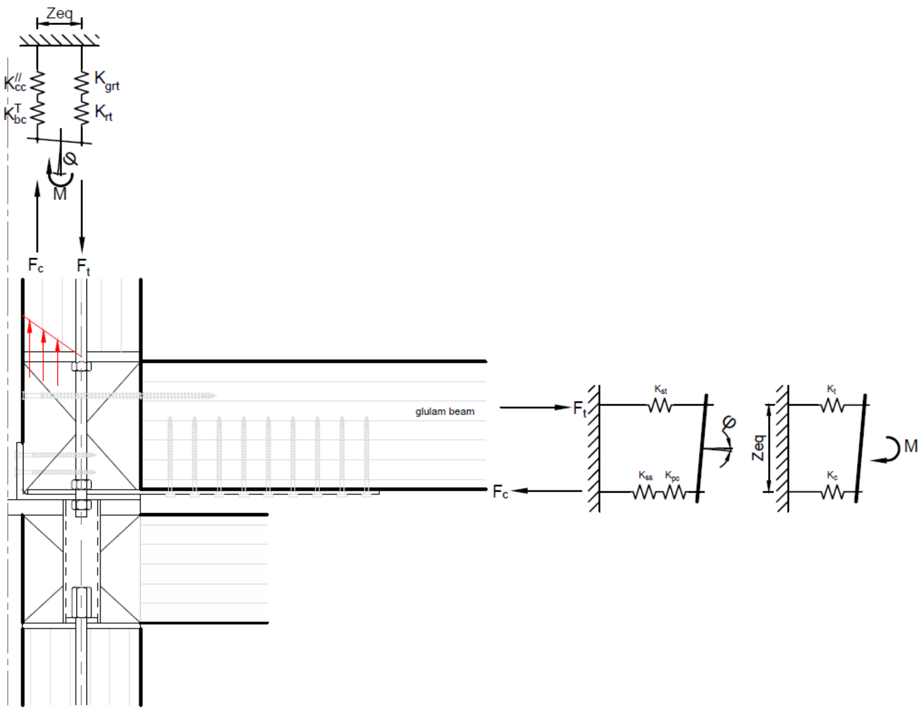


Figure 7.9: Rotational spring model for combination of the intra- and inter modular connection in case of a opening moment θ^+

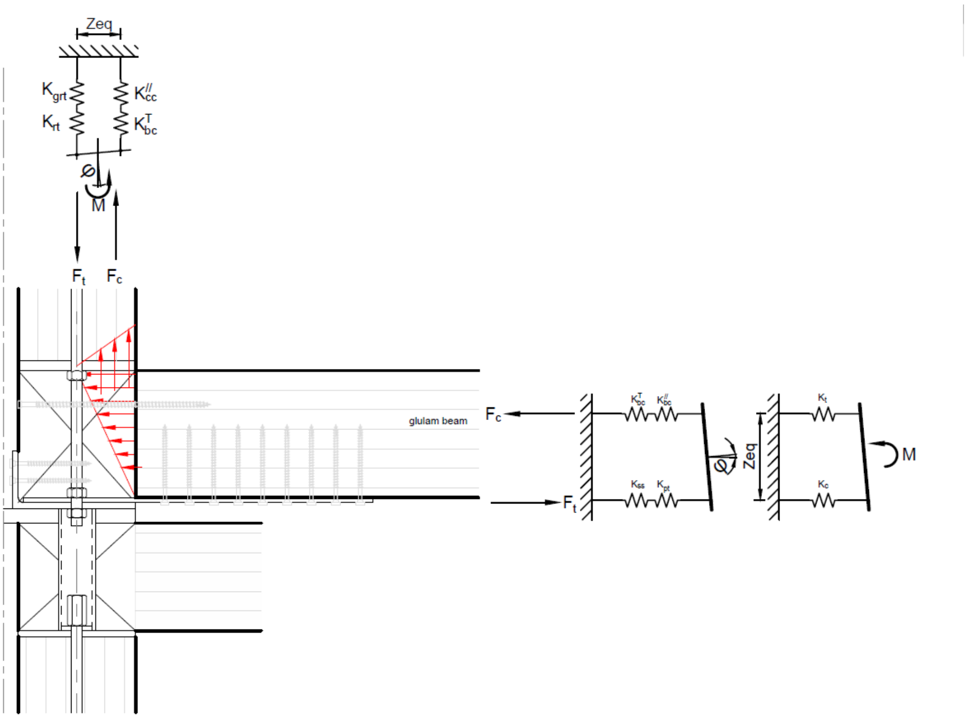


Figure 7.10: Rotational spring model for combination of the intra- and inter modular connection in case of a closing moment θ^-

Table 7.7: Rotational properties for corner connection with intra- and inter modular connection combined

Property	Symbol	x-axis θ^+	x-axis θ^-	Unit
Rotational elastic stiffness	K_r	115.20	101.90	kNm/rad
Rotational resistance	M_y	10.97	10.97	kNm
Plastic rotation	θ_p	0.055	0.042	rad
Ultimate rotational resistance	M_u	10.97	10.97	kNm

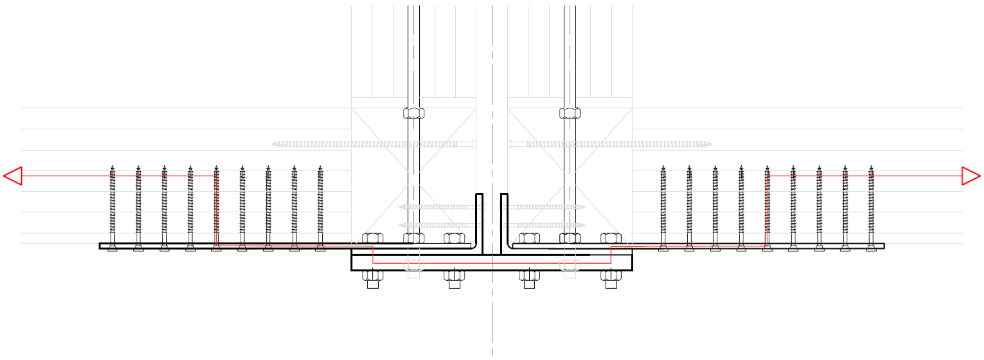


Figure 7.11: Inter-modular connection flow of force for horizontal tie

Table 7.8: Properties for inter-modular connection in case of a horizontal tie force

Property	Value	Unit
Shear resistance threaded rod M18	147.4	kN
Bearing resistance of 6mm plate with threaded rod	155.5	kN
Shear resistance of bolt M16 with 12 mm angle plate	120.6	kN
Bearing resistance of 12 mm angle plate	153.6	kN
Tensile resistance of 6mm plate	126.9	kN
Tensile resistance of net cross section plate	108.9	kN
Shear resistance 10 mm screw (group)	110.2	kN
Shear stiffness screw group	1.25E+05	kN/m
Stiffness 6mm plate	1.53E+05	kN/m
Stiffness 30 mm plate	6.30E+06	kN/m
Elastic stiffness of connection	3.25E+04	kN/m
Yield resistance of connection	108.9	kN
Plastic deformation of connection	3.36	mm

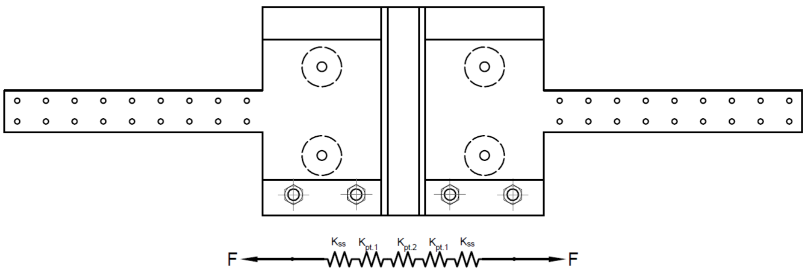


Figure 7.12: Translational spring inter-modular connection

7.4. Analysis Models

Section 7.2 presented the 3D volumetric modular building system. From the design of a single module, we know it has a long side and a short side. In Figure 7.13 below, an illustration of a building of 2x2 modules is presented. It can be seen that the long sides of a module are parallel with the x-axis and facing each other. The short sides are parallel with the y-axis. To analyse the building system for different notional element removal events in the finite element software, the three-dimensional building system is decomposed into two-dimensional plane sub-subsystems.

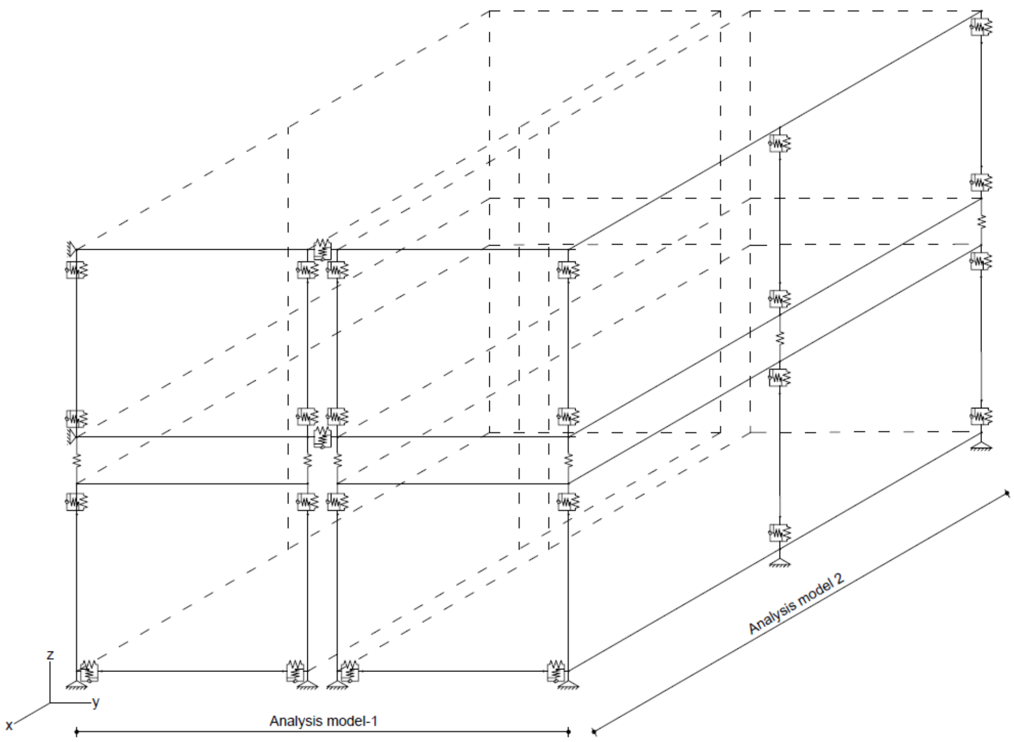


Figure 7.13: Illustration of 3D model with springs replacing the connections

As a result of the decomposition, we are left with two 2D frames. In this study, they are referred to as analysis models 1 and 2, see Figure 7.13. Analysis model 1 is meant to study the behaviour of the structure with several modules next to each other, corresponding to the y-axis. The second variant, i.e. analysis model 2, see Figure 7.15 looks at the lateral side of the building where modules are only stacked vertically onto each other, corresponding to the x-axis. If we further decompose the frames, we are left with linear elements for the columns and beams, and connector elements for the connections. The Figure's connector elements are presented with boxes consisting of two translational springs and one rotational spring.

Boundary conditions are applied at multiple nodes, first, for the foundations, the nodes are restricted in the vertical and horizontal directions. Secondly, for the stacked storeys of modules, the end nodes of the modules' floor beams are restricted in the horizontal direction, as can be seen in the figures below. In the physical building, those supports follow from the steel lateral stability systems and elevator shaft.

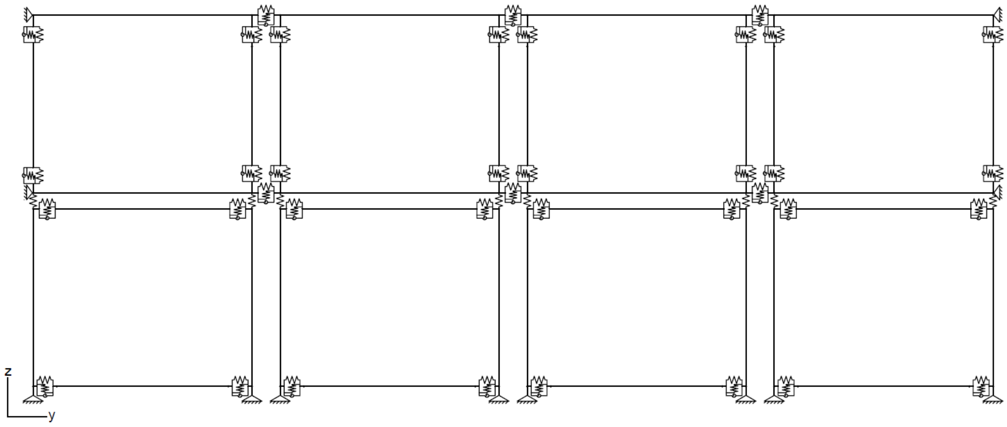


Figure 7.14: Analysis model 1

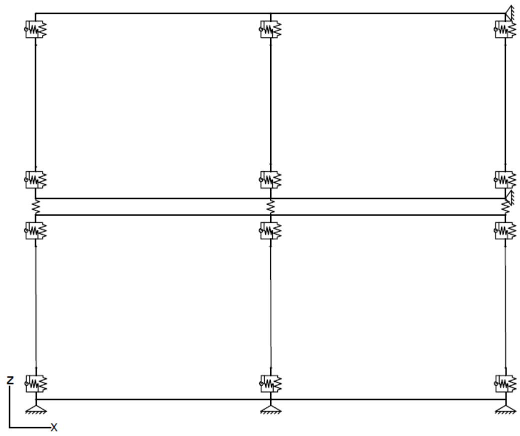


Figure 7.15: Analysis model 2

7.5. Notional element removal

7.5.1. Element removal events

The alternative load path development in the building is studied for 5 different removal events; see Figure 7.16. The removal events consider either the loss of a vertical load-bearing element, i.e. a post, or the loss of a whole module. The events are presented with a number. Besides the figure they are presented with a description in Table 7.9.

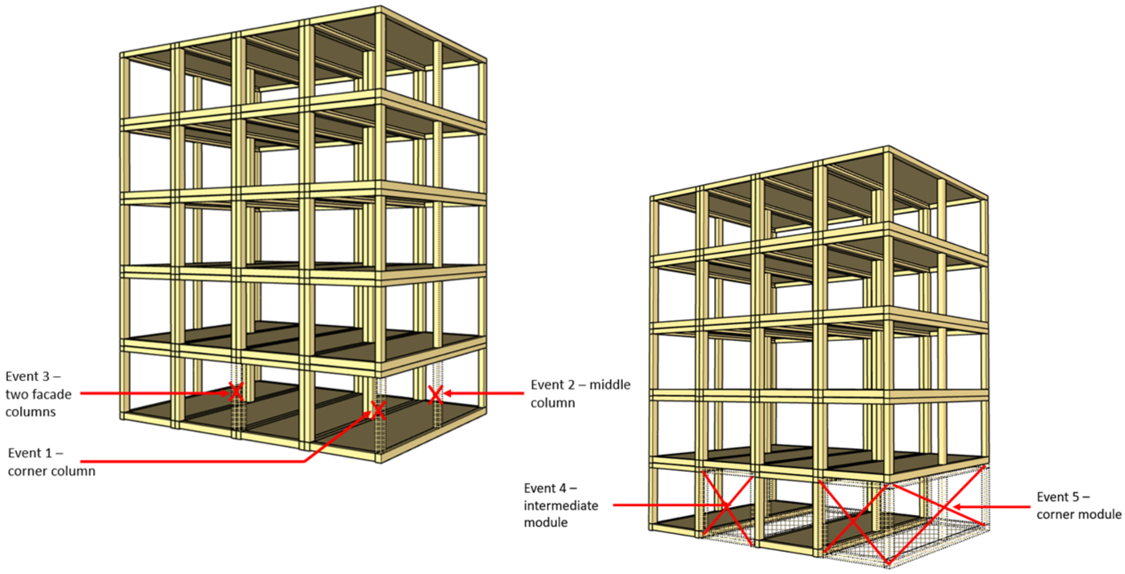


Figure 7.16: Element removal events

Table 7.9: Element removal events

Event	Description
Event 1	considers the loss of a corner column of the building
Event 2	considers the loss of a middle column in the longitudinal side of a module
Event 3	considers the loss of two neighbouring adjacent columns at the front facade simultaneously
Event 4	considers the loss of an intermediate module
Event 5	considers the loss of a corner module

7.5.2. Alternative load path scenarios

Previously, in Section 7.4, it was introduced that the 3D structure is decomposed into two 2D frames. Following that decomposition, with a certain removal event as introduced in the section above, we can study a scenario where an ALP develops in the long side of the modules, corresponding to the y-axis of the structure; see Figure 7.18. And a scenario where an ALP develops in the short sides of the modules, corresponding to the x-axis, see Figure 7.19. As a result of that for the 5 removal events in total 6 scenarios are considered. Some scenarios occur with multiple removal events whilst others only occur for an individual unique event; see Figure 7.17.

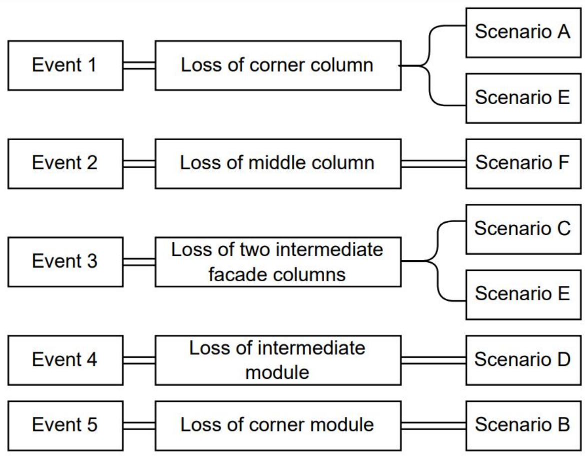


Figure 7.17: Scenarios for ALP following the removal events

Scenarios

Firstly for scenario (a) till (d) the first analysis model is used, see Figure 7.14. Secondly, for scenario (e) & (f), the second analysis model, see Figure 7.15.

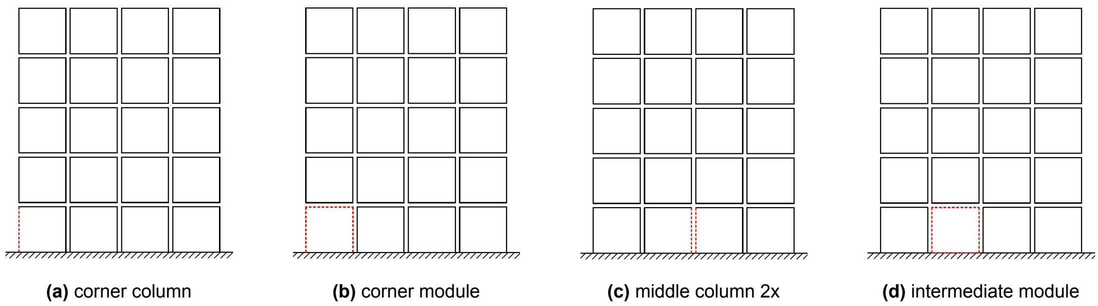


Figure 7.18: Removal scenario's front view in analysis model-1

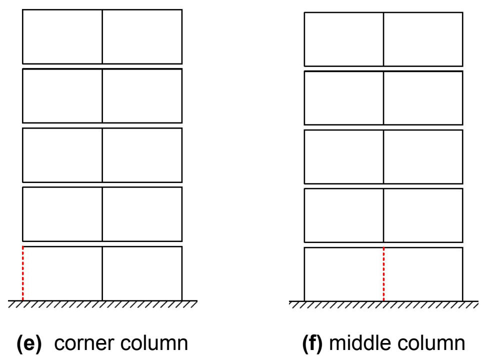


Figure 7.19: Removal scenarios side view in analysis model-2

7.6. Analysis procedures

7.6.1. Static procedure

The loading procedure for the nonlinear static analysis is displayed in Figure 7.20. It consists of two steps, in the first step, the structure with a removed element is quasi-statically loaded with unamplified gravity loads i.e. the design loads. In the second step, the parts of the structure above the locally damaged area are loaded with additional gravity loads equal to the difference between the amplified loads and the unamplified loads. In short, that means that for a maximum dynamic load factor of 2, the damaged areas at the end of step 2 are loaded with twice the design load. [8].

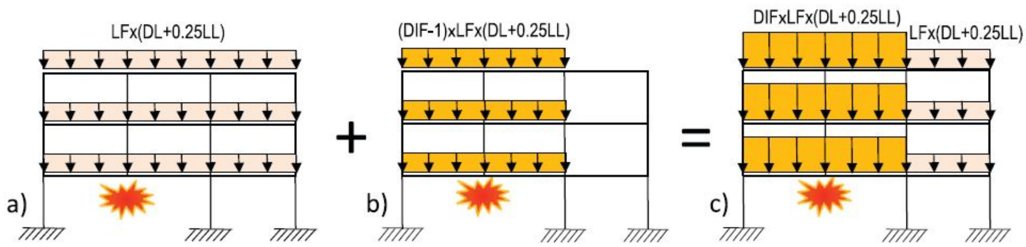


Figure 7.20: a) Push 1: Preliminary push-down analysis under unamplified gravity loads. (b) Push 2: Push-down analysis under additional gravity loads in the bays affected by column removal. (c) Push-down analysis under amplified gravity loads. [8]

7.6.2. Dynamic procedure

Figure 7.21 displays the loading procedure for the nonlinear dynamic analysis. It consists of two steps, in the first step the reaction forces of the intact structure (without a removed element) are statically determined. In the second step, the structure above the locally damaged area is loaded with the inverted reaction forces. Next, to start the dynamic analysis, the reaction forces are instantaneously released, and the structure's response is captured over time.

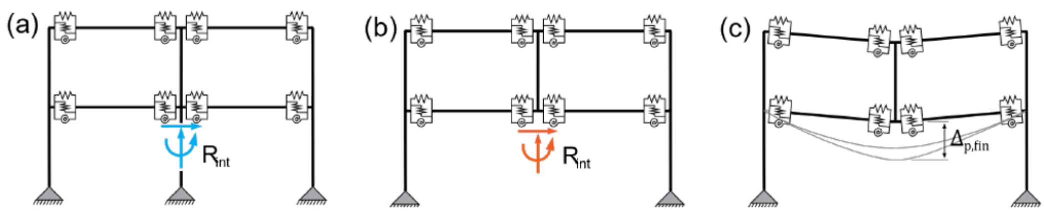


Figure 7.21: Adapted from Cao et al. a) Determining reactions (b) Apply reaction forces in model without column (c) Dynamic response. [4]

7.7. Accidental limit state

In the case of robustness analysis, the structure and connections are assessed for an accidental design situation (ALS). That implies that the loads on the structure are reduced and that the material strength is increased. Therefore, when considering glued laminated timber, the partial factor for the material property and resistance γ_M & γ_R, should be taken as 1.0. Additionally, the modification factor k_{mod} should be taken as 1.1, corresponding to a service class of 1, and an instantaneous load duration. As a result of that, for example, the bending strength of a glued laminated beam in ALS is 26.4 N/mm².

7.7.1. Load combination

Combination of actions for accidental design situations in accordance to clause 6.4.3.3 of EC0

$$E_d = \sum_{j=i}^{n} G_j + \psi_{2,1} Q_j + \psi_{2,i} S \tag{7.1}$$

- E_d = calculation value for load
- S = characteristic value for snow load
- G_j = characteristic value for dead load
- Q_j = characteristic value for live load

According to dutch national annex:

$$E_d = \sum_{j=i}^{n} G_j + 0.3 Q_j \tag{7.2}$$

7.7.2. Loads

The loads applicable for the case study can be found below in Table 7.10 & Table 7.11 below:

Table 7.10: Characteristic G_k dead loads on structure

Property	Value	Unit
Module Floor		
CLT 160 mm ≈ 5 kN/m^3	0.8	kN/m^2
Counterweight 100 mm ≈ 25 kN/m^3	2.5	kN/m^2
Insulation 50 mm ≈ 1.2 kN/m^3	0.06	kN/m^2
Covering floor 30 mm ≈ 12 kN/m^3	0.36	kN/m^2
$G_{k;tot}$	3.72	kN/m^2
Module Ceiling		
CLT 60 mm ≈ 5 kN/m^3	0.3	kN/m^2
Insulation 50 mm ≈ 1.2 kN/m^3	0.15	kN/m^2
Gypsum 25 mm ≈ 10 kN/m^3	0.25	kN/m^2
$G_{k;tot}$	0.7	kN/m^2
Roof Floor		
CLT 160 mm ≈ 5 kN/m^3	0.8	kN/m^2
Insulation ≈ 1.2 kN/m^3	0.3	kN/m^2
Sedum roof ≈ 10 kN/m^3	2	kN/m^2
$G_{k;tot}$	3.1	kN/m^2

Table 7.11: Characteristic live Q_k live loads on structure

Property	value	unit
Module Floor - class A (residential)	3	kN/m^2
Building Roof - class H	1.5	kN/m^2

7.8. Finite elements

The analyses are performed in the software of Abaqus. The structural elements are modelled with one-dimensional beam elements, including a uniform cross-section with linear elastic material properties, see Table 7.12. In Abaqus, the B21 element is chosen, which follows Timoshenko (shear flexible) theory. Abaqus assumes that the transverse shear behaviour of Timoshenko beams is linear elastic with a fixed modulus and, thus, independent of the response of the beam section to axial stretch and bending [6]. For the mesh, an element size is used of approximately 1/6th of the bay size, corresponding to 0.5 meters. An example of the initial intact model can be seen Figure 7.22. The model is constrained at the foundation for vertical and horizontal translation and along the right vertical edge for horizontal translation. The connections are modelled with standard connector elements in Abaqus, also known as CONN2D2 for two-dimensional and axisymmetric analyses. Figure 7.23 below is a close-up of the meshed model at the location of an inter-modular connection. In the figure, the connector wire elements representing the connections are plotted with a dashed line. They have an orange begin point and a yellow end point. The elements have at most two nodes, the position of the second note is relative to the first node. Different types of connectors can be chosen, in this work, the two most general types were used: the *Cartesian* type for translational motion and the *Rotation* type for rotational motion, similar to the work of Huber and Voulpiotis [10], [35]. For the connector, specific behaviour is assigned to the relative motion along each degree of freedom (DoF) to model elastic, plastic and damage behaviour. The DoFs are assumed to be independent, i.e., no coupling is accounted for.

Table 7.12: Analysis input Abaqus

Properties	Values	Unit
Density	460	kg/m3
Modulus of elasticity	11600	N/mm2
Poisson ratio	0.395	-

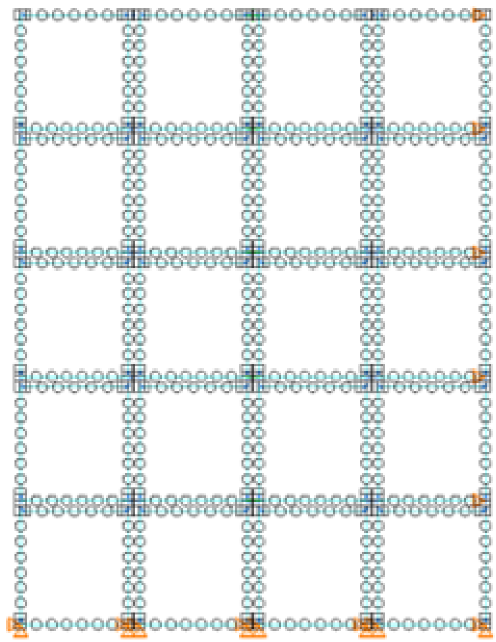


Figure 7.22: Figure with meshed elements representing analysis model 1 in the Abaqus environment

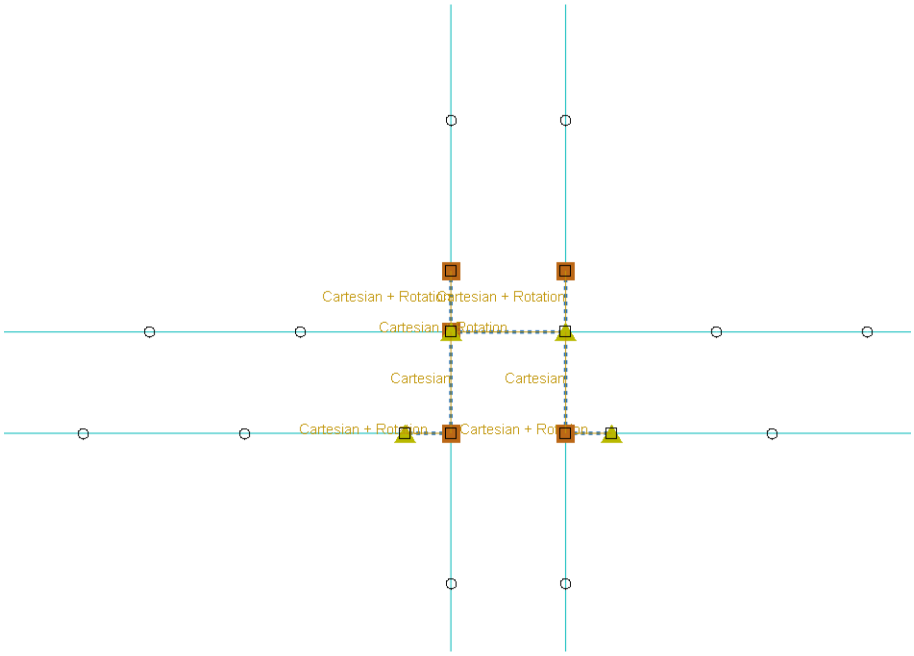


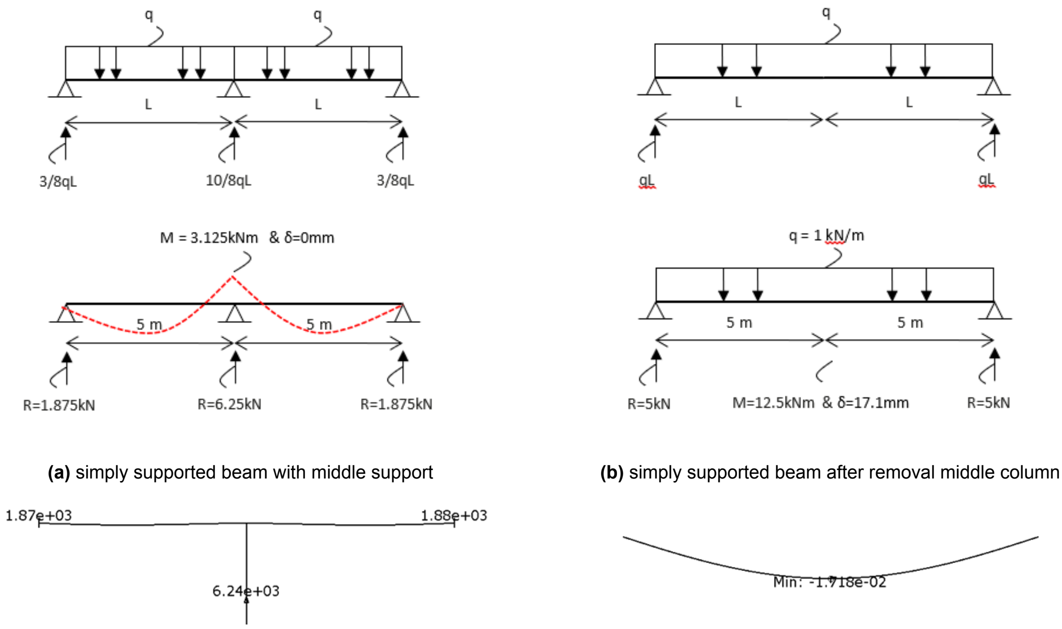
Figure 7.23: Close up from an intermediate module corner with connector wires for intra- and inter-modular connection

7.9. Validation of the method

In section 7.6, two procedures were introduced that will be used to derive the results in this thesis. Before the complex modular structure of this case study will be analysed, first, validation of the software and proposed procedures is necessary. For that reason, a simple structure of a beam on three supports is modelled in Abaqus and analytically validated, see Figure 7.24. The continuous beam with two spans is 10 meters long and is made of laminated timber, GL24h. The input parameters are presented in Table 7.25. First, analytically, the support reactions for the intact model with two spans are calculated. At the location of the middle column, the support reaction is 6.25 kN. Secondly, the deflection at midspan for the structure without a middle column is calculated, resulting in 17.1 mm. Accordingly, the same structure is modelled in Abaqus. The results of the linear static analysis are presented in subfigures (c) and (d), they align with the analytical model.

Table 7.13: Validation input

Properties	Values	Unit
Density	460	kg/m^3
Modulus of elasticity (E)	11600	N/mm^2
Poisson ratio	0.395	-
Area (A)	76800	mm^2
Area moment of inertia (I)	655360000	mm^4
Distributed load (q)	1	kN/m
Length (L)	5	m
Mesh size	100	mm



(a) simply supported beam with middle support

(b) simply supported beam after removal middle column

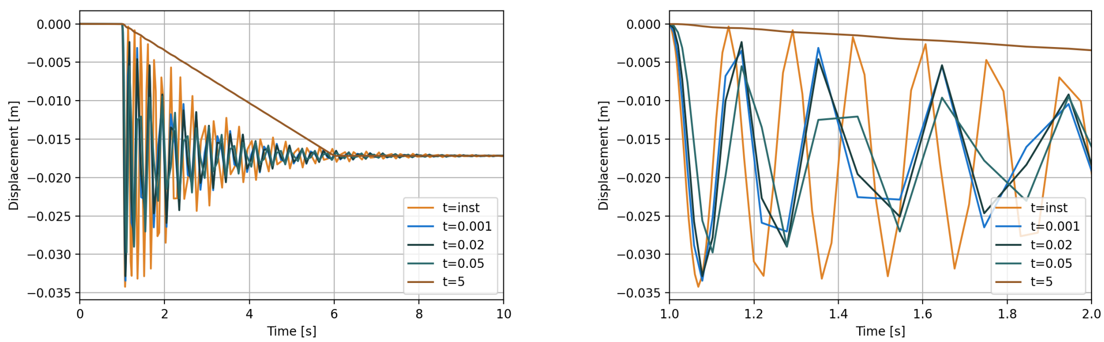
(c) Abaqus support reactions in Newtons

(d) Abaqus deformation in meters

Figure 7.24: Simple model validation with analytical and fem results for static loading

Dynamic effects due element removal speed

In Chapter 5, it was mentioned that the instantaneous loss of column in a linear elastic SDOF system corresponds to a DAF of 2, the upper bound. The factor according to the theory reduces when the removal rate of the column is reduced, having a lower bound at 1.0 for quasi-static column loss. According to the design requirements of the Unified Facilities Criteria (UFC-4-023-03) and GSA specifications, an instantaneous loss corresponds to a removal speed of t = 0.001 seconds. In this study, that theory was validated with the dynamic analysis procedure as presented earlier. The reaction force at the location of the middle column of the intact model was determined, and consecutively the column was replaced by the reaction force in the dynamic model. Then for different removal speeds, the reaction force was reduced to zero in a dynamic implicit analysis in Abaqus. Accordingly, the displacement values up to 10 seconds after removal were recorded. The resulting displacements are presented in Figure 7.25 and the corresponding DAF, i.e. the ratio of the maximum dynamic displacement Δ_{dy} to the final (static) displacement Δ_{fin} in Table 7.14.



(a) vertical displacement response at midspan

(b) peak displacements on time interval 1-2 sec

Figure 7.25: Effects of element removal speed in dynamic response

The results show that for sudden removal,i.e. t<5sec, dynamic effects occur. This was also confirmed by Mpidi Bita [24], he discovered that different removal speeds (t) impose different deformations on the structure. The quicker the removal the higher the peak deflections at midspan and the higher the

DAF. It was observed that the instantaneous removal in Abaqus corresponds to the theoretical upper bound of DAF = 2, which means that the dynamic response is twice the static response. Also is shown that a slow removal speed of t = 5 sec corresponds to a static removal where no dynamic effects occur. Additionally, did all the models for the different removal speeds find the same final equilibrium deflection of 0.0172 meters, the same as the static result in Abaqus and the analytical result, see Figure 7.24. This proves that the proposed models and procedures are valid.

Table 7.14: Effects element removal speeds on dynamic amplification factor

Removal speed t [s]	Δ_{dy} [m]	Δ_{fin} [m]	DAF
t=inst	0.0342	0.0172	1.99
t=0.001	0.0334	0.0172	1.95
t=0.02	0.0329	0.0172	1.91
t=0.05	0.0297	0.0172	1.73
t=5	0.0171	0.0172	1.00

8
Results

8.1. Introduction

This chapter presents the results of a series of notional element removal scenarios applied to the timber building of the case study. The results are obtained with a 2D frame model in the software Abaqus in which connections are replaced by nonlinear springs. The analysis models with corresponding loads and applied loading method, together with the material input parameters and connection properties, were introduced in Chapter 7.

First, in this chapter, the results of the initial design are presented, and then the chapter will continue with a parameter study into the influence of different connection parameters on the global robustness performance/behaviour in case of notional element removal. Finally, the chapter ends with a summary of the findings of the analyses.

8.2. Performance of initial building

The performance of the initial building design is studied with two different analysis methods. At first, multiple nonlinear push-down analyses are performed for the different removal scenarios. The analyses are performed with the software Abaqus with both physical and geometrical nonlinearity. Loads are monotonically increased, i.e. quasi-static analysis. The models are bounded by the resistance and ductility limits of the connections, see Chapter 6

8.2.1. Scenario A

For scenario A, a corner column is removed in the first analysis model, as shown in Figure 8.1. The figure displays the final vertical displacement output of the analysis with a colour plot. The boundary conditions in the model are displayed with an orange triangle. From the figure, it can be seen that the deflection is largest at the location of the removed column.

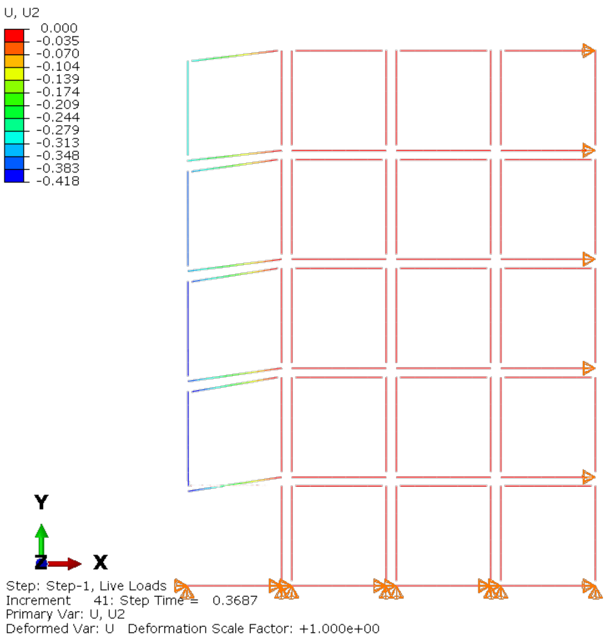
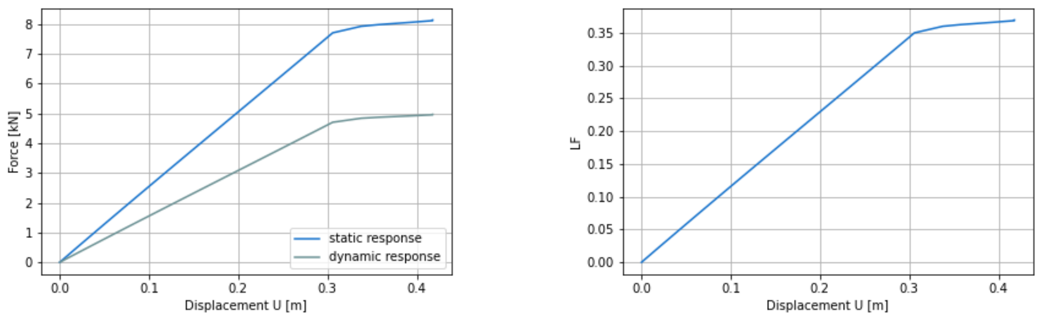


Figure 8.1: Vertical displacement [m] at failure NLS-analysis scenario A

Static response

Subfigure a) of 8.2 shows the response curve of the push-down analysis. On the y-axis, the force in kN is displayed, and on the x-axis, the vertical displacement at the location of the removed column in meters. In the subfigure, two lines are presented, one is the static response, and the other is the pseudo-static response curve, i.e. dynamic response, see section 5.3.1. The dynamic response results from the strain-energy balance, derived with equation 5.4. The ratio between the maximum push-down load $Pstat$ and the dynamic load $Pdyn$ that can be sustained by the structure, see equation 5.2, indicates the dynamic load increase and gives an indication of the structures dynamic amplification factor, see Section 5.2. For this specific scenario, the DAF is found to be 1.63. Meaning that in case a static analysis will be performed, loads should be increased by 1.63 to account for the dynamic effects due to the sudden reduction of internal forces.



(a) pseudo static response

(b) load factor to gravity loads

Figure 8.2: Push-down response curves scenario A

Subfigure b) of Figure 8.2, displays the load factor (LF) of the applied gravity load vs the vertical displacement at the location of the removed column. The LF is obtained as the ratio of $Pstat/Pdesign$. Essentially it is the same curve as the static response of a), the LF, however, tells us what the difference is between the applied load in the analysis and the design gravity loads. From this curve it can be observed that the LF at failure is lower than 1. That means the structure can not resist its design load for this removal scenario because it already fails at a lower load.

Failure initiation
Figure 8.3, allows us to understand the reason why the load displacement curve in Figure 8.2 reaches a plastic plateau, and ultimately fails. Under increasing gravity loads the moment in the floor beam - column connection (intra-modular) increases linearly until a vertical displacement at the removed column of around 0.3 meter. At that point the moment resistance of the connection is reached and the connection starts to yield. Under increasing plastic deformation the structure will continue to display downwards until a displacement of 0.42 meter. At that displacement the rotation in the FB-Col connection reaches the ductility limit of 0.15 rad, the stiffness of the moment connection is then decreased until zero and the analysis stops.

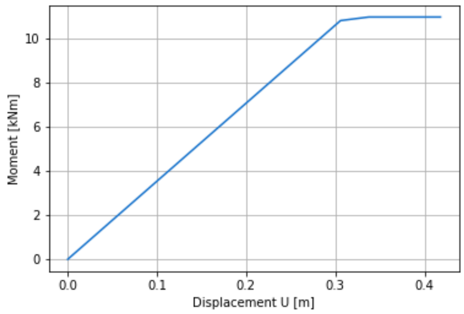


Figure 8.3: Moment response curve of the floorbeam - column connection

Time history response
Besides performing a nonlinear static analysis on the analysis model, also a nonlinear (dynamic) time history analysis is performed. The result of that analysis, for the vertical displacement at the location of the removed column, is presented in Figure 8.4. It can be seen that the displacement reaches a peak of more then 2.5 meters downwards, indicating that the structure must have failed. That can be explained with Figure 8.5, where the moment rotation curve of the intra-modular floor-beam - column connection is plotted. In the figure, it can be seen that connector plastically deforms beyond the ductility limit of 0.15 rad. The local failure in the connection results in a collapse at global structural level since no other alternative load path is present to distribute the load, see Figure 8.6. In that figure the displacement output of the dynamic analysis is presented, it can be observed that a whole vertical stack of 4 modules, above the module with the removed column have collapsed.

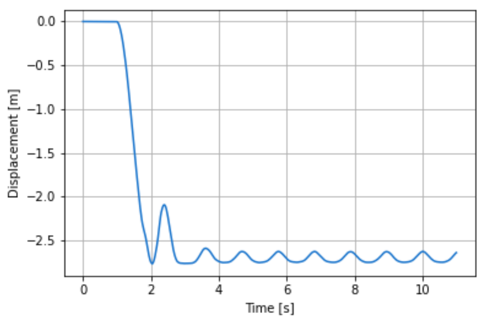


Figure 8.4: Time history response at node removed column scenario A

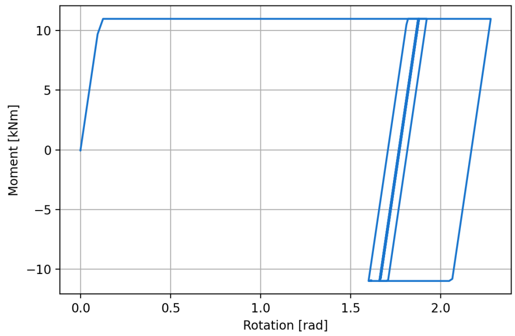


Figure 8.5: Time history response for moment and rotation of intra-modular connection

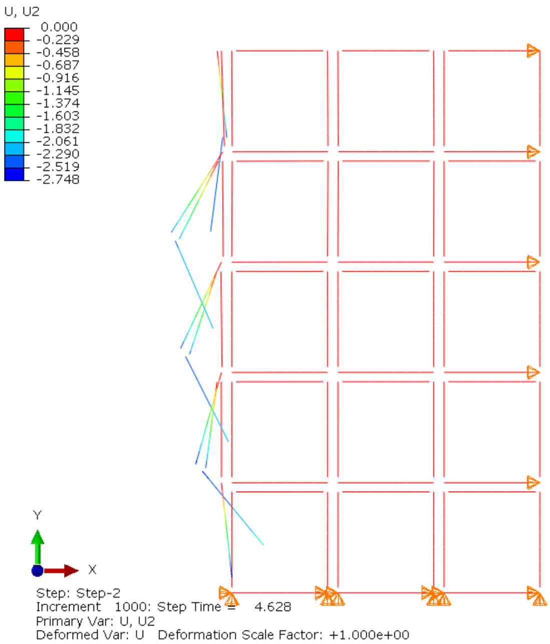


Figure 8.6: Collapsed area in time history analysis scenario A

8.2.2. Scenario B

For scenario B a complete module is removed from the analysis model as can be seen in Figure 8.7 below. The figure displays the final vertical displacement output of the analysis with a colour plot.

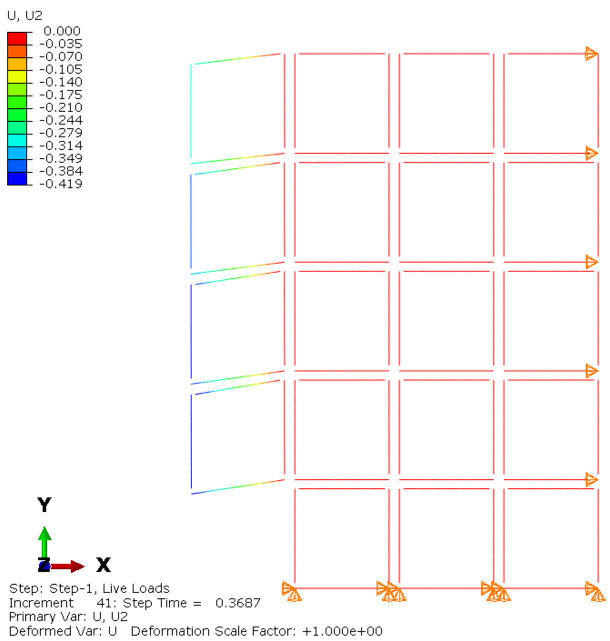
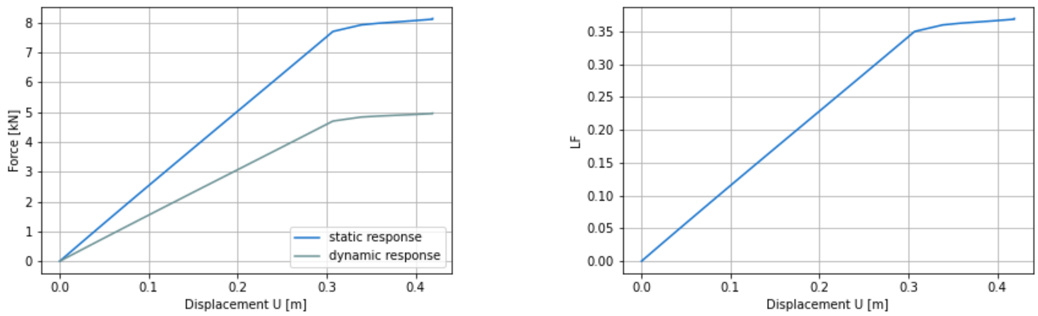


Figure 8.7: Vertical displacement [m] at failure NLS-analysis scenario B

Static response
The pushdown curve for this analysis is presented in Figure 8.8 (a). The resistance in this scenario's is the same as in scenario A. For this scenario the ratio between the dynamic response and maximum push-down load yields a DAF of 1.63.



(a) pseudo static response

(b) load factor to gravity loads

Figure 8.8: Pushdown response curves scenario B

Failure initiation
Likewise, as in scenario A collapse of the structure is induced by the failure of the floor-beam-column connection, from which the response curve is presented in Figure 8.9, allows us to understand the reason why the load displacement curve in Figure 8.2. Under increasing gravity loads the moment in the floor beam - column connection (intra-modular) increases linearly until a vertical displacement at the removed column of around 0.3 meter. Then the connection yields and deforms plastically under increasing load until the final ductility limit of the connection.

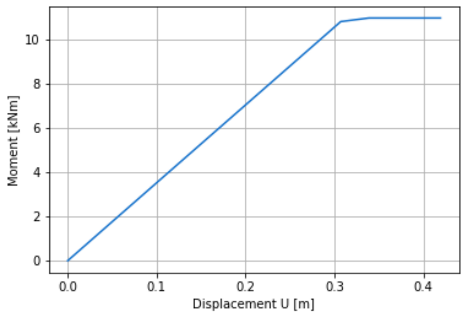


Figure 8.9: Failure initiation scenario B

Time history response

Again a time history analysis is performed from which the response is presented in Figure 8.4. It can be seen that the displacement reaches a peak of more then 2.5 meters downwards, it indicates that the structure has failed. The reason that the analysis continues is due to the fact that the connector is modelled as ideal elastic-plastic without ductility limit.

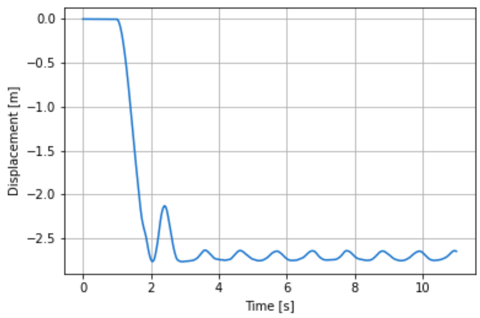


Figure 8.10: Time history response at node removed column scenario B

8.2.3. Scenario C

For removal scenario C, two intermediate columns are removed from the analysis model. The columns are neighbouring components from two different modules. In contrast to the other models, an extra group of horizontal supports is assigned along the left vertical edge. That is done to allow the development of a catenary force in the model. The model can be seen in Figure 8.11 below, it displays the final vertical displacement output of the static analysis with a colour plot.

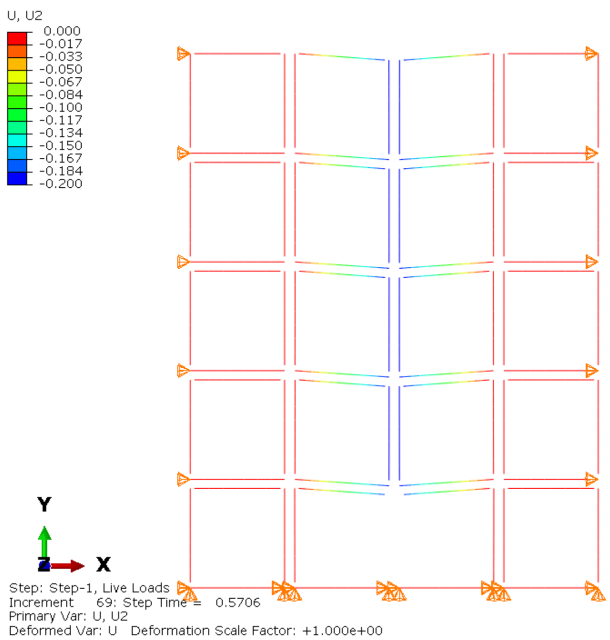
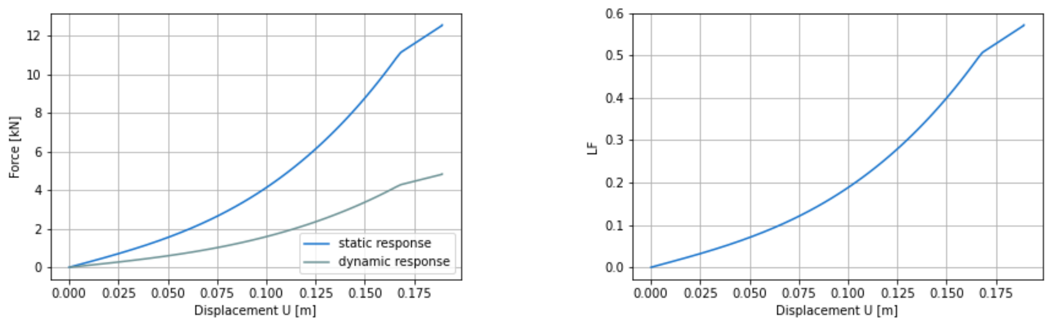


Figure 8.11: Vertical displacement [m] at failure NLS-analysis scenario C

Static response
The results of the push-down analysis are displayed in Figure 8.12. The displacement in this case is measured at the location of left removed column. It can be observed that the structure initially behaves linearly until a certain vertical displacement of 0.025 meters. Consecutively under increasing gravity loads the displacement rate decreases, so a stiffness increase in the response can be observed. At a certain displacement of 0.165 meters, a decrease in stiffness occurs and the response continues linearly until the end of the analysis where a displacement of 0.2 meters is reached. The ratio between the maximum push-down load $Pstat$ that can be sustained by the structure and the dynamic push-down load $Pdyn$ yields a DAF value of 2.6.



(a) pseudo static response

(b) load factor to gravity loads

Figure 8.12: Push-down response curves scenario C

To understand the response curve from scenario C better, the analysis output for the section moment and section normal force are presented in Figures 8.13 & 8.14. It can be observed that the structure and its inter-modular connection facilitate the development of a catenary mechanism with the modules' floor beams in tension, see the red-coloured horizontal beams. The activation of the catenary is the reason the response curve is showing a stiffening, the efficiency of the catenary is increasing with increasing displacement. Additionally, the moment resistance in the intra-modular connections also contributes to some extent to the resistance of the structure, though the contribution is minimal since the moments in the connection have not yet reached their full resistance.

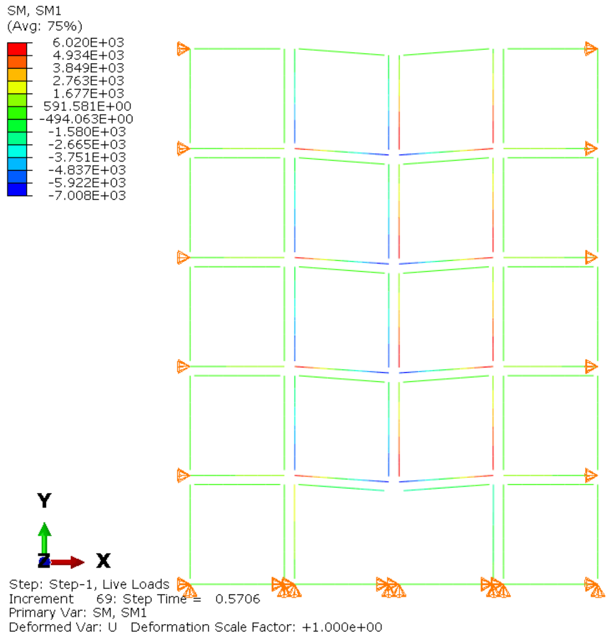


Figure 8.13: Section moments scenario C

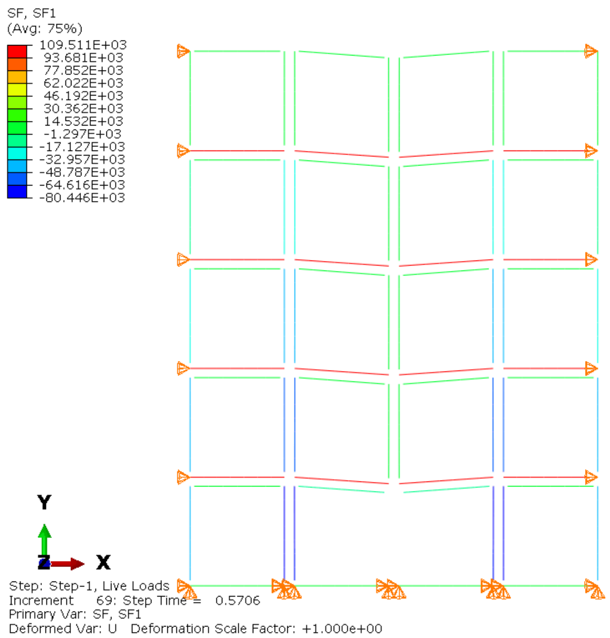


Figure 8.14: Section normal forces scenario C

Failure initiation

Figure 8.15, represents the axial load response curve in the inter-modular connection, above the removed columns, see Figure 7.11. In the figure, on the y-axis the axial normal force is normalised by the connections yield force as presented in Table 7.8. The x-axis contains the vertical displacement at the location of the removed columns. The curve allows us to understand the push-down response of scenario C. It can be observed that the normal force increases exponentially with increasing displacements, that phase of increase lasts until the axial tensile resistance of the inter-modular connection is reached, and that connection starts to yield. After yielding the phase of plasticity last until a vertical displacement of 0.19 meter, at that point, the ductility limit of the connection in axial elongation is reached and the analysis is stopped.

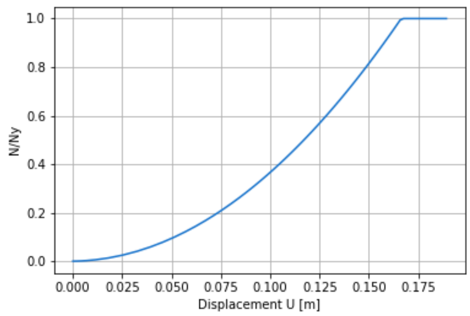


Figure 8.15: Failure inter-modular connection C

Time history response

The time history response for the vertical displacement at the location of the left removed column, for scenario C, is presented in Figure 8.16. Quite rapidly, the structure displaces downwards until a displacement of 1.66 meters is reached, the global minimum. At this moment, the structure has already failed. That can be explained by Figure 8.17, in subfigure (a) the time history moment response of the intra-modular floor beam-column connection is displayed it can be seen that the resistance will be reached, then a phase of plasticity occurs after which the stiffness reduces to zero, i.e. failure. Subfigure (b) contains the normal force of the axial horizontal component of the inter-modular connection. From that response, it can also be observed that the resistance is reached, after which a phase of plasticity starts. The response then continues, which indicates that the catenary is still active. However, in this analysis, the ductility limit for axial elongation was not set, it was checked and appeared to must have failed too.

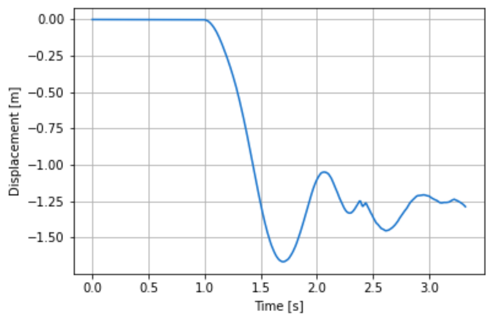
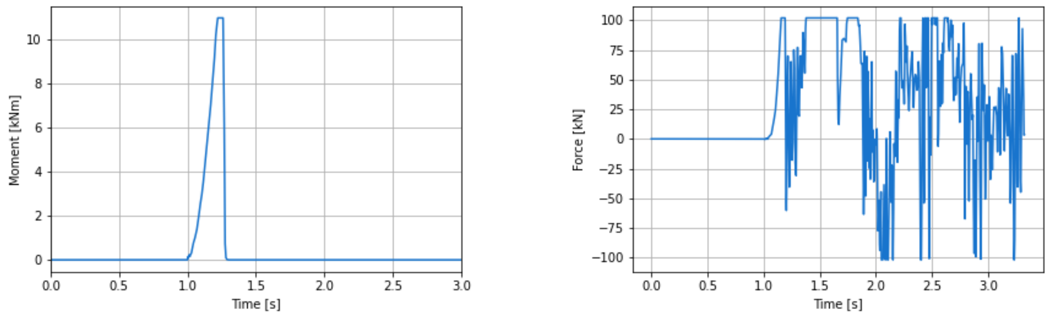


Figure 8.16: Time history response at node removed column scenario C



(a) moment response floorbeam-column connection **(b)** normal force response inter-modular connection

Figure 8.17: Failure initiation and ultimate failure scenario C

8.2.4. Scenario D

For scenario D, a full intermediate module is removed from the analysis model, it represents a total module loss, as can be seen in Figure 8.18. The figure displays the final vertical displacement output of the analysis with a colour plot.

Static response

The pushdown curve for scenario d is presented in Figure 8.19 (a). The push-down curve is bi-linear, and failure in the analysis for the applied load is not reached. The apparent increase in stiffness after reaching LF = 1, see subfigure (b), is due to the fact that additional loads are only applied to the modules above the damaged area. Since the inter-modular connector then is only additionally loaded at one of its ends, the displacement rate reduces with increasing force. Apparently, for this scenario, do the inter-modular connections have enough resistance to transfer the load to another load path. The resulting forces due to the gravity loads would initially flow vertically downwards through the columns towards the foundation. Now they find a new path through shear in the inter-modular connections to a neighbouring column.

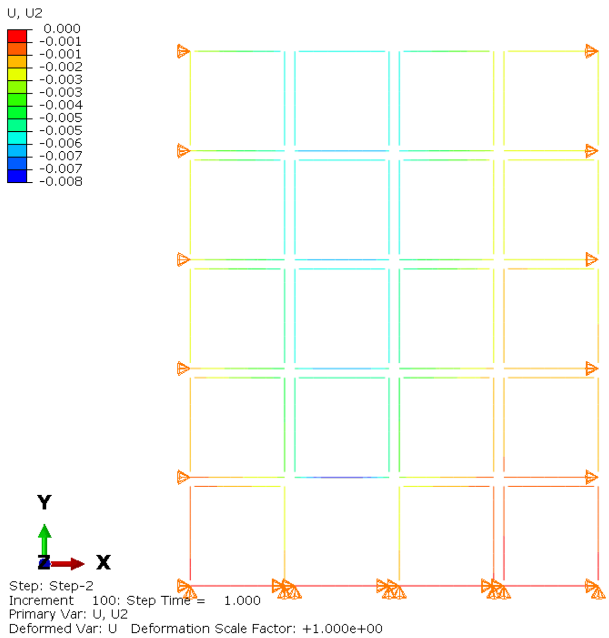
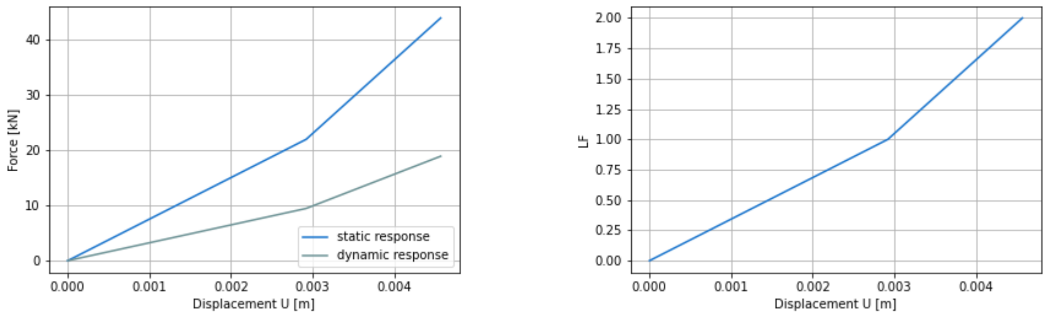


Figure 8.18: Vertical displacement [m] at failure NLS-analysis scenario D



(a) pseudo static response

(b) load factor to gravity loads

Figure 8.19: Push-down response curves scenario D

Time history response

The time history response for the vertical displacement at the location of the left column of the removed module for scenario C, is presented in Figure 8.20. It can be observed that the final response stabilises around 0.003 meters. The structure in this scenario doesn't fail.

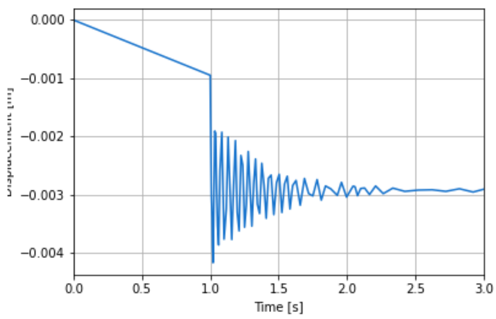


Figure 8.20: time history response at node removed column scenario D

8.2.5. Scenario E

For removal scenario E, a corner column is removed from the second analysis model, see Chapter 7. Figure 8.21 below displays the vertical displacement in a colour plot at the end of the analysis. It can be seen in the figure that the structure is constrained in the horizontal direction at the right side of the modules. Furthermore can be observed that the floor- and ceiling beams are merged together in the output, that is due to the fact of tying in Abaqus. With a tie constraint, the beams work together in flexural behaviour, but the nodes release their initial position.

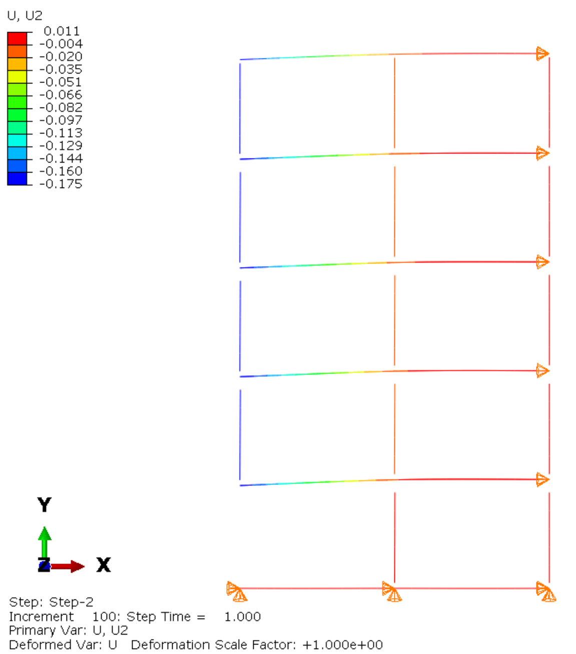
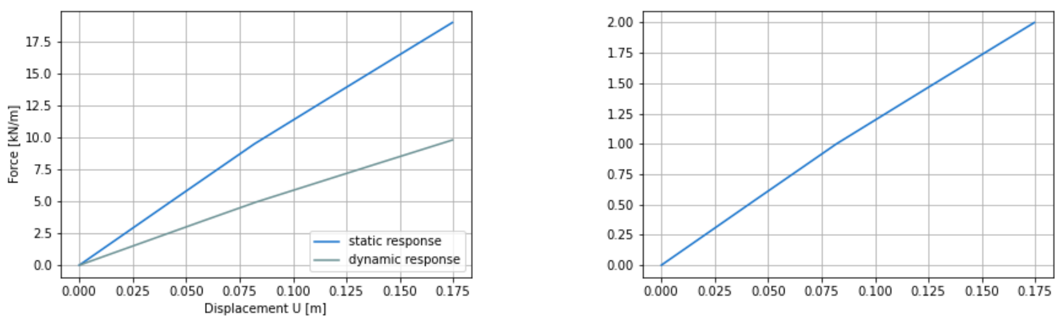


Figure 8.21: Vertical displacement [m] at failure NLS-analysis scenario E

Static response
Figure 8.22 shows the response curve of the push-down analysis. It can be observed that the structure initially behaves linearly until a LF of 1. After this point, a small decrease in stiffness occurs, which can be explained due to the fact that additional loads are applied only above the damaged area. At the end of the analysis, twice the design load is applied above the damaged area, and no failure has occurred in any of the connections. Because the structure was modelled with a linear elastic material model, the stresses need to be checked. For LF=2 the largest bending stress of 26.2 N/mm² occurs at midspan above the middle column of the bottom module. Finally, with the result of the strain-energy balance and the maximum push-down load $Pstat$ that can be sustained by the structure, a DAF value of 1.93 is found.



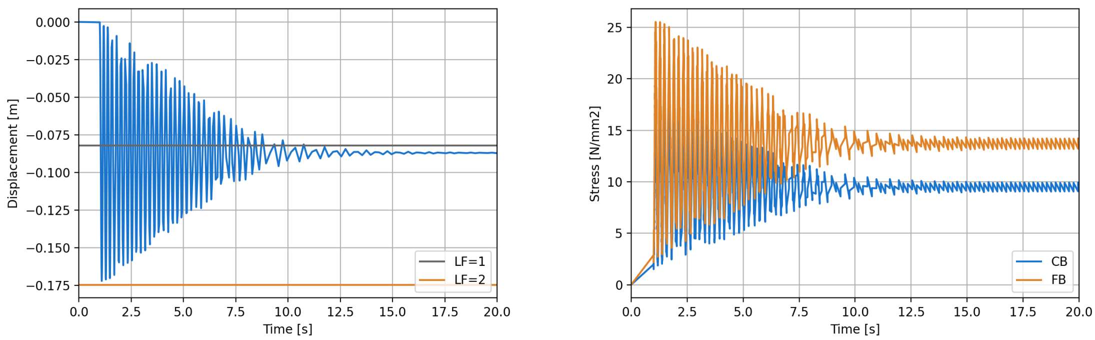
(a) pseudo static response **(b)** load factor to gravity loads

Figure 8.22: Push-down response curves scenario E

Time history response
The time history response for the removal of an edge (corner) column in analysis model 2 is presented in Figure 8.23. The figure consists of two plots, on the left in subfigure (a), the vertical displacement at the location of the removed column is presented, and subfigure (b) shows the flexural stress at the midpoint of the floor and ceiling beam immediately above the middle column of the bottom module.

From the vertical displacement response, it can be observed that after the sudden loss of the column, the structure makes a steep drop downwards to almost -0.175 meters, then it freely vibrates until a new equilibrium position is found at -0.085 meters. The ratio of the maximum dynamic displacement Δ_{dy} to the static displacement Δ_{st} results in a DAF of 2.09. In the figure also two horizontal lines are presented, LF=1 and LF=2, they correspond to the displacement in the static response for that factor. It can be seen that the peak displacements in the dynamic response are lower than the static response for LF=2, but the final response Δ_{fin} of the dynamic analysis is larger than the vertical displacement Δ_{st} of the static analysis for LF=1.

Since the material model of the analysis is linear elastic (the plasticity comes from the connections only) an extra check is necessary to determine if this structure survives or fails within this scenario. Therefore, an extra check is done on the material stresses. From subfigure 8.23 (b) it can be observed that the peak bending stresses, just like the peak displacements, occur immediately after the removal of the element. The floor beam experiences the largest bending stress from both beams, with a value of 25.5 N/mm². Since, for an accidental load case, the allowed bending stress for a GLT24 member is 26.4 N/mm², see subsection 7.2.2, the stress is still below the allowable limit. In this case, the alternative load path is governed by the flexural action of the beams.



(a) vertical displacement **(b)** bending stresses

Figure 8.23: Time history responses for scenario E

8.2.6. Scenario F

For removal scenario F, a middle column is removed from the second analysis model, see Chapter 7. Figure 8.24 below displays the vertical displacement in a colour plot at the end of the analysis.

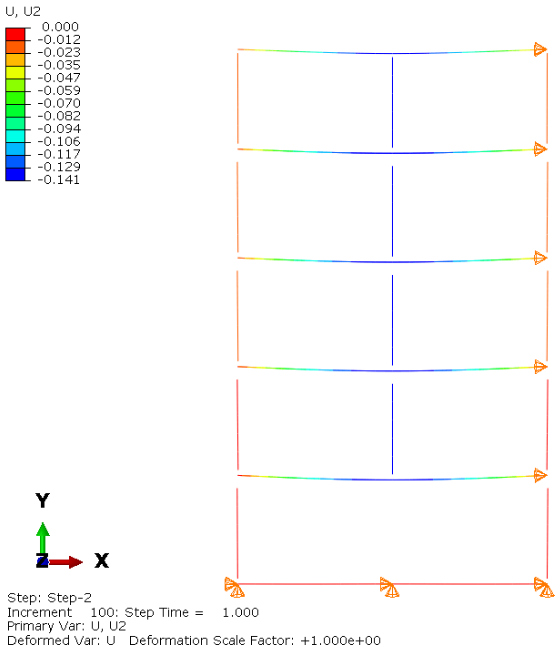
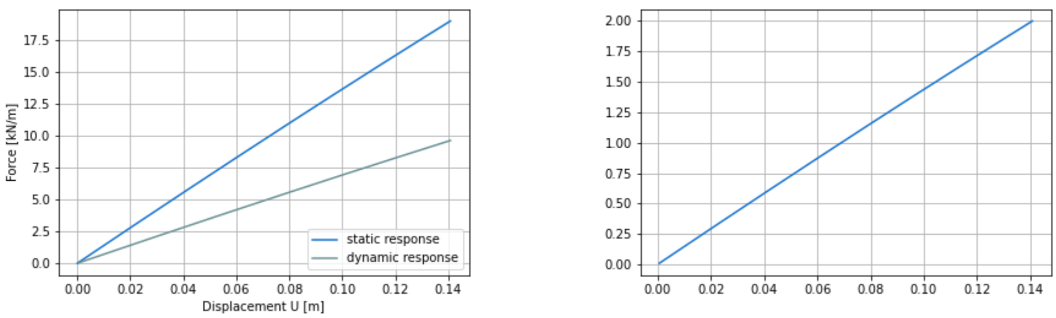


Figure 8.24: Vertical displacement [m] at failure NLS-analysis scenario F

Static response

Figure 8.25 shows the response curve of the push-down analysis. It can be observed that the structure behaves linearly until a LF of 2. No failure was observed in the connections, however, again, an additional check is necessary on the material stresses. For LF=2 the largest bending stress of 30.2 N/mm² occurs at midspan above the middle column of the bottom module. That stress is larger than the allowed bending stress for a GLT24 member of 26.4 N/mm² for this design case. That stress limit was already exceeded by a LF of 1.74. Finally, the result of the strain-energy balance and the maximum push-down load that can be sustained by the structure ($Pstat$), leads to a DAF value of 1.97.



(a) pseudo static response **(b)** load factor to gravity loads

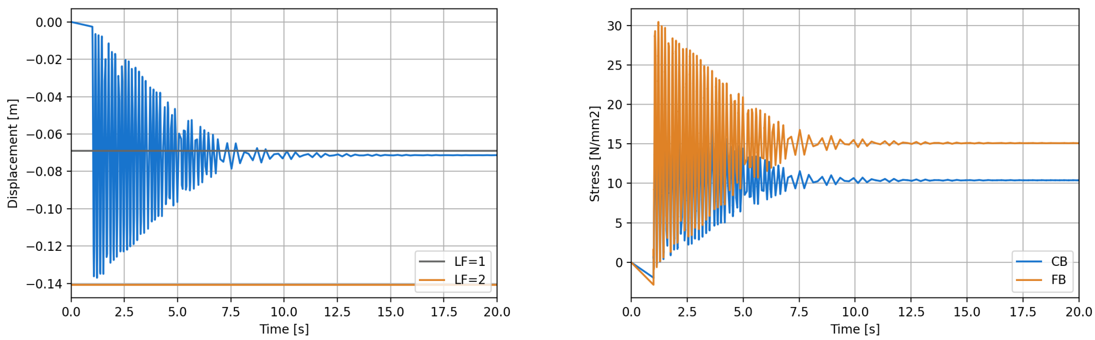
Figure 8.25: Pushdown response curves scenario F

Time history response

The time history response for the removal of a corner column in analysis model 2 is presented in Figure 8.26. The figure consists of two plots. On the left in subfigure (a), the vertical displacement at the location of the removed column is presented. On the right, subfigure (b) shows the flexural stress at the midpoint of the floor and ceiling beam immediately above the middle column of the bottom module.

From the displacement time-history response, it can be observed that after the sudden loss of the middle column, the largest peaks in the displacement occur. Again the structure freely vibrates until a new equilibrium position is found at -0.073 meter. The ratio of the maximum dynamic displacement Δ_{dy} to the static displacement Δ_{st} results in a DAF of 1.89. From the figure, it can be seen that the peak displacements in the dynamic response are lower than the static response for LF=1. The final equilibrium displacement, however, Δ_{fin} of the dynamic analysis is slightly (1 mm) larger than the displacement Δ_{st} of the static analysis for LF=1.

From subfigure 8.26 (b) it can be observed that again the peak bending stresses occur immediately after the removal of the element. The results show that the maximum stress in the ceiling beam is 21 N/mm², and the floor reaches a value of 30.4 N/mm². Therefore, the floor beam experiences the largest bending stress, and the ALS stress limit is exceeded by 15 %. Additionally, the stresses in the floor beams directly above the bottom module reach values of up to 29.5 N/mm² for the top module, indicating that the damage progresses until the top storey.



(a) vertical displacement

(b) bending stresses

Figure 8.26: Time history responses for scenario F

8.3. Results of parameter study

From the initial analyses for the different removal scenarios, it was observed that rotational stiffness and resistance of the intra-modular beam-to-column connections are mainly decisive for the resistance of the system. In this subsection, a parameter study is performed to investigate which properties of that connection especially contribute the most to the development of an alternative load path and to a more robust design. Accordingly, the study is performed by varying the rotational strength and stiffness of the intra-modular floor-beam to column connections in the model. Again the same analysis models are used as earlier, i.e. analysis model 1 for scenarios A & C and analysis model 2 for scenario F, see chapter 7. Additionally, for removal scenario F, the influence of certain connection properties on the system's dynamic amplification factor is treated.

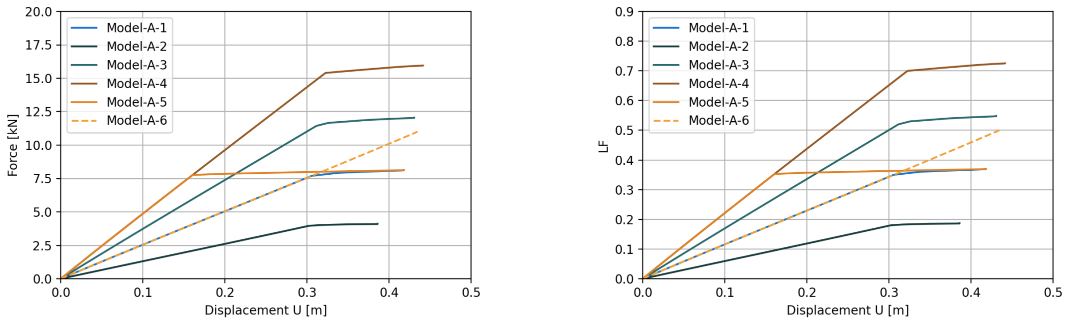
8.3.1. Scenario A

For failure scenario A, the parameters of the respective stiffness and strength values which are used, are presented in Tables 8.1. In the tables, K_0 and R_0, represent the initial values that result from the connection design, see Table 7.5 in Section 7.3.

Table 8.1: Floorbeam-column connection variants - analysis model 1

Variant	Stiffness [K]	Resistance [R]
1	K_0	R_0
2	0.5 K_0	0.5 R_0
3	1.5 K_0	1.5 R_0
4	2 K_0	$2R_0$
5	2 K_0	1 R_0
6	1 K_0	2 R_0

The push-down curves from the parameter study of scenario A are presented in Figure 8.27. Excluding Model-A6, do all response curves consist of two branches. They've all an initial linear path until yielding, and then, a nonlinear path in the plastic region which continues until the rotational ductility limit of the connection is met. The yielding in the curves corresponds to the moment resistance of the connections (R). It can be observed that the rotational resistance of the intra-modular connection is of main influence on the resistance of the system for this specific removal scenario. Variant 3, with an increase of 50 % for K_0 and R_0, results in a 49% increase in the static resistance of the system. Similarly, variant 4, with twice the initial stiffness and resistance results in a 96% increase of the static resistance. Variation in the rotational stiffness of the connection results in a longer or shorter plastic region. When we compare models A-1 and A-5, we see that the linear displacement is halved in A-5, corresponding to the stiffness increase of 2. However, an increase in rotational stiffness only doesn't affect the resistance of the system, it is still the same as for the initial case, i.e model-A-1. On the other hand, increasing only the rotational resistance results in an increase of resistance of the system so does Variant A-6 with $2R_0$ result in a 35% increase of resistance of the system.

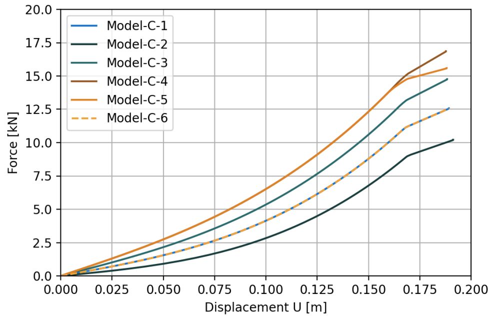


(a) static response curves **(b)** load factor to gravity loads

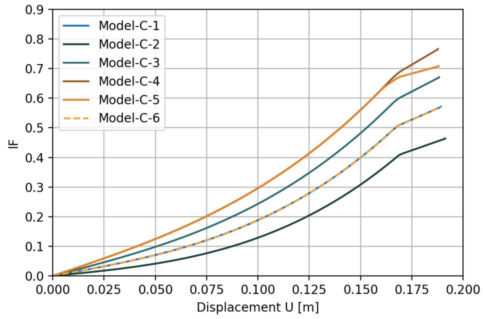
Figure 8.27: Push-down curves parameter study scenario A

8.3.2. Scenario C

For failure scenario C, the same parameters as for scenario A are used, see Table 8.1 above. The push-down curves from the parameter study of scenario C are presented in Figure 8.28. All the response curves of the different models show a transition into a linear path around the same downward deflection of almost 0.17 meters. At that vertical displacement, the inter-modular connection starts to yield. What follows is a linear path provided by the plastic axial elongation of the connection, it stops when the ductility limit of the inter-modular connection is reached, see Chapter 7. From the results, it can be observed that a variation in the properties of the floor-beam-to-column connection influences the resistance of the system. Model C-2 with a reduction of 50% for K_0 and R_0, results in an 18.8 % decrease of the static resistance of the system. Models with increased rotational stiffness, variants C-3, C-4 and C-5 all show an increase in resistance of the system. Variant C-4 with twice K_0 and R_0 results in a 34% increase of the static resistance of the system. If we compare model C-5 with C-6, we observe that when only the stiffness is increased by a factor 2, the static resistance of the system is increased by 24 %. But if R_0, on the other hand, is increased by a factor of 2, the resistance of the system is not affected and the result is the same as for variant C-1, see the dashed line in the plot.



(a) static response curves



(b) load factor to gravity loads

Figure 8.28: push-down curves parameter study scenario

Conclusion/discussion: The results of this parameter study, indicate that the resistance of the system in this removal scenario is governed by the development of a catenary as an alternative load path. The resistance can be increased by increasing the properties of the intra-modular connection. Particularly, an increase in rotational stiffness has a positive influence on the maximum vertical load that can be sustained, which can be seen when models C-4 and C-5 are compared. Nevertheless improving the intra-modular connection alone is not enough to provide an effective alternative load path. The catenary action is developing but it's not sufficient enough for preventing collapse because the load that can be sustained is smaller than the design load.

8.3.3. Scenario F

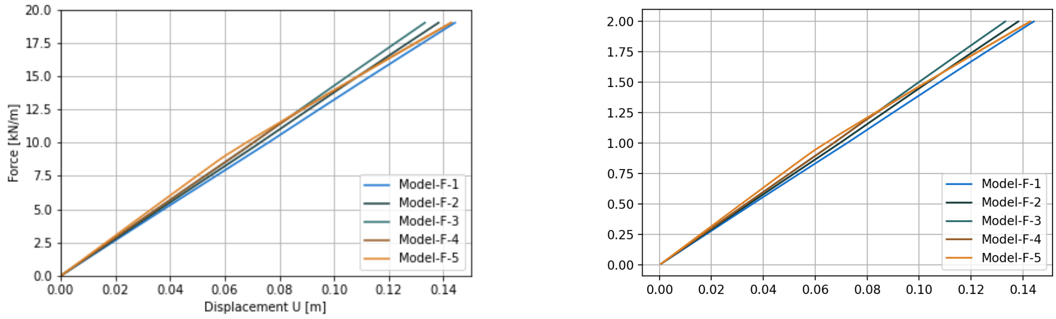
For failure scenario F, the parameters of the respective stiffness and strength values which are used, are presented in Table 8.2. In the tables, K_0 and R_0, represent the initial values used and presented earlier for the floor-beam-column connection in this model see section 7.3. For this scenario in contrast to A and C, also a time history analysis is carried out.

Table 8.2: Floorbeam-column connection variants - analysis model 2

Variant	Stiffness [K]	Resistance [R]
1	K_0	R_0
2	1.5 K_0	1.5 R_0
3	2 K_0	$2R_0$
4	2 K_0	1 R_0
5	3 K_0	1 R_0

Static response

The push-down curves resulting from the nonlinear static push-down analyses for the parameter study of scenario F are presented in Figure 8.29. From the results, it can be observed that a variation in the properties of the floor-beam-column connection barely influences the static response of the system. Increasing the rotational stiffness K_0 by a factor of 2.0 results in a midspan deflection decrease of 7 % for a LF of 1, consecutively for LF=2, the deflection is only decreased by 1%. Increasing both K_0 and R_0 by a factor of 2.0 results in a 7% decrease of deflection for both LF=1 and LF=2.

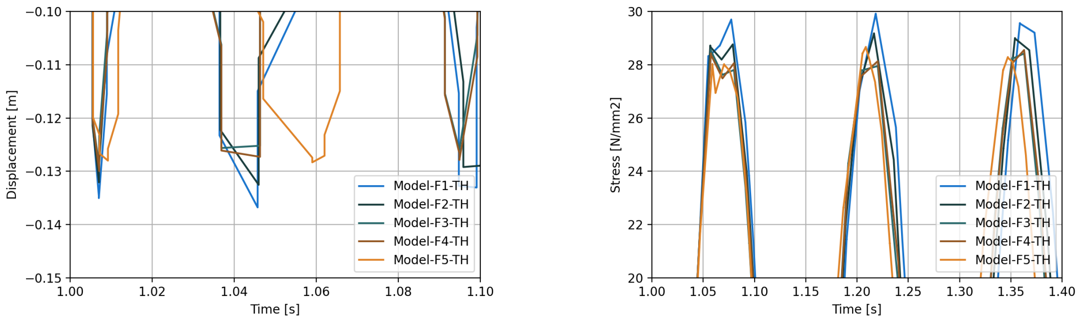


(a) static response curves **(b)** load factor to gravity loads

Figure 8.29: Push-down curves parameter study scenario F

Time history response

The initial result of removal scenario F learnt us that the alternative load path should be provided by the flexural action of the beam. Nonetheless, that proved to be an ineffective mechanism since the beam would fail due to the high bending stresses. In this parameter study, it is explored whether an improved connection can result in a robust design. The time history response curve peaks for scenario F are presented in Figure 8.30. Both the peak downward displacements and peak bending stresses in the floor beam at mid-span are displayed so that the differences can be seen more clearly. It can be observed that the peak responses decrease with increasing rotational stiffness and resistance. The peak dynamic displacements (Δ_{dy}) and final displacements (Δ_{fyn}) of the analyses are presented in Table 8.3. The results show that a doubling of the initial values for K_0 and R_0, see F-3, results in a 5.8 % decrease in peak downward displacement and a 5.4% decrease in bending stress. In the case that only the rotational stiffness K_0 is increased by a factor of 2.0 but the resistance is kept the same, i.e model F-4, it results in a 5.2 % decrease in peak displacement and 6.2 % decrease in maximum stress compared to model F-1. However, for the variants where only the rotational stiffness K_0 is increased the final displacement in the equilibrium state increases again. For example model F-5 shows a 2.4 % increase in final displacement compared to F-1.



(a) Peak displacements **(b)** Peak stresses

Figure 8.30: Time history results for parameter study scenario F

Table 8.3: Results at mid-span for parameter study F

Model	Δ_{st} [m]	Δ_{dy} [m]	Δ_{fin} [m]	σ_{max} [N/mm²]
F-1	0.072	0.137	0.073	30.44
F-2	0.069	0.133	0.069	29.18
F-3	0.067	0.129	0.068	28.81
F-4	0.067	0.130	0.071	28.55
F-5	0.064	0.128	0.074	28.67

Dynamic amplificaton factor
With the results of the static and time history analyses, the influence of the connection parameters on the dynamic amplification factor of the structural system can be determined. Chapter 5, subsection 5.2 presented different definitions for the DAF. Table 8.31 below summarizes them and presents them with a name, i.e. DAF-1, 2 and 3. Those names will be used in the description of the results below.

Table 8.4: Dynamic amplification factors

DAF-1	$\frac{\Delta_{dy}}{\Delta_{fin}}$
DAF-2	$\frac{\Delta_{dy}}{\Delta_{st}}$
DAF-3	$\frac{P_{stat}}{P_{dyn}}$

The resulting DAFs for scenario F can be seen in Figure 8.31. As presented above, three different dynamic amplification factors are shown per variant of the parameter study. It can be observed that initially, the definitions of DAF-1 and DAF-2 lead to almost the same results, having less than 1% deviation for variants F-1 and F-2. Interestingly, the results of F1, F2, and F3 suggest that the force-based dynamic amplification factor, i.e. DAF-3, seems to overestimate the (true) value resulting from the dynamical model by approximately 4.5%. Furthermore, do the results show that when both stiffness and resistance of the connection are increased, DAF-2 and DAF-3 show a slight increase. However, the results are ambiguous when we look at scenarios F4 and F5. There, the results show that when R_0 is kept unchanged but K_0 is increased, the definitions DAF-1 and DAF-3 lead to a decreased dynamic amplification in contrast to DAF-2, which shows an increase. For example take variant F-4 and compare it to the initial design F-1, the definitions for DAF-1 and DAF-3 result in a reduction of approximately 4 %, DAF-2 increases on the other hand by 2 %

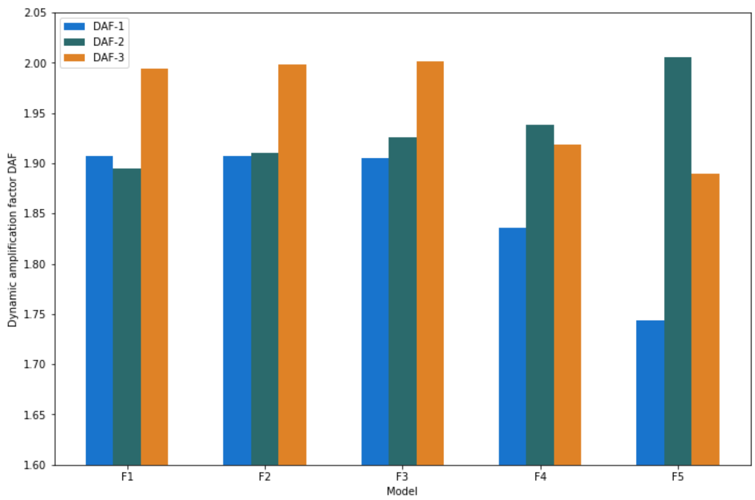


Figure 8.31: Different resulting dynamic amplification factors of variants F

8.4. Summary of the results

Initial design
The results of different removal scenarios, treating the initial design of the modular system, show that alternative load paths in the 2D frames do not develop sufficiently. First, scenarios A and B, treating the removal of a corner column and corner module, respectively, show that the load-carrying capacity of the system after the removal is governed by the intra-modular floor beam-column connection. However, that connection's rotational resistance is insufficient to provide enough resistance against collapse since only 37 % of the design load can be resisted (LF=0.37).

Secondly, the results of scenario C, where two intermediate columns are removed from the model, show that the system transitions from a stage of bending and shear into a catenary stage where a normal force develops in the floor beams and inter-modular connection. The catenary development results in an increase in stiffness, nevertheless, that alternative load is not enough to provide a robust structure because only 57 % of the design load can be resisted. Failure of the inter-modular connection in tension due to the yielding of the plate, there, limits the resistance capacity of the alternative load path.

Thirdly, the results of Scenario D, where a complete module is removed from the model, show that the structure is capable of arresting progressive collapse. The 30 mm thick steel plates of the inter-modular connections facilitate a bridge allowing the vertical force to be transferred to a neighbouring column.

Subsequently, the results of removal scenario E, where a corner column is removed in analysis model 2, show that an alternative load path is present, governed by the flexural action of the beams. The beams cantilever over the middle column, resulting in a large bending moment above that column, leading to a bending stress of 25.5 N/mm².

Finally, the results of removal scenario F, where the middle column is removed in analysis model 2, show that there is no sufficient alternative load path. Resistance against collapse should be provided by the flexural action of the beams. However, the bending stresses in the floor beams above the removed column exceed the acceptable limits by up to 15%.

Parameter study

Adjustments to the rotational- stiffness and resistance properties of the intra-modular connection showed that a sufficient alternative load path could not be achieved with the studied parameters. Speaking of analysis model 1, on the one hand, more considerable rotational resistance will improve the resistance of the structure in scenario A. On the other hand, more rotational stiffness contributes the most to an increase in resistance of scenario C.

Additionally, in scenario F, the results show that the dynamic amplification factor of the system is affected by the rotational stiffness and resistance of the connections. An increase of 100% of both properties results in an 0.6 % increase of the dynamic amplification factor (taking the mean of the three definitions). However, the resulting DAFs for variants F-4 and F-5 are ambiguous, showing a two-way trend of an increase in DAF-2 but a decrease in DAF-1 and DAF-3.

Part III

Research Outcome

9
Discussion

9.1. Discussing the modelling approach and assumptions

Two-dimensional frame approach

This thesis assumes that the 3D volumetric modular structure can be analysed for robustness in a 2D frame approach. There are several reasons why this approach is taken and why it is a valid assumption. First of all, if you directly consider the 3D geometry, you lose insight into the individual behaviours of the failure mechanisms and connections. Therefore, analysing in a 2D setup can provide more insightful conclusions for the system as a whole. Secondly, it allows using conventional analytical calculation methods conveniently used in practical engineering to quantify the behaviour and validate the results. Thirdly, a 2D frame approach is more beneficial for the fast analysis of various variants since the computational time of a 2D model significantly reduces compared to a 3D approach. However, in a 2D setup, you lose potential beneficial resistance mechanisms since the load can only transfer in one direction. For example, in a 3D setup, also the floor and ceiling beams would have been modelled; they can redistribute the load to the undamaged parts through potentially developing membrane action. Additionally, a 2D approach can be seen as more conservative. Still, that doesn't negatively influence any conclusions about the robustness since if a structure in a 2D setup is considered safe, then in 3D, it is too. Nevertheless, the author recommends studying the structure in a 3D setup in a follow-up study. It would be interesting to compare the results and investigate if multiple resistance mechanisms occur simultaneously or not, and if the structure's dynamic amplification differs from the current results.

Removal speed

Within this study, the notional removal scenarios are performed with an instantaneous loss of reaction forces. Instantaneous is a default option in Abaqus. The option was analysed on a simply supported beam, see subsection 7.9 and validated according to the theory and the results of a study by Mpidi Bita [24]. When quantifying the dynamic response, it was observed that this value corresponded to the upper bound response, i.e., a DAF of 2.0 was found. In contrast, the Unified Facilities Criteria (UFC-4-023-03) and GSA guidelines define a specific speed of t = 0.001 seconds as instantaneous. It was found that that is a slower removal speed because the dynamic amplification is reduced by 2% on a linear elastic system. The chosen default value in Abaqus may be a bit too conservative, yet the difference is so small it can be omitted.

Connections

- To utilises the full rotational capability of the intra-modular connections, an assumption was made that the connections would be able to rotate up to 0.15 rad, a value which was found in a review study on experimental tests of ductile moment resisting timber connection by Reboucas et al. [27]. Additionally, that value was also found in the GSA guidelines. The authors suggested that for timber connections in a monotonic test, a value of up to 0.15 rad is achievable. However, none of the review's connection set-ups matches this study's connection design, so it is questionable if that value is non-conservative. Nevertheless, that value serves as a ductility limit and only influences the dynamic results following the energy balance of the non-linear static analysis. From those analyses, we learned that the moment resistance of the intra-modular connection was not enough to arrest collapse in some specific removal scenarios. Those scenarios also failed in the dynamic analyses, so no estimate of the dynamic amplification based on the deformations could be made. Therefore, we can conclude that we should be cautious when looking at the predicted dynamic response of scenarios A and B.

- The resistance of the connections is governed by the weakest link in the connection chain. Within the inter-modular connection, it is questionable if all parts take up an equal force or if some parts are more loaded than others. Currently, the weakest element in that connection chain when considering a tie force is the tensile resistance of the net cross-section of the plate. The shear resistance of the screw group is just slightly larger by 1.2 %. For the screws, it is assumed that all screws take up an equivalent force, though possibly the contribution is not equal but varies. In case a sudden loss of two intermediate columns happens in reality, it could be that the tensile force which develops in the beams is unequally spread amongst the screws. The screws closest to the end of the plate could then start to yield, and like a domino effect, the other screws yield too. Such behaviour will ensure that the screws enter the stage of plasticity. That means that possibly for the current resistance, the ductility and possible elongation would be larger, which would be beneficial because the load-carrying capacity of the catenary would then increase. For example, with the screws of this study, that may result in 13 mm extra elongation. The effect of this extra elongation is not quantified in the models of this study, and it's recommended to analyse that in another study.
- The modules of the studied structure are stacked on top of each other and don't have a tensile connection nor a specific shear connection which prevents them from sliding. Load is transferred vertically downwards by bearing in compression and horizontally by inter-modular connections that connect the floor beams of adjacent modules. Therefore, friction mainly prevents the modules stacked onto each other from sliding. In this study, material contact could not be accounted for in the FEM models. To avoid numerical singularity, the FEM model required a horizontal spring; for that spring, an assumption was made for the stiffness of 100 kN/m. This is a low value and safe assumption. It is smaller than the stiffness of one screw in the inter-modular connection and allows the models to run.

9.2. Discussing the results

9.2.1. Initial results

The results of different removal events, treating the initial design of the modular building system, are derived by considering multiple alternative load path scenarios as introduced in Chapter 7. The results; see Chapter 8 showed that alternative load paths in the 2D frames do not develop sufficiently for scenarios A, B, C and F, whilst they do in scenarios D and E. This is presented Figure 9.1 below, where the successful alternative load paths scenarios are presented in green colour.

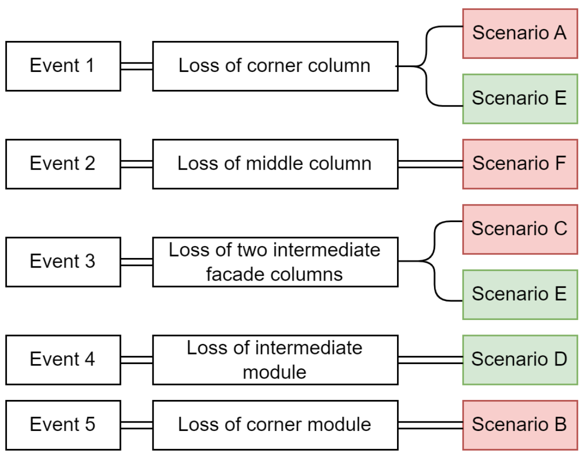


Figure 9.1: Removal events with successful scenarios

If a scenario identifies a successful alternative load path, it can be concluded that the structure is robust enough to arrest progressive collapse for a certain removal event. However, if a certain scenario doesn't identify a sufficient alternative load path, this does not directly mean that the structure is not robust. Remember that the 3D geometry was decomposed into two 2D frames; see section 7.4. If, for example, events 1 and 3 are considered, then although scenarios A & C don't identify a sufficient alternative load path, the structure is still robust because resistance is met in scenario E. Once again, the results for scenario E, indicated that collapse could be prevented by the cantilevering beam. It provided a sufficient alternative load path through flexural action. Additionally, it can be argued that, although scenario E alone is sufficient, the results can be seen as conservative since scenarios A and C showed that load could also be resisted in the other direction, though only up to a certain extent. Under those circumstances, only one removal event doesn't identify a sufficient ALP, i.e. scenario F. That scenario is explained in the paragraph below.

Scenario F

The initial results for removal scenario F, where a middle column is removed from the long axis of the model, showed that bending stresses in the floor beams of the modules exceeded the stress limit, indicating a progressive failure and insufficient ALP. In additionally to the initial analysis, a parameter study was performed on this scenario to study the effects of different connection properties on the dynamic response. Consecutively, dynamic amplification factors were estimated with different definitions, and they showed that differences occur when connections enter the phase of plasticity in the models. However, there is a problem with the conclusions for this scenario. The results are derived with a linear elastic material model for the timber, and since the stresses exceeded the limit, it is questionable if the dynamic response of the model is correct because if the limit is exceeded, the beam should have failed, and the dynamic response would then be different. However, the used bending strength limit is this research is the characteristic (5th-percentile) strength. That means 95 % of the GL24h specimens have a larger strength. In other studies considering robustness, mean strength values are commonly used because when quantifying the probability of collapse, characteristic values are too conservative. In a study by Huber, to derive the mean strength, the characteristic values were multiplied by a factor of 1.49, assuming a typical coefficient of variation (CoV) of 20%. Voulpiotis, in his study, multiplied the characteristic values by 1.2 to estimate the mean based on a lognormal distribution with a CoV of 15%. If the same is done for this study, then the stresses in the floor beams following the middle column's removal will still be acceptable. As a result of that, it can be concluded that the dynamic response of the model is reliable and that the DAF estimates are valid. Furthermore, it could then be concluded that the alternative load path for scenario F is sufficient, resulting that a progressive failure will not occur. Given this, it can be argued whether the structure can be seen as completely robust for all the considered events. Still, the author decided to stick with the 5th-percentile limit for indicating a failure, to be conservative.

Scenario B and D

From the results of the scenarios, it can be observed that the failure in scenario B is similar to scenario A; no differences occur. Yet, more elements, i.e. a corner module of the modular structure is removed. Similarly, for scenario D, a complete module is removed from the structure, yet the system can arrest collapse. That asks for an explanation. A reason why that occurs is due to the way the inter-modular connection is modelled. In the inter-modular connection, the 30 mm thick steel plate can facilitate a bridge for the force to be transferred to the other modules; see Figure 9.5 below. In the Abaqus model, the plate is modelled as rigid and clamped in the neighbouring columns. The connecting floor beams can not transfer a moment through the plate, but a force directly on the plate can. In appendix B, a calculation is presented which demonstrates the action of forces and how resistance is achieved.

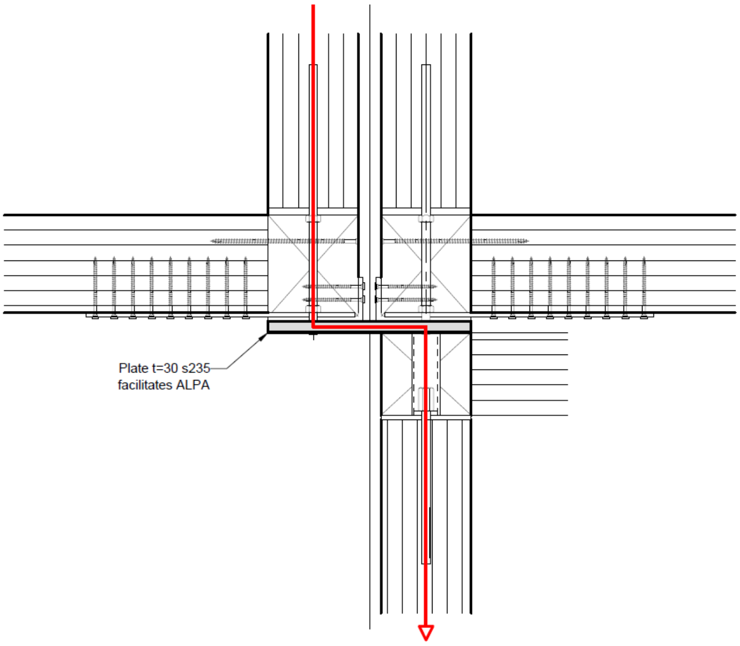


Figure 9.2: Alternative load path of steel plate in the inter-modular connection

DAF

The disparity of the estimates for the dynamic amplification factor, considering variants F-4 and F-5 in contrast to F-1 till F-3, still asks for an explanation. It can possibly be explained due to the permanent displacement resulting from the plastic rotation in the floor beam-column connection. Figure 9.3 presents the moment rotation response in that connection resulting from the dynamic analysis. It can be seen that variants F-4 and F-5 enter plasticity. The response for a load factor of 1, in the static analyses, for the same variants see Figure 9.4 show that the rotation in F-5 entered plasticity and F-4 is still in the linear stage. The plastic straining results in an increase of the peak response in the models. Therefore, does the definition for DAF-2 show an increase for increased plasticity. That is counter-intuitive because the common perception is that plasticity (ductility) decreases the dynamic response since energy dissipates. That was also confirmed by the measured stresses in the models see section 8.3. Therefore, probably the definition for DAF-2 can not be used to quantify the dynamic amplification in models with plasticity. Nevertheless, that needs to be further explored for more variations of the parameters.

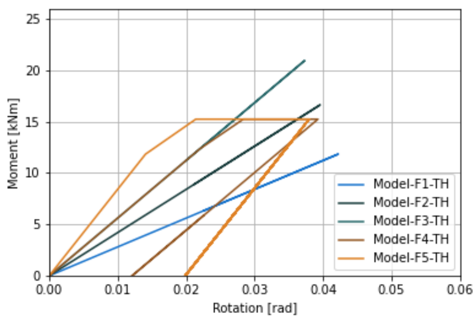
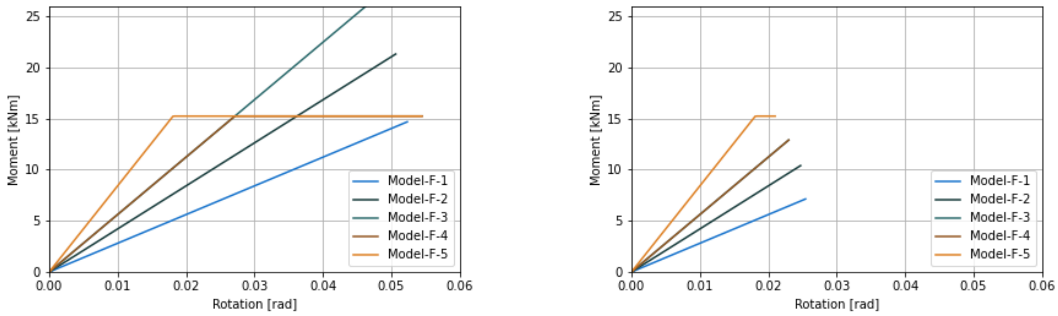


Figure 9.3: Time history results for moment-rotation floor beam-column connection of parameter study scenario F



(a) Moment-rotation until end analysis

(b) Moment-rotation until DAF = 1

Figure 9.4: Moment-rotation output for floor beam-column connection of variants F - NLS

9.3. ALPs and design implications

1. The results of the study indicate that for some removal events, resistance against collapse should be provided with a flexural mechanism, i.e. the bending of the beams facilitates an alternative load path. Generally, a flexural resistance mechanism is considered not the ideal resisting mechanism because it asks for over-dimensioning of the material to guarantee enough resistance. Nevertheless, a change in layout, such as shorter spans or extra columns, could also improve the design. Alternatively, a literature study pointed out that applying braces, for example, in modular frames, could significantly improve the robustness design. Unfortunately, such a design alteration would negatively affect the possibility of creating open apartments with the modules and hampers the flexibility of the design.
2. The results of the study indicate that catenary action cannot develop sufficiently with the inter-modular connection as initially designed. However, it can be improved. For example, allowing for more elongation could result that the axial force required to make resistance could be lowered. Another way is increasing the tensile resistance, though this rather asks for the components to be made stronger, i.e. bigger dimensions of the plate, screws, bolts etc. Therefore it is worthwhile to explore how more elongation could be achieved. An example is presented in Figure 9.5, creating a fuse in the tie-plate, which may plastically strain could potentially develop the desired catenary. The paragraph which follows explores that potential.

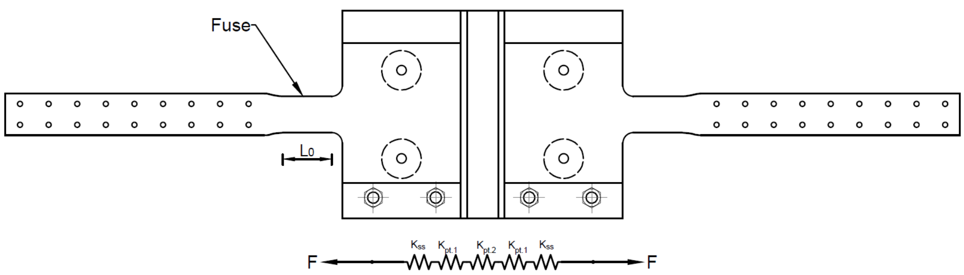


Figure 9.5: Potential solution for inter-modular connection

Required elongation

Figure 9.6, presents a 2D schematic representation of a catenary. The horizontal tie-force can be derived as follows:

$$F_t \cdot \Delta = \frac{q \cdot {L_2}^2}{8} \tag{9.1}$$

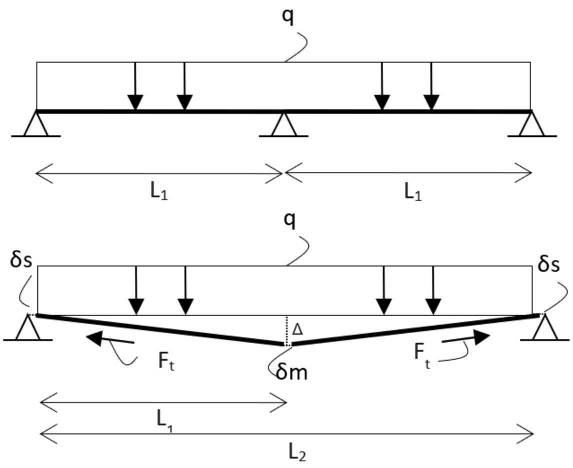


Figure 9.6: Schematic representation of catenary action

With trigonometry, the compatibility between vertical and horizontal deformations can be derived where $\delta_L = \delta_s + \delta_m$ is the elongation based on a single span (L1) only.

$$\delta_{\mathrm{L}} = \sqrt{L_1^2 + \Delta^2} - L_1 \tag{9.2}$$

If a force is taken of Q= 15 kN/m, representative of the design loads, length L1 = 3 meters and the initial tensile resistance of the catenary as 110 kN. The required mid-span displacement Δ results in 0.61 m. From equation 9.2, follows the required elongation, which is δ_l = 62 mm. With a fuse in the inter-modular connection as suggested above and presented in Figure 9.2, 31 mm elongation needs to be accomplished on both sides. Assuming a maximum failure strain of 20 % in S235 steel, the required fuse length will be 155 mm. Nevertheless, the rotation required to achieve a mid-span displacement of 0.61 meters on this span is 0.20 rad, which is a large rotation, and it's questionable if that is achievable with the current connection design. For future work, it is recommended to analyse this further.

10 Conclusions and recommendations

This thesis was initiated with the desire to advance the current academic knowledge on the robustness of modular timber buildings. To do that, a state-of-the-art review on the robustness of modular and timber structures was performed, which initiated the understanding and gathered the desired knowledge of the topic. To narrow down the scope of the research project, a decision was made to investigate volumetric timber post and beam modules. With the help of a case study, which provided the structural design concept of the study, 2D finite element analysis models were developed. Consecutively, based on a notional element removal concept, multiple possible losses of load-bearing elements were considered to examine the structural robustness of a modular timber building. More specifically, this was done by identifying and quantifying the development of alternative load paths necessary to arrest progressive collapse. Additionally, different analysis methods were discussed, and the validity of the theoretical methods and the new proposed robustness prescription for Eurocode 5 were analysed. Based on the results of this work, the main research question can be answered.

To what extent can a timber building constructed out of volumetric post and beam modules develop sufficient alternative load paths to prevent disproportionate collapse in case of notional element removal?

Core observations and recommendations are presented below.

10.1. Conclusions

Alternative load paths

- It was found that the notional removal of an intermediate (middle) column in the long side of a volumetric timber post and beam module is a critical scenario since the alternative load path in this removal case is governed by a single alternative load path, i.e. the cantilever action of the floor and ceiling beams. For this study's volumetric post and beam module, the removal led to an exceedance of the bending stress limit in the floor beams up to the top storey by a maximum of 13 %, indicating a potential progressive collapse to an extent disproportionate to the failure cause. However, the alternative load path can be improved by providing more rotational stiffness and resistance in the connections of the floor beams, the parameter study revealed that will reduce the peak deflections and stresses.
- In the event that a corner column of a modular timber building is lost. Resistance against collapse is provided by two alternative load paths. Firstly, a sufficient alternative load path, capable of redistributing the full accidental load, develops on the long side of the modules, governed by the cantilever action of the floor and ceiling beams. Secondly, in the short side of a module, up to 36% of the accidental design load can be resisted due to the moment resistance in the intra-modular connection. However, such flexural robustness mechanisms are generally considered not the ideal resistance mechanism, so caution should be made that the beams and connections provide enough bending resistance. Nevertheless, it can be concluded that the structure of the case study

is capable of developing sufficient alternative load paths to arrest collapse when a corner column of the structure is lost.

- In the event that two neighbouring columns from the short sides of two modules are simultaneously removed, resistance against collapse is provided by two alternative load paths. First, similarly to the removal of a corner column, a sufficient alternative load path develops on the long side of the modules, governed by the cantilever action of the floor and ceiling beams. Secondly, with the initial design of the inter-modular connection of this study, it can be concluded that the system can resist 57% of the design load in the direction of the short sides of the modules due to the development of a catenary.
- In the event that a complete intermediate module is removed from the structure, resistance against collapse is provided by the inter-modular connections. The plates facilitate a support bridge and transfer the load to a neighbouring column where resistance is met through moment equilibrium.
- Due to the absence of vertical ties, the upper stories in the modular building can not contribute to the robustness of the building. The results, therefore, show that alternative load paths only develop horizontally with flexural mechanisms and catenary action.

Connection properties

- The behaviour of the connections in a modular building system is of crucial importance to quantify the resistance against progressive collapse in the direction of the short sides of the modules. The connections provide resistance by allowing moments to develop in the frames and by transferring a tensile force in the connecting beams to create a horizontal tie, enabling the possibility of developing a catenary. The study revealed that developing sufficient alternative load paths mostly depends on the rotational resistance, the tensile resistance, the ductility and the axial stiffness plus possible elongation of the connections. Aspects which are mainly affected by the steel parts of the connections when considering a timber structure.
- Rotational stiffness and resistance in intra-modular connections influence the alternative load path resistance and dynamic response of a modular system. Based on a notional element removal concept, it was found that increased rotational stiffness in the connections increases the dynamic amplification and resistance of an alternative load path. However, it also led to a reduction of the peak displacements and bending stresses at mid-span in the floor and ceiling beams; a doubling of the initial rotational stiffness and resistance resulted in a reduction of up to 5.8 % of peak stresses and displacement.

Dynamic behaviour

- The results of the study show that the dynamic amplification factor for a modular timber system is very close to the upper bound of 2.0. It was found that the factor depends on the stiffness and resistance of the connections, the removal speed and the removal scenario. It can be concluded that the newly proposed prescription for the DAF, i.e. 2.0, in the working draft of Eurocode 5 for robustness is valid and not too conservative.
- Specifically for the initial structure of this study, considering removal scenario F, a DAF value of 1.90 was found. The value is the result of the ratio between the maximum dynamic displacement Δ_{dy} to the static displacement Δ_{st} following a nonlinear quasi-static and nonlinear dynamic time history analysis, respectively, with an instantaneous removal speed of column.
- The force-based dynamic amplification factor, i.e. the ratio between P_{stat} and P_{dyn} seemed to overestimate the definitions based on the dynamic displacement slightly. It was found that for the structures with connections that remain in the linear stage, the definition, on average, led to a 4.5 % larger value. However, in cases where the connections showed plastic straining, the results for the dynamic amplification factor are ambiguous.
- Rotational ductility influences the dynamic behaviour of the modular system. Models with connections that entered the plastic stage showed a decrease in bending stresses of up to 6.2%. Regarding the dynamic amplification factor of the modular system. It was discovered that more ductility capacity resulted in more energy dissipation due to plastic straining, which led to a growing imbalance between the different dynamic amplification definitions.

Analysis procedures

- The nonlinear static analysis procedure with energy balance, as initially proposed by Izzudin et al. [13], can provide a reliable method to incorporate dynamic effects without the use of a dynamic amplification factor. The results of this study indicate that the dynamic amplification factors derived with that method seemed to slightly overestimate the factor which follows from the dynamic models by, on average 4.5 %. However, that is just an initial indication, and further analysis will be necessary to justify that.

10.2. Recommendations

Based on the research of this thesis, several recommendations are formulated:

- This study is carried out on the essential assumption that a 3D structural system can be decomposed into equivalent 2D planar subsystems for performing the analyses. Consequently, the structural integrity is affected, and the results of the analyses are only an approximation of reality. Essentially, some load-distributing mechanisms, which potentially improve the structural robustness, could not be considered; for example, the contribution of the floor and ceiling slabs and the contact behaviour between the elements could not be accounted for. Therefore, a study considering the complete 3D structure is advised. An illustration considering the full 3D model is provided in Figure 7.13.
- In this research, it was found that quantifying the stiffness and strength of timber connections is rather complicated. Timber connections are complex, and they are still a relatively undeveloped topic compared to their counterparts in traditional materials such as concrete and steel. Especially for novel modular connections of recent innovations like the structure of this study, not much is known. Therefore, several assumptions have been made, and spring models were developed to quantify the connections' stiffness, resistance and ductility. It is advised to analyse further the connections in an experimental study to prove the validity of the models and derive exact properties because they significantly influence the structure's robustness, influencing the alternative load path development and dynamic amplification factor. Alternatively, a further variation of the currently used variables may be necessary to improve the dynamic models and make stronger conclusions.
- In this research, the material model for the timber elements was chosen to be linear elastic so that the plasticity of the structure is lumped at the connections. It would be interesting to model the structure with an inelastic material model to study the differences.
- In this study, an attempt was made to quantify the dynamic amplification factor of a volumetric timber modular building system. It was concluded that a reliable estimation of the factor is troublesome and not adequately well understood. Especially when ductility is provided by the connection, a disparity between the different definitions for the DAF occurs. Further research is necessary in this area to develop suitable dynamic amplification factors for static analysis taking into account the ductility of the connections and mechanisms of resistance needed to prevent collapse.
- It was concluded that the speed of removal of a load-bearing element influences the structure's dynamic behaviour. The Unified Facilities Criteria (UFC-4-023-03) and GSA guideline prescribe a specific rate of t = 0.001 seconds, yet in this study, the used removal speed was instantaneous as default in Abaqus. That results in an estimation which can be seen as an absolute upper bound and conservative. Therefore, it is advised to study the effect of the prescribed removal speed on the structure since it may benefit the collapse resistance.
- The analyses in this research were carried out without the use of damping, i.e structural damping, material damping, viscous damping or modal damping were omitted in the models. Yet damping is known to be beneficial for a structure's collapse resistance. It would be valuable to study to what extent damping influences the collapse resistance of the structure.
- The modular structural system in this study is built up by the stacking of modules The load transfers through the modules towards the foundation by bearing in compression. The modules don't have an inter-modular vertical tensile connection. This means that no alternative load paths are able to develop in the vertical direction. In the literature study, it was found that providing vertical ties is beneficial for the robustness of a modular structure, and it is therefore advised to perform a study which can investigate that influence.

- In the discussion, some ideas were presented to improve the connections. They may lead to improved robustness behaviour of the structure. Further analysis of the influence of those adjustments would be necessary to quantify to what extent they improve the robustness of the structure.

10.3. Recap Literature conclusions

Conclusion - Robustness

Robustness is an intrinsic property of a structure alone which enhances the global survival tolerance of a building to local failure. The general philosophy of a robust building is that it's capable of preventing the disproportionate spreading of failure, independent of the failure cause. To prevent disproportionate collapse, a structure should be able to develop so-called alternative load paths, yet other methods exist too, though they are not always applicable. Robustness-related aspects should be taken into account in the early stages of design, with designing for ductility, redundancy etc. Designing for robustness, therefore, is related to the structural concept, choice of structural materials, and structural detailing. Building codes demand buildings to be designed for robustness based on their consequence class. A higher class means the more severe the risk for loss of lives. Therefore, do the higher consequence classes demand more robustness aspects. Eventually, most international codes require for the higher classes to prove a certain level of robustness by performing a deterministic analysis. With such an analysis, the exact behaviour of a structure after a damage has occurred can be studied. Since the failure cause is unimportant, a scenario-independent approach should be used. In conclusion, should notional element removal be done, it provides the best method to quantify robustness since it can answer the question of whether alternative load paths exist and whether a building is robust.

Conclusion - Modular

- Modular buildings are most vulnerable for progressive collapse in case of the removal of corner modules or corner posts.
- Alternative load path development under column removal scenario of a modular building is highly dependent on the horizontal tie plates
- Floor slabs help to some extent, reduce the deformations of a floor beam and therefore reduce the development of a catenary mechanism.
- Membrane action is not able to develop in a modular building due to the discontinuity of the floor diaphragm between the modules.
- The shorter beams of the modules play a major role in the redistribution of loads in a column removal scenario.
- Rigid inter-modular connections perform better in progressive collapse analysis.
- Strength and rotational stiffness of inter-modular connections should be increased to improve global stability and provide constraint for building motion.
- Corner modules and posts should be given more attention since they are the most vulnerable members.
- Structural solutions that can be considered to enhance the robustness of such vulnerable members are strengthening the column, using bracing systems, and increasing the strength of the inter-module connections.
- Braces can create additional load paths, and the effectiveness of braces in structural robustness is highest when they are in the direction of building movement [32].
- The progressive collapse of modular buildings can be avoided by providing more inter-module connections. The connections between modular units of modular buildings play a significant role in developing alternative load paths during the initial load redistribution phase (more tie links between modules facilitate more load paths). However, the design of these ties must ensure the adequate transfer of load between modules such that undamaged parts fully receive the redistributed load.
- Multiple studies have shown that increasing the amount of modules per floor is an effective solution to enhance the structural robustness of a modular building, this would mean a modular building should become wider. This will ultimately increase the structural redundancy, and in that case, a more effective distribution of the collapse load can be obtained [32].

Conclusion - ALPA

For performing an alternative load path analysis, different analysis procedures are possible. Only some can study the true behaviour of a structure and others are very limited. To include the redistribution of forces and study alternative load path resisting mechanisms, a nonlinear procedure is necessary. That can be done with either a static procedure with dynamic load factors or a dynamic one. Many researchers use the nonlinear dynamic analysis to study the behaviour of a modular building in case of notional element removal. It is a complex and computationally expensive procedure but is most capable of capturing the true behaviour since it considers all relevant aspects and inherently includes dynamic effects. However, a less complex method, which also includes dynamic effects, and is beneficial for the use and identification of load distribution mechanisms with less computational effort required compared to the nonlinear dynamic analysis, is the nonlinear static pushover with simplified dynamic response [21], [11].

Part IV

References and Appendices

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A

Appendix - ALPA studies

A.1. Programs/Software

A.1.1. Studies on Modular Steel Structures

A study performed by Thai et al. used the nonlinear explicit dynamic software ANSYS Workbench with LS-DYNA. Beam and column members of the modules were modelled by explicit 3D hughes-liu beam element BEAM 161, in which co-rotational formulation and integration fibre model are respectively used to capture geometric nonlinearity due to large rotations and inelastic behaviour of material. The Rayleigh damping model with a damping ratio of 2% for steel structures was used based on AS/NZS. 1170.2 (2002c). The connections between modular units were also modelled by the beam element with its cross-section,stiffness and resistance taken from Luo et al. (2019). It should be noted that the floor slab is not modelled in this case in order to be consistent with the numerical model developed by Lou et al. (2019), which is used as a reference solution for verification purpose.

Chua et al performed a numerical study into the robustness of a steel modular building with a story range of up to 30 floors using nonlinear static and dynamic analysis. The performed the analyses with a line model by using the commercial software SAP2000. The robustness performance was analysed by sudden removal of first storey columns at various locations. Column removal was conducted because according to the authors it's more likely to occur under accidental events then module loss. The nonlinear static (NLS) or pushdown analysis can indirectly account for the dynamic effect by considering the dynamic amplification factor (DAF) and material nonlinearity without complex material modelling and time-consuming dynamic analysis [5].

To tie the adjecent modules the connection system as shown in Figure 4.4. The upper vertical and horizontal connections of the spring model are represented using translational link elements where the stiffness parameters of the vertical and horizontal connections of the joint can be easily determined using analytical methods. In this study, the stiffness parameters to be used as the input of the translational spring model (Figure 4.6 are adapted from Chua et al. [5]

A.1.2. Studies on Timber Structures

K. Voulpiotis Recently a PHD study was carried out into Robustness of Tall Timber Buildings in which a nonlinear dynamic procedure was used, since it is the most realistic analysis method to capture collapse behaviour [35]. It was done with the software Abaqus 2021 (Dessault systems simulia corporation). Abaqus allows control through Python language, which is beneficial for parametric studies like this one.

Voulpiotis used both both implicit (Abaqus/Standard) and explicit (Abaqus/Explicit) solvers for the analyses. Implicit solvers are excellent for smooth model responses, like static or (low speed) nonlinear dynamic analyses without material failure (e.g., normal structural analysis). Explicit solvers howevers are a suitable option when the response is non-smooth, like in dynamic analyses where convergence of the implicit solver is difficult or impossible due to element- contacts and failures (e.g., in the collapse of a structure) [Dassault Systemes Simulia Corporation, 2021b] [35].

Voulpiotis used custom connector elements in Abaqus. That is done by assigning properties of every degree of freedom to a wire between nodes. The elastic, plastic, and failure properties of a custom connection, can be applied in all degrees of freedom as wished. Figure A.1 shows an idealised elastic-plastic force-deformation behaviour of such a wire.

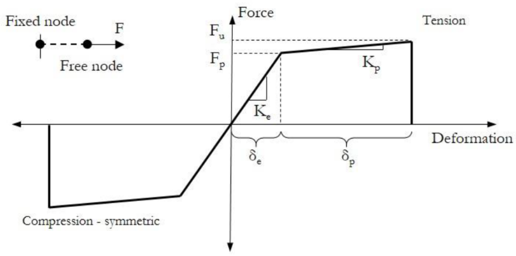


Figure A.1: Idealised connector behaviour for axial degree of freedom [35].

Connection properties can be estimated for all degrees of freedom by using the Eurocodes [EC5]. Each of the six curves (6 curves because 3 along axis and 3 around axis) is characterised by its elastic stiffness (K_e) yield load (F_y) plastic deformation (δ_u) and ultimate load (F_u).

Connection Voulpiotis, assumed symmetry in the negative direction. He mentions that although the behaviour is hysteretic, no pinching or fatigue have been modelled since the collapse analysis is short term dynamic and not long term cyclic. The degrees of freedom are assumed to be independent, i.e., no coupling is accounted for. All properties have been calculated in their design values, however the mean values have been used for the model. In the absence of design-to-mean value expressions, a factor of 1.2 has been applied to estimate the characteristic values, and another factor of 1.2 to estimate the mean values. These factors are determined on average over a number of design calculations without reduction factors, and assuming characteristic resistance values are 5th percentile values following a lognormaldistribution with coefficient of variation of 15% (in line with timber resistance probabilistic properties in part 3.05 of the JCSS code (2002)).

The shear behaviour of both screws and dowels is based on the European Yield Model (EYM) from Johansen (1949). A “mode 2” behaviour is assumed, with a slight ductility Ds = 1.5 according to SIA265:2012 part 6.1.2.3. That is, given elastic deformation ($delta_e$),the plastic deformation is given by ($delta_p$) = (Ds- 1) × $delta_e$ In the ductile phase slight kinematic hardening has been preferred over perfect plasticity, modelled as a 1% increase of the yield force at failure, i.e., F_u = 1.01 × F_y. The axial withdrawal behaviour of screws is assumed to be brittle, therefore failure happens suddenly at the yield load.

J. Huber Another recent study into robustness modelling of timber structures was performed by Huber. He also worked with the software package Abaqus and presents a very elaborate explanation of the custom connector elements in the software. Those elements can replace the mechanical behaviour of a more complex component in the model by constitutive relationships amongst several degrees of freedom. The connector elements can substitute screws, angle brackets or entire compartments of a building. In addition to Voulpiotis idealised connection behaviour, Huber implements a failure motion at the end of the plastic plateau where the stiffness progressively degrades, until total failure (rupture) occurs. According to Huber a too abrupt stiffness change, can jeopardise convergence [10], see Figure A.2.

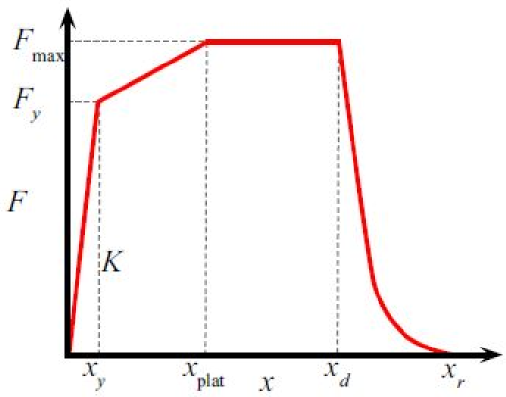


Figure A.2: Generic connector behaviour with failure motion [10].

Thai et al. In a study by Thai et al. [32] a nonlinear dynamic analysis was carried according to the prescriptions of General Service Administration (GSA i.e. the American code). To simulate the progressive collapse, a corner-supported module loss scenario as shown in Figure 4.7 was considered. In order to simulate the removed module, the static equivalent reactions from the initial static analysis were assigned at the removed position to represent the lost module. The equivalent reactions were suddenly reduced zero in 0.02 s to reflect the loss of a module.

Mpidi Bita Mpidi Bita [24], describes a procedure in which he even reduces the load removal time to 0.001 sec, that in turn lead to higher displacement due to the more sudden dynamic effects. He explains that the time history response of the structure is then captured and the the deformation demands on the timber (δ) and the connections on the bay surrounding the removed elements (Δ) are recorded and compared against the respective allowable deformations δlim for timber, and Δlim for connections.

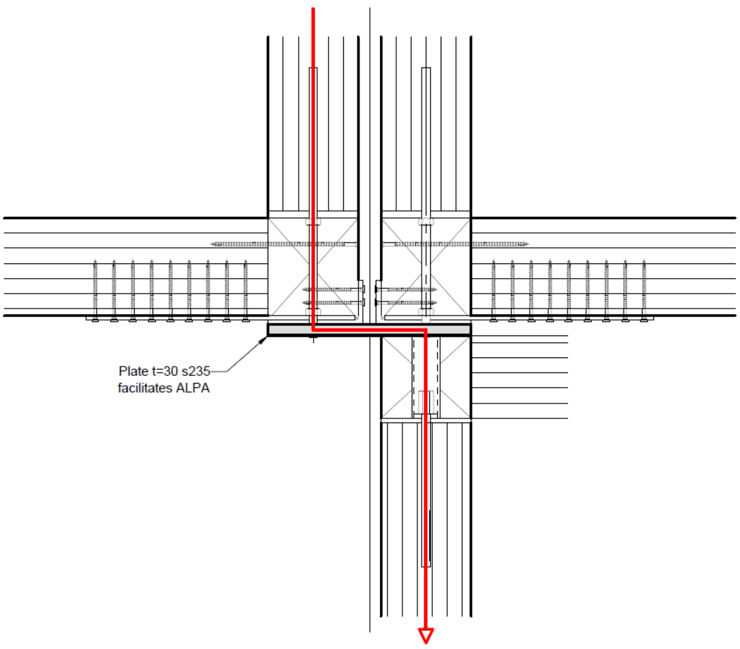
The deformation limit (δlim) can according to Mpidi Bita [24] be determined by deleting a vertical element from the building, without dynamic effect, and applying a vertical downward displacement at the same location until brittle failure (stress exceeding σ0 or σ90) was observed. Therefore, the exceeding of δlim represented a global failure.

B

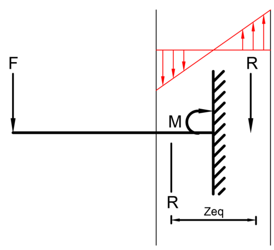
Appendix - Connections

B.1. Inter-modular connection

In Figure B.1 below, it can be seen how the force vertical force is transferred from one stack of modules to the neighbouring column. The moment capacity of the plate is enough to resist the moment which occurs due to the design load. For a plate of 320 mm wide and 30 mm thick, the section modulus is 48000 mm^3. The lever arm of the moment is the distance between the centres of the columns, i.e. 300 mm. Given a force of a load F=22kN, this results in a moment of 6.6 kNm and a unity check for the moment of UC= 0.59. To resist the moment in the beams and connected columns, a reaction force needs to develop in the timber perpendicular to the grain. The moment divided by the internal lever arm, i.e. Z$_{eq}$ = 120 mm, results in a compression force of 55 kN. The allowable compression force, given the dimension of the beam and column, is 159 kN. It can be concluded that the alternative load path is sufficient since the equilibrium is met. However, when we account for a dynamic load factor of 2.0 the stress in the plate exceeds the capacity, because UC = 1.17. Nevertheless, the plastic section modules can be used, following $1/4 \cdot b \cdot h^2$ instead of $1/6 \cdot b \cdot h^2$, resulting in a UC of 0.78 and we see it is sufficient. Therefore it can be concluded that the alternative load path also for the upperbound of 2.0 sufficiently works.



(a) connection flow of force



(b) equilibrium

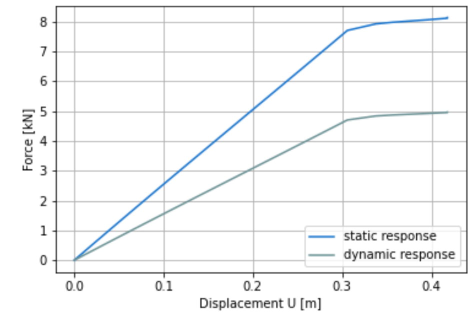
Figure B.1: Alternative load path of steel plate in the inter-modular connection

C

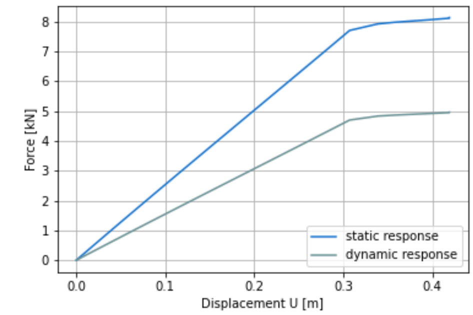
Appendix - ALPA results

C.1. Initial results removal scenarios a till f

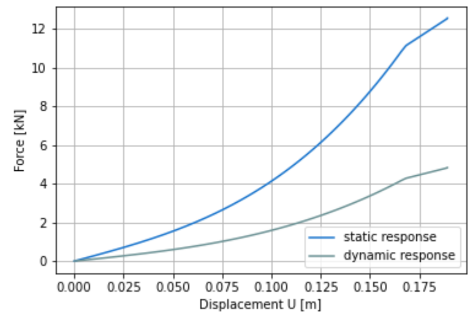
C.1.1. Push down response curves - nonlinear static



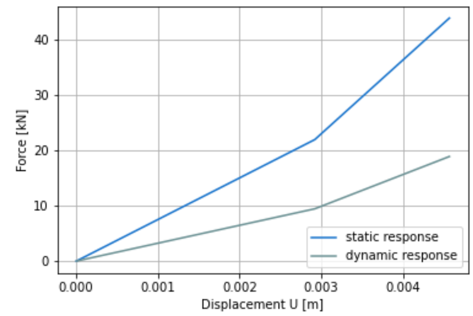
(a)



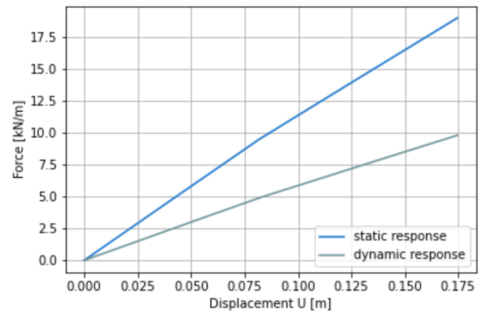
(b)



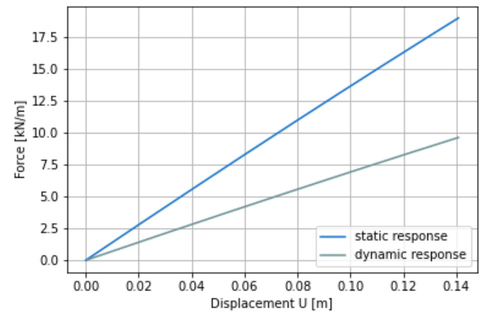
(c)



(d)



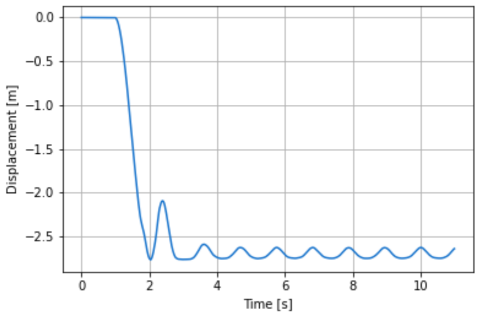
(e)



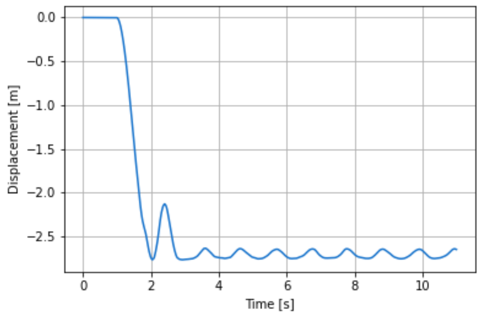
(f)

Figure C.1: Push down curves for column removal scenarios a) till f)

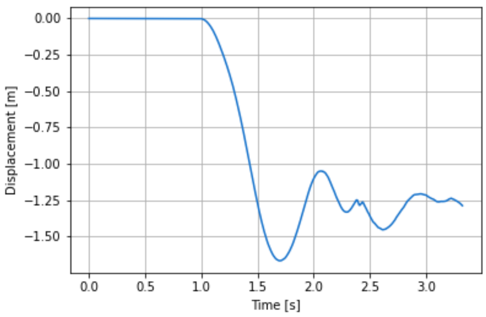
C.1.2. Vertical displacement time history responses - nonlinear dynamic



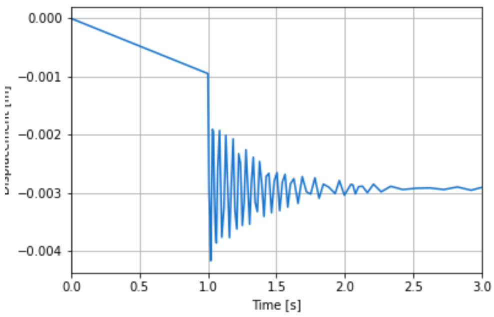
(a)



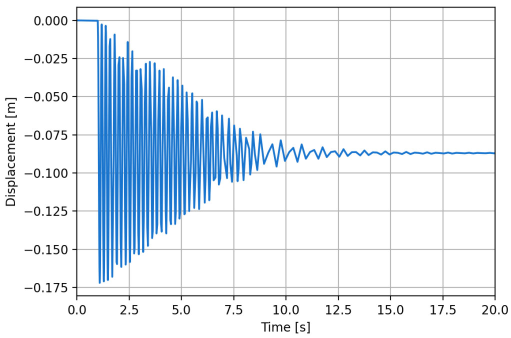
(b)



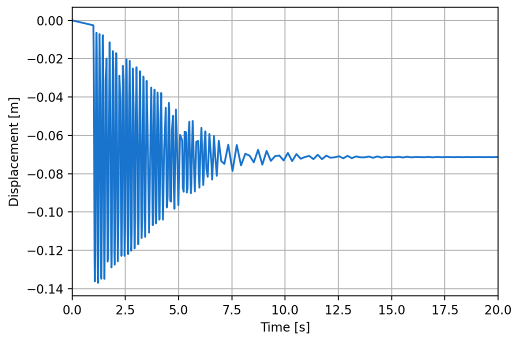
(c)



(d)



(e)



(f)

Figure C.2: Displacement time history response for removal scenarios a) till f)

C.1.3. Vertical displacement color plots - nonlinear static

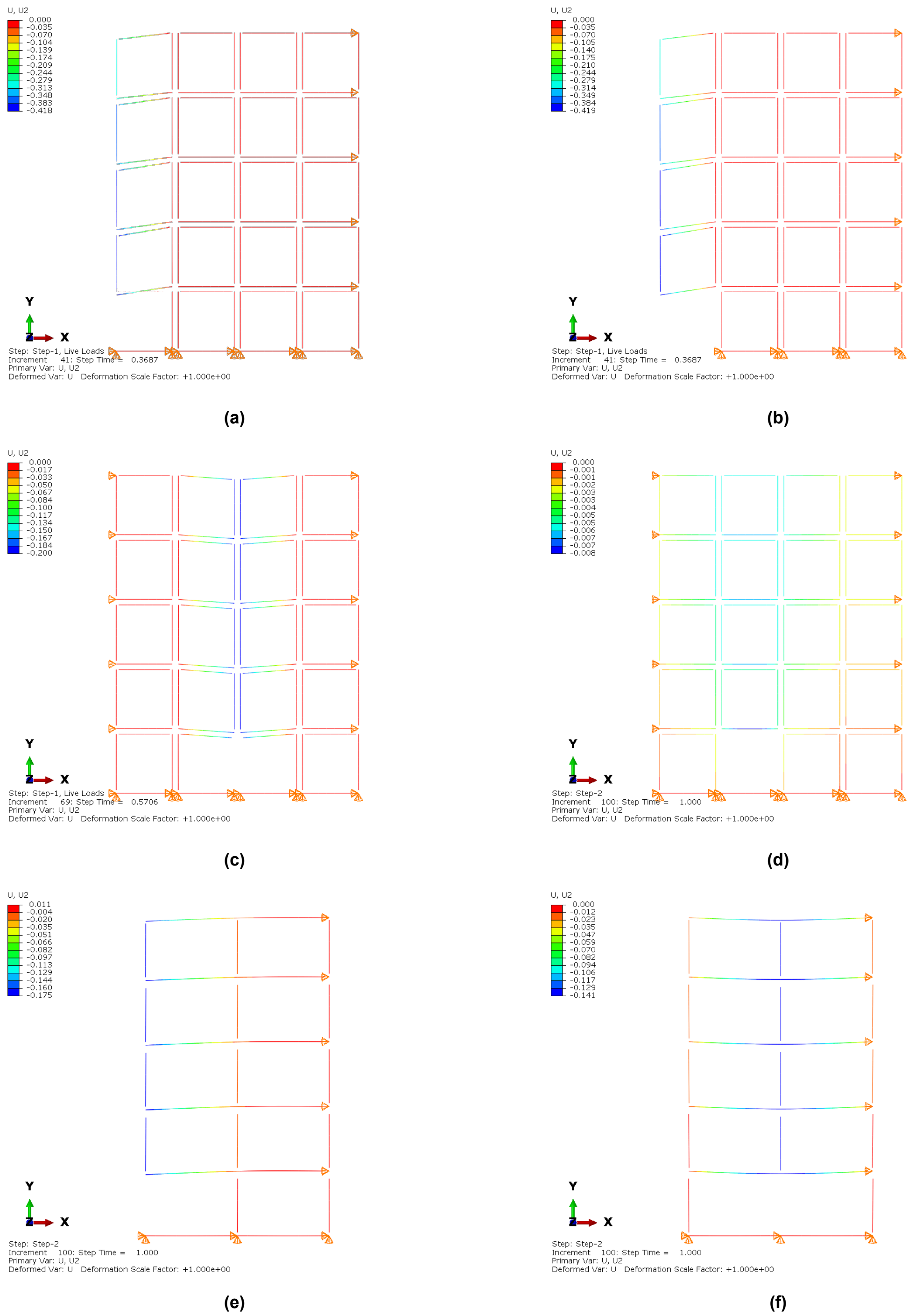


Figure C.3: Vertical displacement output for removal scenarios a) till f)

C.1.4. Section moment color plots - nonlinear static

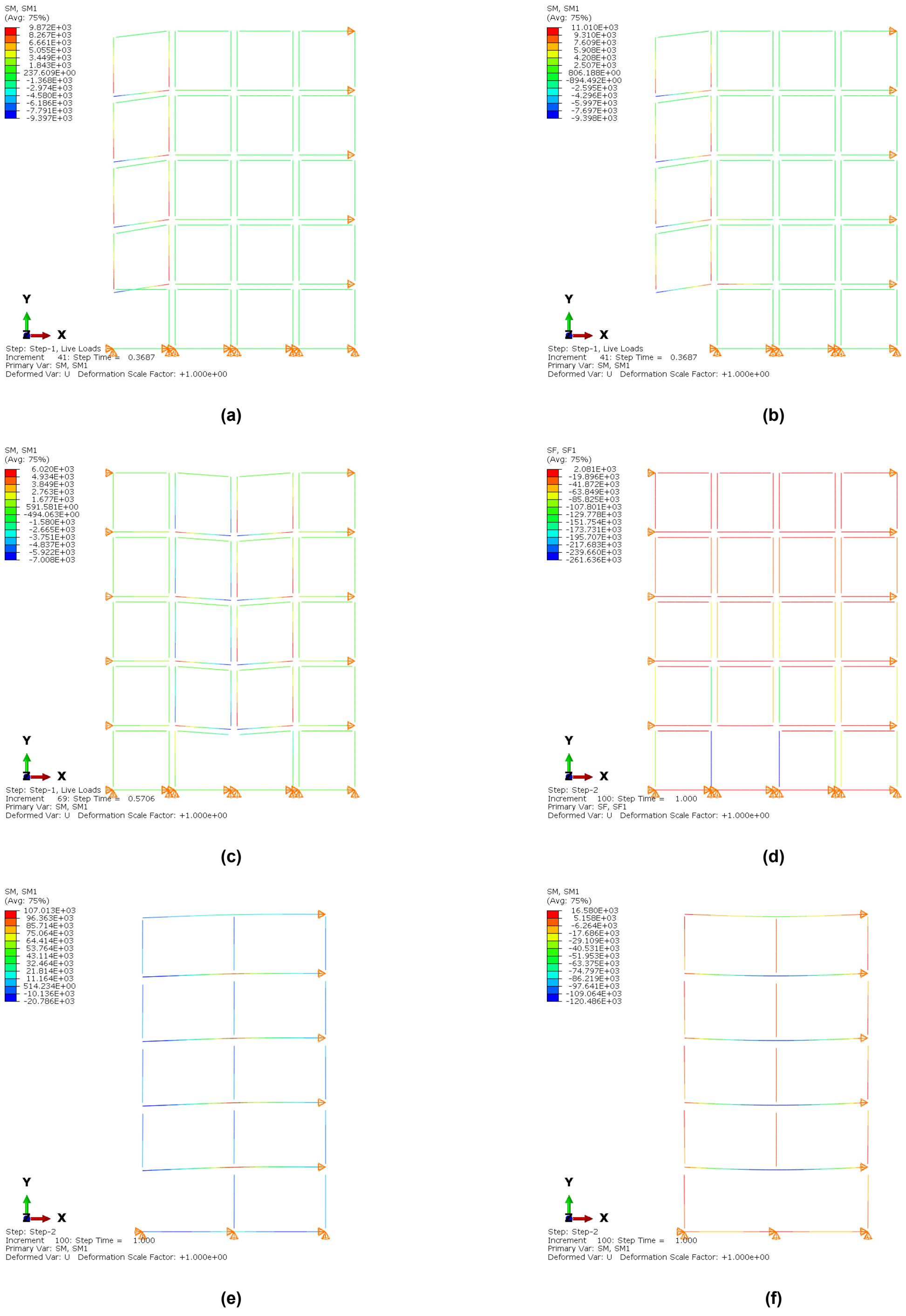


Figure C.4: Section moment output for removal scenarios a) till f)