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Efficiency of new high capacity self-stabilising modules in mid-rise residential buildings

A contribution in the transition to a sustainable way of construction

By

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Preface

In front of you is the thesis *"Efficiency of new high capacity self-stabilising modules in mid-rise residential buildings"*. This thesis has been written as part of the graduation for the master's degree in Building Engineering at the Faculty of Civil Engineering & Geosciences at the TU Delft University of Technology.

The topic of this thesis, modular construction, has been chosen because of my own interests in sustainability and the experience and expertise that Pieters Bouwtechniek has in the field of modular construction. Previous projects in my bachelor and master's study have had their focus on sustainable solutions in the building industry and modular construction fits in nicely with this.
The research field and application of modular construction expands every year rapidly in terms of diversity and size and I would like to contribute to the transition to a more sustainable way of construction.

This thesis was written under the supervision of prof.dr.ir. Bert Sluys, ir. Sander Pasterkamp and dr. Florentia Kavoura. As an external supervisor from the company Pieters Bouwtechniek, ing. Jan Berkhout was also part of the thesis committee. The author would like to thank all of them for their ideas and recommendations during the various meetings that we have had. Receiving feedback from different perspectives has helped a lot during the process of writing this thesis.

Abstract

The aim of this thesis is to investigate and quantify the efficiency of high capacity self-stabilising modules in mid-rise residential buildings. These modules have a higher stabilising capacity than modules that are currently being used in the Netherlands and other countries and can therefore be used for more storeys without requiring an additional stabilising structure such as a concrete core.

In the first part, reference projects and case studies are looked at to get a good understanding of the current applications in The Netherlands and the United Kingdom. After analysing four case studies, an assessment is done on the functional efficiency, structural capacity and environmental impact of these modules. By doing so, the load-bearing structure of self-stabilising modules that can be used at a greater height can be identified. Design variants can now be drafted with different bracing configurations, which are later verified on strength and stability requirements. To effectively design a suitable braced frame, it has been researched what the displacement components for braced frames are. This has been done for simple frames without eccentricity as well as frames including eccentricity. Apart from single-cross frames with a relatively large span, double-cross frames are also looked into due to their increased stiffness.

As part of the total structure of the building, a design for the foundation as well as the inter-module joint, which is required to be demountable, has been made. These parts of the design are required to calculate the horizontal displacement during lateral loads.

A structural assessment is done on the stabilising capacity of each variant at 8 storeys. The design adjustments that are required to further increase the number of storeys up to 10 are looked into as to see whether or not an efficient structure can be maintained. It turns out that each design variant requires adjustments that reduces the efficiency. These changes are the result of a large increase of braced span, resulting in either inefficient use of beam profiles or a too large length when there is more than one braced span along the length.

Apart from a structural assessment, the functionality and environmental impact of the design variants has been analysed as part of the overall efficiency of the modules. The functional assessment includes several criteria such as wall-to-floor area and space efficiency factor. Using the required material use in partition structures and load bearing elements, the environmental impact is calculated, resulting in values for the embodied energy and embodied carbon per square meter in each design variant. Since the differences between the design variants are relatively small, they are also compared to four case studies that were done before.

On the basis of the results of this research, it can be concluded that self-stabilising modules can be constructed with different possible bracing layouts and an efficient load-bearing structure up to 8 storeys.

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Glossary

Bracing	A structural steel member that is subjected to lateral loads.
Bracing eccentricity	The distance from the intersection of the direction of the bracing and the centre of the column to the inter-module joint.
C-section	A steel profile used for beams and columns that has the shape of the letter C.
Cross Laminated Timber	A panel made from gluing multiple layers of wood in which each layer is oriented perpendicular to the adjacent layer.
Demountability	The ability of an element that is able to be removed from a supported position.
Floor system	A group of components that are fastened together to create a floor.
Emissions	The amount of a substance that is produced and sent out into the air that is harmful to the environment.
Embodied energy	The sum of all energy required to produce any goods or services.
Energy performance	The energy that is permitted during the service life of a building.
Environmental impact	The change to the environment resulting from emissions and energy use.
Erection speed	The time required to install and connect individual modules on-site.
Flexibility	The capacity of an element to adapt to changing requirements or circumstances.
Floor joists	Steel or timber elements that are used to support a floor in the transverse direction.
Floor slab	A floor that has been formed using reinforced concrete.
Gross floor area	The sum of all floor areas in a module.
High-rise	A building that has more than twelve storeys.
Integration	The action of coupling two or more elements in an effective way.
Inter-module connection	The connection between two modules that are either horizontally or vertically connected or both.
Low-rise	A building that has up to three storeys.
Mid-rise	A building that has between four to twelve storeys.

Modular construction	A construction method in which the building consists of off-site constructed modules that are assembled on site.
Modules	Three-dimensional units that are fitted out in a factory and assembled on-site as the main structural elements of a building.
Net Floor Area	A portion of the gross floor area of a module excluding the area of partition structures.
Self-stabilising	A structure that has the capacity to be stable during lateral loads without the addition of an external structure to help stabilise the total structure.
Space efficiency factor	A factor that is calculated by dividing the net floor area by the gross floor area and that determines how well the space is used.
Structural capacity	The capability of load bearing elements to resist against vertical and lateral loads.
Wall-to-floor ratio	The ratio between the area of the external walls and the floor area.

1 Introduction

1.1 Background information

In recent years, a lot has happened in the Netherlands with an effect on the building industry. In June 2019, the Dutch cabinet presented the *Klimaatakkoord*, as part of the Dutch climate policy (Ministerie van Economische Zaken en Klimaat, 2019). It is an agreement between many parties and companies to reduce the emissions of greenhouse gases. The most important goal of the *Klimaatakkoord* is to reduce the CO_2 emissions by 49% in 2030. Each industry will have to take its own measures to comply with the goals. Since the goals have only been set last year, many innovative ways to reduce the CO_2 emissions are still too expensive or not fully developed yet. The construction industry is globally responsible for 40% to 50% of all the greenhouse gases that are generated. During the construction of a building, nearly all CO_2 emissions come from the material production phase. The other phases, transportation and on-site construction account for only a small percentage of the emissions, roughly 5% to 10% on average (Designing with Vision: A Technical Manual for Material Choices in Sustainable Construction, 1999). During the life cycle of a building, a lot of additional CO_2 emissions are generated in the operation process. Several measures need to be taken to reduce the CO_2 emissions significantly in the building industry.

Subsequently, all permit applications for new construction must comply with the *Bijna Energieneutrale Gebouwen* (BENG) requirements for energy performance. The performance will be assessed on the total required energy, use of primary fossil energy and the share of renewable energy (Rijksdienst, 2020).

A major problem in the Netherlands is the housing shortage. There is a current shortage of 331,000 houses and this shortage is expected to be increased to 419,000 in the year 2025. This shortage is due to wrong projections of population growth and the banking crisis of 2008, which lead to a decrease of construction companies. The lack of construction employees and materials lead to higher construction costs. This effect will only increase the housing shortage since it is harder to make a project financially feasible. There seems to be only one solution for the housing shortage, which is simply to build more, according to Johan Conijn. Real estate entrepreneurs and investors have warned for the scarcity of building land, causing prices to rise even higher (Obbink, H. 2020).

1.2 Introduction to modular construction

Building for the future nowadays means constructing in an environmentally responsible way for those who are in need. The above-mentioned issues lead to the necessity of a different construction approach. Over the years, the traditional on-site block construction has shifted towards more off-site activities, where 2D panels and hybrid panels are constructed. Nowadays it is also possible to construct 3D elements in a factory, which are assembled on site. This new way of construction is called modular construction and offers many advantages compared to traditional construction. A modular building is defined as a building that is built up of volumetric units which are prefabricated in a factory. These units are assembled on-site using large cranes, shown in Figure 1. The modules are often accompanied by a separate stability system such as a braced steel structure or a concrete core to complete the structure of the building.

Modular construction is defined as three-dimensional or volumetric units that are generally fitted out in a factory and are delivered to the site as the main structural elements of the building (Lawson et al., 2010). Each volumetric unit is called a module and can have a load-bearing structure of either steel, concrete, timber or a combination of those materials. The load-bearing structure consists of a floor, ceiling and walls or only edge beams if one side is kept fully or partially open. A modular building can either be constructed with the purpose of being a temporary building with a user time of one or multiple years, or a permanent building in which the user time is multiple decades. Although the building has an end of its lifetime, it can still be disassembled at the end of its initial lifetime and used in another project when demountable connections are made between the modules.

Figure 1. Installation of modular units (Courtesy of Yorkon and Cartwright Pickard architects).

1.3 Advantages of modular construction

Modular construction is in the first place a faster and safer construction method than the traditional on-site construction. A faster construction method is deemed necessary to cope with the increasing housing shortage. In the modular construction process, 3D units are fabricated in a factory under ideal circumstances. The specialism of the factory workers ensures rapid assembling of the 3D units. These modules are then transported by trucks and assembled on-site using heavy cranes. The total construction time can be reduced up to 60% compared to traditional methods (Murray-Parkes, J. et al). Another advantage of the factory production is the reduction in waste. Due to the experience and knowledge of the factory employees there is only minimal material waste. Since material production is the main source of CO_2 emissions, the reduction in waste is an important aspect in dealing with CO_2 emissions. The scarcity of building land in many cities indicates the necessity to build in the height. Modular construction lends itself for high-rise towers, often using concrete cores as the main stabilizer.

1.4 Application

The application of modular construction is currently very little compared to traditional construction. This is due to the low number of skilled workers, experienced contractors and transportation difficulties (Ferdous, W. et al., 2019). The applications, visible in Figure 2, vary from student residences and family homes to mixed residential and commercial buildings and even health sector buildings.

Sectors for which OSM is most relevant	*Levels of off-site manufacture (OSM)* 2. Elemental or planar systems: Structural frames	2. Elemental or planar systems: 2D panels	3. Mixed-construction systems	4. Fully modular systems
Housing		✓✓		✓
Apartments—multistorey	✓✓	✓✓	✓	✓✓
Student residences	✓	✓✓	✓	✓✓✓
Military accommodation				✓✓✓
Hotels	✓	✓	✓✓	✓✓✓
Office buildings	✓✓✓		✓	✓
Retail buildings	✓✓✓		✓	✓
Health sector buildings	✓✓✓	✓	✓	✓✓✓
Educational buildings	✓✓✓		✓	✓✓
Mixed use, e.g., retail/residential	✓✓	✓	✓✓✓	
Industrial, e.g., single storey	✓✓✓		✓	
Sports buildings	✓✓✓	✓	✓	✓
Prisons and security buildings	✓		✓	✓✓✓

Note: ✓✓✓, widely used; ✓✓, often used; ✓, sometimes used.

Figure 2. Most relevant construction sectors for off-site manufacturing (Lawson et al., 2014).

Modular construction has been conventionally used for low and mid-rise buildings in which the walls can be load-bearing and provide stability. High-rise buildings up to 25 storeys height, with modules clustered around a central core are less common yet. The first high-rise modular building has been constructed in 2007 in London and is called the Paragon. Since then, many more buildings have been constructed, most of them being apartments or student residences (Lawson et al., 2014).

1.5 Research problem

The benefits of modular construction are focused on those sectors where disturbance to the neighbourhood is unwanted, a fast construction method is required and an economy in manufacture is important in the business requirement. Ideally, a construction strategy involving modularization should be incorporated as early as possible in the project so a design can be made that is suitable for modularization. Some important aspects are reducing the interdependency between elements, allocation of tolerances and standardizing the design so that the cost benefits can be obtained during the works in the factory (O'Connor, T. et al. 2014). Modular construction is currently used as 'one of the alternatives' in the design process. Therefore, it often turned down since it is not feasible in the given design requirements such as an irregular grid of the building that has been chosen for aesthetical reasons. This is contrary with the design principles of modular construction where regularity improves the effectiveness.

This research is aimed at investigating different possible structural layouts of high capacity self-stabilising modules and to find out the efficiency of these layouts in terms of functionality, material use and environmental impact.

An important parameter for choosing a specific concept design is the space efficiency of the floor. Therefore, the floor slab shape and total floor area needs to be designed. The more efficient the floor slab is, the more usable space the client gets and therefore the more income he can get. In a building, the space efficiency is calculated by dividing the Net Floor Area (NFA) by the Gross Floor Area (GFA). A tool to achieve a high space efficiency is to use a certain shape, such as a square, circle or octagon instead of using an irregular shape. Another advantage of this shape is the reduced wind loads on the building. This concept can be used in making design variants using modules (Sev & Özgen, 2009).

Another important parameter is the wall-to-floor ratio, expressing the ratio between external walls and the floor area. From a cost perspective, the lower this ratio the better since less walls need to be constructed. Typical values for this ratio are in the range between 0.35-0.60 with the majority being above 0.45. The decisive factor of the wall-to-floor ratio is a maximized size of the floor plate, while the articulation, the way multiple surface form the total shape, is minimized.
It seems needless to say that the highest space efficiency and wall-to-floor ratios is always the aim. However, there are factors preventing this from happening. For example, the wishes of the architect to have a certain shape of the building that fits well into its surroundings or the surroundings being so small and irregular that the plot is constrained. High values for the space efficiency and wall-to-floor ratio may lead to the necessity to construct using more material and complex connections, resulting in additional costs which make the design not the most optimal one (Barton, J. et al. 2013).

1.6 Research questions

The research objective leads to the following main question:

What is the efficiency of new high capacity self-stabilising modules that are used at a greater height than is currently done in mid-rise residential construction?

To adequately answer the main question, five sub-questions were formulated. These sub-questions can be divided into the study on modular systems and design research of new self-stabilising modules. The first two questions on modular systems are:

1. *What are the different module types that are currently used for mid-rise residential buildings?*
2. *Which self-stabilising module types have the capacity to be used for extra storeys compared to what is currently done in low- to mid-rise residential buildings?*

Three more sub-questions are set up which will be answered during design research of new self-stabilising modules in residential buildings, these are:

3. *Which configurations of bracing systems are possible for new high capacity self-stabilising modules?*
4. *What is the structural capacity of new high capacity self-stabilising modules?*
5. *How does the functional efficiency and environmental impact of the new high capacity self-stabilising modules relate to existing self-stabilising modules?*

1.7 Goal

The research field of this thesis is mid-rise residential buildings consisting of modular units. The goal is to find out what the possibilities of high capacity self-stabilising modules with an efficient use of space and a load-bearing structure with an efficient use of material. This will be done for a design case of around 8 storeys, which is multiple storeys higher than is currently being done in the Netherlands and other countries.

1.8 Work approach

During the literature research, information will be gathered about how buildings can be constructed using volumetric modular units. How they behave structurally, individually and grouped and how the construction is done off-site and on-site. Relevant reference projects of modular buildings will be looked into to understand how modular buildings are constructed.
After that, a few projects will be subjected to a more detailed case study in which the structure will be analysed. This will be done for projects within the scope of the research which each use a different load-bearing and stabilising structure in the modules. When the behaviour of modules is known, design

principles of an efficient load-bearing structure and sustainable design will be investigated. These principles will help to come up with design variants in the later stage.

These case studies will be subjected to an assessment in which the properties of the building, the functional efficiency of the module, the use of material and environmental impact are calculated. Conclusions can be drawn on the advantages and disadvantages of each module on the above mentioned subjects.

The next step is generating variants, and this will be done based on the case studies as well as the design principles of modular construction such as having repetition in design.

2 Methodology

Type of research
A combination of quantitative and qualitative research will be performed. Important aspects such as the dimensions of structural elements are quantified while the efficiency of load-bearing systems is also analysed.

Data collection method
A lot of data on the structure of modules will be collected from research papers on modular construction to find out how buildings can be constructed, using volumetric modular units. Additional data will be collected from the archives at Pieters Bouwtechniek to find out how existing modular buildings are constructed. Apart from these structures in the Netherlands, that were designed by Pieters Bouwtechniek, other case studies from the United Kingdom will be looked into as well. This is done because they use a different modular system, in lightweight steel compared to steel-concrete composite modules and fully concrete modules in the Netherlands. For these projects it is unknown how the modules are build-up in detail. Therefore, standard lightweight steel modules will be examined as well as the known properties of the case studies to determine a load-bearing structure for these modules.

Explanation calculation method lightweight steel modules
Performing structural calculations on lightweight steel modules of which the load-bearing system is unknown has the following vale. It is known what kind of wall, floor and ceiling system lightweight steel modules have and whether or not it is self-stabilising. The build-up of the partition structures has to comply with requirements for insulation and fire safety. To design these partition structures, similar partition structures in traditional construction as well as modular construction will be looked into. It is also known that preventing tension in columns and meeting displacement requirements are the governing criteria for the stability of modular buildings. When a design is made for lightweight steel modules that complies with these requirements, a result will be obtained that is plausible. Even though the structure differs on some parts in reality, it is now known which sizes of the elements and build-up of partition structures are required. This knowledge can be used in a later design stage for designing a new type of module.

Data characteristics
There are several characteristics of the case studies which will be investigated. First of all, the general properties of the building such as the number of floors and allocation of modules are examined. Secondly, the dimensions of the modules themselves are important such as the length and width of various parts of the module and the elevation of the facades. These are highly important when designing structural members. Thirdly, the exact dimensions of partition structures will be looked into. Lastly, the stabilising elements and types of connections are examined.

Scope of research
The scope of the research is mid-rise modular buildings which are self-stabilising. Therefore, projects with more than 3 storeys will be examined. Another criterion is that the function of these buildings is residential use.

Data analysis criteria and method
The criteria that will be part of the analysis are the dimensions of partition structures as well as external dimensions of the module. These data values will be used to determine the efficiency of the module as well as the material use and environmental impact that they have.

Reliability
In this research as many sources as possible are literary sources. These sources are mentioned in the list with references. Apart from the information sources, the structural verification will be done by applying the Eurocode. The hand calculations that are done will be explained based on the steps that have been taken. Repeated calculations however will only be shortly explained at most to reduce repetition. Most of the structural hand calculations are done using Excel. The verification are done using Technosoft in which the same structure will be modelled. This will be done for the stability calculations since these are more complex than strength calculations of a single element.

3 Modular construction

Using existing building materials, many different module types can be made, each with their own floor, ceiling and wall elements. The elements that will be mentioned are retrieved from the book Design in modular construction by Lawson et al. The different elements can be combined into modules with different load-bearing capacities. In the paragraph about stability systems, the use of the module types will be explained for reference buildings with its corresponding maximum height.

3.1.1 Horizontal elements

The horizontal elements are the ceiling and floor members. Ceiling members are designed to support the self-weight of the ceiling as well as loads that are applied during installation, equal to 1 kN/m^2. The sizes of C-sections are often chosen equal to the size of the floor joists in case of lightweight steel modules, to have the same production system. Ceiling joists with a height of 100 mm for example are sufficient along a span of 3.3 m. The temporary construction load is higher than the snow load and therefore the upper module does not require a different load-bearing structure in the ceiling. An example of the build-up of a ceiling system with a floor system of the module above is given in Figure 3.

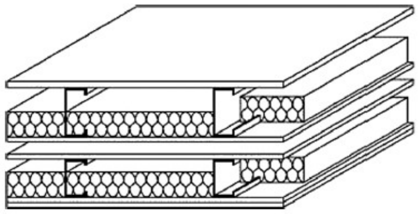


Figure 3. Light steel joists for ceiling with next floor on top (Lawson et al).

Steel joists

The first floor system comprises of C-section joists in the transverse direction at around 400 mm centres, which can be placed individually or manufactured as a floor cassette. This section is commonly used for loads that are uniformly distributed and have small bending moments. The advantages of using C-sections are the high structural capacity in a multiple member system such as a floor joist system or a module. It is also highly compatible for connections to other internal steel members and the concrete surfaces of the central core. A C-section requires almost half the amount of steel compared to an I-section and offers good properties when flexure is not a critical factor. Due to the asymmetrical Y-Y axis, it is susceptible to buckling and the top flange often needs to be braced for that reason (Liang, 2020). In the longitudinal direction, edge members of larger size can be used. This system is always paired with steel corner columns in the form of hot-rolled steel angles of square hollow sections. The build-up of the floor consists of rigid boards and insulation which are both supported by steel joists along the length. A single board can cover the insulation from the bottom side. The ceiling consists of similar joists with a lower height since the applied load is also lower. The dimensions of the structural elements highly depend on the function of the building as well as the floor span and will therefore not be estimated beforehand. The combined system is visible in Figure 4.

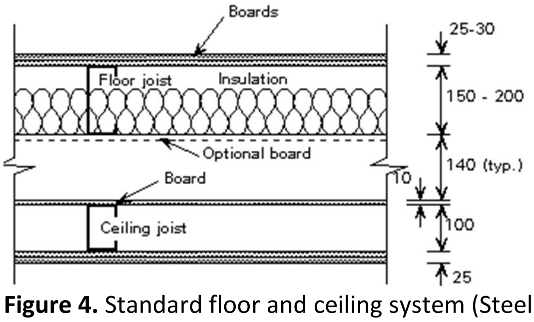


Figure 4. Standard floor and ceiling system (Steelconstruction.info).

Precast concrete slabs

Instead of using steel C-sections along the length of the module, concrete slabs can also be used. When the slab is the only fire resisting element in the floor to ceiling structure, a minimum depth of 120 mm is necessary to obtain 120 minutes of fire resistance (NEN-EN 1992-1-2). The steel reinforcement is calculated based on the active bending moment due to its self-weight and live load on the floor. The slab can be integrated with the edge beam in several ways. When I-section or C-section beam is used, a conventional method is to place the slab on top of the beam using a vertical shear stud, visible in Figure 5. Modern methods include integrated supports, in which the floor slab can be integrated with or without a horizontal shear stud. The depth of the floor slab increases towards the location of the beam to get a connection across the whole height of the beam.

Figure 5. Structural depth of open steel beam and concrete slab (Liew et al, 2009).

3.1.2 Wall elements

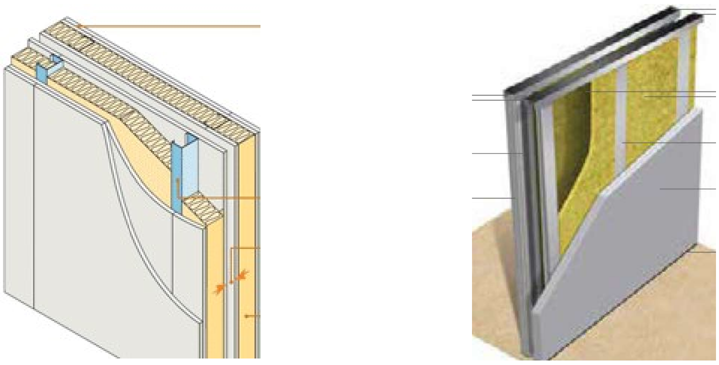
There are three general types of wall systems. The first two systems are steel infill walls and concrete walls, and both are considered to be four-sided closed modules due to their linear load path. The walls transfer all vertical loads to the foundation and also help resist horizontal load. The third system uses a steel frame in which the floor and ceiling loads are transferred from the floor and ceiling structural elements onto the edge beams and then taken down to the foundation through corner columns.

Steel infill walls

A four-sided module is continuously supported on the longitudinal walls, which bear on the walls of the module below. When steel elements are used, the wall consists of 70 to 100 mm deep C-section studs either singly or in pairs at 600 mm centres. In between, mineral wool is often used as insulation material and a rigid board is added on the inside as well as a sheathing board on the outside of the wall. The exact dimensions depend on the insulation and structural requirements. An estimation of these values, visible in Table 1 below, can be made using existing steel infill walls such as in Figure 6 and Figure 7.

Table 1. Dimensions of commercial steel infill walls.

Layer	Robustdetail	Twin Metal Stud Wall	Standard infill walls
Cavity	50 mm	72 mm	50 mm
Sheathing board	10 mm, 7.5 kg/m^2	2 layers of 15 mm plasterboard	Optional board
Absorbent material	75 mm wool	75 mm 'Hush slab'	100 mm
Metal frame	Min 72 mm C-section	70 mm C-stud	C-stud
Wall lining	2 layers of gypsum-based board	2 layers of 15 mm plasterboard	2 layers of 12,5 mm board



(**a**) (**b**)

Figure 6. Examples of commercial steel infill walls: (**a**) Robustdetail, (**b**) Hush Acoustics.

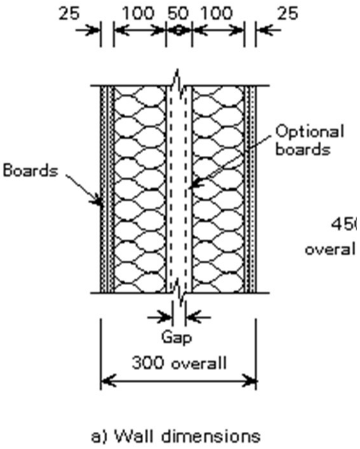


Figure 7. Standard steel infill wall (Steelconstruction.info).

Edge columns

At its corners, large corner columns are used either in the form of a square hollow section or a hot-rolled steel angle. The walls transfer all vertical loads to the foundation and help resisting horizontal loads. They provide attachments for other structural elements and local lifting points during construction on-site. Fire resistance is provided by the fire resisting boards, combined with mineral wool between the C-sections, visible in Figure 8 below. The prevention of passage of smoke to other modules is done horizontally by outer sheathing boards over full length and fire barriers at edges of module. Vertical prevention is also done by horizontal fire barriers at the floor level of each module. (Lawson, 2007).

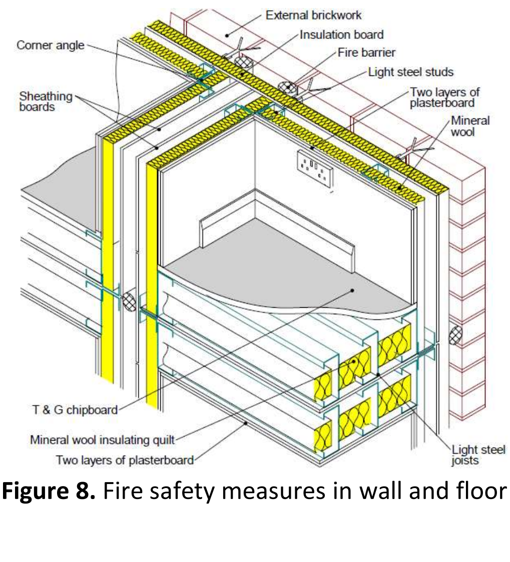


Figure 8. Fire safety measures in wall and floor structure (Lawson, 2007).

Concrete Walls

Concrete walls can also be used as an alternative to obtain four-sided modules and they have several benefits. Electrical conduits and service voids can be built into the concrete and a plaster skim coat is all that is needed on site to finish it. The minimum dimensions of the concrete walls are based on the fire resistance. Since walls are part of the primary structure, in mid-rise to high-rise construction the fire resistance is 120 minutes. An example of a fully concrete modules is shown in Figure 9 below.

Figure 9. Installation of a precast concrete module (Oldcastle Infrastructure).

Timber studs

Similar to steel infill walls, timber studs can also be used to resist vertical loads. These studs are quite small, standard dimensions are 38 by 89 mm with a 9 mm thick sheathing board on the outside, using an oriented strand board. A typical depth of the floor is 385 mm when 250 mm deep floor joists are used. The walls are insulated in between the studs using mineral wool and a rigid insulation board. A standard layout of a timber wall is shown in Figure 10 below.

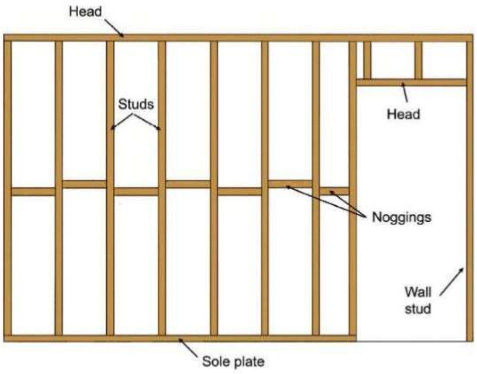


Figure 10. Timber stud wall.

CLT-walls

Cross Laminated Timber (CLT) walls are able to resist larger lateral forces than timber studs. CLT elements can be made from coniferous wood lamellae that are crosswise laminated where one layer is in the longitudinal direction and the other one in the transverse direction, visible in Figure 11. After lamination, the lamellae are glued to form a larger and solid timber element. The walls are able to achieve sufficient sound insulation. Depending on the height of the building and horizontal dimensions, additional stability in the form of a concrete core could be necessary (Lignas). An example of a module with CLT walls is shown in Figure 12.

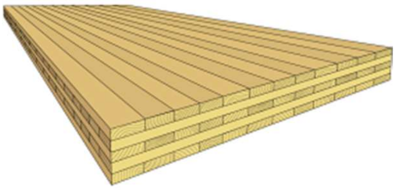


Figure 11. Lamination of lamellae into a CLT panel.

Figure 12. Installation of module with CLT walls (Ursem).

Open sided modules

Open-sided or corner supported modules use corner posts its edges and sometimes at intermediate lengths as well, depending on the dimensions of the module. Edge beams span between the posts, allowing for open sides in the module as shown in Figure 13. In case of open sides, corner posts are in the form of square hollow sections and edge beams are parallel flange channels. The beam-to-post connections are weak in bending resistance and therefore additional bracing is required, often located around the stair and lift core. Standard dimensions are edge beams with a depth between 200 and 350 mm and a span of 6 up to 12 m. The depth of the floor and ceiling varies between 450 and 700 mm.

Figure 13. Open-sided module (Kingspan).

Other modular systems

In other modular systems, panels are combined into hybrid systems and are used for higher-valued areas such as kitchens and bathrooms. Modules can also be combined with a primary steel structure to form a podium structure that supports the modules above. This podium structure is then used as a communal or commercial space at the ground floor and a parking garage can be constructed underneath.

3.2 Modular building principles

3.2.1 Factory production

The modules are produced in a factory under ideal circumstances, this leads to less material being used and less wastage compared to traditional on-site construction. Another advantage is the increased productivity in the factory due to skilled workers (Lawson et al., 2010). The production of concrete elements and steel or timber elements are different and will be explained below.

The first step in the production of a concrete floor is to fix the reinforcement into the moulds or formwork. This has to be done securely so that the pouring of the concrete will not displace the reinforcement. Lifting attachments are added to the modules so that they can be lifted by a crane after the casting. Self-compacting concrete is often used because of its superior qualities. When placed correctly, it allows for a more consistent finished product with barely any defects. After the concrete has been poured, a concrete finisher is added. The ideal circumstances in the factory ensure that a high-quality concrete product is being made.

The production of steel or timber modular elements is done in different stages and is shown in Figure 14 below. In order to make a wall or ceiling element, framing stations are used that are either manual or semi-automatic. The first station uses C-sectional elements as its input and these elements will be connected with nails or rivets over multiple stations into panels. In the next step, the planar elements are moved onto turning tables that can rotate and raise the panels in order to add insulation and electrical cabling behind a sheathing board. A second table is used to add plasterboard to close the panel. The individual panels are assembled into 3D modules by using overhead cranes where fixing are made from the outside using self-tapping screws or bolted connections. After the planar elements are connected, the modules are finished at another workstation where finishing operations such as painting are done.

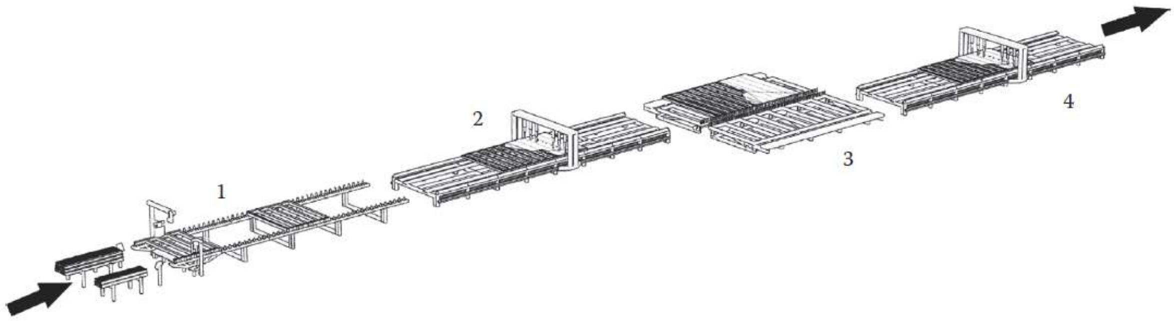


Figure 14. Semi-automated line for wall and ceiling panels (Lawson et al., 2010).

3.2.2 Sustainability of modules

The factory production in modular construction offers many benefits for the sustainability during the construction process and in-service performance. These advantages can be categorized into social, environmental and economic advantages. The social and economic advantages are related to the superior working conditions in the factory. The main environmental benefits during construction are less pollution, less material wastage and more recycling of materials. The production of concrete panels leads to a reduction in construction waste up to 65% compared to in-situ concrete. This waste mainly comes from over-ordering, damage and losses on site and additional work due to errors. (Jaillon et al, 2008).

During the service life, the improved energy performance, such as better airtightness, results in lower CO_2 emissions. The shrinkage and long-term movement of concrete is also reduced due to the dry factory conditions and multiple checks that are done before delivering the concrete to the site (Buildoffsite, 2021). Modules that have open ends are more flexible and adaptable than fully closed modules. Structural components can easily be replaced due to the openings, potentially leading to a longer service life.

3.2.3 Service interfaces

A building has multiple services that are necessary to provide a comfortable, functional and safe living conditions. These services are part of the design and are distributed horizontally and vertically across the building. The services include energy supply and distribution, escalators and lifts, façade engineering, fire safety detection and protection, heating, ventilation and air conditioning (HVAC), lighting and water drainage. In modular construction, most of the services are tested and installed off-site in the modules. Connections need to be made in modules to the service distribution in rest of building. A module with a concrete floor can have electrical services cast in conduits in the concrete itself. Steel floor joists however require openings to prevent fraying of cables. In steel wall systems, vertical service ducts can be incorporated in the design in three ways:

1. A corner post with vertical service ducts incorporated and not connected to adjacent walls.
2. The wall has an opening for vertical services incorporated.

3. A service riser is located outside the line of modules, resulting in additional width of the corridor but leaving the stability of the module corner unaffected.

At floor and ceiling levels, vertical fire stoppings are required to prevent the passage of smoke in a fire compartment. In a design for a high-rise building, additional services can be implemented in the structural core of the building. The horizontal distribution of services can be done by using the corridors or floor and ceiling voids where pipes, cables and air circulation ducts can be installed. An enclosed roof space can be used for chillers as well (Lawson et al., 2010).

3.3 Stability system

Various modular stability systems are possible, using different materials and dimensions. The strength of the structural materials determines the possible dimensions and the number of floors that can be achieved. The resistance against lateral wind forces is often critical when it comes to stacking modules as high as possible.
The first two systems of modular construction are modules with steel as the main structural material. The first system is self-stabilising while the second system uses an external steel structure to provide stability to the building. The next two systems use concrete as main structural material. The third system is again self-stabilising using concrete walls, while the fourth system uses a concrete core for the majority of lateral resistance. The last stability system is a timber structure with cross-laminated timber walls, accounting for most of the lateral resistance combined with a concrete core when necessary.

3.3.1 Internal stability

The first type of stability system has stability measures inside the steel module. These measures are bracings, either X-shaped or K-shaped, diaphragm action of the walls or moment resisting connections between beams and columns. The weight of the walls are low compared to concrete since small steel profiles are used, combined with insulation and board protection. Since bolted connections are often used to connect the steel modules, on-site inspection during the operation phase is necessary to check the bolts on corrosion. Steel offers a high degree of flexibility in the design and large spans can be created using steel beams. Openings in the walls are necessary for connections to pipes and cables. The construction speed is fast, since only bolted connections are usually necessary to connect the modules (Liew, Y. et al).

Bracings

Stability of an internal steel structure can be obtained using X-bracings longitudinal walls of the modules, visible in Figure 15 below. K-bracings can be used in transverse walls as part of the wall system, next to a door or window when limited width is available.

Figure 15. Closed module with bracings in both directions (Lawson et al, 2005).

Horizontal bracings can be placed in the corridor, visible in Figure 16 below, to transfer horizontal forces through the module to corridor connections towards the access core where it is taken down to the ground.

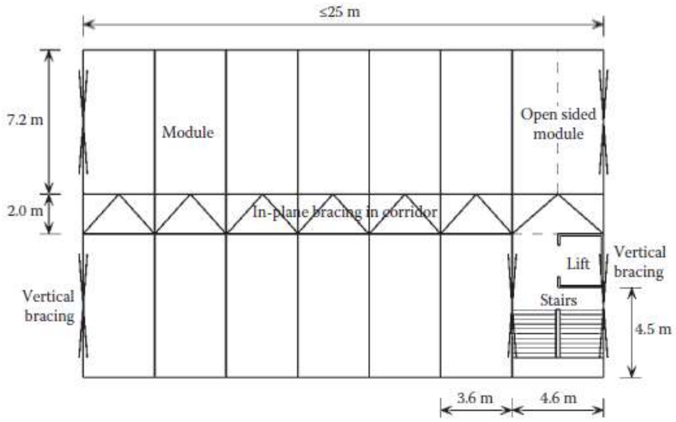


Figure 16. Location of vertical and horizontal bracing (Lawson et al., 2010).

Generally, X-bracings are able to resist forces in the order of 25 kN, making it a suitable solution for medium-rise buildings. K-bracings can resist much lower forces, up to around 5 kN. However, when two K-bracings are installed on either side of a window opening, a total shear force of 10 kN can be resisted.

Diaphragm action

Diaphragm action is the resistance to shear forces of sheathing boards. These boards are fixed to the light steel framework such as cement particle board, moisture-resisting plywood and orientated strand board. Higher in-plane shear resistances can be obtained using unperforated longitudinal walls compared to X-braced walls. Boards can resist a shear force of approximately 4 kN/m of wall length for a cement particle board and 3 kN/m for an orientated strand board, visible in Table 2. The governing design criteria is the deflection limit, equal to 1/500 times the module height.

Table 2. Tests on wall panels with sheathing boards (Lawson et al, 2005).

Configuration of 2.4m square wall panel	Service load (kN) based on stiffness:		Failure load (kN)	Design load (kN/m)	
	Spacing of fixings at perimeter of boards (mm)				
	300	150	150	300	150
No openings					
Plasterboard (Pb)	3.7	–	–	1.5	–
Plywood and Pb	7.2	8.0	26.8	3.0	3.3
Cement particleboard and Pb	9.0	11.0	34.6	3.7	4.6
Steel sheeting (19mm) and Pb	5.8	7.5	23.8	2.4	3.1
With window opening					
Plasterboard (Pb)	3.0	–	13.5	1.2	–
Plywood and Pb	6.0	8.6	–	2.5	3.6
Steel sheeting (19mm) and Pb	4.0	4.4	18.9	1.6	1.8

Note: Design load is the unfactored load per unit length resisted in shear

Moment-resisting connections

Between edge beams and corner posts, moment-resisting connections can be made using end plates or deep fin-plates. The edge beams are usually in the form of C-sections while the columns are hot-rolled steel posts in a SHS section. These open-sided modules can be used for buildings up to 3 storeys high. Additional height can be achieved by adding additional stability measures such as X-bracing or intermediate posts.

3.3.2 External steel structure

When the modules themselves are unable to resist all lateral loads, an external steel structure can be used to increase the lateral resistance. A square core area for vertical transportation or a walkway for horizontal transportation can both be used as an additional stabilising structure. Modules are usually installed perpendicular to the walkway. By doing so, the walls of the modules are able to resist wind in the longitudinal direction and wind in the transverse direction is transferred to the braced walkways.

3.3.3 Precast concrete module

A precast concrete module can be made using concrete walls paired with a concrete slab as the floor system. The high mass helps meet the requirements for fire resistance and acoustic separation as well as controlling internal temperatures. Another advantage is that there is no need for a separate ceiling and floor since the ceiling of the lower module is used as the floor of the second module. Cables and ducts are built into the concrete, as well as the reinforcement. Both the walls and slabs have two layers of mesh reinforcement for a faster construction speed compared to individual bars. Concrete modules have a high weight, on average 20 tonnes and up to 40 tonnes. The building height ranges from a single storey up to 6 or 7 storeys. Contrary to steel wall systems, there is only little design flexibility when using concrete. Concrete walls have a large weight compared to timber of steel infill walls and are approximately 40% heavier. During the operation phase, concrete walls are low on maintenance, no inspection is required to check the behaviour of the material. (Liew, Y. et al)

The most common applications of precast concrete modules are prisons, hotels and secure accommodations. The construction speed is slow, due to the in-situ grouted joints between the modules. In hotels, a corridor layout is used with a repetitive use of precast modules. The corridor can be manufactured as extensions to the modules or as separate planar elements. These modules allow for open sides, using rigid connections in the floor and ceiling structure, allowing for a wider use. For example, a school building can also be constructed using concrete modules, with spans up to 12 m using a ribbed concrete roof slab. Instead of using the modules as a room, they can also be used to form a core area in any type of modular building. When doing so, attachments for stairs and lifts are part of the modules. Stairs can then be installed along with the modules.

3.3.4 Timber structure

There are two types of modules consisting of timber as the main load-bearing material. These two types are timber-framed modules and a module consisting of cross-laminated timber (CLT) walls and a concrete floor slab. Timber-framed modules can be used for 1 to 2 storey buildings, such as educational buildings and housing. In the case of residential modules, the standard specifications are 38 by 89 mm timber studs with a 9 mm thick sheathing board on the outside, using an oriented strand board. A typical depth of the floor is 385 mm when 250 mm deep floor joists are used. The walls are insulated in between the studs using mineral wool and a rigid insulation board. A module consisting of CLT walls and concrete floor slab can be used to achieve a larger height. The CLT walls are able to resist lateral forces as well as vertical forces. They are also able to achieve sufficient sound insulation. Depending on the height of the building and horizontal dimensions, additional stability in the form of a concrete core could be necessary. Both systems have a low weight due to the low timber volumetric weight and a fast construction speed compared to concrete modules. During the operation phase, maintenance is required to check the timber on shrinkage and swelling.

3.3.5 Summary of stability systems

By summarizing the different build-ups of floor, roof and wall elements, it will be made clear what the possibilities are to configure different types of modules. The different stability options will be mentioned which are often related to a certain number of storeys and are shown in Table 3 to Table 5. For low-rise structures, the options for types of modules are the largest, since they can all be self-stabilising up to a certain height. For mid-rise structures there is often an additional external stabilising structure necessary for stability in the transverse direction of the modules, as stated before. In high-rise structures there are the least options for module types since the stability is always provided by a central core and the vertical elements need to be able to carry high loads.

Table 3. Low-rise module types.

Option	Floor system	Wall system	Stability
1	Timber studs	Timber joists	Diaphragm action
2	Steel joists	Steel infill walls	Diaphragm action
3	Steel joists	Edge beams	Moment-resisting connections
4	Concrete slab	Concrete walls	Shear walls

Table 4. Mid-rise module types.

Option	Floor system	Wall system	Main stability
1	Concrete slab	CLT walls	Walls and optional core
2	Steel joists	Steel infill walls	Bracings
3	Concrete slab	Steel infill walls	Bracings
4	Concrete slab	Concrete walls	Walls

Table 5. High-rise module types.

Option	Floor system	Wall system	Main Stability
1	Concrete slab	Steel infill walls	Core
2	Concrete slab	Concrete walls	Core

3.4 General reference projects

Across the world, many modular buildings have been constructed over the past decades. These projects all have different properties in terms of function, size and structure. Seven projects are looked into to get an idea of the possibilities with modular buildings. These buildings are located in the United Kingdom, the Netherlands and in Sweden. The buildings can be categorized into two categories. The first two buildings, the Norra Tornen and Croydon towers are some of the tallest modular buildings constructed so far. The other five buildings are considered mid- to high-rise with a number of floors between 6 and 17. The reference projects can be found in Appendix B.

3.4.1 Conclusions

Some high-rise buildings want to stand out against the rest, which is done using segmentation, this is clearly visible in the Norra Tornen and Wembley cases which either have horizontal segmentation or horizontal curvatures. Modular construction lends itself for segmentation since small units can be placed on various locations. The façade structure is often separate from the modules itself and therefore do not hinder a certain appearance. Other high-rise building full use of floor area, no segmentation, uses different structural system with steel walls. The buildings which are less tall show no vertical segmentation and only minor horizontal segmentation when multiple wings are present. This is done to speed up the construction process and to profit from the advantages that pre-fabricated modules offer during installation.

3.5 Structural reference projects

To gain more insight in the possibilities of modular buildings, several reference projects are looked into. They are analysed to find out the following topics:

- User function of modular buildings
- Relation between number of storeys and stability system
- Relation between materials and stability system
- The use of different building shapes: gallery, corridor, cluster

The buildings that are looked into have different load-bearing systems. The first type is mid-rise buildings that use lightweight steel modules and an internal or external steel structure for stability. The second type of buildings uses a concrete core as the main stabilising element, these are mid-rise buildings as well as high-rise buildings. To finish it off, a building which uses CLT as well as a concrete core is looked into. The goal is to find a relation between the height of the building and the stability system that has been used. Additionally, the different materials that are used for each type of buildings will be analysed.

3.5.1 Conclusions

The buildings investigated show a variety of shapes. The mid-rise steel buildings use the following shapes:

- Single, L-shaped or T-shaped gallery
- Square corridor with internal courtyard
- U-shaped with a gallery shape at its two ends and a gallery in the middle

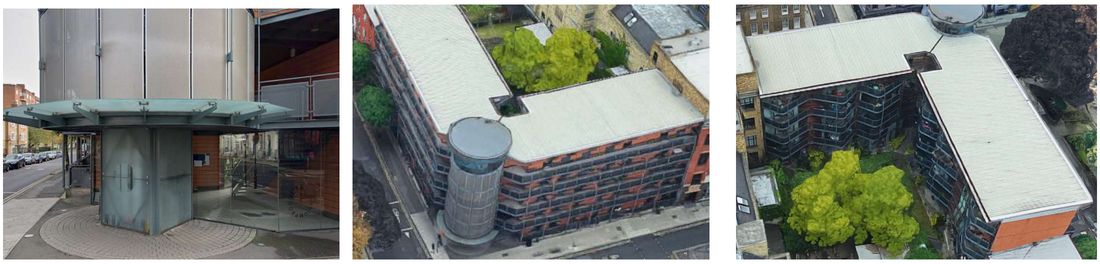
The buildings with a concrete core are either clustered partially or completely around the core or between two cores along a horizontal length. The placement of modules is almost always perpendicular to the hallway. However, in the case of the MoHo building, the modules are placed parallel to the hallway. Using open sides along the length of the module, multiple modules can be stacked parallel to the façade to create large open areas. Some of the buildings have a ground floor with a separate function. This function is either office or commercial and retail. To make an accurate comparison between different load-bearing systems for mid-rise buildings, an extensive case study needs to be done. This will be done for two of the reference projects as well as two other projects. These projects require a form of self-stabilisation, have different materials used and have a residential function.

4 Case studies

A case study has been done to four modular projects across the Netherlands and the United Kingdom. These projects are within the scope of the research, they are all mid-rise residential apartment buildings. They vary in construction materials as well as the number of modules per apartment and the external dimensions of the modules. For each case study it will be explained what the properties of the whole building are and how the floor plan and cross-section of an apartment look like. After that, an overview will be given of the sizes of the load-bearing materials as well as a build-up of the partition structures. In the end, an analysis has been done on the load-bearing structure and stabilising system.

4.1 Murray Grove

The Murray Grove project was the first apartment building in the UK which uses modular construction and was finished in 1999. It is used for low-rental housing for small families. The aim was to create a high-quality architectural image which has been obtained by having an L-shaped building with a central cylindrical stair tower with perforated aluminium screens that closes off a glazed lift, visible in Figure 17. Each apartment has a large balcony area at the rear side of the building with a view on a private courtyard. These balconies are ground supported by a tubular column and connected to the modules on each floor level (Lawson et al., 2010).



(**a**) (**b**) (**c**)

Figure 17. Views on the Murray Grove building: (**a**) Entrance, (**b**) Front façade, (**c**) Back façade (Google Maps).

4.1.1 Front façade

The front of the building shows a total of 7 modules, each with either a window or a door, making up for 3 apartments in total, visible in Figure 18.

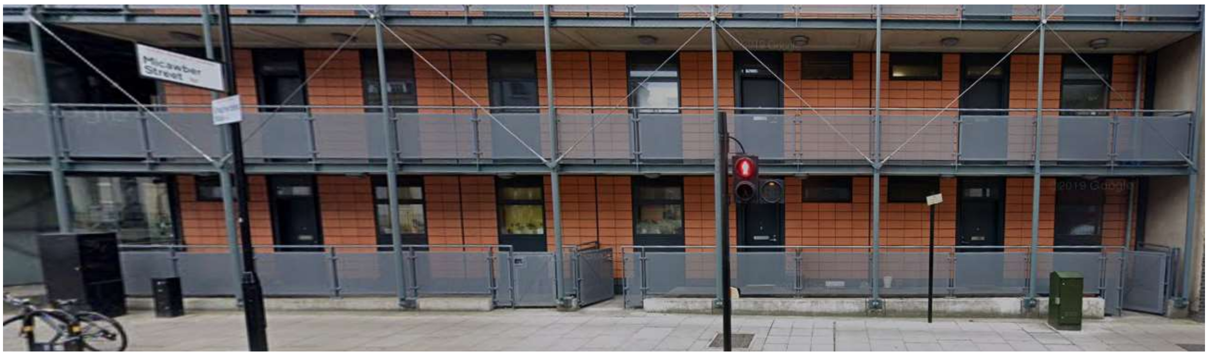


Figure 18. Murray Grove elevation of apartment entrance (Google Maps).

To find out how the building can be stabilised in the transverse direction, the upper right module is considered. It is deemed necessary that there are bracings along the front façade since it is the only location in which a continuous placement of bracings is possible since the façade walls are all aligned. Each module has a width of 3.2 meters and therefore the edge of each module can be derived from the façade view. The module that contains the entrance of the apartment has a door in the middle, leading to a horizontal bracing length of approximately 1 meter. The window in the adjacent module, which contains the kitchen and bedroom, is located at the edge of the module and therefore there is an additional horizontal length available for the bracing. However, the position of an additional window reduces the available bracing height.

4.1.2 Rear façade

The back of each module consists of large windows and a balcony for each module. Therefore, it is not possible to place stabilising elements along this façade.

4.1.3 Floor plan

The two-module apartments have an open section between the kitchen and the living room. The module on the left contains the kitchen and a large bedroom and the module on the right contains a large living room and a bathroom. Figure 19 shows the installation of the right module in which the open section is closed off by a protection fabric. The section of the module which is open has the structure of an open-sided module in which a thick edge member is used to support the roof and an extra internal column is used to support it.

Figure 19. Murray Grove module installation (Lawson et al).

The module requires longitudinal bracings and with only one suitable location, based on the floor plan of the module. Due to the open side between the kitchen and the living room, the only possible location is along the length of the bedroom. This leads to internal columns at 3.1 meter length seen from the bottom. Double crossed bracings will be used with twice a span of 2.4 meter. The functional floor plan as well as the structural floor plan are shown in Figure 20.

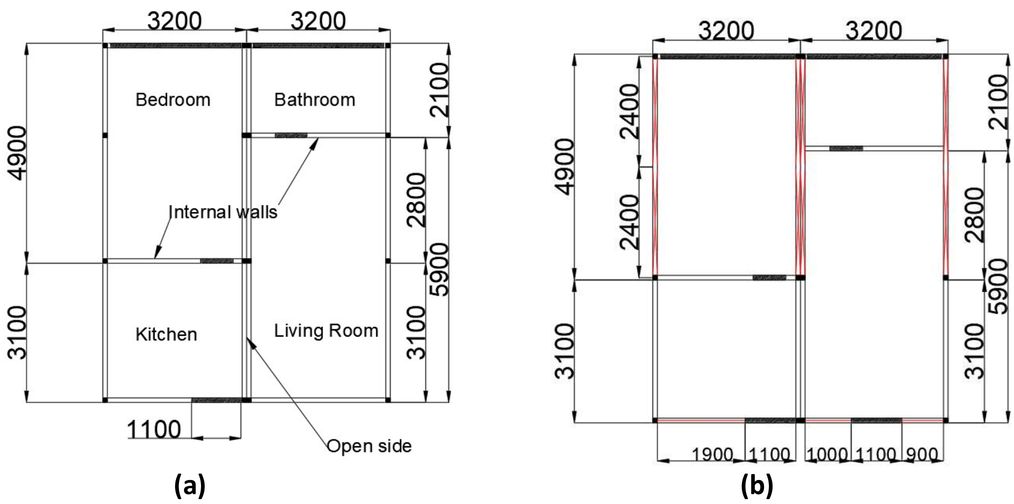


Figure 20. Murray Grove floor plans: (**a**) Functional, (**b**) Structural (own work)

4.1.4 Cross-section

The cross-section of the module shows the location of the various parts of the module and their size, visible in Figure 21 below. It can be seen that the insulation of the floor runs across the whole module length and that the roof structure runs between the wall posts and therefore has a smaller length.

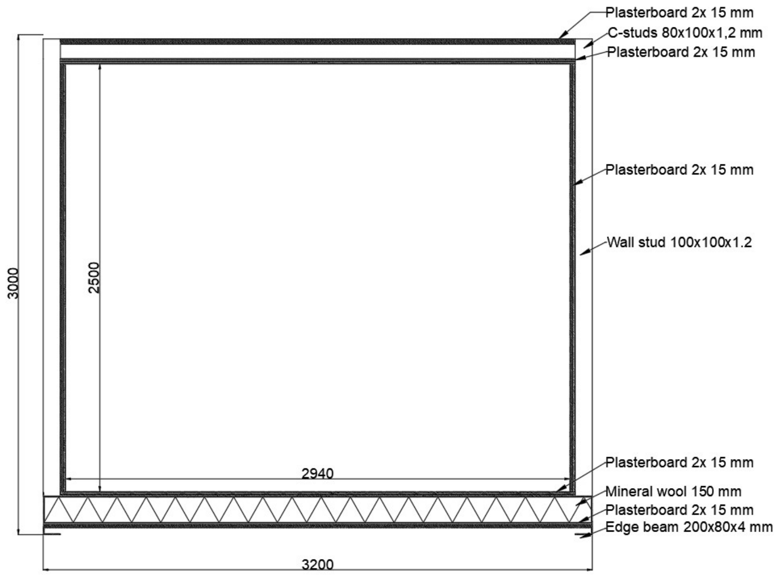


Figure 21. Cross-section Murray Grove (own work).

4.1.5 Column tension

The wind load has been calculated according to the UK National Annex (EN1991-4). The calculation method considers the following parameters:

- Fundamental basic wind velocity, based on UK national grid coordinates $V_{b,map}$: 21.5 m/s
- Altitude of the site above mean sea level A: 10 meters
- Directional factor c_{dir}: 1.0, conservative approach
- Upwind distance to shoreline d_{shore}: 100 km
- Height above ground at which peak velocity pressure is calculated z: 15 meters
- Displacement height for buildings in town h_{dis}: 6 m
- Orography factor at reference height z, $c_0(z)$: 1.0, conservative approach
- Calculated $q_p(15\text{ m})$ is 0.75 kN/m^2.

The given q_p value is used to calculate the tension force at ground floor of the column that supports the bracing.

$$F_{tension,ed} = \frac{M_{wind}}{h_{bracing}} * 1.2 * 1.5 = 38.54 \text{ kN}$$

The load on the critical column, which carries the smallest width of the module has been calculated to compare it to the tension force and is shown in Table 6. The characteristic loads per storey are multiplied number of floors, 5, and the favourable load factor, 0.9, in order to get the design value for the compression load. This value is equal to 42.1 kN

Table 6. Column load components.

	Value in kN
F,k,floor	1,98
F,k,wall	3,01

F,k,ceiling	1,87	
F,k,var,40%	2,10	
F,k,column	0,14	
F,k,edge beam	0,27	+
F,k,storey	9,36	
F,ed	42,11	

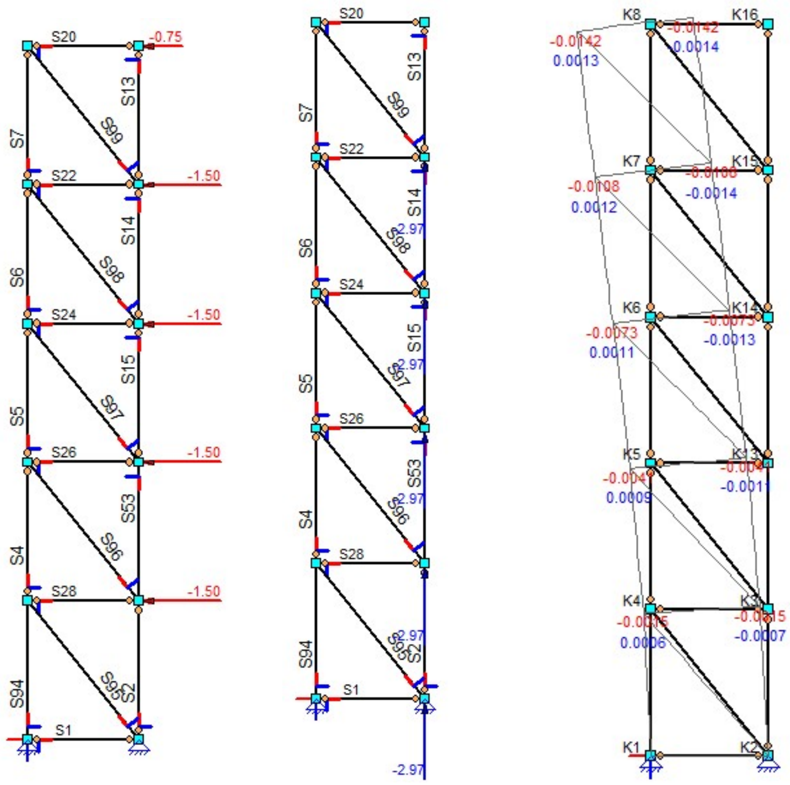
4.1.6 Longitudinal stability

The detailed calculation with the displacement of the different components on the different floors is visible in the Appendix D. The impact of each component on the total displacement, as well as second order displacements are shown in Table 7 below. It can be seen that the rotation of the floor has the biggest impact on the displacement while the bracing and horizontal displacement due to the compression of the columns have a significant lower impact. The second order effect is also small and only two iterations were necessary.

Table 7. Murray Grove horizontal displacement.

First order components	Displacement (mm)
u1,bracing	2,19
u2,phi,wind	7,71
u3,phi,perm	2,64
u5,hor,column	1,57
u,tot,1st	14,12
Second order	Displacement (mm)
Iteration 1	
u,extra	0,49
u,tot,2nd	14,61
Iteration 2	
u,extra	0,03
u,tot,2nd	14,64
Verification	
u,rd	30,00
UC	0,49

A verification of the first order displacement has been made using Matrixframe. The schematization of the permanent column load is done in an unconventional way. Since the two columns have a difference in permanent load, the floor above will rotate and thus additional displacement along the height will occur. However, when adding the permanent load acting downwards on a node that is connected to a bracing element, this has a negative effect on the force distribution. Some of the load will be taken up by the bracing since it is not possible to model a 'tension-only' element in Matrixframe. Another problem is that the compression of the column will lead to a horizontal displacement of the bracing in order to maintain the original geometry which does not occur in reality. Therefore, the impact of the permanent load is schematized as the difference in vertical load between the two columns, acting on the opposite column in the upward direction. By doing so, the bracing will be unaffected since the load is taken up by the column on which it acts. It can be noted that the total displacement is equal to the modelled displacement. The individual storey displacements are also equal and can be found in Appendix D.



(**a**) (**b**) (**c**)

Figure 22. Matrixframe longitudinal braced frames: (**a**) Wind load, (**b**) Change in permanent load, (**c**) Horizontal displacement.

4.1.7 Transverse stability

In the transverse direction, in one of the two modules bracings can be placed on either side of a central window. In the other module, bracings can be placed next to a door on the side of the module. The displacement of a single storey, loaded by wind and permanent loads is modelled in Matrixframe. The assumed dimensions for the floor beam, ceiling beam as well as the wall post and the bracings are used in the model. Having wall studs with low centres highly reduces the displacement since the bracing in the left pulls the ceiling beam downwards while the other bracing pulls the floor beam upwards.

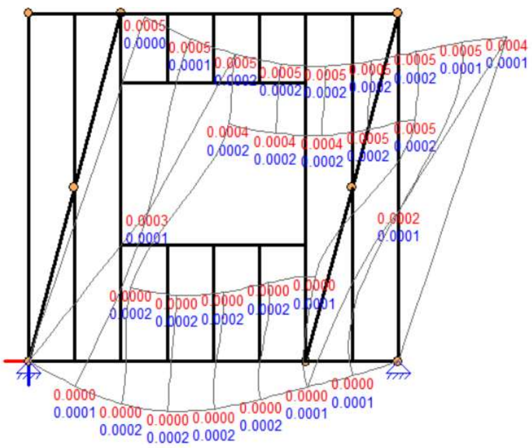


Figure 23. Matrixframe transverse single storey frame horizontal displacements.

4.2 Raines Court

The Raines Court project in north London had the aim of achieving architectural variety as well as maximising the available space on site. The apartment block consists of 6 storeys and has a T-shape with a private courtyard which can be accessed at the rear walkways, visible in Figure 24. The ground floor contains eight working and living units while the floors above are all two-bedroom apartments with a single smaller wing of three-bedroom apartments to the rear.

Figure 24. Birdview of Raines Court building (Google Maps).

Contrary to the Murray Grove project, these modules are fully closed along their length. An impression of a standard 4-sided module is given in Figure 25 below.

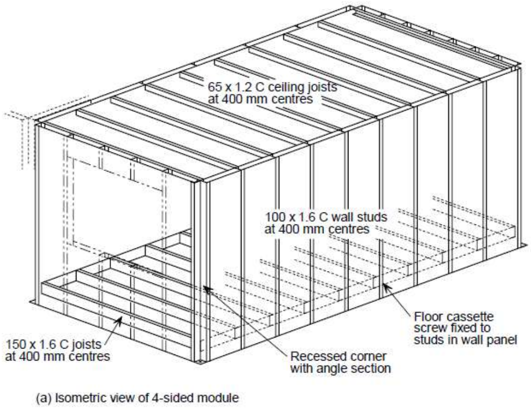
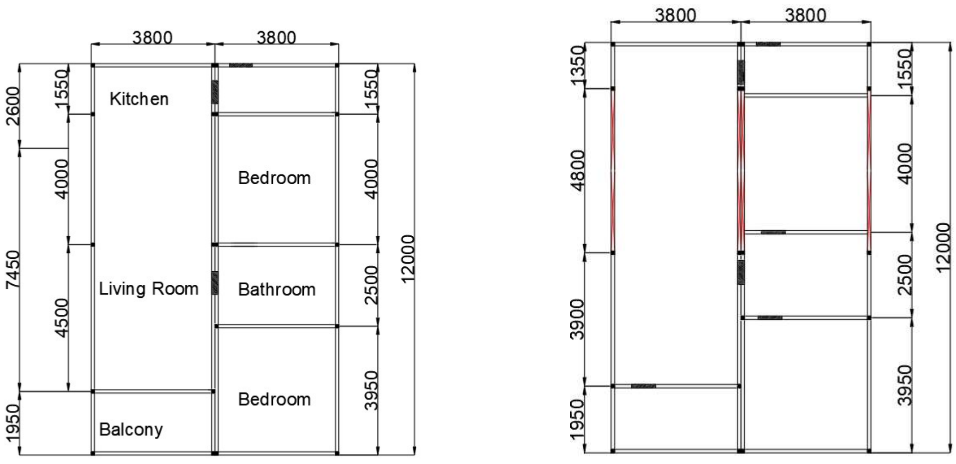


Figure 25. Isometric view of a 4-sided module (Lawson et al., 2010).

4.2.1 Floor plan

The floor plan of the two-bedroom apartment consists of two modules with external dimensions 3.8 meters by 12 meters. The module on the left comprises a large living room and a kitchen as well as a small balcony next to the courtyard, visible in Figure 26. The module on the right has two bedrooms and a large bathroom as well as the entrance at the top. The position of two doors along the length of the module hinders the placement of longitudinal bracings near the edges of the module. This is logical since larger compression forces on the columns which support the bracings are necessary to counteract the tension force from the wind. The placement of bracings is estimated along the length of the bedroom and part of the living room since this area does not have any doors. The stability in the transverse direction is provided by X-bracing around the access cores. This is necessary since there are no closed walls in transverse direction which can be used for the placement of any bracings. Therefore, no calculations will be done on transverse stability of the modules.



(**a**) (**b**)

Figure 26. Raines Court floor plans: (**a**) Functional, (**b**) Structural (own work).

4.2.2 Cross-section

The cross-section of the module in Figure 27 shows the location of the various parts of the module and their size.

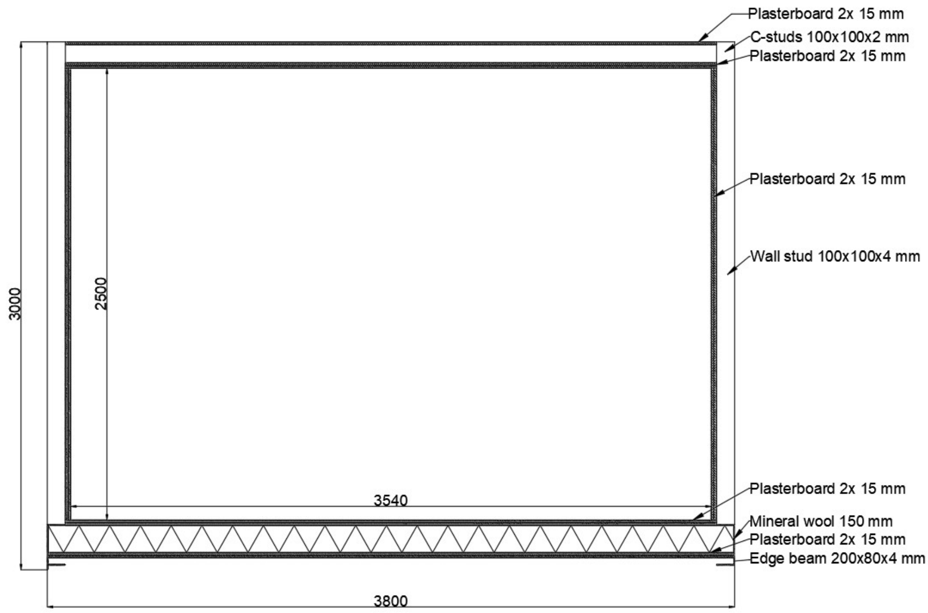


Figure 27. Cross-section Raines Court (own work).

4.2.3 Column tension

Wind load

- Fundamental basic wind velocity, based on UK national grid coordinates $V_{b,map}$: 22 m/s
- Altitude of the site above mean sea level A: 10 meters
- Directional factor c_{dir}: 1.0, conservative approach
- Upwind distance to shoreline d_{shore}: 100 km
- Height above ground at which peak velocity pressure is calculated z: 18 meters
- Displacement height for buildings in town h_{dis}: 6 m
- Orography factor at reference height z, $c_0(z)$: 1.0, conservative approach
- Calculated q_p(15 m) is 0.61 kN/m^2.

$$F_{tension,ed} = \frac{M_{wind}}{h_{bracing}} * 1{,}2 * 1{,}5 = 77{,}3 \text{ kN}$$

Table 8. Raines Court critical column load.

	Value in kN	
F,k,floor	3,32	
F,k,wall	4,53	
F,k,ceiling	3,01	
F,k,var,40%	3,33	
F,k,column	0,27	
F,k,edge beam	0,27	+
F,k,storey	14,74	
F,ed	79,58	

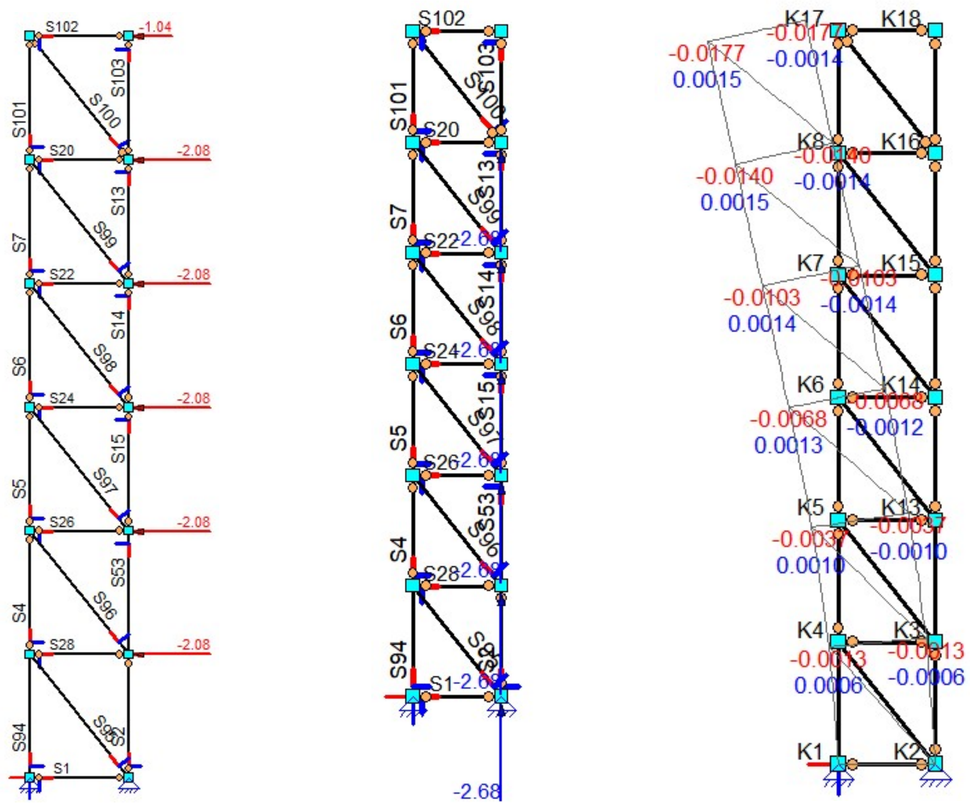
4.2.4 Longitudinal stability

The result of the calculated longitudinal displacement show that the rotation of the floor again has the highest impact on the displacement. The values of the other displacement components are also similar to the other case and shown in Table 9.

Table 9. Raines Court horizontal displacement.

First order components	Displacement (mm)
u1,bracing	1,76
u2,phi,wind	15,47
u3,phi,perm	2,77
u5,hor,column	2,45
u,tot,1st	22,45
Second order	Displacement (mm)
Iteration 1	
u,extra	0,84
u,tot,2nd	23,29
Iteration 2	
u,extra	0,05
u,tot,2nd	23,34
Verification	
u,rd	36,00
UC	0,65

A verification has been made again using Matrixframe in a first order analysis, visible in Figure 28. The same schematization as in the Murray Grove case has been used. The displacement at the top is 100% accurate as well as the other storeys.



(**a**) (**b**) (**c**)

Figure 28. Matrixframe horizontal verification: (**a**) Wind load (kN), (**b**) Shifted permanent load, (**c**) Horizontal displacements.

4.3 North Orleans

The North Orleans building in Amsterdam is a modular building which shows a unique architectural style in the Netherlands. As the name suggests, the style of the building is based on the American city of New Orleans which is known for its zest of live of its artists, jazz musicians and romantics. The city therefore attracts young people from various places and these people are also the target audience for this building. The first thing you notice at the building is the ornamental balustrades which is the clearest resemblance of the French Quarter style in the city of New Orleans. The internal courtyard and various plants on the balustrades also add to the livelihood of the area around the building.

Figure 29. Impression North Orleans (North Orleans).

4.3.1 Access core

An external structure is used to provide access to the different floors, and it is located at the edge of two of the rows of apartments, shown in Figure 30.

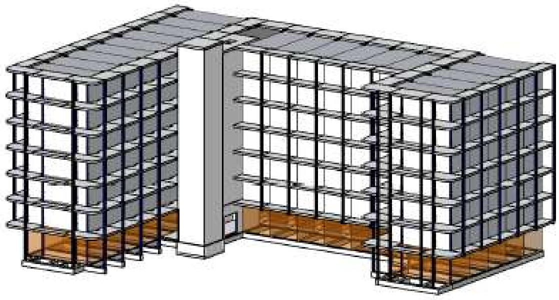


Figure 30. North Orleans building (Pieters Bouwtechniek).

4.3.2 Floor plan

The apartment consists of a single concrete module with a large balcony attached to it, visible in Figure 31 and Figure 32. The layout is different from the previous projects since it is used for starters instead of couples or families. When you enter the apartment, there is a small bedroom on one side and the kitchen furniture on the other side. The rest of the apartment consists of the living room with large windows at the façade. As just mentioned, the back façade is fully open with a balcony attached to it. On the front of the module is a single door at the gallery.

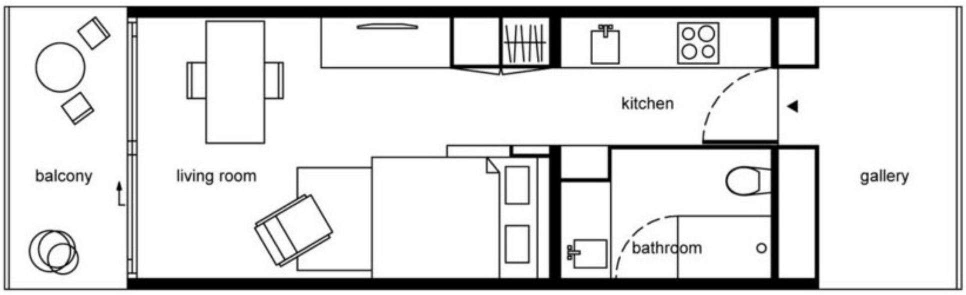


Figure 31. Functional floor plan (North Orleans).

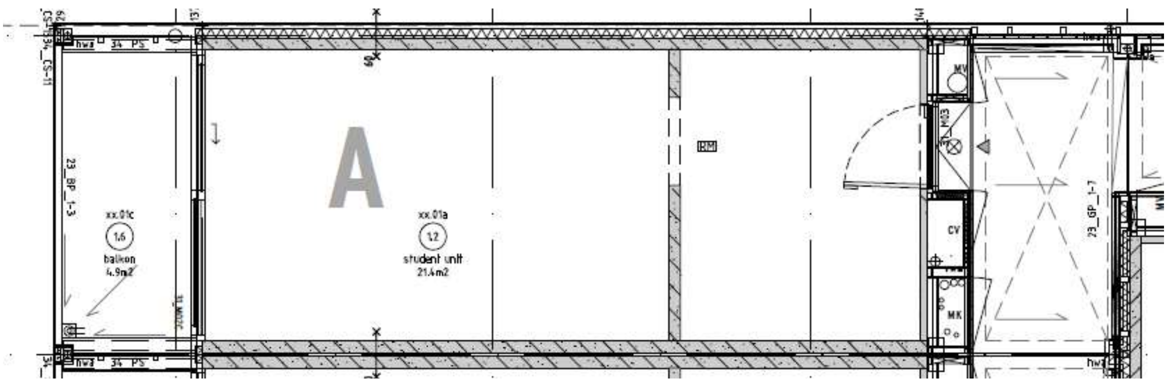


Figure 32. Structural floor plan (Pieters Bouwtechniek).

4.3.3 Cross-section

In the cross-section of the module, visible in Figure 33, the different concrete elements are clearly visible. Below the floor and along the length of the module, small concrete blocks are placed, marked with an X-sign in the drawing, which are used to provide a large cavity in which insulation will be placed on-site along the width of the modules.

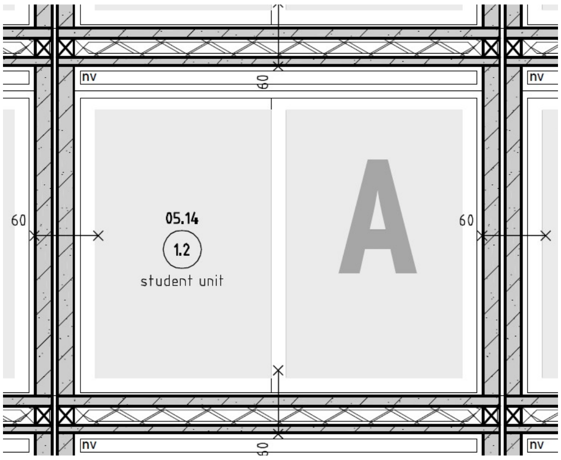


Figure 33. Cross-section of a module (Pieters Bouwtechniek).

4.3.4 Longitudinal stability

The stability in longitudinal direction is provided by the concrete walls.

4.3.5 Transverse stability

In the transverse direction there are concrete walls at two locations in the module. At the entrance there is a wall which has an opening for the door and another wall is located next to the bedroom over a length of around 1,5 meters. These walls combined provide the stability in transverse direction.

4.4 Regioplein

The modular project at the Regioplein in the city of Schagen in the Netherlands consists of three modular blocks with two and three bedroom apartments with a corresponding living area between 60 to 90 m^2, visible in Figure 34. The modules are fully fitted out at the factory and therefore minimal work on site is necessary. There are two gallery flats of which one has 8 modules in row while the other one only has 6. This leads to the requirement of additional stabilising elements in transverse direction in this flat. Only the two gallery flats are considered in this case study.

Figure 34. Regioplein modular buildings (Ursem).

4.4.1 Access core

The gallery flats have external access via a staircase in a steel structure which is braced for its own stability, visible in Figure 35 below.

Figure 35. External access (Ursem).

4.4.2 Floor plan

The floor plan is visible in Figure 36 below and consists of a hallway with on one side entrances to a small bedroom, a large bedroom with a bathroom in between and on the other side entrance to the technical room and the living room. The living room is large with one side entrance to a balcony and on the other side the kitchen area. The entrance façade of the apartment has a door as well as two intermediate windows, one in each module. The back façade has a door to the side with a window next to it and another window in the adjacent module.

Figure 36. Functional floor plan (Pieters Bouwtechniek).

4.4.3 Longitudinal stability

The longitudinal stability consists of two single bracings along the 3 internal spans. By doing so, the façade columns are loaded in compression during wind which highly reduces the displacements. The floor will tilt as a result of a change in displacements between the internal column and edge column. By adding additional compression on the edge column and tension on the internal column, the tilt of the floor will be significantly reduced. The upper floor wall structure differs from the lower storeys. A fully timber structure is used which acts as a stiff plate along the full module length. The schematization in Technosoft is shown in Figure 37.

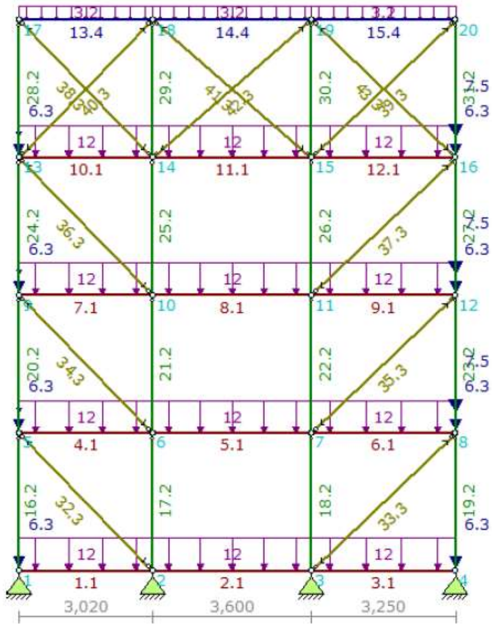


Figure 37. Longitudinal stability scheme (Pieters Bouwtechniek).

4.4.4 Transverse stability

In the transverse direction, obstruction of windows hinders the placement of strips along the full span. Therefore, box profile are required which can take up compression as well as tension. One of the two gallery flats consists of 8 modules in row, while the other one only has 6. A single box profile with an angle of around 70° in either façade is sufficient to stabilise the flat with 8 modules in row. The structure is modelled as a single row of bracings per storey since it is repetitive along the length and on both facades. The wind load is equal to 1/16 of the total façade load since there are 8 modules in row and two bracing per module. Since the other flat has lower modules, the forces in the bracings are larger and additional stability is required. To solve this problem, an additional bracing is placed in three of the six modules. The bracing is located in the middle of the module and an additional column is placed at the location. The position is chosen based on the required floor and wall area that it carries to counteract tension forces. Since there is less obstruction at this location, the bracing can have a larger horizontal length equal to 2 meters and the angle is reduced to circa 55°. The overall stability is modelled by considering two modules, equal to one apartment, with 4 small bracing and one large bracing. The wind load is equal to 1/3 of the façade load since there are 3 blocks of two modules in row. A Technosoft schematization of the transverse stability scheme is shown in Figure 38 below.

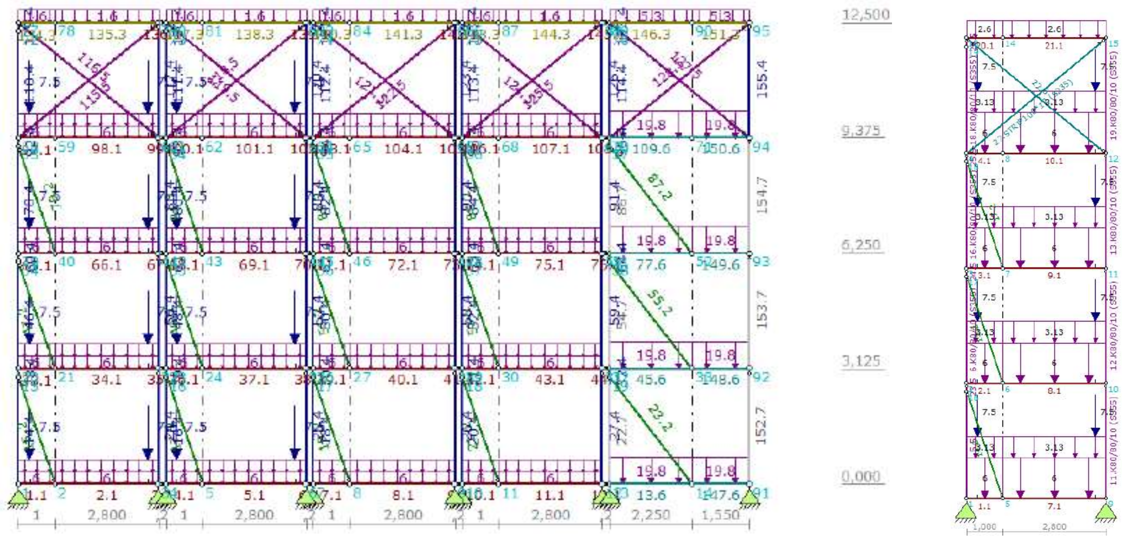


Figure 38. Transverse stability scheme (Pieters Bouwtechniek).

The location of the bracings can be seen in the floorplan in Figure 39 below. At either façade, the small strip bracings can be seen in the upper module as well as the lower module. The internal box profile bracings can be seen next to the internal separation wall in both modules as well.

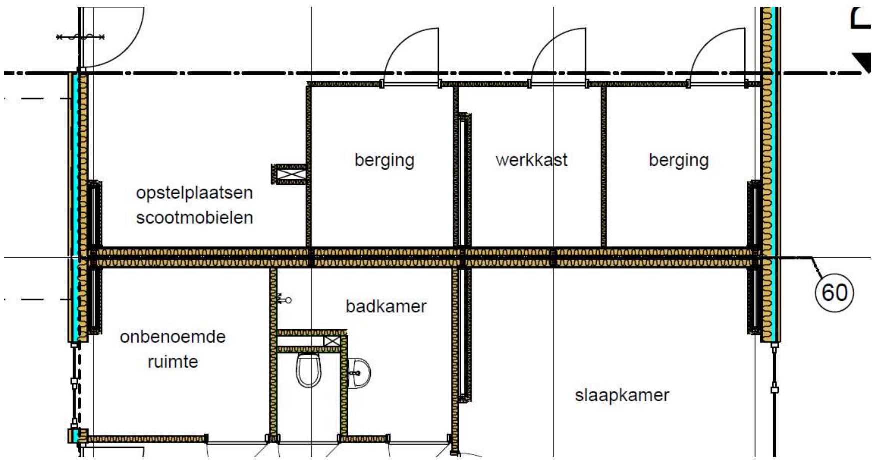


Figure 39. Location of bracings in floor plan (Pieters Bouwtechniek).

4.5 Assessment of case studies

The four studied cases will be subjected to a quantitative assessment to gain insight in the differences between the modules. The result of the individual assessment will be compared to determine the advantages and disadvantages of each module. The steps of the assessment will now be explained.

4.5.1 Properties

The assessment starts with a list of the building properties. These are the shape of the building, the number of blocks of modules and the number of modules per row as well as per storey. It is also mentioned whether or not there is a balcony present. However, the area of the balconies will not be considered in the assessment since it is not clear for each project what the exact area of the balcony is. Furthermore, the size of the balcony is usually standard, equal to the width of the module with a length of around 1.5 meters. Next, the sizes of the module are listed, external dimensions as well as the structural sizes. The usable area is calculated by reducing the total area with the area of longitudinal walls, façade walls as well as internal walls. The space efficiency factor can then be calculated, equal to the usable area divided by the total area of the module. The area of the façade is used to calculate the wall-to-floor area. The number of required stabilising elements is mentioned as well. Since concrete walls are self-stabilising, no additional stabilising elements are necessary for this type of module. An overview of the properties and efficiency of the modules can be found in Appendix E. The number of stabilising elements is also mentioned. These are the total number of additional stabilising elements in each direction. X-bracing is considered as 2 elements since both strips need to be connected.

4.5.2 Material use and environmental burden

To gain insight in the division of the material use for each module, the volume of each material in the wall, floor and ceiling structure is separated. Only the materials with a substantial volume are considered. These are the plasterboard cover, insulation, wood, concrete and the steel used in studs, beams and columns. The total volume is calculated based on the thickness multiplied by the length and width of the element. The façade walls are excluded in this comparison. This is done since most of the façade walls consist of large windows and it is unknown what type of window system is used. The internal walls are included in the calculation.
The impact of each material on the environment is calculated based on the embodied energy in MJ/kg and the embodied carbon in $kgCO_2/kg$. Therefore, the corresponding density of the materials is required. A study has been conducted by Hammond and Jones on the embodied carbon and energy of building materials and these values are representative for the building industry. They also provide average densities of the materials, and these values will be used as well. The values for the weight, embodied energy and embodied carbon of each material are marked with a bar as a percentage of the total value of the weight, embodied energy or embodied carbon of the module. This is done to provide a clear overview of where the environmental impact comes from per module type. A similar visualisation has been done for the comparison between the different modules. The values of the weight, embodied energy and carbon are calculated per square meter of the module, by dividing the value by the total area.

4.5.3 Conclusions

Based on the quantitative assessment of the four modules, several conclusions can be drawn for the various criteria of the assessment.

Functional

The width of the walls are more or less equal for each module. This is because the governing design requirement is based on the insulation capacity and this requirement is equal for each design. The concrete module differs from the other modules since concrete walls are used which are self-insulating with a slightly lower thickness, 130 mm compared to 160 mm for the other variants. The slight advantage that the concrete walls offer in terms of thickness is nullified by the reduced length of the walls, resulting in a more or less equal space efficiency factor as the other modules. The large differences in length between the four modules result in a significantly different wall-to-floor ratio. The Raines Court module has a ratio of only 0.29, while the North Orleans and Murray Grove modules have ratios of respectively 0.42 and 0.45 due to their small length. A comparison of the weight of the modules is done by dividing the total weight by the usable area. This is done to make an accurate comparison and to disregard the impact of the surface area of the modules on the total weight. This result shows a more or less linear increase in weight when extra concrete is used. The lightweight steel modules have an equal weight per square meter, the steel-concrete module with a thick concrete floor has a 150% increase in weight and the full concrete module has 260% increase in weight compared to the lightweight steel modules due to the additional concrete walls and ceilings.

Environmental

The embodied energy of the lightweight steel modules are significantly larger than the other modules due to the sheer amount of steel that is used. The embodied carbon is the highest in the full concrete module, simply because of the high concrete weight. The steel-concrete modules scores the best in terms of combined embodied carbon and energy.

Structural

Lightweight steel modules that are partially open, can be constructed in which the closed section has sufficient length to allow for two cross-bracings. By doing so, the intermediate supporting column will be loaded in tension by one of the bracings and in compression by the other bracing. Therefore, this column will not be subjected to any additional wind load compared to the edge columns and a continuous frame can be fabricated. The weight of the steel structure is very small and only has a small impact on the total weight of the module. When taller buildings are constructed using low-weight steel modules, the low weight of the steel structure results in the necessity of adding additional stabilising elements to prevent tension forces from occurring in the columns. Since there are only so little locations to place bracing elements in the modules due to open sections, lightweight steel modules have a maximum height in which they can be constructed.

The use of concrete in one or multiple sections of the module has a high impact on the total weight of the module. There is less room for optimisation in concrete use since it requires a certain thickness to achieve sufficient fire resistance and insulating capacity. When stacking up concrete masses to higher levels, the additional weight will result in less efficient use of the concrete in the load-bearing structure.

Each module type uses its own system to achieve longitudinal stability. Lightweight steel modules use one or multiple X-bracings along the steel wall system and concrete modules do not require any additional stabilising elements due to the presence of concrete walls. The Regioplein case has three internal spans and depending on the requirement of open sections, either single bracing or cross-bracings are used. When using single bracings, they are located at either side of the module for a favourable load distribution. The façade column which has a lower permanent load than the internal column, will be loaded in compression only during wind.

It is striking that each case uses a different system to achieve stability in the transverse direction. These systems are listed below:

- Murray Grove: X-braced façade walls
- Raines Court: Transverse stability using an access core, possibly combined with internal bracings
- North Orleans: Internal concrete walls
- Regioplein: Square tubes between column and floor at the façade walls and internal walls

The use of the stability system is highly dependant on the façade layout. When the facades consist of mostly walls, stability must come from either internal wall, such as in the North Orleans case or from an access core such as in the Raines Court case. When the facades however are partially closed, either a box profile is used as a single stabilising element, or the steel frame is stabilised with one or multiple X-bracings.

The advantages of using steel and concrete elements can be combined to design modules that are efficient at a greater height than is currently being done in lightweight steel or fully concrete modules. Using strategically placed bracing elements combined with a concrete floor slab will be necessary to design a module that is suitable for a height of at least 8 storeys. The assessment which includes the impact of the material use on the environmental burden can be used to lower the environmental impact.

5 Design concept

In this chapter, the demarcations for the design research will be explained as well as the materials and build-up of partition structures and load-bearing elements that are chosen based on the case studies. After these choices have been made, a summary of the design case is shown and functional requirements as well as sustainability methodologies are given to which the design should adhere.

5.1 Design demarcation

5.1.1 Function

A distinction between two types of apartments can be made for mid-rise buildings, residential and student apartments. Residential apartments consist of modules that can be combined to create larger living areas. Student apartments are smaller and consist of a single module. Since the goal of this project is to come up with a design that can be used to reduce housing shortage, it has been chosen to design for residential apartments. Student apartments are fixed to certain cities with universities and the application of modular design therefore is limited.

5.1.2 Module type

The internal width of the module is set to 3.5 m due to transportation requirements. The values for the length of the module are less limited for road transport. The length of the module will be determined based on the required area of the apartments (RDW, 2021). These are the external dimensions of the module, which means that the usable area will be smaller. The floor area of load-bearing walls as well as separation walls need to be considered to find out the usable area. The number of modules per floor depends on the capacity of the stability systems. Additional modules in the transverse direction lowers the horizontal forces on the stabilizing elements.

5.1.3 Height

The design case will have a height based on around 8 storeys. Self-stabilising steel modules currently go up to 5 floors in the Netherlands and use steel columns. Critical aspects when going up in height are preventing the occurrence of tension in stabilising columns as well as preventing large deformations due to the angular deflections and considering tolerances.

5.2 Module elements

5.2.1 Comparison of main load-bearing system

A comparison has been made between the three building materials steel, concrete and timber. This has been done to find out which material is most suitable for the design. An overview of the comparison is visible in Table 10 below. This table is used to come up with the materials of the partition structures and the load-bearing elements and these will be explained next (Wagemans, L. et al).

Table 10. Comparison of main load-bearing system.

	Steel	Concrete	Timber
Reasoning for	High design flexibility	Good insulating properties	Low environmental impact
	Low self-weight	Maintenance free	Low self-weight
	Many existing connections	Helps stability	
	Fast erection speed		
Reasoning against	Maintenance against corrosion	High self-weight	Few existing buildings
	Wall system needed	Low design flexibility	Low flexibility
		Less connection examples	Existing knowledge
		Slow erection speed	Shrinkage and swelling

5.2.2 Floor slab

A reinforced concrete slab provides support to the module floor and leads to high values for the acoustic insulation as well as fire resistance. The required reinforcement is calculated based on the bending moment acting on it due to its self weight and live load. This calculation can be found in Appendix P. The spacing of the reinforcement should not be greater than three times the depth of the slab. The initial sizing of a floor slab and the required reinforcement per m^2 can be done based on the imposed load. When a solid slab is used, the minimum depth is 120 mm based on the fire resistance requirement for R120 (NEN-EN 1992-1-2).

5.2.3 Ceiling

The design criteria for ceiling members is being able to support the self-weight of the ceiling itself as well as the loads applied during installation. The temporary construction load is 1 kN/m^2. Steel ceiling joists with a height between 100 and 150 mm are suitable for a structure with a span of 3.5m. The thickness of these joists is usually 1.5 mm with a centre-to-centre distance of around 500 mm. The temporary construction load is more or less equal to the snow load, therefore the upper module does not require a different ceiling system (Lawson et al., 2010). An example of how the ceiling structure looks like is shown in Figure 40 below.

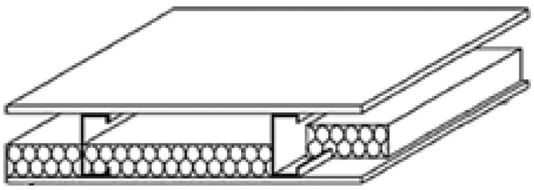


Figure 40. Ceiling structure (Lawson et al., 2010).

5.2.4 Walls

Steel infill walls will be used in each module and offer several advantages:

- Design flexibility, variation between small and large spans.
- Construction speed, favourable to tackle housing shortage.
- Low weight, less material is favourable for environmental reasons.
- Knowledge, many innovative connections available as well as reference buildings.
- Demountable option, ability to demount connections when bolts are used.

A partially open module is supported on two or more columns, which bear on the columns of the module below. When steel infill walls are used, the wall consists of 70 to 100 mm deep C-section studs either singly or in pairs at 600 mm centres. In between, mineral wool is used as insulation material and a rigid insulation board is added on the inside as well as a sheathing board on the outside of the wall, visible in Figure 41 below.

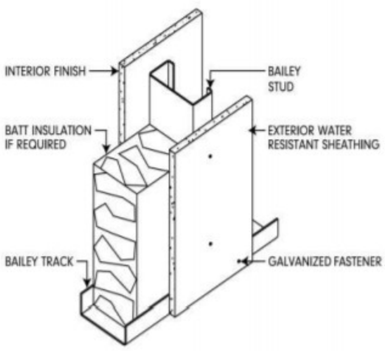


Figure 41. Layers in a steel infill wall (Bailey Metal Products, 2019).

The C-section studs are parallel flange channels with a very low thickness and only support the plasterboard on both sides. Standard thicknesses for infill walls consisting of mineral wool and plasterboard are 100 mm and 30 mm, respectively.

5.2.5 Columns

At the corners of each wall, larger corner columns are used either in the form of a square hollow section or a hot-rolled steel angle. The walls transfer all vertical loads to the foundation and help resisting horizontal loads. They provide attachments for other structural elements and local lifting points during construction on-site. Important design considerations are the number of internal columns along the longitudinal wall, based on the length of the wall and the required bracings in the wall, the span of edge beams and the difference in profile that internal columns have compared to edge columns.

5.2.6 Façade

An indication has been made of the weight of a glass-wooden façade structure. The façade has a roughly equal distribution of glass and wooden elements as visible in Figure 42 below. Standard thicknesses and the volumetric weight of both materials have been used to calculate the total weight in Table 11 below. This weight is used as additional load on the edge columns.

Figure 42. Impression of a glass-wooden facade (CirQ Wood, 2021).

Table 11. Facade properties.

	Unit	Glass panel	Wooden plate
Thickness	mm	15	25
Volumetric weight	kN/m^3	25.5	6.5
Façade area	m^2	10	10
Total weight	kN	1.9	0.8

1. $F_{glass} = 50\% * 10\ m^2 * 0.015\ m * 25.5\ kN/m^3 = 1.9\ kN$
2. $F_{plate} = 50\% * 10\ m^2 * 0.025\ m * 6.5\ kN/m^3 = 0.8\ kN$

The total weight of the façade therefore is 2.7 kN and counts as permanent load on the edge columns of the module.

5.3 Design case

In Table 12 below, a summary of the design case is given in which the important design choices are listed.

Table 12. Summary design case.

Function	Starters apartments
Floors	Around 8
Module width	3.5 m
Module length	Around 10 m
Vertical elements	Steel columns
Floor	Reinforced concrete slab
Wall	Steel infill walls
Stabilising elements	Internal bracings
Inter-module connections	Demountable

5.3.1 Partition structures

The build-up of the three partition structures, wall, ceiling and floor is shown in Table 13 to Table 15 below. The thicknesses of the mineral wool insulation and plasterboard cover layers are derived from standard partition structures and insulation requirements. The sizes of the steel studs are calculated using the occurring loads. The calculations can be found in Appendix N.

Table 13. Wall build-up.

	Thickness (m)	Height (m)	Weight (kN/m^3)	Resulting load on wall/m
Plasterboard	0,030	2,6	5,00	0,39
Mineral wool	0,100	2,6	0,45	0,12
	Area (m^2)	Height (m)	Number/m	Resulting load on wall/m
C-studs	0,00019	2,6	2,50	0,10
Plasterboard	0,015	2,6	5,00	0,20
Total summation				0,80

Table 14. Ceiling build-up.

	Thickness (m)	Span (m)	Weight (kN/ m^3)	Resulting load on wall/m
Plasterboard	0,030	3,5	5,00	0,26
Plasterboard	0,015	3,5	5,00	0,13
	Area (m^2)	Span (m)	Number/m	Resulting load on wall/m
C-studs	0,00040	3,5	2,50	0,14
Total summation				0,53

Table 15. Floor build-up.

	Thickness (m)	Span (m)	Weight (kN/ m^3)	Resulting load on wall/m
Concrete slab	0,120	3,500	25,00	5,25

5.4 Functional requirements and wishes

To effectively design floor plan layouts, a few required functional demands are considered. Some of these demands are measurable while others are immeasurable. All of these aspects are considered when designing concepts.

The required user demand is the income of daylight. Every apartment needs a window area of at least 10% of the floor area of the room. When an inner wall of the room is further away from the façade than 6 meters, the area is perceived as dark. These rooms can be used best for functions that do not

require daylight such as a bathroom. The desired user demands are there to increase the willingness of potential residents to choose for a specific building.
The first user demand is publicity. An iconic building is often an aim for a high-rise building. Since height is an important factor in creating an iconic building, the most iconic buildings are often the tallest buildings in large cities because they stand out against the other buildings. Apart from the building height, other important factors are the shape and façade of the building. A certain shape that is not yet present in the surrounding area makes that building stand out against the other buildings. The second user demand is outside view. A tall building gives views that are unavailable in other places of the city. The value of the view is increased by a difference in elevation. Having a unique view gives a feeling of exclusivity (Riad, J.).

5.5 Sustainability methodologies

To achieve a design for a sustainable building, the environmental impact and energy use needs to be reduced. There are four methodologies, set up by Joseph Danatzko and Halil Sezon, which can be used to achieve a sustainable design and these methodologies can also be combined.

5.5.1 Minimizing material use

The number of required materials for the design can be reduced in two ways during both the design of the floor layout, generally the task of the architect, and during the engineering phase. When designing the floor layout, the goal can be either to generate a layout that has the largest amount of usable space or to minimize the required material by making the layout as efficient as possible. During the engineering stage, different materials can be combined to create a more efficient structure, or the use of a single material can be optimized using complex calculations such as topology optimisation. The goal of a minimized material use is achieved in both ways by the architects and engineers. The complex material optimization has a few downsides. It is attributable to iterations and the complexity requires more time and additional drawings to complete the design. Additional resources may be required for the fabrication and the approval process becomes longer. These negative aspects are expected to increase the project costs.

5.5.2 Minimizing Material Production Energy

The energy costs that are required during the production of materials such as the gathering, mixing and refining of materials determine the total sustainability costs of the material. When industries and engineers better define the properties of materials, including those of the production, the sustainability of the structure will be increased. The use of the most sustainable material is reduced when a specific structural system is chosen beforehand, such as in the case of a complex lateral resisting reinforced concrete frame, when a more sustainable construction material could have been a masonry shear wall.

5.5.3 Minimizing Embodied Energy

The concept of minimizing the total energy used is based on evaluating the energy during the construction and operation phases and trying to find a minimum between them. In terms of the structure, a balance needs to be made between the building use and the façade design. This goal can be achieved by including adjacent structures in the design or by splitting the design into multiple smaller structures to allow for a better structural use. The structure needs a balance between the structural and architectural form to reduce the energy envelope of the structure. Applying this methodology is tied to the location, since the advantages of for example certain façade systems or energy generation depend on the solar conditions.

5.5.4 Maximizing Structural System Reuse

This concept is to design structural layouts that contain materials that will eventually produce the lowest amount of possible waste at the end of their life. Contrary to the most material efficient design, the goal is to achieve a layout that allows different structural uses and longer structural lifespans and the option to include reused elements in the design. The goal is to achieve more sustainability by including multiple uses for a structural system during the design phase. The engineer will have to assess the materials to be used beforehand and consider how it can be possibly reused after its initial service life. The possible reuse of a structure gives the owner financial incentives he can reoccupy it for a new use. The downside of including various possible functions of the design is that an optimum in functionality is not achieved compared to having a single function (Danatzko & Sezen, 2011).

6 Structural Design

The structural design possibilities are researched for a modular building with around 8 storeys. The internal module parts are set beforehand and will not change between the variants. These are the build-up of partition structures as well as the width of the module which is set to 3.5 m due to transportation constraints. These structures depend on the stability system and are designed based on fire safety and insulation requirements. An additional requirement is given to the design, at least one column bay needs to be open along the length to provide an open section between adjacent modules in an apartment. This is wished for in residential apartments in terms of functionality. By having these similarities in all variants, an accurate comparison can be made.

6.1 Design method

A structural design has been made for the module in both directions, longitudinal and transverse. The longitudinal direction is most important, since there are multiple parameters such as the length of the module as well as number of columns and stabilising elements. The transverse direction offers less freedom of design since the width is set to 3.5 m and the presence of doors and windows hinders the placement of bracing elements along the full width at some walls. Furthermore, the wind load can be reduced when more modules are placed in a row. Therefore, the design in longitudinal direction is the focus in the research. For the design in the transverse direction, the different options for stability will be mentioned and later verified on the two stabilising criteria.

The structural design in longitudinal direction consists of the following steps. First, the layout of columns and bracings along the length is designed based on the column load per meter of beam length. Since the width is standard, the column load in kN only depends on the loaded length between two adjacent columns for internal columns and the one adjacent column for stabilising edge columns. In case of stabilising edge columns, additional weight comes from the weight of the transverse façade. The first stability verification is preventing the occurrence of tension in the stabilising column. The tension force during wind needs to be counteracted by the dead load of the module. This verification has been done for multiple layouts and a safe design can be made in four variants. The layout of columns and stabilising columns for these variants are shown in Figure 43 below. It is shown for a bracing layout that uses a double frame with an intermediate column. These columns are marked in blue and are not continuous along the height. Therefore, they do not carry any permanent weight. Instead of using a double frame, a single frame with only one bracing in each direction can also be used, however that results in a less stiff frame. The variants are given names based on the layout in which 'O' refers to 'Open section, 'S' refers to 'Single bracing' and 'F' refers to 'Frame using cross-bracings. The figures below do not have the actual element lengths and are only shown to visualize the different layouts.

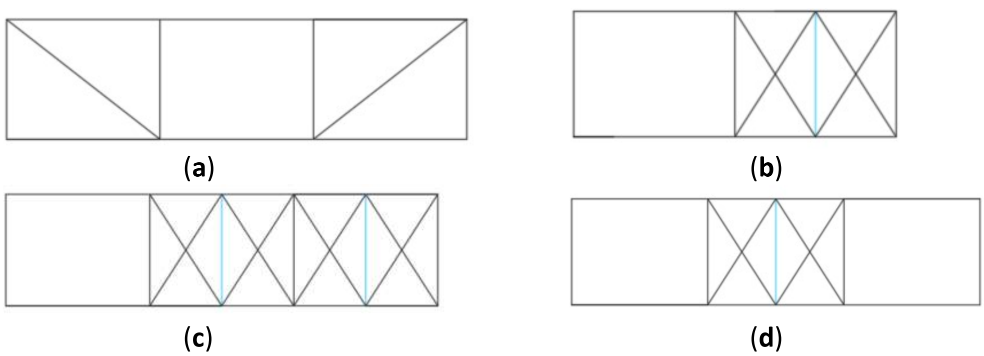


Figure 43. Design variants: (**a**) S-O-S, (**b**) O-F, (**c**) O-F-F, (**d**) O-F-O.

The next step in the design is the dimensioning of the beam and column profiles based on strength verification. These calculations can be found for each variant in Appendix P. The difference in loaded width for the columns and span for the beams result in different dimensions and thicknesses. The second stability verification is on horizontal displacement. This verification can be found in chapter 7.3.

6.2 Longitudinal direction

There are four variants within the boundaries of the starting points which differ from each other on the following points. The number of column bays is either 2 or 3. Having additional fields result in either an inefficient structure or a length that is too long. Some variants have open walls at either the entrance or at the end of the length as well as both parts, which is favourable compared to open section in the middle due to the allocation of the living room in the module. Single bracings, which only use a single bracing per side, as well as cross bracings can be used to obtain additional stabilising capacity when necessary. Cross-bracings however have a less favourable distribution of forces compared to single bracings at both ends of the module. The span of the bracing elements is determined based on the required length to prevent tension in the columns. The different lengths for the bracing elements result in a difference in total length between the variants.

6.2.1 Variants

The first variant S-O-S has a length of 12 m and only requires two single bracings while the other variants require additional bracings. The tension load acts on the internal columns which carry twice as much floor area compared to the edge columns and therefore have high compression forces which leads to an efficient structure. The second variant O-F is the only variant with only one internal column, this variant has the smallest length of all variants, 10 m. One section uses cross-bracings with an extra column to reduce the displacements. An open section at the edge is favourable since a large living room can be created with direct sunlight. The third variant O-F-F is similar to the O-F variant but has one open section and twice as much stabilising elements. This is done to reduce the span of the beam and have additional stabilising capacity. The fourth and last variant O-F-O stands out from the other variants by having two open sections and a length of 10 m. The internal columns are high in compression to counteract tension load during wind.

6.2.2 Modelling

Each layout that has already been verified on column compression by hand, has been modelled using Technosoft to calculate the horizontal displacement at maximum height. Different load combinations as well as wind directions are used to obtain the maximum horizontal displacement. The model for each variant has a few characteristics and is shown for variant 4 O-F-O in Figure 44.

The first characteristic is that the bracings are not supported at full height but eccentrically due to the height of the horizontal partition structures, leading to additional displacements. The vertical connection has only been made at the vertical grid lines of the three supports. This means that the column in between the double-crossed bracings is not continuous. The vertical connection between two storeys is modelled with the same stiffness as the columns. The ceiling and floor elements of two modules above each other are modelled as separate elements with a small difference in height. They both have hinged connections to the columns. The intermediate column along the span of the beam has been given the same stiffness as the adjacent stabilising column to facilitate equal lateral force distribution between the active bracings.

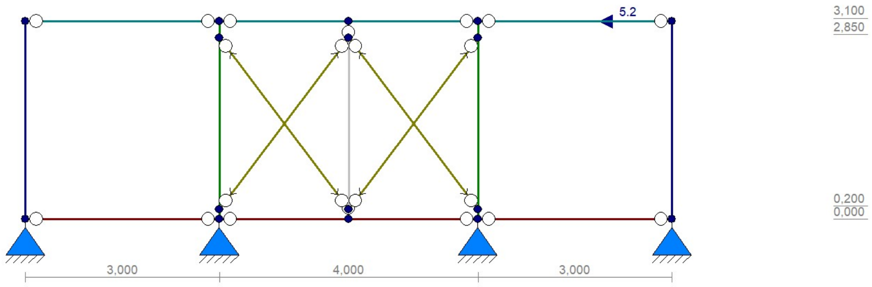


Figure 44. Single module structural model in (Technosoft).

6.3 Transverse direction

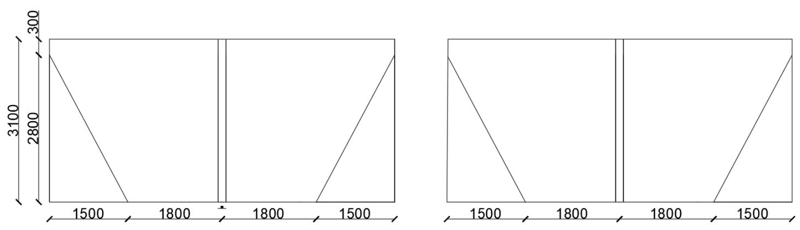
For the stability in the transverse direction, several design alternatives are drafted. The design has several changes from the longitudinal direction. Since an apartment consists of two modules, a repetitive design will be made for two modules in width for both facades. On the front façade, one module is used as the entrance and requires a large opening. The adjacent module has the possibility to be fully closed. On the back façade there are usually several windows present.

6.3.1 Variants

For the stabilising elements, either strip elements or box profiles can be used. The alternatives are visible in Table 16 below as well as Figure 45 to Figure 49. The first alternative consists of two box profiles with size 100*100*8 mm with a width of 1.5 m on either facade. By doing so, a width of 1.8 m is left for the entrance or a window. The second alternative uses strip profiles with size 150*10 mm, of which one module is braced over its full width and the other one along parts of the width. The third alternative uses a mix of box profiles as well as strips for maximum stabilising capacity.

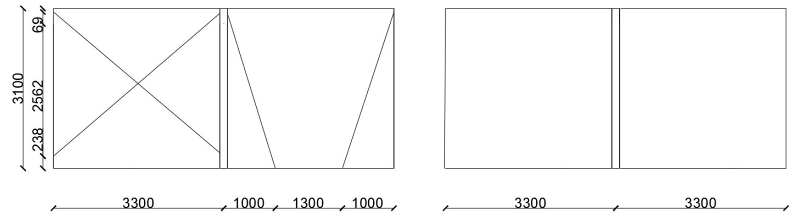
Table 16. Transverse frame alternatives.

	Front façade		**Back façade**	
	Left module	Right module	Left module	Right module
1. Box profiles	Box 1.5 m	Box 1.5 m	Box 1.5 m	Box 1.5 m
2.1 Strips	Strip 3.2 m	2x Strips 1.0 m	Open	Open
2.2 Strips	Strip 3.2 m	2x Strips 1.0 m	2x Strips 1.0 m	Open
3.1 Mix	Strip 3.2 m	Box 1.5 m	Box 1.5 m	Open
3.2 Mix	Strip 3.2 m	Box 1.5 m	Box 1.5 m	Box 1.5 m



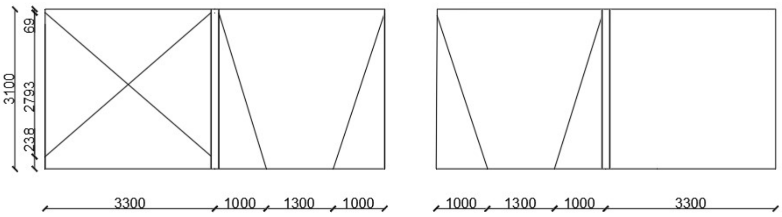
(**a**) (**b**)

Figure 45. Variant 1 Box profiles: (**a**) Front facade, (**b**) Back facade**.**



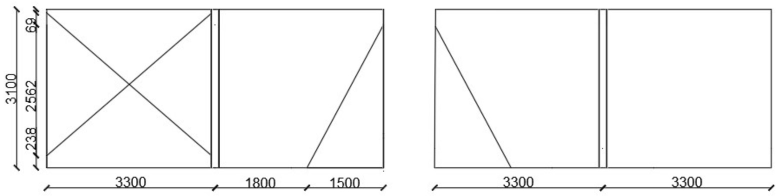
(**a**) (**b**)

Figure 46. Variant 2.1 Strips: (**a)** Front facade, (**b**) Back facade.



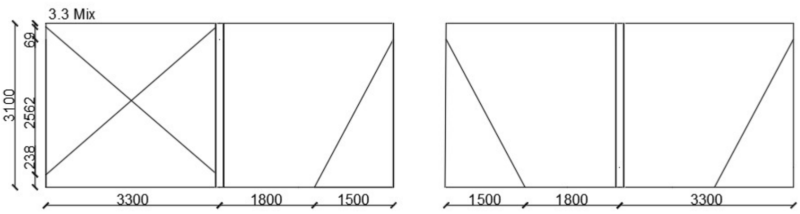
(**a**) (**b**)

Figure 47. Variant 2.2 Strips: (**a)** Front facade, (**b**) Back facade.



(**a**) (**b**)

Figure 48. Variant 3.1 Mix: (**a)** Front facade, (**b**) Back facade.



(**a**) (**b**)

Figure 49. Variant 3.2 Mix: (**a)** Front facade, (**b**) Back facade.

6.3.2 Modelling

A longitudinal length of 12 m is used since this is the largest length across the variants. The connection between the modules is modelled as a hinge to prevent the transfer of shear forces through and to have an equal force distribution between the modules when the stiffnesses are equal. The total number of modules in row is taken as 8. This number may be increased in case additional capacity is required. The wind load is taken as equal on each storey floor, except for the upper storey in which it is halved.

6.4 Floor support

Three different floor slab supports are proposed, visible in Figure 50 below. These supports differ from one another in terms of height, material use and complexity of the manufacturing. The aim is not to find the most suitable type, but the implications on the module. The design verification of supports 1 and 2 can be found in appendix P. The support has been dimensioned based on a beam span of 4 m. In the first system, the concrete slab is supported by a PCF-section. This is a simple connection in which the slab is supported at its edges by the top flange. Vertical shear studs may be required for the transfer of shear forces. It has a very low material use due to the stiffness of the steel section. However, a large height is required compared to the other connections. The second floor system is an integrated concrete floor slab and concrete support beam. It has a low complexity due to sole use of concrete, however a large number of materials is used due to the dimensions of the concrete beam. It results in a lower height compared to the steel section. The third system consists of a concrete beam integrated in a C-section, resulting in a reduced structural height and additional load-bearing capacity. Therefore, lower structural height and total height is necessary. However, there is a high complexity due to the addition of shear studs and integration of steel and concrete.

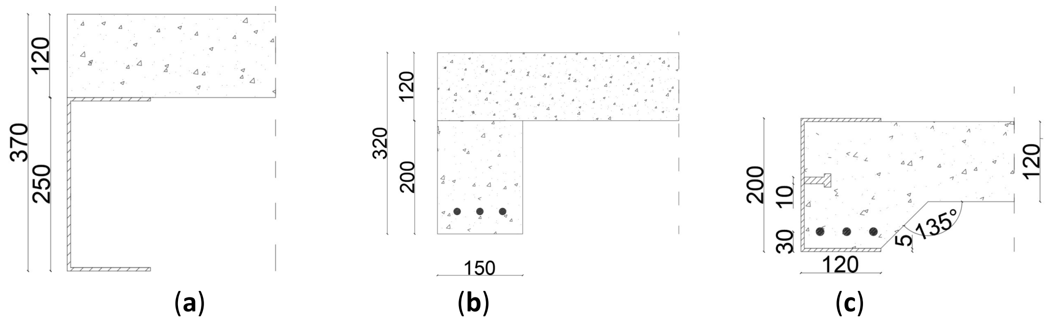


Figure 50. Floor slab supports: (**a**) PFC-section, (**b**) Concrete beam, (**c**) Integrated steel-concrete beam (own work).

6.5 Comparison of floor supports

The three systems are compared on three criteria, height of the floor zone, material use and environmental impact. The environmental impact is calculated by multiplying the area of the material by the total beam length in a module, followed by the corresponding density in kg/m^3 and then the embodied carbon in kgCO_2/kg.

Table 17. Comparison of floor support systems, values per module.

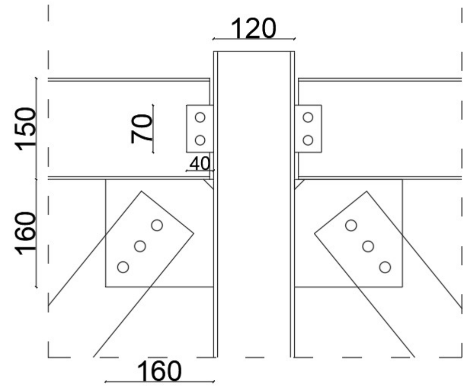
	Unit	**System 1 C-section**	**System 2 Concrete**	**System 3 Integrated**
Floor zone height	mm	370	320	200
Material use	kN	3,4	15,2	9,7
Steel		*3,4*	*0,7*	*3,1*
Concrete		*0*	*14,5*	*6,6*
Environmental impact	kgCO_2	523	428	623
Steel		*523*	*108*	*478*
Concrete		*0*	*319*	*145*

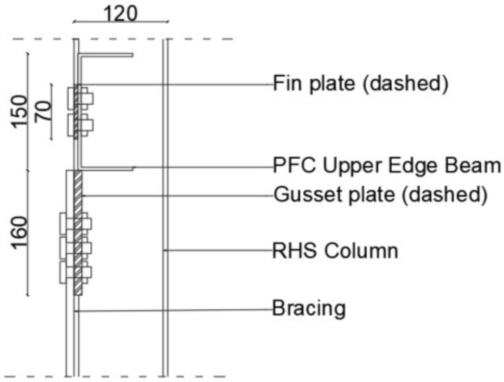
Depending on the wishes of the client, a choice of the floor system can be made. The C-section has low scores for floor zone height and environmental impact; however, it offers some other advantages. The reduction in weight can be beneficial when a module with a large length is used since it is harder to transport and install a module on-site with a large weight. Furthermore, the module itself is demountable when bolted connections are made in steel.

6.6 Bracing connections

The upper connection of the bracing to the column-ceiling beam joint can either be made at same height as the ceiling or below the ceiling level. In case a bolted fin-plate is used to connect the upper edge beam with the column, there is no space left for the connection of the bracing at same height.

Figure 51 shows how the connection of the bracing can be made below the upper edge beam, leading to some eccentricity.





(**a**) (**b**)

Figure 51. Bracing connection below ceiling height: (**a**) Side view, (**b**) Cross-section (own work).

Another option is to use an end-plate to connect the upper edge beam with the column. This leads however to bolts running through the hollow column instead of only on the outside. The advantage is that the bracing is connected at a larger height and there is no eccentricity.

The lower bracing support is depicted in Figure 52 and consists of a bolted connection of the bracing with a gusset plate, which is welded on the edge between the floor slab and the column. The eccentricity of the lower connection depends on which floor system is used. The three before mentioned floor systems each have a different structural height and therefore a different eccentricity of the bracing. The calculation of the eccentricity can be found in chapter 6.8.

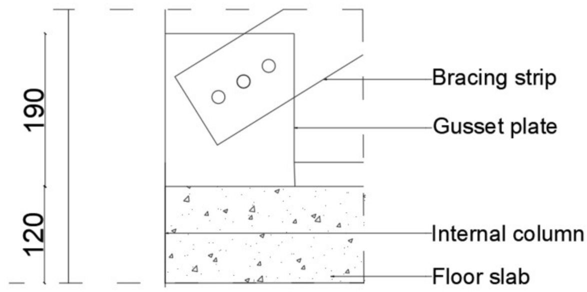


Figure 52. Side view of lower bracing connection.

6.7 Inter-module connection

6.7.1 Comparison of connection types

There are many existing connection types that can be used to transfer horizontal and vertical forces between light steel elements. In order to do so, several techniques can be applied. Columns often have a welded base plate which are connected using a transfer plate. A comparison of over 20 connection types for column connections in modules by Srisangeerthanan et al (2020) has been studied to find the most suitable type of connection for this research. The connection types in this paper are weighted based on structural (S), manufacturing (M) and construction (C) requirements. The complete list of connections and the comparison between them can be found in Appendix H.

Two different joints are to be designed and these are shown in Figure 53 below. The first joint is at four edges of the modules and connects four columns and lateral force transfer in the transverse direction is considered. The second joint is along the span of the modules in which only the columns above each other are connection and lateral force transfer in the longitudinal direction is considered. Only in case of transverse stabilising elements along the length of the module, the joint should connect also adjacent columns. The lateral forces in the transverse direction are smaller since the length of a single module is considerably smaller than the width of all modules combined.

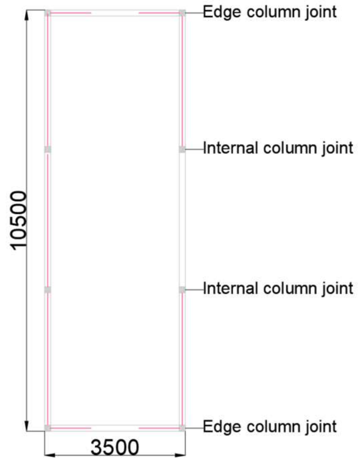


Figure 53. Location of joints in a module.

To find the most suitable connection type, the studied connection types are assessed based on the following requirements. The structural requirements are transfer of vertical as well as lateral forces. The manufacturing requirements are low complexity of connection parts and low complexity and requirements of post-manufacturing integration of parts. This is important since the project consists of many dozens of modules. High complexity and intensive on-site work will highly reduce the efficiency of modular construction. The construction requirements are a low number of tools and operations for the inter-module connectivity and the capability to be easily demounted after its initial lifetime for sustainability reasons. These requirements are used to find the most suitable type of connection.

For the stabilising edge joint the connection type by Gunawardena et al has been chosen. For the internal joint, two connection types are selected with clear differences. These are the connection type by Styles et al and Lacey et al. These connection types score the best on the requirements that were set beforehand. The edge joint design is more complex than the internal joint since it involves more elements and will be explained more extensively.

6.7.2 Stabilising edge joint

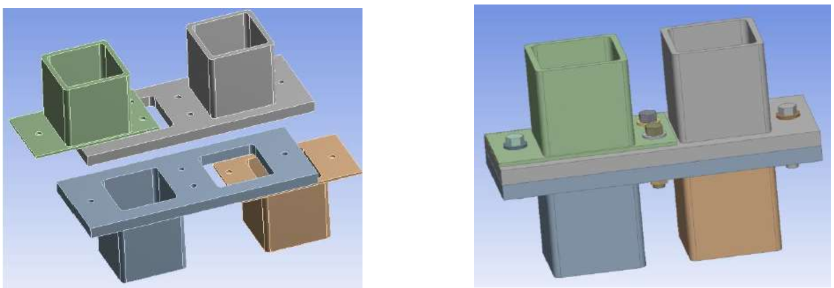
The chosen type of connection by Gunawardena et al connects in total 4 modules over two floors which are adjacent. The 3D schematization as well as the top view and side view of the joint are shown in Figure 54 and Figure 55 to give a clear visualisation of how the joint looks like. Two of the four columns have a thin welded end plate, and the other two columns have thick welded end plate with a larger length and a hole at the location of the adjacent column. The exact dimensions of the joint are shown in Table 18 below.

Table 18. Gunawardena connection element dimensions.

Bolt type	M12, 8.8
Thin plate thickness	6 mm
Thick plate thickness	25 mm
Plate width	200 mm
Plate length	500 mm

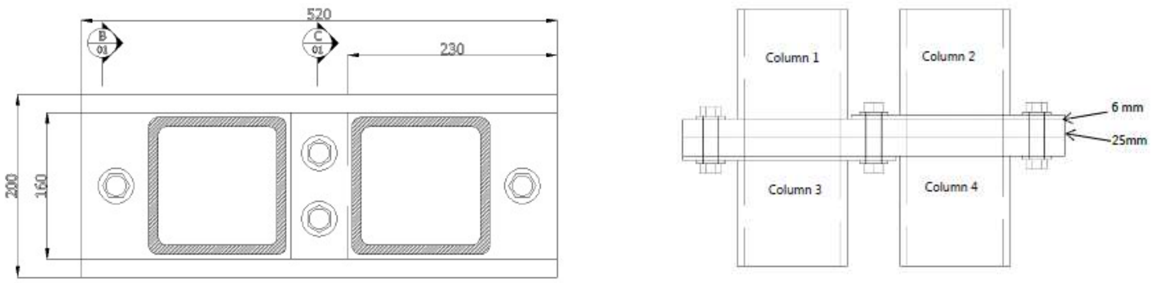
These sets of columns are diagonally the opposite of each other so that the total plate thickness is equal on both sides.

The construction sequence of positioning the four module is visible in Figure 54 and will briefly be explained. First, the lower right module is positioned, followed by the lower left module and the thick plate is positioned above the lower thin plate. Next, the upper right module is placed with the end plate again positioned above the end plate below it. At the end, the upper left module is positioned, and the bolted connection can be made (Gunawardena et al, 2016).
Due to the presence of adjacent modules and horizontal elements below and above the joint, there is only little space available to connect the modules on-site. The two bolts in the middle can be installed from the outside since there are no elements above the holes of the bolts. The two edge bolts however can not be installed from the outside since the presence of floor beams above the holes hinders the placement of the bolts. A solution to this problem is to lower the position ceiling beams below the holes to provide sufficient space to make the bolted connection from the outside.



(**a**) (**b**)

Figure 54. Gunawardena Joint 3D schematizations: (**a**) Order of module installation, (**b**) Post installation and bolting (Gunawardena et al, 2016).



(**a**) (**b**)

Figure 55. Scheme of bolt interaction: (**a**) Top view, (**b**) Side view (Gunawardena et al).

Verification

The connection will first be checked on its capacity to resist the applied loads. The checks that are done are in accordance with the norm EN 1993-1-8 Design of steel structure, part Design of joints. The first check is to calculate the nominal shear capacity which depends on the number of shear planes. The number of plates is 3 for the bolts in the middle and 2 for the bolts at the edges of the connection. However, there is only 1 common interface along the full connection length which is between the two thick plates. Therefore, only one shear plane is considered for the design. Two more checks are done for the bearing and tear-out of the plies, with the thin plies being critical. Due to the geometric position of the connection, it is treated as slip critical. Stiffness is created by tightening the bolts to hold the connection together. The tension force needs to be large enough so that the shear is transferred by the structural members and not the bolts. The slip coefficient of a steel surface needs to be above 0.30 and this value can be reached by treating the surface with a wire brush or by painting. After the slip stage, the connection turns into a bearing-type joint, and the loads are transferred through the bolt shear and connection plate bearing. The displacement stiffness of the overall connection is different for the slip stage and load-bearing stage. In the load-bearing stage, the displacement stiffness is calculated by adding up the shear stiffness and the tension stiffness. The calculation can be found in Appendix F.

6.7.3 Longitudinal non-stabilizing joint

Two alternative connections are proposed for the inter-module connection for the internal columns. The first alternative by Styles et al is depicted in Figure 56 and is similar to the previous joint by Gunawardena et al. The joint consists of a base plate welded to each column and these plates are connected by two bolt rows. During the manufacturing on-site, the bolt row on the outside of the module has sufficient space available to install the bolts since there is no adjacent module. When the adjacent module is placed, there is no more space left to install the outer bolt row of the second module which can be seen in Figure 57 below. A solution to this problem is to make a provision in the wall to create space to reach the connection. The internal bolt row has no direct access as well for both modules. Therefore, another provision is required and this time in the floor slab.

The slip stiffness of the connection is high compared to the connection by Gunawardena and have a value for k_{slip} of 21 kN/mm. The calculation for the stiffness against displacement of this connection can be found in Appendix H. This displacement stiffness is low compared to the connection by Lacey et al since bolts are used in this connection. The displacement in the slip stiffness is 1 mm, equal to the hole clearance.

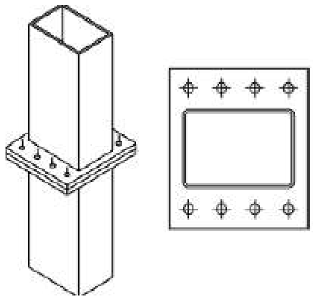


Figure 56. Bolted plates inter-module connection (Styles et al).

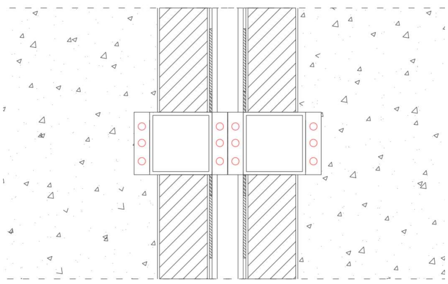


Figure 57. Top view of internal column joint (own work).

The rotational stiffness of the inter-module joint has been calculated as well. This has been done to see if it classifies as a simple connection, semi-rigid connection or a rigid connection. Using the stiffness coefficients that are used for a connection between columns, it has been calculated that the connection can be considered semi-rigid. This calculation can be found in appendix I.

The second inter-module joint consists of a shear key that is made up of two square hollow sections (SHS) with a transfer plate welded in the middle of the SHS and can be seen in Figure 58. The transfer plate (P1) has a hole in its centre that allows a threaded rod to pass through the shear key and the module columns that it connects. These columns both require an access opening and a second plate (P2) that is welded within the columns. The assembly on-site consists of the following steps. The shear key component is placed on top of the lower module column. Then, the upper module is lifted above the lower module. The shear key is used to position the upper module and then the module can be lowered onto its final position. After it has been placed, the opening in the SHS columns allows the tie rod to be tensioned from inside the modules.

The slip stiffness of the joint is high, compared to the previous mentioned joint by Styles et al. The slip load is for different specimens around 50 kN with a slip of only 0.05 to 0.01 mm. This displacement is neglectable when comparing it to the allowable storey displacement which is in the order of 5 to 10 mm (Lacey et al., 2019).

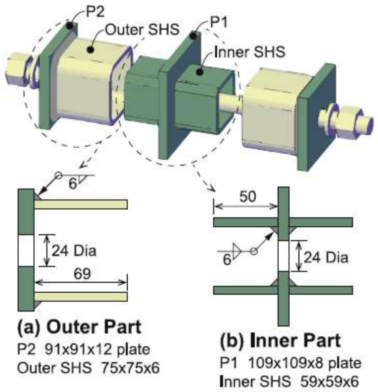
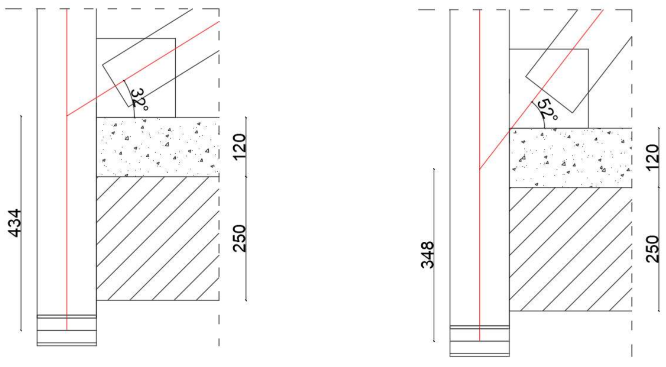


Figure 58. Components of shear key inter-module joint (Lacey et al., 2019).

6.8 Eccentricities

It has been researched what the values for eccentricity are when using different options for the floor slab support, ceiling connection and whether or not single or double frames are used. The calculation of the lower and upper bracing eccentricity consists of the following steps. First, the angle of the bracings to the horizontal is calculated for single as well as double frames. A free height of 2600 mm is used and a width of 4000 mm which is standard for most variants. The direction of the centre of the bracing is extended to the intersection of the column centre, visible in Figure 59 and Figure 60 below. Next, the distance from this intersection to the end of the storey level is measured, considering the thickness of the inter-module joint which is equal to 62 mm as well an open space of 30 mm per side. This procedure can be done for the three different floor systems as well as the two options for ceiling connection.



(**a**) (**b**)

Figure 59. Lower bracing eccentricities using PFC-section support: (**a**) Single frame, (**b**) Double frame (own work).

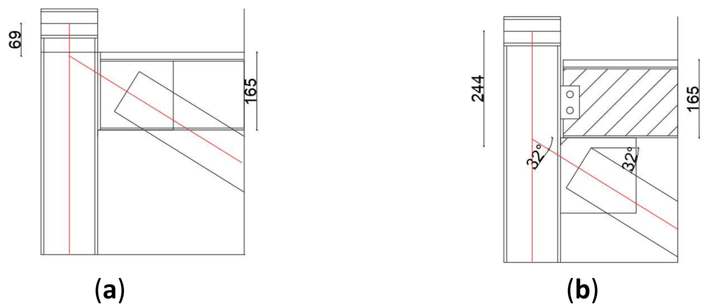


Figure 60. Upper bracing eccentricities using PFC-section support: (**a**) Single frame, (**b**) Double frame (own work).

The lower and upper eccentricities for the three floor supports as well as the options for the ceiling connection are shown in Table 19 and Table 20 respectively below.

Table 19. Lower eccentricities for three floor systems using either a single or a double frame.

		Lower eccentricity (mm)
C-section	*Single frame*	434
	Double frame	348
Concrete beam	*Single frame*	384
	Double frame	298
Integrated beam	*Single frame*	264
	Double frame	178

Table 20. Upper eccentricities for ceiling connection using either a single or double frame.

		Upper eccentricity (mm)
Below ceiling height	*Single frame*	244
	Double frame	102
At ceiling height	*Single frame*	69
	Double frame	0

The results of all 12 possibilities are shown in Table 21 in which the total eccentricity is shown. It can be seen that there is a large variation in eccentricity with the lowest eccentricity being only 178 mm per storey and 678 mm being the highest eccentricity.

Table 21. Total eccentricity for different combinations of floor systems and ceiling connections.

		Below ceiling height (mm)	**At ceiling height (mm)**
C-section	*Single frame*	678	503
	Double frame	450	348
Concrete beam	*Single frame*	628	453
	Double frame	400	298
Integrated	*Single frame*	508	333
	Double frame	280	178

The behaviour in terms of horizontal displacement of a single storey with eccentricity is different from the whole building. This is due to the small internal moments that occur in the columns which reduce the impact of eccentricity. To calculate the impact of a single storey eccentricity on the whole building, the total eccentricity has been reduced by the displacements due to rotation, bracing elongation and column compression. By doing so, the effect of the eccentricity can be isolated. This has been done for eccentricities up to 600 mm and can be found in Chapter 7.3.

6.9 Foundation

A design for a foundation of a modular building will be made to find out the base rotation, which will result in horizontal displacement along the height of the building. The design of the foundation is based on the foundation load due to the permanent and variable load of the module as well as wind load. An extensive calculation in which a strong first sand layer is present is shown in appendix J. This method is also used to find out the rotation of the foundation when a weaker first soil profile is present and longer piles are used.

6.9.1 Load

The load on the foundation piles has been calculated based on the weight of a single module in which wind load is the governing variable load. The SLS values of the permanent and variable load have been calculated based on a standard module with a width of 3.5 m and a length of 12 m. This value can change between the variants but for the foundation calculation only one value will be considered. The final load on the foundation pile can be found by multiplying the G_{total} and $Q_{2,res}$ by the number of floors, 8 in this case and dividing it by the number of piles along the length which is 4. The wind load has been calculated based on the moment of inertia I_p of the pile group, this will be explained later in this chapter.

6.9.2 Displacement

The horizontal displacement of a pile group in longitudinal direction will be calculated based on the governing wind load on the foundation. The change in pile load during wind loading results in a difference in length and a rotation φ at the base of the building, visible in Figure 61. This value is used to calculate the horizontal displacement u_h at the top of the building (Structural Calculations of Highrise Structures, 2017).

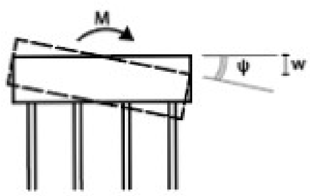


Figure 61. Base rotation of the foundation during wind.

6.9.3 Result

The foundation on a weak and strong soil with a difference in pile length and diameter show similar results for the horizontal displacement u_h at maximum height. When a strong soil is present, round piles with a length of 12 m and 0.25 m diameter are required and the obtained horizontal displacement is 2.88 mm. In case of a weaker soil, the piles require a length of 19 m and 0.28 m diameter, resulting in a larger horizontal displacement, 3.66 m. These values are both very small when compared to the maximum allowable displacement of a 24.8 m tall building, which is 49.6 mm.

7 Verification

This chapter covers the structural verifications for each variant. This will be done first for the strength of the load-bearing elements, followed by the connections between elements and then the stability of multiple storeys. At the end, an overview is given for the stabilising capacity of each variant for different number of storeys.

7.1 Element strength verification

The load-bearing elements in each variant are verified on strength first. The calculations that are done will briefly be explained in this chapter. All calculations can be found in Appendix P.

7.1.1 Load combinations

Three load combinations are considered. In the first combination the permanent load of the module is governing (Fu.C.1) , in the second combination the variable load in residential apartments (Fu.C.2) and in the third combination the wind load is governing (Fu.C.3). A resulting force in kN/m is calculated that acts on the floor beam due to permanent load and variable load. This force is only dependant on the weight of the partition structures and width of the module and is therefore applicable to all variants. The resulting force on each column can be found by multiplying this load by the loaded span and adding the self weight of the column.

7.1.2 Floor slab

The floor slab is designed based on the criteria of fire safety for 120 minutes. The corresponding concrete thickness is 120 mm (NEN-EN 1992-1-2). The required reinforcement is calculated based on the self-weight and variable floor load.

7.1.3 Beams

The two strength verifications that are done for the ceiling joist, upper edge beam and lower edge beam are on bending stress and deflection. A similar verification has been done when using a concrete edge beam instead of a steel section.

7.1.4 Columns

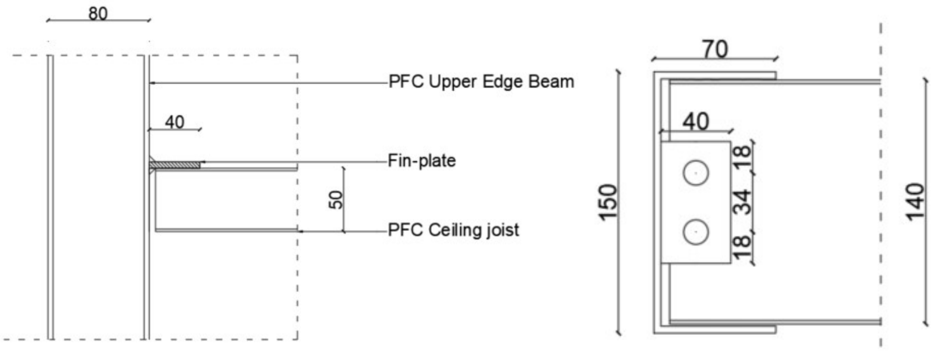
The ultimate limit state verification of the columns will be done for flexural buckling when using a closed Square Hollow Section (SHS) as well as a SHS with an access opening for different load combinations. Apart from this verification, it has also been verified whether or not the column is able to withstand the combination of axial for and internal bracings, resulting from the eccentricity of the bracings.

7.2 Internal connections verification

A verification has been done for the different internal connections in the module. These are the ceiling connections between the ceiling joist and the edge beam, the lower and upper edge beam and column and the connection between the bracing and the gusset plate near the edge beam-column joint. The end, edge and spacing distances are chosen based on the minimum lengths and required resistance of the connection (Design Manual Steel Structures II). The same goes for the other properties of the plated connection, such as the bolt strength, plate thickness and number of bolts. The values for the occurring load are from one of the variants and are based on the standard module partition structures and module width. The complete calculation are visible in Appendix Q. The Excel sheets that are used to do these calculations are visible in Appendix R.

7.2.1 Connection 1 - Ceiling joist to upper edge beam

The connection between the C-section ceiling joist and the C-section upper edge beam will be a bolted connection using a fin plate, welded to the edge beam. The steel horizontal frame consists of these elements and can be manufactured as a flat frame in which each connection is bolted. Figure 62 below show the fin-plated connection between the upper edge beam and the ceiling joist. The height of the ceiling joist is 10 mm smaller than the edge beam so the connection can be made.



(**a**) (**b**)

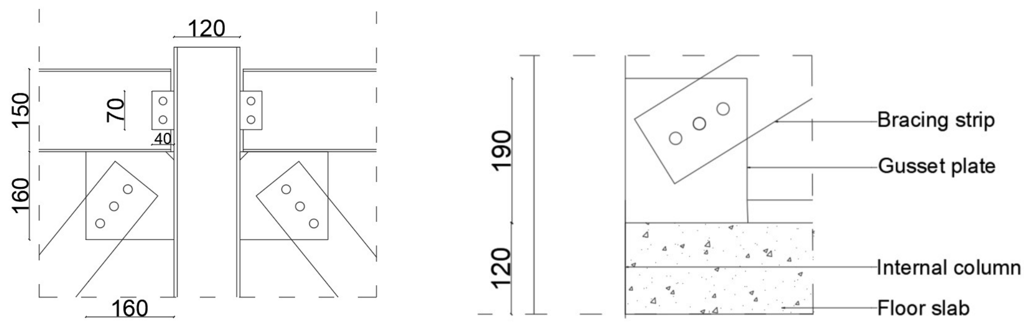
Figure 62 Ceiling joist-Edge beam connection: (**a**) Top view, (**b**) Cross-section (own work).

7.2.2 Connection 2 - Upper edge beam to column

The connection between the PFC upper edge beam and the RHS edge column uses a fin plate that is welded to the column. This is done to prevent bolts going through the column in case of a bolted end plate in which either bolts go completely through the column or access hole are required to tighten bolts that go through the column flange. The detail of this connection in Figure 63 is visible after the next connection is treated since the edge beam-column connection is part of the joint in which the bracing is also attached to these elements through a gusset plate.

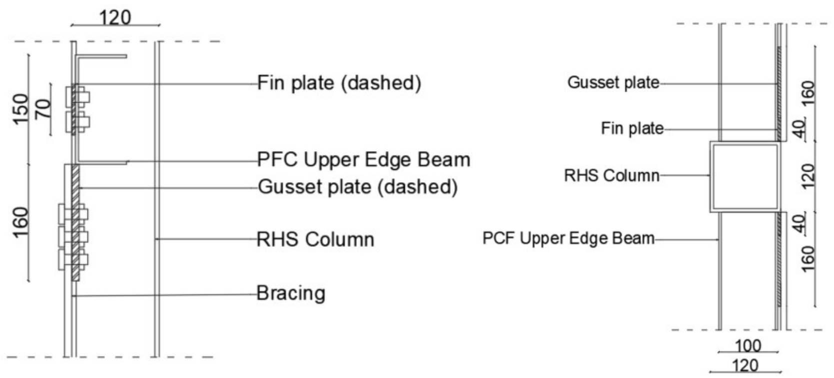
7.2.3 Connection 3 - Bracing to gusset plate

The diagonal bracings will be connected at the top and bottom using a gusset plate and three bolts shown in Figure 63 and Figure 64. This is done to facilitate an equal stress distribution from the bracing towards the column. The joint at the top of the column shows the fin plate connection of the upper PFC edge beam to the column as well as the gusset plate connection of the diagonal bracing. The internal column is connected to the edge beam and bracing on both sides.



(**a**) (**b**)

Figure 63. Bracing to gusset plate connection: (**a**) Upper connection, (**b**) Lower connection (own work).



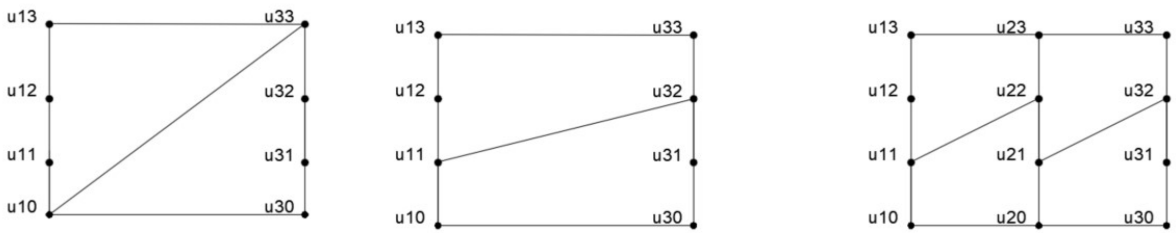
(**a**) (**b**)

Figure 64 Connections of fin plate and bracing to column: (**a**) Cross-section, (**b**) Top view (own work).

7.3 Stability verification method

The two main verifications for stability are checking whether tension can arise in one of the stabilising columns and calculating the maximum horizontal displacement during wind loading. When calculating whether tension will occur in the column that carries the lowest amount of permanent load, a favourable load factor of 0.9 will be applied. Additionally, a load factor for second order effects and imperfections will be used as well. The horizontal displacement will be calculated for the longitudinal as well as the transverse direction. In the longitudinal direction, three frames are considered as shown in Figure 65. The first frame is the simplest frame in which the bracing is connected concentric to the beam-column joint. In the second frame the bracing is supported eccentrically with its distances exaggerated for visibility. The third frame consists of two eccentrically supported bracings with an internal, non-continuous column.

It should be noted that the bracing in opposite direction of the drawn bracing is not shown for simplicity and that the eccentricities are exaggerated. The nodes that are part of the system are numbered to clarify the explanation that continuous below.



(**a**) (**b**) (**c**)

Figure 65 Schematization of three braced frames: (**a**) Concentrically single braced, (**b**) Eccentrically single braced, (**c**) Eccentrically double braced frame (own work).

7.3.1 Frame 1: Concentrically braced

The horizontal displacement of a concentrically braced frame can be calculated by summing up the individual storey displacements due to the components u_1 to u_3, these components will be explained in Table 22 below.

Table 22. Total displacement components.

	Element involved	Cause of displacement	Wagging-tail effect
U1	Bracing elongation	Tension force in bracing	No
U2	Column rotation	Displacement of connection between compressed column and bracing	No
U3	Floor rotation	Difference in N_{column} from wind and live load in left and right column	Yes

U1 Bracing elongation

The joint of the bracing and the column moves horizontally as a result of the elongation of the bracing. The horizontal displacement u_1 can be calculated using the equation 7.1 below.

$$\boldsymbol{u_1 = \frac{H_{wind} * {L_d}^3}{A_b * E_s * L_h^2}} \tag{7.1}$$

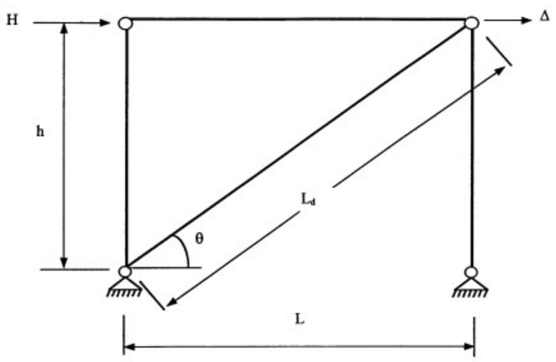


Figure 66. Parameters in a braced structure.

U2 Column translation

The wind loads results in compression in one column and tension in the other column. The weight of the module will result in compression in each column and the value of the load is dependant on the area that the column carries. The horizontal displacement due to column shortening or elongation can be calculated using equation 7.2.

$$u_{2,h} = \frac{N_{column} * h_{storey}^2}{E_s * I_c * L_{h,bracing}} \tag{7.2}$$

U3 Floor rotation

The difference in load acting on the stabilising columns results in a difference in vertical sag of the column and as a result the connecting beam will be tilted. Since the upper floors are vertically aligned, they will follow this rotation. The rotation φ_i can be calculated on any floor as shown in equation 7.3 below. The total rotation consists of the rotation due to difference in variable load between the braced columns and the rotation due to tension and compression forces that arise during wind loading.

$$\varphi_i = \frac{\left(\frac{N_{c,i,left}}{A_{c,left}} + \frac{N_{c,i,right}}{A_{c,right}}\right) * h_{storey}}{E_s * L_{storey}} \tag{7.3}$$

Displacement at maximum height

The displacement at the top of the building is the summation of the non-wagging-tail displacements u_1, u_2, and u_3 added up with the rotation φ_i of each storey floor times the height towards the top. Except for the upper floor, each floor rotates as a result of vertical tension and compression forces during wind. In case of an 8 storey building, where n is 8, the rotation of the first floor is multiplied by the number of storeys, 8, minus 1. Adding up these rotations of each floor results in the total displacement at maximum height as shown in equation 7.4.

$$u_{max} = \Sigma(u_1 + u_2) + \varphi_1 * (n-1) + \varphi_2 * (n-2) + \varphi_3 * (n-3) + \varphi_4 * (n-4) + \varphi_5 * (n-5) + \varphi_6 * (n-6) + \varphi_7 * (n-7) \tag{7.4}$$

The maximum allowable displacements are $1/300 * h_{storey}$ for a single storey and $1/500 * h_{total}$ for the total height.

7.3.2 Frame 2: Eccentrically braced

The displacement of a single eccentrically braced frame consists of the three displacement components mentioned before and two additional components.

- u_4 Column displacement due to nodal rotation
- u_5 Column displacement due to cantilevering load

The displacement of node 11 needs to be considered as well when calculating the storey displacement. This calculation will be explained after the explanation of components u_4 and u_5.

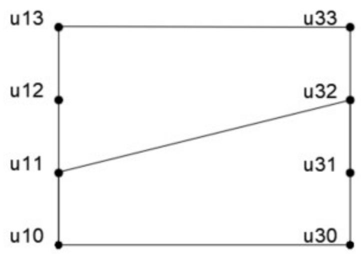


Figure 67. Eccentrically braced frame.

U4 Displacement due to nodal rotation

The lever arm 'a' of the horizontal wind load h_{wind} is equal to the distance between nodes 32 and 33. The internal moment as well as the internal rotation at intermediate height at node 32 can be calculated using a *forget-me-not*, visible in equations 7.5 and 7.6.

$$M_{32} = H_{wind} * a \tag{7.5}$$

$$\varphi_{32} = \frac{1}{3} * \frac{M_{32} * l_{32-33}}{EI} \tag{7.6}$$

The displacement at node 33 due to the internal rotation is as follows:

$$u_{33} = \varphi_{32} * a$$

U5 Displacement due to cantilevering load

Node k_{37} is modelled as a clamped support since it is able to take up moment. The displacement due to wind load H can be calculated using another *forget-me-not* in equation 7.7.

$$u_5 = \frac{1}{3} * \frac{H_{wind} * l_{32-33}{}^3}{EI} \tag{7.7}$$

Displacement at bracing nodes

The left column is modelled as an element that is supported on both ends with a point load at node 11, perpendicular to the direction of the element. The displacement u_{11} can now be calculated parametrically using equation 7.8. The full calculation of the equation below and subsequent equations can be found in Appendix O.

$$u_{11} = \frac{F_H * l_2 * l_1{}^3}{6 * EI * (l_1 + l_2)} + \frac{F_H * l_2 * l_1{}^2 * (l_1 + 2 * l_2)}{6 * EI * (l_1 + l_2)} \tag{7.8}$$

- $l_1 = length\ between\ node\ 10\ and\ 11$
- $l_2 = length\ between\ node\ 11\ and\ 13$
- $EI = column\ stiffness$

The displacement at the end of the bracing, u_{32}, is equal to the displacement of node 11 plus the elongation of the bracing and the horizontal displacement due to the compression in the column. This displacement leads to an internal rotation ϕ1 in the right column.

Initial displacement at maximum height

The cantilevering load on the upper element leads to an additional displacement of the column as well as an extra rotation ϕ_2 at node 32. The displacement at the top of the column is depicted in equation 7.9.

$$u_{33} = u_{32} + (\varphi_1 + \varphi_2) * l_3 + u_{cantilever} \tag{7.9}$$

- $l_2 = length\ between\ node\ 32\ and\ 33$

Iteration

An iteration is required since the displacement of the lower bracing support is modelled as an element that is supported on both sides. In reality, the upper support displaces, equal to the displacement of node 33 and the support of bracing will therefore displace more. Due to the stiffness of the system, the additional displacement is neglectable after 1 iteration since the displacement is only a few millimetres at maximum. The iterative displacement of node 11 can be calculated as shown in equation 7.10 below.

$$u_{11} = \frac{F_H * l_2 * l_1{}^3}{6 * EI * (l_1 + l_2)} + \frac{(F_H * l_2 * l_1{}^2 + 2 * F_H * l_1 * l_2{}^2 + 6 * EI * u_{13}) * l_1}{6 * EI * (l_1 + l_2)} \tag{7.10}$$

The final displacement of node 33 can be calculated using equation 7.9 again.

7.3.3 Frame 3: Eccentrically double braced

The calculation method for the displacements of the left bracing supports is equal to the single bracing system. The displacement of the lower support of the right bracing can be calculated by modelling the internal column element as a simple column with two forces perpendicular to the element and in the opposite direction, at the locations of nodes 21 and 22 respectively. Since there is an equal force distribution between the bracings, these forces are both called F_H. The displacement calculation of node 23 as shown in equation 7.11 below, does not affect the displacements of the lower nodes since the displacement of node u_{22} is fixed due to the left bracing displacement.

$$u_{21} = \frac{F_H * l_2 l_1^3}{6 * EI * (l_1 + l_2)} + \frac{(F_H * l_2 l_1^3 + 3 * F_H * l_1^2 l_2^2 + 2 * F_H * l_1 l_2^3 - F_H * l_1 l_2^3 + 6 * EI * u_{22} * (l_1 + l_2 + l_3)) * l_1}{6 * EI * (l_1^2 + 2 * l_1 l_2 + l_1 l_3 + l_2^2 + l_2 l_3)} \tag{7.11}$$

- $l_1 = length\ between\ node\ 20\ and\ 21$
- $l_2 = length\ between\ node\ 21\ and\ 22$
- $l_3 = length\ between\ node\ 22\ and\ 23$

The displacement of the right column nodes is equal to the method for single bracings.

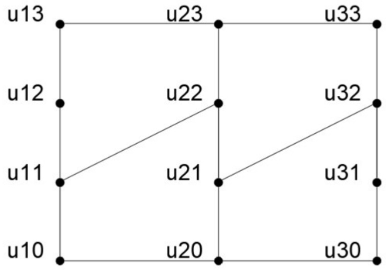


Figure 68. Displacement nodes in a double braced system.

Iterations

Using the displacement of the upper node 33 as the support displacement for the left column, the new displacement of nodes 11 and 22 can be calculated again. The final displacement of node 33 can be calculated using equation 7.9 again.

7.3.4 Transverse direction

To stabilise the modules in transverse direction, either a box profile or a strip can be used. Contrary to a strip bracing, a box profile can also be loaded in compression. Therefore, less bracings along the length are necessary. It is recommended to use either one of the two instead of both to have an equal load distribution on either side which also makes it easier to model. The elements can usually be placed at three locations. Along the front façade, rear façade or in an internal wall that is supported by strong columns. A box profile is often used due to the hinder of doors and windows along the façade. When using box profiles, additional displacements occur due to the connections that is has with the column as well as the floor. The full list of components is shown in Table 23 below. These displacements are considered and are called u_4 to u_7 and they will be explained below.

Table 23. Total displacement components.

	Element involved	Cause of displacement	Wagging-tail effect
U1	Bracing elongation	Tension force in bracing	No
U2	Column horizontal displacement	Connection compressed column with bracing	No
U3	Floor rotation	Difference in N_{column} from wind and live load in left and right column	Yes
U4	Column horizontal displacement	Nodal rotation of upper column part	No
U5	Column horizontal displacement	Cantilevering H_{wind} on upper column part	No
U6	Bracing uplift	Bulging of floor	No
U7	Bracing pulled downwards	Sagging of floor	No

Cantilevering load & intermediate floor support

A schematization has been made of a bracing element which is connected to the column and supported at a lower level by a floor beam and is shown in Figure 69 below. In the example, a horizontal load of 17.4 kN is applied at full height and the bracing is connected to the column at 2 m height.

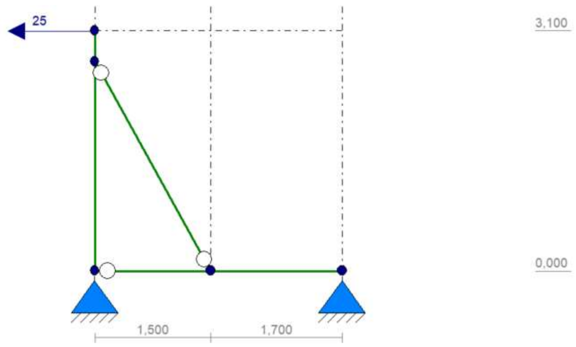


Figure 69. Geometry and applied load on a braced column (Technosoft).

The displacement of the intermediate node 4 consists of the following four components:

- u_4 Column displacement due to nodal rotation
- u_5 Column displacement due to cantilevering load
- u_6 Bracing uplift
- u_7 Column compression due to H_{wind}

The displacement due to the elongation of the bracing is not modelled since this displacement component has been explained in the previous paragraph.

U6 Floor beam loaded by wind

The horizontal wind load leads to either a compression or tension force in the bracing element. The vertical component of the internal bracing force is taken up by the floor beam, which will either deflect upwards or downwards as a result of it. To calculate the deflection at the connection between the bracing and the floor beam, a parametric equation, shown in equation 7.12, has been calculated using Maple. The complete calculation can be found in Appendix O. The parameters that are used to calculate the deflection are as follows:

$$w(x) = \frac{-F * (l - x) * x^3}{6 * EI * l} + \frac{F * x^2 * (2 * l^2 - 3 * l * a + a^2)}{6 * EI * l} \tag{7.12}$$

- $F = vertical\ component\ of\ the\ bracing\ force\ (kN)$
- $L = span\ of\ the\ floor\ beam\ (m)$
- $x = horizontal\ distance\ from\ left\ support\ to\ the\ applied\ force\ (m)$
- $EI = stiffness\ of\ the\ floor\ beam\ (kN/m^2)$

The horizontal displacement at the connection of the bracing with the column is equal to the ratio of the vertical bracing length divided by the horizontal bracing length times the vertical displacement.

U7 Floor beam loaded by floor load

The floor beam is also loaded by the permanent floor load as well as variable load. This load results in a downwards deflection w(x) of the beam. The same calculation method is used to calculate the deflection at the connection between the bracing and the floor beam as shown in 7.13. In this case a distributed load q is applied. The method used to obtain the formula below can be found in Appendix O.

$$w(x) = \frac{q * a^4}{24 * EI} - \frac{q * l * a^3}{12 * EI} + \frac{q * l^3 * a}{24 * EI} \tag{7.13}$$

- $q = floor\ load\ (kN/m)$

Deformed state

The deformed state of the column-bracing-beam structure is depicted in Figure 70. The internal moments are also highlighted. It can be seen that the beam deflects upwards due to a tension force in the bracing and that the horizontal displacement of the column is amplified at the cantilevering part.

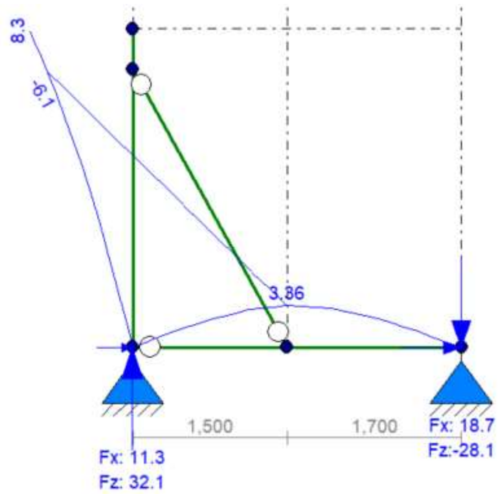


Figure 70. Deformed state of a braced column (Technosoft).

Determination of displacement at top

The calculation of the displacement at the top of the building is similar to the longitudinal method. The new displacement components do not lead to additional wagging-tail effect and can therefore be summed up with the other displacements u_1 and u_4. The equation for the total displacement is depicted in equation 7.14 below.

$$u_{max} = \Sigma(u_1 + u_4 + u_5 + u_6 + u_7 + u_8) + \varphi_1 * (n-1) + \varphi_2 * (n-2) + \varphi_3 * (n-3) + \varphi_4 * (n-4) + \varphi_5 * (n-5) + + \varphi_6 * (n-6) + \varphi_7 * (n-7) \quad (7.14)$$

7.3.5 Second order effects

The second order effect will be considered when calculating the maximum displacement. The additional horizontal force that arises from vertical loads on a displaced column will be added to the first order horizontal load to calculate the additional displacements. Iteration will be done until the additional displacement is less than 0.1 mm. Figure 71 below visualises the new horizontal force H_2 that arises from the second order effect.

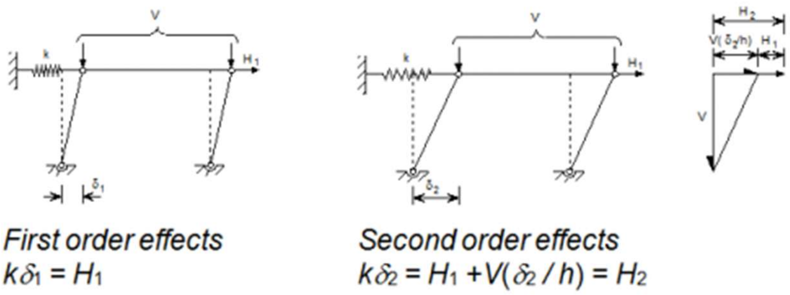


Figure 71. Second order effect visualisation.

7.4 Variant stability

The stability calculations have been performed on all variants and will be explained in this paragraph. First, the tension and compression forces that arise during wind are calculated based on two formulated equations that are applicable to all variants. It has been visualised how the tension and compression forces relate to each other for various number of total storeys. After that, an overview is made of the horizontal displacement for three critical number of storeys, these are 7, 8 and 9. This is the maximum number of storeys possible for most of the variants. When going up higher, in 3 out of 4 variants, the design fails on both stabilising criteria.

7.4.1 Longitudinal wind tension force

A formula has been drafted to calculate the tension force in the stabilising column for each variant. Using the standard values for the wind load, storey height and width of the module, the tension force can be calculated for different number of storeys and length of the bracings as shown in equation 7.15. The Excel sheet that has been used to calculate the forces can be found in Appendix N.

$$F_{ed,tension} = \frac{\frac{1}{2} * q_{wind} * (h_{storey} * n_{storey})^2}{l_{bracing}} * \frac{w_{module}}{2} * 1{,}10 * \gamma_{Q;1} \quad (7.15)$$

- $q_{wind} = variable$
- $h_{storey} = 3.1\ m$
- $w_{module} = 3.5\ m$
- $\gamma_{Q;1} = 1.5$
- $l_{bracing}, n_{storeys} = variable$

7.4.2 Longitudinal compression load

Another formula has been drafted to calculate the compression force due to the permanent loads acting on the stabilising column for different loaded widths and number of storeys, visible in equation 7.16. In some of the variants, the stabilising column is the edge column which has additional permanent load due to the transverse wall and the presence of a corridor or façade and this load is taken as 5 kN.

$$F_{ed,compression} = \left(\left(q_{slab} + q_{wall} + q_{ceiling} + q_{beam} + q_{var,red}\right) * w_{load} + F_{column}\right) * n_{storeys} * \gamma_{G,fav} \qquad (7.16)$$

- $q_{slab} = 5.25\ kN/m$
- $q_{wall} = 0.80\ kN/m$
- $q_{ceiling} = 0.67\ kN/m$
- $q_{var,red} = 1.13\ kN/m$
- $\gamma_{G,pos} = 0.9$
- $w_{load},\ F_{column},\ n\ = variable$

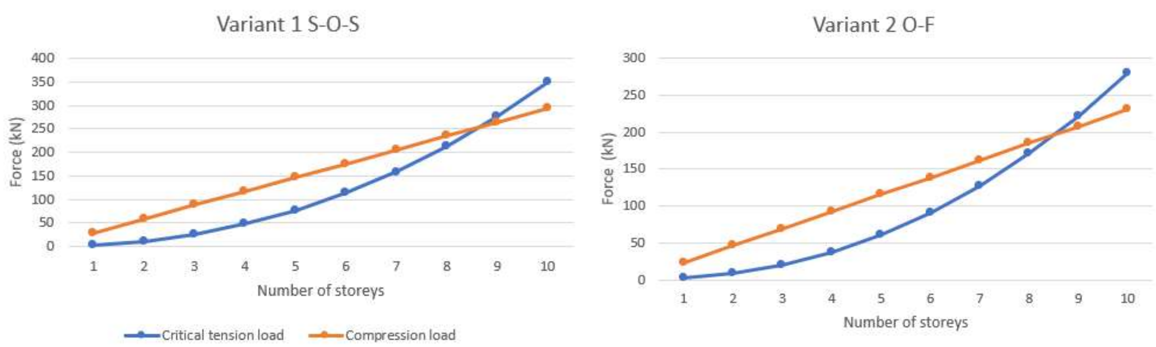
7.4.3 Longitudinal relation compression and tension forces

The ratio between the compression and tension load on the stabilising columns has been calculated for up to 10 storeys in which the wind load in kN/m^2 is increased when a higher building is used while the other parameters do not change. This has been done for the four variants which have different braced lengths and loaded widths, depicted in Table 24.

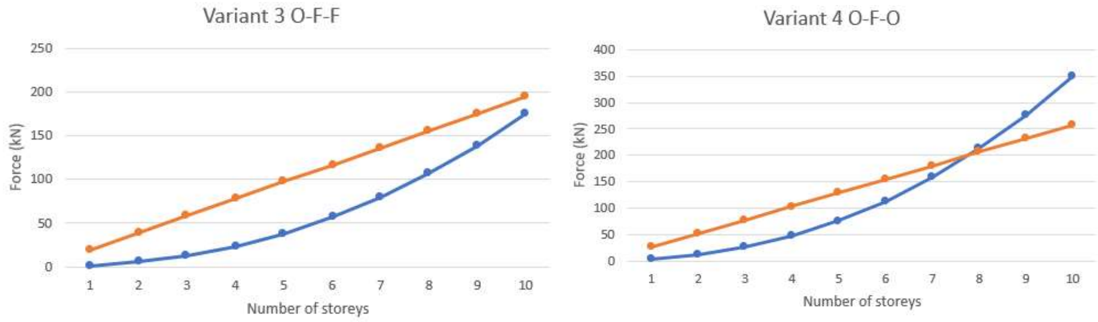
Table 24. Input values for calculation loads during wind for all variants.

	Stabilizing edge columns	Braced length (m)	Loaded width (m)
Variant 1 S-O-S	No	4.0	4.0
Variant 2 O-F	Yes	5.0	2.5
Variant 3 O-F-F	Yes	8.0	2.0
Variant 4 O-F-O	No	4.0	3.5

The graphs in Figure 72 show a clear relation between the compression and tension loads. It can be seen that the value of the forces differs a lot between the variants due to the difference in braced length, loaded width and whether or not the edge column are stabilising. The quadratic increase in tension load causes at some point a net tension force in the column. This happens at 8 storeys for variant 4 and at 9 storeys for variants 1 and 2. Variant 3 has the lowest forces and tension occurs above 10 storeys. This is an important turning point since column tension is to be prevented since it can destabilize the building.



(a) (b)



(**c**) (**d**)

Figure 72. Relation between critical tension load and compression load for design variants: (**a**) S-O-S, (**b**) O-F, (**c**) O-F-F, (**d**) O-F-O (own work).

7.4.4 Longitudinal horizontal displacement

The horizontal displacement has been calculated using the following starting points.

- The inter-module joint displacement can not be integrated into the Technosoft model and is calculated separately based on the stiffness of the joint in the slip stage as well as the load-bearing stage.
- Translation due to rotation of the base of the building can be calculated separately, using the base rotation of the foundation during wind loading.
- The factor for the second order has been calculated by comparing the first and second order displacement of the Technosoft model and amplifying it to consider the joint displacement. A factor of 5% is applied for this effect.

The Technosoft models that were used to perform these calculations can be found in Appendix L.

Table 25. Variant displacement components (values in mm).

	Number of storeys	Model	Joint	Base rotation	Second order	Total	UC
Variant 1 S-O-S	7	27,4	9,0	2,6	2,0	41,0	0,94
	8	37,1	10,3	3,0	2,5	52,9	1,07
	9	54,0	11,6	3,3	3,4	72,4	1,30
Variant 2 O-F	7	17,8	9,0	2,6	1,5	30,9	0,71
	8	26,7	10,3	3,0	2,0	42,0	0,85
	9	38,7	11,6	3,3	2,7	56,3	1,01
Variant 3 O-F-F	7	8,5	9,0	2,6	1,0	21,1	0,49
	8	12,3	10,3	3,0	1,3	26,8	0,54
	9	17,2	11,6	3,3	1,6	33,7	0,60
Variant 4 O-F-O	7	17,9	9,0	2,6	1,5	31,0	0,71
	8	27,3	10,3	3,0	2,0	42,6	0,86
	9	40,2	11,6	3,3	2,8	57,9	1,04

7.4.5 Transverse wind tension force

The wind load that has been used on each storey is calculated as shown in equation 7.17 below for the worst case scenario. That is the situation in which the bracing is supported along the full module width and the tension force is not reduced on the stabilising column due to a reduced width of the bracing.

$$F_{ed,tension} = \frac{\frac{1}{2} * q_{wind} * (h_{storey} * n_{storey})^2}{l_{bracing} * n_{rows}} * \frac{l_{module}}{2} * 1{,}10 * \gamma_{Q;1} \tag{7.17}$$

7.4.6 Transverse compression load

The compression forces that act on the edge column come from the partition structures on each storey as well as the reduced variable load. The distributed load is dependant on the loaded width, w_{load}, in the longitudinal direction. The lowest loaded width between the design variants is 2.0 m and will therefore be used for this calculation, visible in equation 7.18 below.

$$F_{ed,compression} = (q_{tot} * w_{load} + F_{column} + F_{trans-wall}) * n_{storeys} * \gamma_{G,fav} \tag{7.18}$$

- $q_{tot} = (q_{slab} + q_{wall} + q_{ceiling} + q_{beam} + q_{var,red})$

7.4.7 Transverse relation compression and tension forces

The result of the calculations is visible in Figure 73 below. It can be seen that the tension force becomes dominant when the number of storeys is above 9. This is a desirable result since that is also the number of storeys in which the tension force becomes dominant in the longitudinal direction.

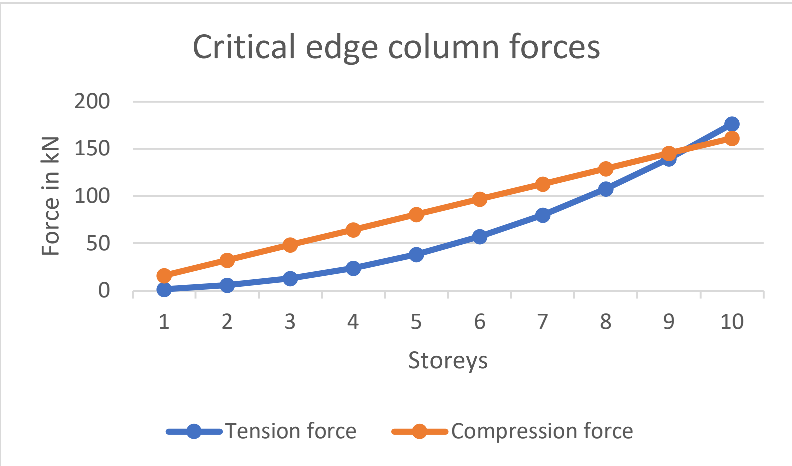


Figure 73. Calculation of critical column forces in transverse direction.

7.4.8 Transverse horizontal displacement

The calculation of the horizontal displacement is done using Technosoft. The model incorporates the following displacement components:

- Bracing elongation
- Column compression
- Storey rotation
- Eccentricity
- Floor uplift or sag

The following components are not included and are therefore calculated by hand:

- Inter-module connection stiffness
- Translation due to base rotation
- Second order effect due to vertical load

These displacements will be added to the model displacement as visible in Table 26 as part of the total displacement and are calculated similarly to the longitudinal stability. The Technosoft models that were used to perform these calculations can be found in Appendix M.

Table 26. Transverse horizontal displacements (values in mm).

Variant	Technosoft	Joints	Foundation	Second order	Total	UC
1.1 Box profiles	44	8,2	2,5	2,7	57	1,16
2.1 Strips	43	8,2	2,5	2,7	56	1,14
2.2 Strips	41	8,2	2,5	2,6	54	1,09
3.1 Mix	37	8,2	2,5	2,4	50	1,01
3.2 Mix	30	8,2	2,5	2,1	43	0,87

It can be seen that when a design is made using the current starting points, the unity checks are slightly above 1.0 for most variants. Therefore, changes in the design are required. These will be mentioned in the upcoming chapter.

8 Assessment

In this chapter, the design variants will be assessed on their structural capacity as well as functional efficiency and environmental impact. The structural capacity is assessed to get a proper image of the stabilizing capacity on preventing column tension and the horizontal displacement. Several options for optimization are given on both stabilizing criteria and their impact is visualized.
After the structural assessment, a functional and environmental assessment is done to gain insight in the functional efficiency as well as the environmental impact. In the functional part of the assessment, variant properties are listed and properties such as the wall-to-floor ratio and the space efficiency factor are calculated. An overview is made of the strength verification, in which the load-bearing element sizes are listed as well as their respective unity checks. The sizes of the load-bearing elements as well as partition structures together form the structure of the module. The material use for plasterboard, insulation, steel and concrete are calculated to find out the environmental impact. The full assessment is visible in Appendix Q. As well as an internal assessment, the variants are also compared functionally and environmentally with the previously done case studies.

8.1 Structural Assessment

The four variants are assessed on the two criteria for stability in longitudinal direction, preventing column tension and adhering to the maximum allowable horizontal displacement. The calculations on stability have resulted in a critical number of storeys in which the stability is no longer guaranteed. For variants S-O-S and O-F-O this is at 8 storeys, 9 storeys for variant O-F and above 10 storeys for variant O-F-F. A favourable result has been obtained in which both criteria's, column tension and horizontal displacement are fulfilled for similar storeys.

Table 27. Critical number of storeys for longitudinal stability.

	Column tension	Exceeds horizontal displacement
Variant 1 S-O-S	9	8
Variant 2 O-F	9	9
Variant 3 O-F-F	> 10	> 10
Variant 4 O-F-O	8	9

8.1.1 Longitudinal optimization parameters

Based on the design method, there are several parameters of the module design that can be changed to increase the stabilising capacity. These parameters will briefly be explained on what their impact is.
The first option is to increase the beam span at the location of the bracings. This increases the compression load and reduces the tension load as well as the horizontal displacement. This requires however a larger beam profile which is inefficient when the same stiffer beam profile is used for the other column bays since the beams with a smaller span are then over-dimensioned.
Furthermore, the length of the module becomes larger and therefore also the wind load in the transverse direction, requiring additional stabilising elements in that direction.
Another option is to use heavier partition structures. This simply adds more permanent load onto the stabilising columns and larger column profiles are required.
Three more options are possible to reduce the horizontal displacements only. The eccentricities of the bracings can be reduced. Figure 74 shows the horizontal displacement due to bracing elongation for several eccentricities. The horizontal values are the total eccentricities in which the lower and upper bracing eccentricity are added up. It can be seen that the displacement of single cross-bracings increase exponentially while the double cross-bracings are more stiff and show a near linear increase. Both systems lead to similar displacements for eccentricities up to 300 mm. For eccentricities larger than 400 mm, the differences are rather high. The advantages of using double-cross bracings are that there are less forces in the bracings, resulting in less extension of the bracing and less displacement of the bracing support. Furthermore, the horizontal force is lowered in the right upper beam, resulting in

less displacement due to the cantilevering load and additional internal rotation. The similarity between both systems is the floor rotation. Since the outer columns are continuous in both systems and the span between them is equal, the forces are equal, as well as the elongation and compression of the columns, resulting in an equal floor rotation. Figure 74 shows on the right how much the displacement due to bracing elongation is reduced when a larger bracing area is used. An area of 1500 mm^2 is used for the variant designs and it can be seen that a larger area results in only a small reduction in displacement.

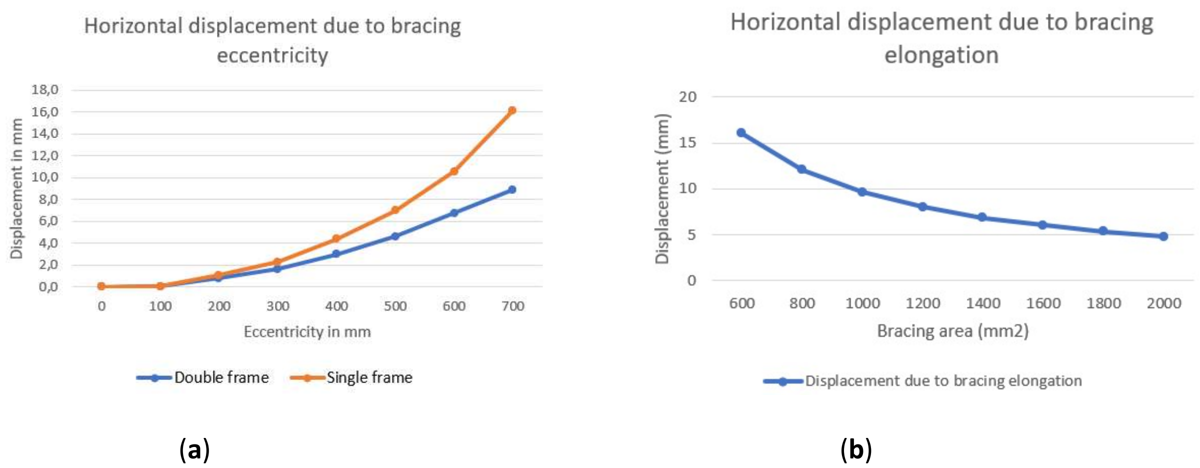


Figure 74. Horizontal displacement due to bracing geometry: (**a**) Support eccentricities, (**b**) Profile area.

The last option is to increase the stiffness of the stabilising columns. By doing so, there is less translation due to storey rotation as well as column shortening. Both effects are visualised in Figure 75. The current column profile in the design variants is a 120*120*6 mm SHS profile with an area of 2664 mm^2.

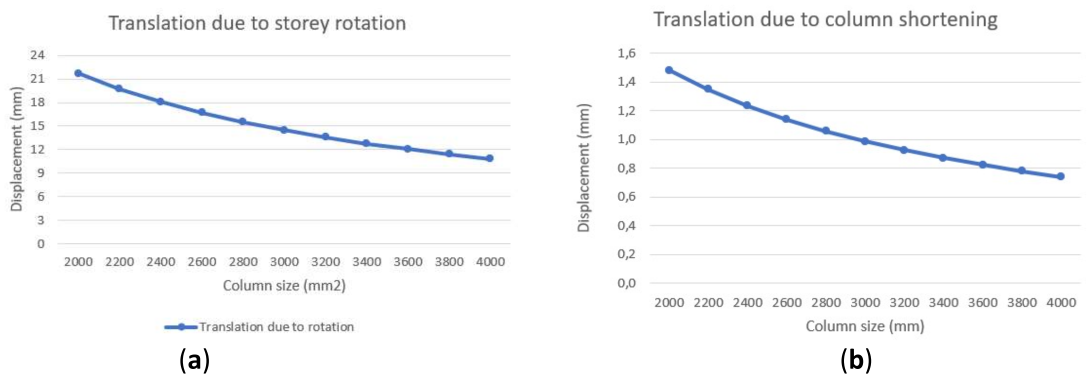


Figure 75. Displacements for different column sizes: (**a**) Due to storey rotation, (**b**) Due to column shortening.

8.1.2 Transverse optimization

Several options are possible to increase the stabilising capacity in transverse direction. There are two possibilities to reduce the wind load without changing the stabilising elements. The first option is to use additional modules in row, 10 instead of 8. This leads to a reduction factor for the wind load as well as horizontal displacement of 20%. This reduction factor is applied to the Technosoft model displacement and does not result in less displacement of the additional components. The second option is to use a design variant with a smaller longitudinal length, 10 m instead of 12 m. This leads to a reduction factor of 17%. The third option is to simply reduce the number of storeys.

Instead of changing the geometry of the building, additional stabilising elements can also be used. This is however less efficient since additional material is used and more space is required. The first option is to add an additional box profile at an intermediate column. The module will then become more complex, and an extra horizontal connection is required as well. The second option is to use larger element sizes for either the columns or bracings. Stiffer columns result in less rotation and larger bracings result in less elongation. In Table 28 the unity checks for the design alternatives are given when either additional modules in a row are used or a reduction in longitudinal length as well as using both options.

Table 28. Unity checks for design alternatives.

	Current	Option 1 or 2	Option 1 and 2
1.1 Box profiles	1,22	1,02	0,81
2.1 Strips	1,19	1,00	0,80
2.2 Strips	1,13	0,94	0,75
3.1 Mix	1,34	1,12	0,89
3.2 Mix	1,03	0,85	0,68

8.2 Increased stabilising capacity

To get a better understanding of the structural capacity of the variants, it has been looked at what changes to the structural system are required to obtain a stable structure up to 10 storeys.

The first verification is again preventing column tension. The increase in height and therefore also the wind load in kN/m^2 requires an increase in the horizontal length of the bracings. Increasing the span of the bracing is much more effective than increasing the unbraced span since that only adds loaded width onto the stabilising columns while increasing the braced span also reducing the tension load due to wind.

8.2.1 Force increase

Equations 7.15 and 7.16 are used again to calculate the wind load and vertical loads for 9 and 10 storeys, these values can be found in appendix S. In the load combination in each variant, the wind load is the governing variable load. Due to the large increase in forces, the braced length needs to be increased by several decimetres for 9 storeys and up to 1 meter for 10 storeys to prevent column tension. This also increases the loaded length of the stabilising columns, further increasing the column load. Table 29 shows the required bracing lengths, the new loaded length on the stabilising columns and the increase in total length. These new lengths are used to calculate the new tension and compression forces in which column tension has been prevented in each variant as well as each storey.

Table 29. Values for braced length, loaded column length and total length for 8 to 10 storeys.

	Braced length (m)			Loaded length on column (m)			Total length (m)		
Storeys	8	9	10	8	9	10	8	9	10
Variant 1 S-O-S	4,00	4,3	4,65	4,00	4,15	4,33	12,0	12,6	13,3
Variant 2 O-F	5,00	5,35	5,75	2,50	2,68	2,88	10,0	10,4	10,8
Variant 3 O-F-F	8,00	8,00	8,00	2,00	2,00	2,00	12,0	12,0	12,0
Variant 4 O-F-O	4,30	4,65	5,00	3,50	3,83	4,00	10,3	10,7	11,0

8.2.2 Horizontal displacement

The horizontal displacement has been calculated for 9 and 10 storeys, considering the increased bracing length and increased column profiles based on the increase in total load. Both changes have a positive effect on reducing the horizontal displacement. Table 30 shows these column profiles as well as the maximum horizontal displacement. It can also be seen that all variants, except for variant 1 S-O-S are safe. This variant requires a reduction in displacement of around 10 millimetres. This can be accomplished by using a thicker bracing profile or further increasing the column area. The Technosoft models that were used for these calculations can be found in appendix T.

Table 30. Column area and horizontal displacement verification for 8 to 10 storeys.

	Storeys	**Column area (mm^2)**	**u_{ed} (mm)**	**UC**
Variant 1 S-O-S	8	2664	55,0	1,11
	9	3161	62,1	1,11
	10	3671	71,4	1,15
Variant 2 O-F	8	2250	44,6	0,90
	9	2622	48,8	0,87
	10	3009	50,8	0,82
Variant 4 O-F-O	8	2664	45,2	0,91
	9	3043	47,3	0,85
	10	3581	51,9	0,84

8.2.3 Result

The increase in lateral load at 9 and 10 storeys has a large effect on the stabilising structure. In variant 1 S-O-S, the length needs to be increased on both edges which results in a larger increase in total length. This variant has the largest total length and therefore requires the heaviest trucks and cranes to install. Furthermore, the large increase in length results in additional loads in the transverse direction. This requires even heavier or additional stabilising elements in that direction which is inconvenient. The second variant consists of only two column bays and a beam span of 5 meters in the 8 storey layout. This length needs to be increased by circa 400 mm per additional storey. Since the span was already large, this further increase results in even larger beam profiles and additional storey height. As mentioned before, the third variant has the largest stabilising capacity and no changes to the module structure are required at 9 and 10 storeys.

The fourth variant has its bracings positioned in the middle of the module. Increasing this length by 350 mm per storey results in a large difference between the beam at the braced span and the unbraced spans. This requires different beam profiles to be used with the same height since they need to be levelled.

8.3 Internal functional and environmental assessment

The design variants show clear functional differences due to their geometry and positioning of stabilising elements. The two variants with a smaller length of 10 m, variant O-F and variant O-F-O, have a large length of open wall as well as a high wall-to-floor ratio. The reduction is cost-efficiency due to the high wall-to-floor ratio can be countered by the freedom in design due to the open wall area as well as the fact that there is only one internal wall in transverse direction, compared to two for the other two variants. There is a small variation in storey height due to the difference in beam height. Variant O-F has an additional storey height of 9 cm since the beam span is 5 m instead of 4 m for the other variants. The difference in total material use and corresponding values for embodied energy in MJ/m^2 and embodied carbon in $kgCO_2/m^2$ is very low since the same load-bearing and partition structures are used. Therefore, the environmental result will be discussed when comparing it to the case studies in the upcoming paragraph.

8.4 Comparison between design variants and case studies

To get a better idea of the efficiency of the four design variants, they are compared to the four case studies. The 8 module types will not be compared on their structural sizes, since the module geometry is very different, and an accurate comparison can therefore not be made. They will only be compared for their functionality and environmental impact.

8.4.1 Functional comparison

The functional assessment starts with the number of storeys which is between 4 and 6 for the case studies using steel columns and 7 for the North Orleans case which uses stacked concrete walls. The North Orleans case is also the only case which only uses 1 module per apartment since it is used for students while the other modules are used for starters. The percentage of open wall differs a lot between all modules and is between 34 % and 50 % for the steel cases and 33% to 60% for the own design variants. The difference in total area also differs a lot and can be categorised into three sizes. The Murray Grove and North Orleans modules have a total area of around 26 m^2, the area of own variants 2 O-F and 4 O-F-O is 35 m^2 and the other modules, Raines Court, Regioplein and variants 1 S-O-S and 3 O-F-F all have areas around 44 m^2. Since the width and storey height are similar for most modules, the modules with a low total area also have a high wall-to-floor ratio resulting in a cost-inefficient structure.

8.4.2 Environmental comparison

The values of the module weight, embodied energy and embodied carbon per square meter of total area are calculated. This has been done by multiplying the total weight of each material in the module by their corresponding embodied energy in MJ/kg and embodied carbon in $kgCO_2/kg$. These calculations can be found in Appendix U. Table 31 shows the total values as well as per square meter of total area for all 8 module types. The two English lightweight steel modules have a very low weight due to the sole use of steel as load-bearing material. Although the weight is very low, due to the high embodied energy of steel the total embodied energy per square meter is similar to the other module types. The North Orleans fully concrete module has the largest values for the weight, embodied energy as well as embodied carbon due to the large use of concrete. The Regioplein steel-concrete module also has a large weight per square meter since a concrete edge beam is used as well as a concrete floor slab. The four design variants use a steel edge beam, reducing the module weight by a lot. However, since they are used for a larger number of storeys, thicker steel stabilising elements are required, resulting in a slightly larger values for the embodied energy and embodied carbon per square meter than the other modules which use steel columns.

Table 31. Comparison of design variants with case studies.

	Total area (m^2)	Weight (kg)		Embodied Energy (MJ)		Embodied Carbon ($kgCO_2$)	
		Total	**Per m^2**	**Total**	**Per m^2**	**Total**	**Per m^2**
Murray Grove	25,6	5281	206	56096	2191	3691	144
Raines Court	45,6	8862	194	92262	2023	6046	133
North Orleans	27,3	26327	964	65194	2388	6822	250
Regioplein	42,4	19698	722	85465	2016	6910	163
S-O-S	42,0	19095	455	95059	2263	7479	178
O-F	35,0	15896	454	81772	2336	6442	184
O-F-F	42,0	19091	455	95160	2266	7487	178
O-F-O	35,0	15532	444	75183	2148	5948	170

9 Conclusions

The main question of this thesis is what is the efficiency of new high capacity self-stabilising modules that are used at a greater height than is currently done in mid-rise residential construction?
To answer the main question, five sub-questions have been composed which will be answered first.
To answer the first question, there are currently many different module types used in mid-rise residential buildings and these can be categorized into four categories. The first module type uses a concrete slab and CLT walls in which the stability often comes from the CLT walls combined with a core in the transverse direction. In the second and third module either steel floor joists or a concrete slab is used, and they both use steel infill walls as well as steel edge columns and bracings to stabilise the building. The fourth module type is a fully concrete module in which the walls in both directions are used to stabilise the building.
To answer the second question, the module type that has the capacity to be used for extra storeys uses a concrete slab and steel columns. The concrete slab offers high fire safety and the load-bearing capacity of the steel columns, and the added dead load of slabs is used to prevent column tension which is critical for larger number of storeys.
To answer the third and fourth question, there are four configurations of bracings possible for the new high capacity self-stabilising modules.
The simplicity of variant 1 S-O-S and the favourable load distribution that is has during wind makes it the most efficient variant for low number of storeys, up to around 6. For slightly higher number of storeys, around 7 to 8, variants 2 O-F and 4 O-F-O are both feasible options due to their reduced length compared to variant 1. For even higher number of storeys, these variants become less efficient since the required increase in the span of the bracings results in inefficient beam profiles along the length. Furthermore, the total length becomes 5% larger per storey, requiring even more stabilising capacity in the transverse direction while there is limited space available. Variant 3 has the largest stabilising capacity and can be used up to 10 storeys without changing the structure. However, it has a high complexity due to the large number of stabilising elements.
To answer the fifth and last question, the functional efficiency of the design variants is similar to the existing self-stabilising modular buildings. The storey height is slightly larger while the space efficiency factor and wall-to-floor ratio is comparable to the case studies. The weight per square meter of the design variants falls in between the lightweight steel case studies and the case studies which use mainly concrete. The embodied energy of the building materials is increased by only 7 to 16% while the embodied carbon is increased by 28 to 39% compared to the case study with the lowest value.

The calculations in this research are done for an urban area since mid-rise buildings are nearly only present in urban areas. Wind area II is used since several provinces in the Netherlands along the coastline are classified as this wind area. In case of wind area I, the wind loads are increased by 19%, reducing the number of storeys to be constructed.

This research has shown that self-stabilising modules with a steel-concrete load-bearing structure can be used in wind area II up to 8 storeys efficiently using different bracing configurations.

10 Recommendations

10.1 Design changes

The current design variants use C-sections as floor support and double cross bracings due to their high stabilising capacity. A design alternative has been drafted which has been optimized on environmental impact. Due to this optimization, the structural capacity will be lower.
The floor support will be a concrete beam. In chapter 6.5 the different floor supports are compared and6.4 it can be seen that a concrete beam has less environmental impact than the C-section, based on the required area. The reduced height of the concrete beam compared to the C-section, lowers the eccentricity of the lower bracing support. The eccentricity of the upper bracing support can be reduced as well when connecting the bracing at ceiling height, instead of below. This requires however a change in the connection of the upper edge beam with the column. Instead of using double-cross bracings, single bracings can be used which requires less elements. However, the stabilising capacity is reduced as well. From the possible design variants, variant 4 has the lowest environmental impact per square meter. An additional advantage is that due to its reduced length compared to the other variants, transportation and installation requires less heavy trucks and cranes which is also beneficial to the environment.

10.2 Concrete changes

The new design in which changes are made to reduce the environmental impact uses more concrete due to the edge beam which is now also in concrete. The environmental impact of concrete can be reduced by using more sustainable concrete in which the standard aggregates and cement ingredients can be replaced. Standard aggregates can be replaced by ground granulated blast-furnace slag, sintered fly ash or lytag lightweight aggregate. This reduces the CO_2 emissions as well as the density. The reduced weight leads to a density of only 2020 kg/m^3. The application of lightweight concrete in self-stabilising module types results in less dead load, reducing the stabilising capacity.
Apart from changing the type of aggregates, standard Portland cement CEM I can be replaced by multicomponent cements CEM II and CEM VI. In these types of cement, the Portland clinker has been partially replaced by a mixture of limestone, siliceous fly ash or granulated blast furnace slag. Standard Portland cement has a CO_2 emission ranging from 825 to 890 kgCO_2 per Mg of clinker. These mixtures roughly have a Portland cement content of 45% to 60% and either one or two non-clinker components. This results in low CO_2 emission levels between 340 to 453 kgCO_2 per Mg of clinker, reducing the emissions by up to 50% while there is only a small reduction in weight compared to Portland cement.

10.3 Topics for further research

Four topics will be mentioned that can be used for further research studies. These topics are all about mid-rise modular construction.

Stiff transverse frame

In this thesis, the focus was on the design possibilities in longitudinal direction rather than the transverse direction. That is because there is less freedom in design due to the restricted width and the option to add additional modules in row, reducing the lateral forces. In some areas of application however, it is not possible to build a modular building in which many modules can be placed next to each other. Therefore, an interesting topic is to research how self-stabilising modules can be constructed with a high stabilising capacity in the transverse direction. By using a very stiff frame, other facades can be left open without stabilising elements, which creates more user comfort.

Material optimization in mid-rise modular construction

One of the main advantages of modular construction is the reduction in material waste due to the off-site fabrication. However, modular construction thrives when the exact same module can be produced in large numbers. This requires the same profiles for load bearing elements on each storey. The upper storeys in which the lateral and vertical forces are low are therefore over-dimensioned. An interesting topic is to research how the material use can be optimized across the different storeys while still having an efficient manufacturing process.

Minimising the environmental impact of modules

The environmental impact of modular construction is low compared to traditional construction. Suggestions are already given in this thesis on how to further reduce the emissions, by changing parts of the module and to use different types of concrete. Another interesting topic for further investigation is to minimise the environmental impact of modules. Instead of using steel for the main load bearing elements, timber columns and CLT walls can be used as well, combined with green concrete in the floors or even timber joists. Changing these materials has large influences on the structural capacity and this requires extensive research.

Manufacturability inter-module joint on-site

An important issue in modular buildings is the type of inter-module connection. A demountable connection is often desired since it enables the building to be disassembled after its initial lifetime. However, a demountable connection often requires bolted connections between multiple modules. The lack of access to the connection at the construction site can provide problems during the installation. An interesting topic is to look into the changes to the module that are required such as access openings in the walls or columns to make such a connection. Different existing inter-module joints can be examined and assessed on the complexity of connecting them on-site.

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12 Appendices

A. Functional requirements

Table 32. Functional requirements for modular components (Reprinted from: Design in Modular Construction).

Functional consideration	Comment on modular construction
Plan form	Dependent on module size, the strategy for stability, and issues such as fire evacuation of the building. Additional braced cores are often required for taller buildings.
Circulation space	Means of access to the modules require design of corridors or external walkways, and braced stair and lift cores.
Cladding	Cladding may be in the form of ground-supported brickwork (up to 3 storeys high) or lightweight cladding. In both cases, the cladding is normally attached to the modules on site. The modules are designed as watertight insulated units.
Roofing	Roofs may be manufactured as modules or using conventional roof trusses. Flat roofs are not normally recommended in modular construction unless provision is made for water runoff in the module design.
Thermal insulation	High levels of thermal insulation are generally provided within the modules, which can be supplemented by additional insulation on the outside of external walls.
Acoustic insulation	Double-layer walls, and combined floors and ceilings, provide excellent acoustic separation.
Fire safety	90 min fire resistance is generally achieved by the measures adopted for acoustic insulation. 120 min fire resistance is achieved by additional boards. Fire spread between the modules is prevented by use of fire stops.
Services distribution	Modules are generally manufactured as fully serviced units, and service connections are made externally to the modules. Corridors provide useful zones for service distribution.

B. Reference projects

General reference projects

Norra Tornen

The Norra Tornen project in Stockholm consists of two high-rise towers of 110 and 125 meters comprising 138 and 182 modular units, respectively. The towers stand out because of their vertical and horizontal segmentation and the whole structure is made out of concrete to give the building a brutalist appearance. The exterior of the building has a rough concrete skin with an alternating pattern between protruding floors of living areas and outdoor spaces. This results in a very high wall-to-floor ratio, close to 1.0. Since such a large ratio is cost-inefficient for the structure, it needs to be outweighed by some additional value. This value comes from the unique layouts of the apartments as well as multiple orientations and additional window area compared to a homogenous façade over the height. These assets are precious since Sweden has a scarcity of daylight for half of the year. The size of the apartments varies between a 44 m^2 one-bedroom apartments to 271 m^2 penthouses. The ground floor has a double height and is used as a leisure area with a cinema, dining and event rooms as well as a gym and sauna (Norra Tornen, 2020). The structural design has been made on the 4.8 by 4.8 m grid, with layout differences between each floor. Prefabricated elements of ribbed coloured concrete were used around the central concrete core. The core of the smaller tower has a rectangular shaped while the larger tower has a squared core.

Figure 76. Norra Tornen buildings (Norra Tornen, 2020).

Floor plan

The floor plan of the building shows the position of the core, corridor and layout of apartments. The core is eccentric from the center of the building with a single corridor on the right side. There are in total 8 apartments on this floor, which are marked by a light or dark hue. Each apartment has a balcony for additional floor area. The core has three openings of which two are the entrance to two different apartments and another opening for the corridor to which the doors of other six apartments are connected. The core comprises three elevators, a single staircase and additional area for building services (Wilner, 2020).

Figure 77. Norra Tornen Floor Plan (Norra Tornen, 2020).

Croydon Tower

In a district called Croydon, part of the South of the Greater-London region, the worlds tallest modular building has been built. The building consists of two concrete cores with a difference in height. The first core is 38 storeys high, and the second core is 44 storeys high with a height of 135 meters. These cores were made using slipform construction and finished before any of the other constructed had been started. The two lower storeys are used as a podium with room for communal facilities such as a reception area, laundry room, meeting rooms and a gym. The other floors contain 15 apartments across both towers consisting of 38 modules in total. The modules have concrete bases because of the greater flexibility on the module's sizes and the excellent acoustic performance. These concrete bases were linked together on site to make it part of the stability system of the building. The modules have a steel frame consisting of 60 mm square sections with heavier duty sections in the corners of the modules for vertical load takedown, varying from 150 mm at the top up to 300 mm at the base. Between the modules there is a gap of 16 mm for tolerances. The walls between the steel sections consist of fire-rated plasterboard over sheet of fibreboard, rockwool between the framing sections and outside clad in cement fireboard sheets resulting in the required fire resistance of 2 hours. The external cladding is 200 mm thick rockwool layer with green glazed terracotta cladding. The configuration of using steel frames and modules results in a 6% gross to net gain compared to traditional construction (Lane, 2019). Contrary to the Norra Tornen towers, the Croydon Tower has no horizontal or vertical segmentation across its height. Figure 31 below shows the stacking of modules in both towers. The modules are placed in two directions, resulting in a completely closed perimeter of the building and a low wall-to-floor ratio.

Figure 78. Croydon Tower installation of modules (Tide Construction).

Wembley, London

In Wembley, London a modular building with 17 storeys for students has been built by the company Futureform. This building is, contrary to most other modular buildings, designed with a circular shape. Therefore, it is not expected to be characterized as a modular building from the outside. The only characteristic that may reveal its structure is the repetition in windows horizontally. The concrete core is surrounded by a circular floor plan with modules in the north, east and west wings radiating from the core. The north and east wings only have 4 and 6 storeys respectively while the west wing consists of 16 storeys of modules as well as a podium level on the ground floor. The modules have a length of 16m and therefore contain two separate modules with part of the corridor in between. The modules are delivered with extra plasterboards to finish the corridors after connections to building services are made. There are two types of modules, study bedrooms with a width of 2.7 m and kitchens with a width of 3.8 m. All modules have a steel structure for the floor and ceiling joists consisting of 150 mm C-sections, resulting in a combined depth of only 380 mm. A set of 5 study bedrooms is connected to a communal kitchen. The construction of the concrete core and modules has been done parallel with the installation of three floors per week. Over a total period of 15-weeks all modules installed and finished (Lawson et al., 2010).

Figure 79. High-rise building in Wembley, London (Courtesy of Futureform).

Laan van Spartaan

The highest modular building in the Netherlands is located in Amsterdam-West at the Laan van Spartaan. The building has 16 floors with a total of 361 apartments. On the ground floor commercial rooms as well as car and bicycle parking are facilitated. The modules have concrete floor as well as concrete walls and are made by the company Ursem. Some of the modules have steel braced walls that make up the stability system together with a concrete core. The modules are placed on either side of a corridor consisting of concrete plates. This system resulted in a total construction time of only 12 months (Pieters Bouwtechniek, 2017).

Figure 80. Student housing Laan van Spartaan, Amsterdam (Ursem Modulaire bouw, 2017).

Campus Uilenstede

Commissioned by the Dutch student housing company DUWO, a total of 233 student apartments have been build on the campus Uilenstede in Amstelveen, using modules made by the company Ursem. The design consists of a high-rise building with 11 storeys and a low-rise building with 5 floors. These buildings are interconnected using a transparent connection. Most of the apartments use 1 module and have a living area of 28 m^2. The other apartments consist of two modules with an area of 42 m^2.
The structure of the building consists of modules with a steel structure and an external steel supporting structure. Due to the close proximity of Schiphol airport, there are strict rules for the sound insulation of the modules (Pieters Bouwtechniek, 2013).
This was the first project where a 11 storey high-rise building has been made for student apartments using modules only. The housing project of campus Uilenstede consisted of two stages with a total of 700 new apartments. Modular construction was chosen for the second stage for various reasons. The use of prefabricated modules cut the construction time in half while improving the technical qualities as well as reducing the costs (Ursem, 2013).

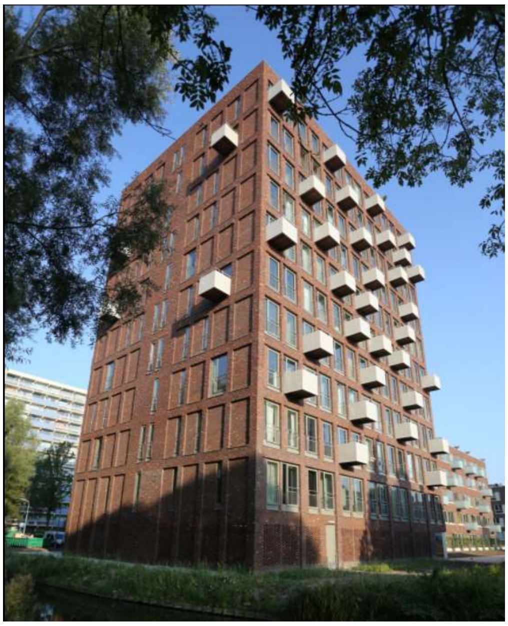


Figure 81. Student housing Campus Uilenstede, Amsterdam (Ursem Modulaire bouw, 2013).

Sentinel Housing Association Basingstoke

The development of residential apartments in Basingstoke is split into three modular building blocks, ranging from 6 to 11 floors. Modular construction was chosen as the construction method because of its speed of manufacture and minimum disturbance to a nearby hospital. The three blocks have a total number of 360 modules and the total building was constructed in only 5 months. The apartment sizes are either 48 m^2, using two modules or 60 m^2 using three modules. All modules had a length of 7.2m and a width of either 3.0m or 3.6m. The Vision modular building system was used for this project. This system has the following materials: concrete floors that are supported by parallel flange channel (PFC) sections and a wall profile of 60x60 square hollow sections (SHS) placed at 600 mm centres, which support the 9 storeys of modules. The modules are partially open-sided and utilize the spanning capabilities of the PFC edge beams and balconies were attached to the perimeter PFC sections. This structure has a total fire rating of 120 minutes.

The lateral stability was provided by the reinforced concrete cores that were used for the stairs and lift. The shape of the modules varied from rectangular elements to irregular shapes with flared corners. The depth of the floor and ceiling was only 350 mm and has a ceiling truss, allowing for building services to pass through. The arrangement of the modules was on either side of a corridor that was accessed from the stairs and lift (Lawson et al., 2010).

Figure 82. Eleven-storey modular building, Basingstoke (Lawson et al., 2010).

Royal Northern College of Music

In Manchester, the Royal Northern College of Music needed extra student accommodation near its campus. The desire was to complete a new building in 12 months, so it would be finished before the new academic year started. Therefore, modular construction was chosen with the additional advantage that it could be dismantled and relocated to another part of the campus. The building has a square plan around a central courtyard with a height that varies between 6 and 9 storeys. There is no concrete core for the stability of the building, instead the modules were placed on either side of a central corridor with stairs and lifts on the four corners of the building. These elements were braced to provide the stability of the building. Wind loads are transferred to the braced cores laterally by the group of modules on each floor. A rain screen cladding was pre-attached to the modules which meant that there was no additional cladding and scaffolding of the building on site. Part of the cladding system are the joints between the modules that required a high degree of accuracy in manufacture. The connected modules provided double-layered walls as well as floors, providing excelling acoustic insulation that was necessary since music students practice in their rooms (Lawson et al., 2010).

Figure 83. Royal Northern College of Music Modular Building (Liberty Living at Sir Charles Grove Hall, Manchester, 2020).

Internal steel structure projects

Project 1 Lillie Road, London

Table 33. Lillie Road project characteristics (New steel construction).

Building properties	
Characteristics	Green roofs and a mix of modular construction and panel construction
Function	Residential use
Layout	3 Gallery flats
Height	6 Storeys
Stability	North-South: X-braced walls and floor diaphragms
	East-West: Bracing in spine wall at east end and braced lift core modules.
Load transfer	Horizontal load taken down by braced module walls
Apartment area	25 m^2
Module properties	
Floor system	Cassette using 200 mm C-section
Wall elements	Braced light gauge steel panels, 100 mm C-sections, 1.2-2.4 mm thickness, mineral wool and plasterboard

Figure 84. Lillie Road, London (Greenroofs.com).

Figure 85. Bird view of Lillie Road, London (Google Maps).

Project 2 Murray Grove, London

Table 34. Project characteristics Murray Grove, London.

Building properties	
Layout	L-shaped gallery flat
Height	5 storeys
Stability	X-braced walls
Function	Low-rental housing
Module properties	
Area	Width 3.2 m, length 8.0 m, height 3.0 m
Floor system	C-section joists
Wall elements	C-section studs

Figure 86. Construction of Murray Grove (Cartwright Pickard).


Figure 87. Braced access walkways of Murray Grove (Cartwright Pickard).

H3 External Steel structure projects

Project 3 Sir Charles Groves Hall, Manchester

Table 35. Project Characteristics Sir Charles Groves Hall, Manchester.

Building properties	
Layout	Corridor shape in all four sides and an internal courtyard
Height	6 to 9 storeys.
Stability	Steelwork staircases on four sides
Load transfer	Wind loads transferred laterally to the cores on each face
Function	Student apartments
Module properties	
Floor system	C-section joists
Wall elements	C-section studs


Figure 88. Sir Charles Grove Hall (Liberty Living at Sir Charles Grove Hall, Manchester, 2020).


Figure 89. Bird View Sir Charles Grove Hall (Google Maps).

Project 4 MoHo, Manchester

The characteristics of this project are that the modules are placed parallel to façade rather than perpendicular and open sides using 1 or 2 intermediate posts SHS 100 mm. Steel braced frame connected to corners of modules to transfer loads between them. Apartment length is extended using an additional second bedroom module.

Table 36. Project characteristics MoHo, Manchester.

Building properties	
Layout	U-shaped with modules placed parallel to the facade
Height	Commercial ground floor and 6 storeys of modules
Stability	Internal stability combined with external steel frame for transverse stability
Load transfer	All horizontal forces in transverse direction transferred through inter-modular connections to steel structure
Function	Commercial ground floor, other floors one or two-bedroom apartments
Apartment area	38-54 m^2
Module properties	
Dimensions	Width 4.1, length 9.1 m, height 3.0 m
Wall support	1 or 2 intermediate posts using 100 mm SHS

Figure 90. Bird view MoHo, Manchester (Google Maps).

Figure 91. MoHo, Manchester (Barbour Product Search).

Figure 92. Braced walkways of MoHo (Steelconstruction.info).

Project 5 Raines Court, London

Table 37 . Project characteristics Raines Court, London.

Building properties	
Layout	T-shaped gallery flat
Height	6 storeys
Stability	Braced longitudinal walls and x-bracing around steel-framed access core
Load transfer	Wind transferred through transverse walls to core
Apartment area	40 m^2
Function	Two- and three-bedroom family apartments
Module properties	
Dimensions	Width 3.8 m, length 9.6 -11.6 m, height 3.0 m
Floor system	Cold formed galvanized steel ‘plate floor’ with structural board floor deck
Wall system	Hot rolled columns and insulated cold formed galvanized steel frame

Figure 93. Rear facade of Raines Court (Allford Hall Monaghan Morris).

Figure 94. North Elevation of Raines Court (Allford Hall Monaghan Morris).

Figure 95. Bird view Raines Court (Google Maps).

Concrete core projects

Project 2 Sentinel Housing Association Basingstoke

The Vision modular building system was used for this project. This system has the following materials: concrete floors are supported by parallel flange channel (PFC) sections and a wall profile of 60x60 mm square hollow sections (SHS) placed at 600 mm centres are used, which support the 9 storeys of modules. The modules are partially open-sided and utilize the spanning capabilities of the PFC edge beams and balconies were attached to the perimeter PFC sections. This structure has a total fire rating of 120 minutes.

Table 38. Project characteristics Sentinel Housing Association Basingstoke**.**

Building properties	
Layout	Corridor
Height	6 to 11 floors
Stability	Longitudinal braced module walls and a core for lateral wind forces
Load transfer	Lateral forces transferred to concrete core through floor diaphragms
Function	Residential use
Apartment area	48 m^2 or 60 m^2
Module properties	
Dimensions	Length 7.2 m, width 3.0-3.6 m, height 3.0 m
Floor system	150 mm deep concrete floor, PFC sections around perimeter SHS roof
Wall elements	Structural hollow sections welded into frames

Figure 96. Bird view Sentinel Housing Association Basingstoke (Google Maps).

Project 2 Allegro, Dublin

Table 39. Project characteristics Allegro, Dublin.

Building properties	
Layout	Corridor type, irregular plan form due to non-rectangular shaped modules
Height	Commercial ground floor and 4 to 9 floors of apartments
Stability	Reinforced concrete podium, concrete access core
Function	Ground floor office, residential use on floors above
Module properties	
Dimensions	Width 3.3-4.2 m, length 6-11 m, internal height 2.45 m
Wall elements	Vision modular system SHS 60x60 mm at 600 mm centres
Floor system	Vision modular system, concrete floor, PFC edge beams

Figure 97. Installation of modules at the Allegro project (Lawson et al., 2010).

Figure 98. Allegro Dublin (Google Maps).

Project 3 Student residences, Wolverhampton

Table 40. Project characteristics Wolverhampton.

Building properties	
Layout	Combination of gallery and cluster shape around concrete core
Height	8 to 25 storeys
Stability	Concrete core
Load transfer	Vertical loads resisted by module walls. Horizontal loads transferred to core in-plane by the modules.
Function	Student apartments
Module properties	
Dimensions study room	Width 2.5 m, Length 6.7 m
Dimensions communal room	Width 4.2 m, Length 6.7 m
Floor system	Vision modular system
Wall elements	Vision modular system

Figure 99. Modular building in Wolverhampton (O'Connell East Architects).

Project 4 Croydon Tower, Croydon

Table 41. Project characteristics Croydon Tower, Croydon (Lane, 2019).

Building properties	
Layout	Cluster around core
Height	38 to 44 storeys
Stability	Concrete core
Load transfer	All horizontal loads transferred to the core in-plane by the modules
Function	Residential use
Module properties	
Floor system	Concrete slab
Wall elements	Corner posts varying from 150 mm to 300 mm at the bottom and intermediate posts of 60 mm.

Figure 100. Construction of Croydon Tower (Lane, 2019).

Project 5 Laan van Spartaan, Amsterdam

Table 42. Project characteristics Laan van Spartaan, Amsterdam (Pieters Bouwtechniek, 2017).

Building properties	
Layout	Corridor type
Height	6 to 16 floors
Stability	Combined system a concrete core and braced module walls in some of the modules
Load transfer	Transfer of horizontal forces through single floor diaphragms to stabilising elements
Function	Student apartments
Module properties	
Floor system	Concrete slab
Wall elements	Small concrete walls


Figure 101. Student housing Laan van Spartaan, Amsterdam (Ursem Modulaire bouw, 2017).

Timber structure project

Project 1 Hotel Jakarta, Amsterdam

Table 43. Project characteristics Hotel Jakarta, Amsterdam (Pieters Bouwtechniek).

Building properties	
Layout	V-Shaped with modules perpendicular to both legs of the V and central area in between.
Height	Ground floor and 5 to 9 storeys of modules
Stability	X-lam timber walls combined with concrete table structure on lower floors and concrete core for vertical services.
Load transfer	Horizontal load taken down through x-lam walls towards concrete structure
Apartment area	30-42 m^2
Function	Hotel
Module properties	
Floor system	Concrete slab
Wall elements	X-laminated 5 plate layered timber walls

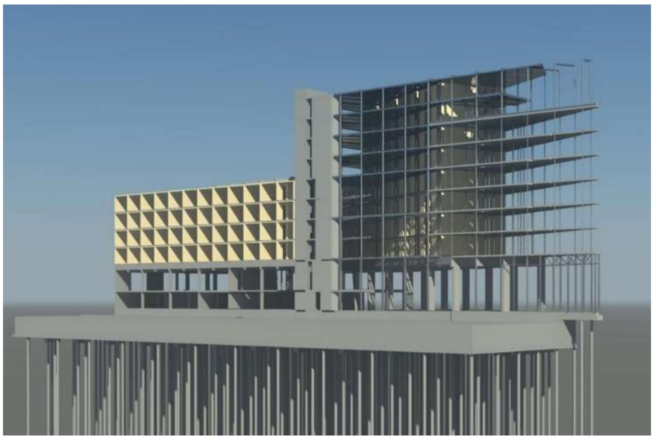


Figure 102. Concrete and Timber structure of Hotel Jakarta (Pieters Bouwtechniek).

C. Case study project properties

Murray Grove

Structural sizes

The module type that has been used in the Murray Grove project is from the company Yorkon. Since the dimensions of the structural elements are unknown, they will be based on the standard sizes of these Yorkon type modules. The standard sizes are visible in the table below (Lawson et al., 2010).

Table 44. Standard Yorkon module element dimensions (Lawson et al., 2010).

Vertical elements	Height (mm)	Centres (mm)	Thickness (mm)
Steel wall studs	60 to 100	400 to 600	1,2 to 2,0
Steel edge column	100	3000 to 6000	1,2 to 4,0
Horizontal elements			
Steel edge beam	200 to 350	3000 to 6000	2,4
Steel floor and ceiling joists	100 to 200	400 to 600	1,2

Partition structures

The element sizes of the wall, floor and ceiling elements, as well as edge beams and wall posts are chosen based on their critical capacity such as bending stress, deflection and normal stress. The British code of practice for dead and imposed loads (BS 6399-1: 1996) has been used to obtain the applicable uniformly distributed and concentrated loads in residential buildings. These loads for domestic and residential activities are as follows:

- Uniformly distributed load: 1.5 kN/m^2
- Concentrated load: 1.4 kN

Table 45. Element sizes Murray Grove modules.

Wall post C-section			*Floor joist C-section*		
b	0,10	m	b	0,06	m
h	0,10	m	h	0,15	m
t	0,002	m	t	0,0012	m
Wall stud/Ceiling joist C-section			*Edge beam C-section*		
b	0,08	m	b	0,08	m
h	0,10	m	h	0,20	m
t	0,0012	m	t	0,004	m
Steel bracings					
Bracings/module	2				
t	0,005	m			
h	0,10	m			

The build-up of the wall, floor and ceiling structure has been based on the previously mentioned build-ups for steel lightweight modules.

Table 46. Wall build-up Murray Grove (own work).

	Thickness (m)	Height (m)	Weight (kN/m^3)	Resulting load on wall/m
Plasterboard	0,030	2,5	8,50	0,63
Mineral wool	0,100	2,5	0,60	0,15
Plasterboard	0,015	2,5	8,50	0,32
	Area (m^2)	**Height (m)**	**Number/m**	**Resulting load on wall/m**
C-studs	0,00036	2,5	2,50	0,15
Total				1,25

Table 47. Floor build-up Murray Grove (own work).

	Thickness (m)	Height (m)	Weight (kN/m^3)	Resulting load on wall/m
Plasterboard	0,030	3,2	8,50	0,41
Mineral wool	0,15	3,2	0,60	0,14
Plasterboard	0,015	3,2	8,50	0,20
	Area (m^2)	**Height (m)**	**Number/m**	**Resulting load on wall/m**
C-studs	0,00032	3,2	1,67	0,067
Total				0,82

Table 48. Ceiling build-up Murray Grove (own work).

	Thickness (m)	Span (m)	Weight (kN/m^3)	Resulting load on wall/m
Plasterboard	0,030	3,2	8,50	0,48
Plasterboard	0,015	3,2	8,50	0,20
	Area (m^2)	**Span (m)**	**Number/m**	**Resulting load on wall/m**
C-studs	0,00036	3,2	2,50	0,10
Total				0,78

Table 49. Building properties Murray Grove.

Number of floors	5	
Rows of modules	1	
Braced column span	2,4	m
Entrance span	3,1	m
Number of bays along length	2	
Module length	8,00	m
Loaded module width	2,9	m
External width	3,20	m
Free floor height	2,5	m
Floor height	3,00	m
Total height	15,00	m
Internal columns	2	
Longitudinal bracing length	3,84	m
Door width	1,10	m
Window width	1,10	m

Case study Raines Court

Partition structures

As in the previous case study, the element sizes of the wall, floor and ceiling structures are calculated based on the critical load-bearing capacity.

Table 50. Element sizes Raines Court modules.

Wall post C-section			**Floor joist C-section**		
B	0,10	m	B	0,10	m
H	0,10	m	H	0,15	m
t	0,004	m	t	0,0012	m
Wall stud/Ceiling joist C-section			**Edge beam C-section**		
B	0,10	m	B	0,08	m
H	0,10	m	H	0,20	m
t	0,002	m	t	0,003	m
Steel bracings					
Bracings/module	2				
t	0,010	m			
H	0,10	m			

Table 51. Wall build-up Raines Court (own work).

	Thickness (m)	**Span (m)**	**Weight (kN/m^3)**	**Weight (kN/m length)**
Plasterboard	0,030	2,52	8,50	0,64
Mineral wool	0,100	2,52	0,60	0,15
Plasterboard	0,030	2,52	5,00	0,38
	Area (m^2)	**Height (m)**	**Number/m**	**Weight (kN/m length)**
C-studs	0,00059	2,52	2,50	0,29
Total	0,13			1,47

Table 52. Floor build-up Raines Court (own work).

	Thickness (m)	**Span (m)**	**Weight (kN/m^3)**	**Weight (kN/m length)**
Plasterboard	0,030	3,80	8,50	0,48
Mineral wool	0,15	3,80	0,60	0,17
Plasterboard	0,030	3,80	8,50	0,48
	Area (m^2)	**Height (m)**	**Number/m**	**Weight (kN/m length)**
C-studs	0,00059	3,80	1,67	0,14
Total				1,29

Table 53. Ceiling build-up Raines Court (own work).

	Thickness (m)	**Span (m)**	**Weight (kN/m^3)**	**Weight (kN/m length)**
Plasterboard	0,030	3,80	8,50	0,48
Plasterboard	0,030	3,80	8,50	0,48
	Area (m^2)	**Height (m)**	**Number/m**	**Weight (kN/m length)**
C-studs	0,00059	3,80	2,50	0,22
Total				1,19

Table 54. Raines court building properties.

Number of floors	6	
Rows in N-S direction	1	
Braced column span	2,40	m
Entrance span	1,35	m
Unbraced internal span	3,90	m
Facade span	1,95	m
Number of bays along length	3	
Module length	12,00	m
Loaded module width	3,47	m
External width	3,80	m
Free floor height	2,52	m
Floor height	3,00	m
Total height	18,00	m
Internal columns	3,00	
Longitudinal bracing length	3,84	m
Door width	1,10	m
Window width	1,10	m

North Orleans

Properties

The building has the following properties:

Table 55 North Orleans building properties.

Number of floors	7	
Rows of modules	1	
Front floor length	2,7	m
Back floor length	5,1	m
Module length	7,8	m
Intermediate walls	1	
Structural module length	7,8	m
External width	3,5	m
Storey height	2,9	m
Ground floor height	3,4	m
Total height	20,7	m
Transverse wall length	1,6	m

Partition structures

The different layers of the wall, floor to ceiling structure, façade and roof will be shown in the table below. The build-up is done from the inside to the outside.

Table 56. Wall build-up North Orleans.

Concrete wall	130	mm

Table 57. Combined floor to ceiling build-up North Orleans.

Floor slab	80	mm
Insulation	150	mm
Ceiling slab	60	mm

Table 58. Closed side facade build-up North Orleans **.**

Concrete wall	130	mm
Insulation	100	mm
Wooden framework	50x90 and 50x50	mm
Water-resistant foil		
Horizontal wooden framework		

Table 59. Roof build-up North Orleans.

Ceiling slab	60	mm
Insulation layer 1	120	mm
Insulation layer 2	120	mm
2 layers of bitumen cover		

Regioplein

Properties

The building has the following properties:

Table 60. Regioplein building properties.

Number of floors	4	
Total height	13	m
Rows of modules	1	
Internal columns	2	
Number of bays along length	3	
Entrance braced column span	3,25	m
Internal span	3,60	m
Back braced façade span	3,02	m
Structural module length	9,87	m
Total module length	10,6	m
External width	4,0	m
Floor height	3,25	m
Longitudinal bracing length	3,7	m

Element sizes

The modules have an internal load-bearing structure of steel and concrete. The modules are standard and fabricated by the company Ursem. The maximum number of storeys possible with this module is 5 and the maximum dimensions are 4.0 m in width and 12.5 m in length. The vertical structure along its length consists of a timber frame and steel columns at the four corners and midspan along the length. In transverse direction the façade is also connected to a timber frame. The floor consists of a single floor slab with a rib support along its edges. The ceiling is also a timber frame with insulation between the studs. Stabilising bracing can be placed along its length between the columns and box profile can be used in transverse direction in front of the closed section of facades and near internal transverse walls if necessary.

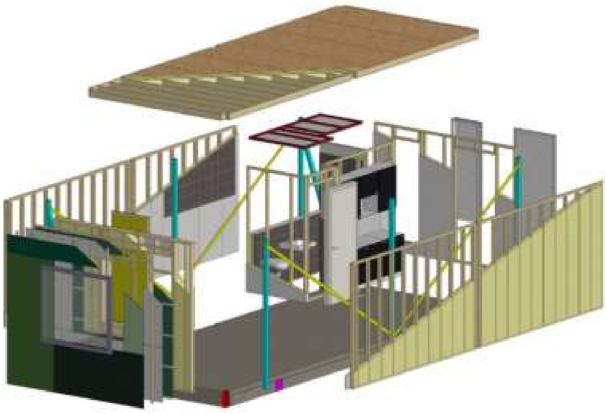


Figure 103. IDS System (Ursem).

Partition structures

The different layers of the wall, floor to ceiling structure, façade and roof will be shown in the table below. The build-up is done from the inside to the outside.

Table 61. Floor build-up Regioplein.

Slab thickness	100 mm
Edge rib	200 x 320 mm

Table 62. Wall build-up Regioplein.

Plasterboard	2x 15 mm
Timber studs	38x89, c.t.c. 600 mm
Rockwool (type 201)	90
Water resistant foil	
Bracing	6x80 mm

Table 63. Ceiling build-up Regioplein.

Plasterboard	15 mm
Timber beams	38x120 C18, c.t.c 400
Rockwool (201 vario)	120 mm
OSB-3	18 mm
EPDM (synthetic rubber)	Thin layer

Detailing

The vertical connection between the steel columns consists, from bottom to top, of a rubber support with a coupling plate above it and a cone attached to it. During the installation on site, the column of the module above will be positioned over the cone to provide vertical continuity. The coupling plate provided horizontal continuity between two adjacent modules.

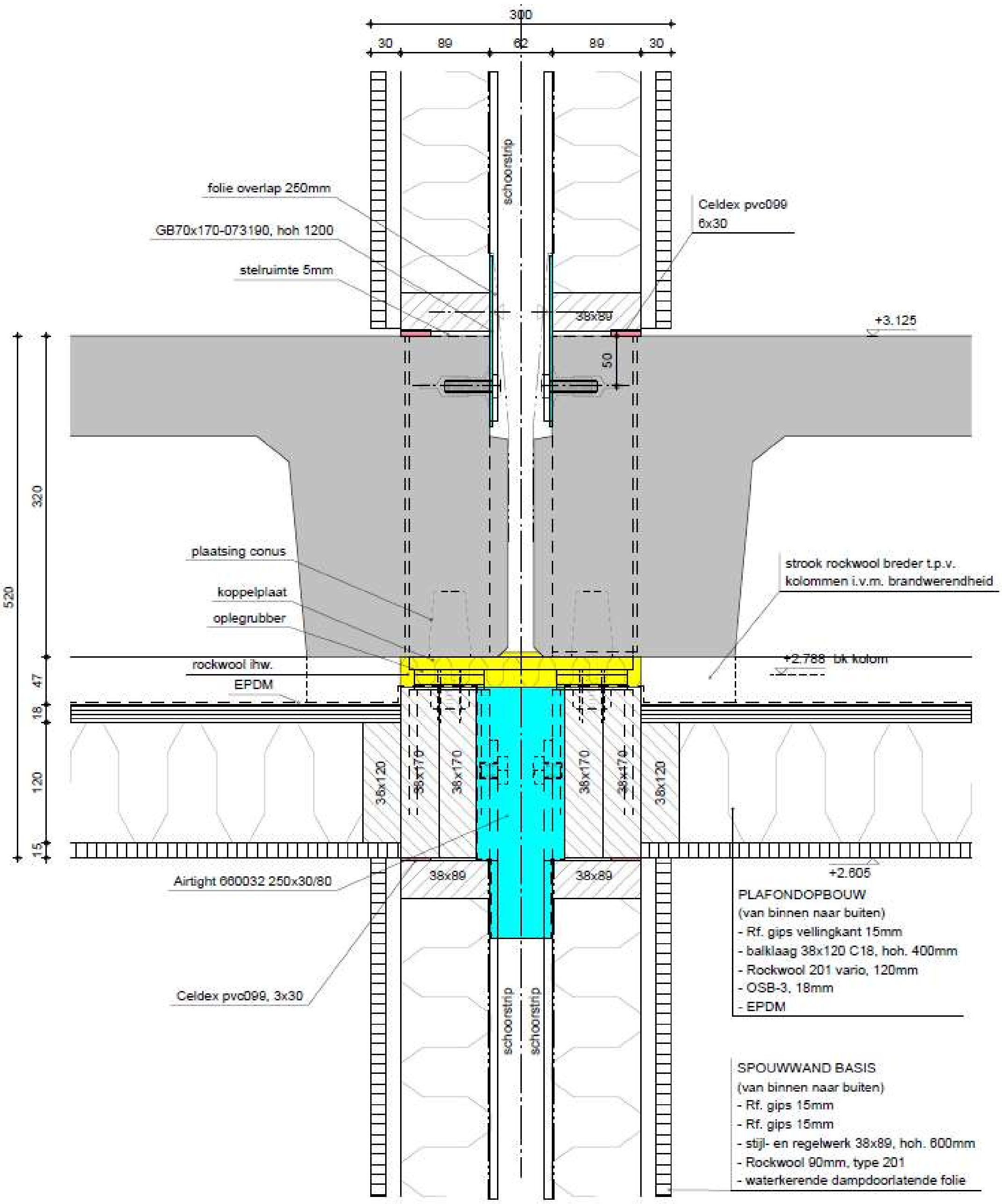


Figure 104. Connection detail (Pieters Bouwtechniek).

D. Case study calculations

Murray Grove Longitudinal displacement

	Loads						Storey rotation		Internal forces
	Column in tension			Column loaded by compression			Due to wind	Due to weight	
Storey number	F,wind,left (kN) tension	F,perm,left (kN)	F,left,tot (kN)	F,wind,right (kN)	F,perm,right (kN)	F,right,tot (kN)	phi (mm/m-storey)	phi (mm/m-storey)	H, hor
5	0,00	0,00	0,93	0,93	0,00	0,93			0,75
4	-0,93	9,36	8,42	3,74	15,31	19,05	-0,143	-0,091	2,24
3	-3,74	18,72	14,98	8,41	30,62	39,03	-0,371	-0,182	3,74
2	-8,41	28,07	19,66	14,95	45,93	60,88	-0,714	-0,273	5,23
1	-14,95	37,43	22,48	23,36	61,25	84,60	-1,171	-0,364	6,73

Displacements										
Bracing elon.	Column rotation			Column compression		F,wind				
u1	u2, H,wind	u3,F,module	**u,hor,phi,total**	u,vert,column	u4,F,module	u,vert,column	u5, F,wind	**u,hor,kolom total**	u,h,uit H total	u,h,tot,storey (mm)
0,09	7,71	2,73	10,44			0,02	0,03	0,03	11,47	**14,20**
0,26	5,31	1,82	7,13			0,09	0,11	0,11	8,96	10,78
0,44	3,06	1,00	4,06			0,21	0,26	0,26	6,32	7,33
0,61	1,17	0,36	1,54			0,37	0,46	0,46	3,74	4,11
0,79	0,00	0,00	0,00			0,57	0,71	0,71	1,50	1,50

Second order											
First iteration						Second iteration					
Adjoining load (kN	H,extra (kN)	% of H	u,storey	u,total	u,h,tot,2e orde	Adjoing load (kN)	H,extra (kN)	% of H	u,storey	u,total	u,h,tot,2e orde
34,03	0,00	0,00	0,00	0,51	14,71	34,03	0,00	0,0000	0,00	0,04	14,75
68,05	0,06	2,665	0,24	0,51	11,29	68,05	0,005	0,2352	0,02	0,04	11,32
102,08	0,09	2,350	0,17	0,27	7,60	102,08	0,006	0,1531	0,01	0,01	7,61
136,11	0,10	1,944	0,08	0,10	4,21	136,11	0,004	0,0679	0,00	0,00	4,21
170,13	0,09	1,267	0,02	0,02	1,52	170,13	0,001	0,0158	0,00	0,00	1,52

Raines Court Longitudinal displacement

	Loads						Storey rotation		Internal forces
							Due to wind	Due to weight	
Storey number	F,wind,left (kN) tension	F,perm,left (kN)	F,left,tot (kN)	F,wind,right (kN) compression	F,perm,right (kN)	F,right,tot (kN)	phi (mm/m-storey)	phi (mm/m-storey)	H, hor
6	0,00	0,00	1,30	1,30	0,00	1,30			1,04
5	-1,30	14,74	13,43	5,21	20,09	25,30	-0,102	-0,042	3,12
4	-5,21	29,47	24,27	11,72	40,18	51,90	-0,266	-0,084	5,21
3	-11,72	44,21	32,49	20,83	60,27	81,10	-0,512	-0,126	7,29
2	-20,83	58,95	38,12	32,54	80,36	112,90	-0,839	-0,168	9,37
1	-32,54	73,68	41,14	46,86	100,45	147,31	-1,248	-0,210	11,46

Displacements										
Bracing elon.	Column rotation			Column compression		F,wind				
u1	u2, H,wind	u3,F,module	**u,hor,phi,total**	u,vert,column	u4,F,module	u,vert,column	u5, F,wind	**u,hor,kolom total**	u,h,uit H total	u,h,tot,storey (mm)
0,05	11,77	2,31	14,08			0,02	0,02	0,02	15,39	**17,70**
0,15	8,80	1,68	10,48			0,07	0,08	0,08	12,35	14,03
0,24	5,93	1,09	7,03			0,15	0,18	0,18	9,26	10,35
0,34	3,34	0,59	3,92			0,26	0,33	0,33	6,23	6,82
0,44	1,25	0,21	1,46			0,41	0,51	0,51	3,47	3,68
0,54	0,00	0,00	0,00			0,59	0,74	0,74	1,27	1,27

Second order											
First iteration						Second iteration					
Adjoining load	H,extra (kN)	% of H	u,storey	u,total	u,h,tot,2e orde	F,perm op die verdieping (kN)	H,extra (kN)	% of H	u,storey	u,total	u,h,tot,2e ord
0,00	0,00	0,00	0,00	0,58	18,28	0,00	0,00	0,0000	0,00	0,03	18,31
47,84	0,05	1,579	0,19	0,58	14,61	47,84	0,003	0,0980	0,01	0,03	14,64
95,69	0,10	1,854	0,19	0,38	10,73	95,69	0,006	0,1154	0,01	0,02	10,75
143,53	0,13	1,809	0,12	0,19	7,01	143,53	0,006	0,0795	0,01	0,01	7,01
191,37	0,14	1,497	0,06	0,07	3,75	191,37	0,004	0,0370	0,00	0,00	3,75
239,21	0,10	0,887	0,01	0,01	1,28	239,21	0,001	0,0078	0,00	0,00	1,29

E. Case study assessment

Properties and material quantities

Project	Building properties					
	Shape	Number of blocks	Modules in row	Modules per storey	Number of storeys	Balcony included
Murray Grove	L-shaped gallery	2	7	15	5	Yes
Raines Court	T-shaped gallery	3	10	21	6	Yes
North Orleans	U-shaped gallery	3	9	20	7	Yes
Regioplein	Single gallery	1	6	6	4	Yes

Module Dimensions (m)							
Length	Width	Height	Total modules	Total area	Structural height	Wall width	Internal walls
8,00	3,20	3,00	75	25,6	0,5	0,16	1
12,00	3,80	3,00	126	45,6	0,5	0,16	2
7,80	3,50	2,90	140	27,3	0,29	0,13	1
10,60	4,00	3,25	24	42,4	0,5	0,16	1

Efficiency				Stabilising elements	
Usable area (m2)	Space efficiency factor	Facade area (m2)	Wall-to-floor ratio	Longitudinal	Transverse
21,50	0,84	9,60	0,45	1	4
39,33	0,86	11,40	0,29	1	2
23,91	0,88	10,15	0,42	0	2
37,09	0,87	13,00	0,35	1	2

Material use	Structure	Plasterboard		Insulation		Wood	
		Thickness layer (m)	Total volume (m3)	Thickness layer (m)	Total volume (m3)	Area (m2/m length)	Total (m3)
Murray Grove	Wall	0,045	2,12	0,1	4,72		
	Floor	0,045	1,15	0,15	3,84		
	Ceiling	0,045	1,15				
Raines Court	Wall	0,045	3,48	0,1	7,74		
	Floor	0,045	2,05	0,15	6,84		
	Ceiling	0,045	2,05				
North Orleans	Wall						
	Floor			0,15	4,095		
	Ceiling						
Regioplein	Wall	0,06	4,11	0,09	6,1578	0,0056	0,27
	Floor						
	Ceiling	0,015		0,12	5,088	0,0114	0,48

Steel studs		Steel beam		Steel columns		Concrete	
Area (m2/m length)	Total (m3)	Area (m2/m length)	Total (m3)	Area (m2/m length)	Total (m3)	Thickness (m)	Total (m3)
0,0008	0,036	0,0029	0,044	0,0048	0,0288		
0,0005	0,014						
0,0008	0,020						
0,0015	0,114	0,0022	0,050	0,0096	0,0576		
0,0007	0,031						
0,0015	0,067						
						0,13	6,38
						0,08	2,18
						0,06	1,64
				0,0205	0,1331	0,2	1,27
						0,1	3,82
							0

Environmental analysis and result

Environmental analysis		Unit	Plaster/Gypsum	Insulation	Wood	Steel	Concrete	Total
	Embodied Energy	MJ/kg	6,75	16,60	10,00	21,50	1,90	
	Embodied Carbon	kgCO2/kg	0,39	1,28	0,46	1,53	0,22	
	Volumetric weight	kN	8,50	0,50	6,00	79,00	25,00	
Murray Grove	Volume	m3	4,43	8,56	0,00	0,14	0,00	13,1
	Weight	kg	3.764	428	0	1.130	0	5.322,1
	Embodied energy	MJ	25.406	7.105	0	24.301	0	56.811,0
	Embodied carbon	kgCO2	1.468	548	0	1.729	0	3.745,0
Raines Court	Volume	m3	7,59	14,58	0,00	0,32	0,00	22,5
	Weight	kg	6.449	729	0	2.531	0	9.708,8
	Embodied energy	MJ	43.530	12.101	0	54.413	0	110.044,8
	Embodied carbon	kgCO2	2.515	933	0	3.872	0	7.320,4
North Orleans	Volume	m3	0,00	4,10	0,00	0,00	10,20	14,3
	Weight	kg	0	205	0	0	25.502	25.706,9
	Embodied energy	MJ	0	3.399	0	0	48.454	51.852,8
	Embodied carbon	kgCO2	0	262	0	0	5.610	5.872,5
Regioplein	Volume	m3	4,11	11,25	0,75	0,13	5,09	21,3
	Weight	kg	3.489	562	452	1.052	12.720	18.275,7
	Embodied energy	MJ	23.554	9.334	4.524	22.610	24.168	84.189,6
	Embodied carbon	kgCO2	1.361	720	208	1.609	2.798	6.696,1

Environmental result		Unit	Per m2	Module weight (tonnes)	Truck fuel use (L)	Truck carbon use (kg)
	Embodied Energy	MJ/kg				
	Embodied Carbon	kgCO2/kg				
	Volumetric weight	kN				
Murray Grove	Volume	m3				
	Weight	kg	208	5,3	72,33	188
	Embodied energy	MJ	2.219			
	Embodied carbon	kgCO2	146			
Raines Court	Volume	m3				
	Weight	kg	213	9,7	89,29	232
	Embodied energy	MJ	2.413			
	Embodied carbon	kgCO2	161			
North Orleans	Volume	m3				
	Weight	kg	942	25,7	100,25	261
	Embodied energy	MJ	1.899			
	Embodied carbon	kgCO2	215			
Regioplein	Volume	m3				
	Weight	kg	669	18,3	114,29	297
	Embodied energy	MJ	1.986			
	Embodied carbon	kgCO2	158			

F. Comparison of existing inter-module connections

Connections	Structural (S) Requirements		Manufacturing (M) Requirements					Construction (C) Requirements						
	S1	S2	M1	M2	M3	M4	M5	C1	C2	C3	C4	C5	C6	C7
ISO [115, 117]	■	■	▩	■	▩	■	■	■	□	■	■	■	■	■
ATLSS [139, 140]	■	■	■	▩	■	▩	□	■	□	■	▩	■	▩	□
Annan [85]	■	■	■	■	■	■	□	□	□	□	□	□	□	■
Lawson *et al.* [4, 123]	■	■	■	■	■	■	□	□	□	■	■	■	▩	■
Farnsworth [141]	■	■	▩	▩	■	▩	□	■	■	▩	□	▩	▩	■
VectorBloc™ [142-144]	■	■	▩	▩	■	▩	■	■	□	■	■	■	▩	■
Hickory [15]	■	■	▩	▩	■	□	□	■	□	□	▩	▩	□	■
Styles *et al.* [146]	■	■	■	■	■	■	■	□	□	■	▩	■	▩	□
Gunawardena [73]	■	■	■	■	■	■	■	□	□	■	■	■	▩	□
Choi *et al.* [74]	■	■	■	■	■	■	□	□	□	■	■	■	▩	■
Heather *et al.*, 1 [147]	■	■	▩	▩	□	▩	■	■	□	■	■	■	▩	■
Heather *et al.*, 2 [148]	■	■	▩	▩	□	▩	■	■	□	■	■	■	▩	■
Chen *et al.*, 1 [149, 150]	■	■	■	▩	□	■	□	■	□	■	■	■	▩	■
Chen *et al.*, 2 [151]	■	■	▩	■	□	□	■	□	□	□	□	□	□	■
Deng *et al.*, 1 [152]	■	■	■	▩	□	■	▩	■	□	■	▩	■	▩	■
Deng *et al.*, 2 [153]	■	■	▩	■	□	▩	□	□	□	▩	▩	■	□	■
Doh *et al.* [154]	■	■	■	■	■	■	■	□	□	■	▩	■	▩	■
Lee *et al.* [155]	■	■	■	■	□	■	▩	□	□	■	▩	■	▩	■
Sharafi *et al.* [156]	□	■	■	■	■	■	▩	□	■	■	■	■	■	■
Sanches *et al.* [157]	■	■	▩	▩	□	▩	□	■	□	▩	□	▩	▩	■
Yu *et al.* [158]	■	■	■	▩	■	■	■	■	□	▩	■	■	□	■
Chen *et al.*, 3 [159]	■	■	■	▩	▩	■	■	■	□	▩	▩	▩	▩	■
Dai *et al.* [160]	■	■	□	□	▩	■	□	■	■	■	■	■	□	■
Lacey *et al.*, 1 [161]	■	■	■	■	□	■	■	■	□	■	■	■	▩	□
Lacey *et al.*, 2 [162]	■	■	■	■	□	■	■	■	□	▩	□	▩	▩	■
Generic (**Fig. 7**)	■	■	■	■	■	■	■	□	□	■	■	■	▩	□

□	Requires modifications (0 ≤ weighted score (WS) < 0.34)
▩	Can partially meet requirements (0.34 ≤ WS < 0.67)
■	Can meet requirements (0.67 ≤ WS ≤ 1)
S1	Capable of withstanding vertical plane tension
S2	Capable of horizontal plane or diaphragm axial and shear resistance
M1	Number of unique parts in a connecting system to achieve vertical and horizontal connectivity
M2	Complexity of parts and the manufacturing process complexity as per assinged modifiers
M3	Complexity and requirement of post-manufacturing integration of parts as per assigned modifiers
M4	The final number of unique off-the-shelf parts after integration
M5	Ease in pre-attaching the connecting system to modules, as per assigned modifiers
C1	Incorporates self-aligning or self-guiding features
C2	Capable of achieving inter-module connectivity remotely without requiring direct access
C3	Complexity of engaging inter-module connectivity as per assigned modifiers
C4	The number of operations to engage a type-b inter-module connectivity as per assigned modifiers
C5	The number of tools required to engage connectivty as per assigned modifiers
C6	Capable of being easily demounted
C7	Capable of minimising non-usable space between modules

G. Gunawardena inter-module joint verification

Nominal shear capacity

$$F_{v,Rd} = \frac{\alpha_v * f_{ub} * A}{\gamma_{M3}} = \frac{0{,}6 * 800 * 84.3}{1.25} = 32.4\ kN$$

- α_v = Reduction factor for the bolt class
- f_{ub}= Ultimate bolt strength (N/mm^2)
- A = Bolt gross cross-section (mm^2)
- γ_{M3}= Partial safety factor

Conclusion

Since the joint consists of 4 bolts, the total shear capacity is 129.5 kN. Since it is experimental, reduction factor for the capacity of 0.8 is applied and the capacity is reduced to 103.6 kN.

Unity Check

$$F_{v,Ed} = \frac{q_{wind} * w_{module} * h_{storey}}{2} * n_{storeys} = \frac{0.96*3.5*3.1}{2} * 8 = 41.7\ kN$$

$$UC = \frac{F_{v,Ed}}{F_{v,Rd}} = \frac{41.7}{103.6} = 0.40$$

Plate bearing

Shear failure is anticipated to be most critical. Checks for bearing and tear-out are done for the plies. 6 mm thick ply most critical.

$$F_{b,Rd} = \frac{k_1 * a_b * f_{up} * d * t_p}{\gamma_{M2}}$$

$$k_1 = \min\left(2.8 * \frac{e_2}{d_0} - 1.7; 1.4 * \frac{p_2}{d_0} - 1.7; 2.5\right)$$

$$a_b = \min\left(\frac{p_1}{3 * d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}\ or\ 1.0\right)$$

k_1= Function of edge distance (e2) and pitch perpendicular (p2) [-]
a_b= Function of pitch parallel (p1) [-]
d = Bolt diameter (mm)
t_p = Plate thickness (mm)
f_{up} = Strength of plate (N/mm^2)

In case the internal and edge distance are large enough to not reduce the k_1 and a_b factors, the reference value for $F_{b,Rd}$ is equal to:

$$F_{b,Rd} = \frac{2.5 * f_{up} * d * t_p}{\gamma_{M2}} = \frac{2.5*800*12*6}{1.25} * 10^{-3} = 115.2\ kN$$

The result is a pre-tension that needs to be below $115.2\ kN$.

Tear-out failure

$$V_p = a_e * t_p * f_{up}$$

a_e= Minimum distance from ply edge to centre of the hole in direction of bearing load

$V_p = 35 * 6 * 450 \ * 10^{-3} = 94.5\ kN$

Result: Tear-out capacity of full connection is 4 times V_p . This value is higher than the shear capacity, so the tear-out capacity is not critical.

Slip Resistance

The joint is treated as slip critical due to geometric position of the connection in the structure.

$$F_{s,Rd} = \frac{\mu * n * k_s * (F_{p,c} - 0.8 * (F_{t,Ed,ser})}{\gamma_{M3}}$$

- μ = Coefficient of friction between plies (EN 1090-2)

Table 64. Surface treatments.

	Friction coefficient μ
Blasted with shot or grid and not pitting	0.5
Surface blasted with shot or grid and painted	0.4
Cleaning with steel brush and removing rust particles	0.3
Not treated	0.2
Surface blasted with shot or grid and hot dip galvanised	0.1

n = Number of shear planes

$F_{p,c}$ = Minimum pretension on bolts during installation

$$F_{p,c} = \ 0.7 * f_{ub} * A_s$$

$$F_{p,c} = \ 0.7 * 800 * \frac{84.3}{1000} = 47.2\ kN$$

k_s = Factor or hole type (1.0 = standard, 0.85 = oversize, 0.7 = long slotted)

γ_{M3}= 1,1 (No slip in SLS)

$$F_{s,Rd} = \frac{0.2 * 1 * 1 * (47.2 - 0)}{1.1} = 8.58\ kN$$

The result is a slip at a load of 8.58 kN for one bolt. Entire connection is 4 times 8.58 kN is 34.3 kN.

Transverse stiffness

An accurate estimate of the stiffness of the connection will be made for both stages of the bolt deformation, the initial slip stage and the shear deformation stage. The values are estimated for each bolt first and then combined to find the overall stiffness of the joint. The significance of this value is related to the global structural system and this value can be used in the modelling of the connection as a spring or link type element that has a spring stiffness. The schematization of the bolt stiffness is shown in Figure 105.

(**a**) (**b**)

Figure 105. Gunawardena inter-module connection: (**a**) Top view, (**b**) Schematization of transverse bolt stiffness.

It can be seen that the stiffness of bolts 2 and 3 are parallel and together are in series with bolts 1 and 4. The resultant stiffness therefore is calculated as follows:

$$\frac{1}{k} = \frac{1}{k_{b1}} + (\frac{1}{k_{b2} + k_{b3}}) + \frac{1}{k_{b4}}$$

Slip stiffness single bolt

The slip capacity of a single bolt is simply the slip force divided by the hole clearance.

$$k_{b,slip} = \frac{P_{slip}}{\Delta_{slip}}$$

Δ_{slip}= 1 mm slip to edge of hole clearance

P_{slip} = 8,58 kN

$$k_{b,slip} = \frac{8{,}58\ kN}{1\ mm} = 8{,}58\ kN/mm$$

Slip stiffness entire connection

Since all bolts are equal, the slip stiffness of the entire connection is equal to:

$$\frac{1}{k_{slip}} = 2 * \frac{1}{8{,}58} + \left(\frac{1}{2 * 8{,}58}\right) = 0{,}29\ mm/kN$$

$$k_{slip} = 3{,}34\ kN/mm$$

Shear stiffness single bolt

$$k_{\tau} = \frac{G * A_s}{L}$$

A_s= Tensile stress area of bolt

G= Shear modulus of steel

$$k_{\tau} = \frac{80000 * 84{,}3}{31} = 218\ kN/mm$$

Shear stiffness entire connection

$$\frac{1}{k_\tau} = 2 * \frac{1}{217{,}5} + \left(\frac{1}{2 * 217{,}5}\right) = 0{,}0115\ mm/kN$$
$$k_\tau = 87{,}0\ kN/mm$$

Tension stiffness

The stiffness of a single plate in the connection is calculated first, then combined in series for the overall stiffness.

$$k_m = A * E * d * e^{B*\left(\frac{d}{l}\right)} \qquad (Wileman\ et\ al\ (1991))$$

A, B = Numerical constants, for steel these constants are 0.787 and 0.629 respectively
d = diameter of bolt clearance hole (mm)
l = grip length (mm)

$$k_{m(6mm)} = 0{,}787 * 210000 * 14 * e^{0{,}629*\left(\frac{14}{6}\right)} = 10035\ kN/mm$$
$$k_{m(25mm)} = 0{,}787 * 210000 * 14 * e^{0{,}629*\left(\frac{14}{25}\right)} = 3291\ kN/mm$$

Edge bolt: plates in series

$$\frac{1}{k_{m,edge}} = \frac{1}{k_{m(6m\ \ )}} + 2 * \frac{1}{k_{m(25mm)}} = 1414\ kN/mm$$

Middle bolt

$$\frac{1}{k_{m,middle}} = 2 * \frac{1}{k_{m(6mm)}} + 2 * \frac{1}{k_{m(25mm)}} = 1239\ kN/mm$$

Overall tension stiffness

$$\frac{1}{k_m} = \frac{1}{k_{m,edge}} + \frac{1}{2 * k_{m,middle}} + \frac{1}{k_{m,edge}} = 550\ kN/mm$$

Overall stiffness

$$\frac{1}{k_{br}} = \frac{1}{k_m} + \frac{1}{k_\tau} = 75\ kN/mm$$

Longitudinal stiffness

An accurate estimate of the stiffness of the connection will be made for both stages of the bolt deformation, the initial slip stage and the shear deformation stage. The values are estimated for each bolt first and then combined to find the overall stiffness of the joint. The significance of this value is related to the global structural system and this value can be used in the modelling of the connection as a spring or link type element that has a spring stiffness. The schematization of the bolt stiffness is shown below.

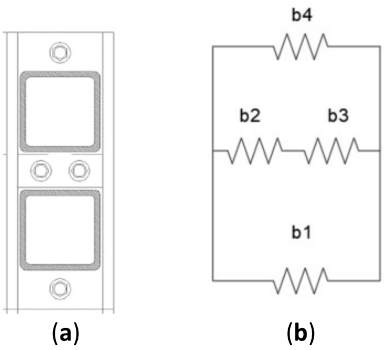


Figure 106 Gunawardena inter-module connection: (**a**) Top view, (**b**) Schematization of longitudinal bolt stiffness.

It can be seen that the stiffness of bolts 2 and 3 are parallel and together are in series with bolts 1 and 4. The resultant stiffness therefore is calculated as follows:

$$k = k_{b1} + (\frac{1}{\frac{1}{k_{b2}} + \frac{1}{k_{b3}}}) + k_{b4}$$

Slip stiffness single bolt

The slip capacity of a single bolt is simply the slip force divided by the hole clearance.

$$k_{b,slip} = \frac{P_{slip}}{\Delta_{slip}}$$

Δ_{slip}= 1 mm slip to edge of hole clearance
P_{slip} = 8.58 kN

$$k_{b,slip} = \frac{8.58\ kN}{1\ mm} = 8.58\ kN/mm$$

Slip stiffness entire connection

Since all bolts are equal, the slip stiffness of the entire connection is equal to:

$$k = k_{b1} + (\frac{1}{\frac{1}{k_{b2}} + \frac{1}{k_{b3}}}) + k_{b4}$$

$$k_{slip} = 21.45\ kN/mm$$

Shear stiffness single bolt

$$k_{\tau} = \frac{G * A_s}{L}$$

A_s= Tensile stress area of bolt
G= Shear modulus of steel

$$k_{\tau} = \frac{80000 * 84.3}{31} = 218\ kN/mm$$

Shear stiffness entire connection

$$k_{\tau} = k_{\tau 1} + (\frac{1}{\frac{1}{k_{\tau 2}} + \frac{1}{k_{\tau 3}}}) + k_{\tau 4}$$

$$k_{\tau} = 217.5 + \left(\frac{1}{\frac{1}{217.5} + \frac{1}{217.5}}\right) + 217.5 = 544\ kN/mm$$

Tension stiffness

The stiffness of a single plate in the connection is calculated first, then combined in series for the overall stiffness.

$$k_m = A * E * d * e^{B*\left(\frac{d}{l}\right)} \qquad (Wileman\ et\ al\ (1991))$$

A, B = Numerical constants, for steel these constants are 0.787 and 0.629 respectively
d = diameter of bolt clearance hole (mm)
l = grip length (mm)

$$k_{m(6mm)} = 0.787 * 210000 * 14 * e^{0.629*\left(\frac{14}{6}\right)} = 10035\ kN/mm$$

$$k_{m(25mm)} = 0.787 * 210000 * 14 * e^{0.629*\left(\frac{14}{25}\right)} = 3291\ kN/mm$$

Edge bolt: plates in series

$$\frac{1}{k_{m,edge}} = \frac{1}{k_{m(6mm)}} + 2 * \frac{1}{k_{m(25mm)}} = 1414\ kN/mm$$

Middle bolt

$$\frac{1}{k_{m,middle}} = 2 * \frac{1}{k_{m(6mm)}} + 2 * \frac{1}{k_{m(25mm)}} = 1239\ kN/mm$$

Overall tension stiffness

$$\frac{1}{k_m} = \frac{1}{k_{m,edge}} + \frac{1}{2 * k_{m,middle}} + \frac{1}{k_{m,edge}} = 550\ kN/mm$$

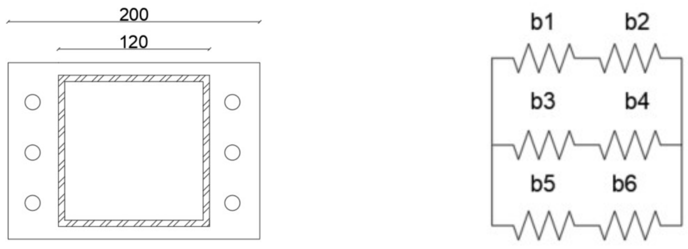
Overall stiffness

$$\frac{1}{k_{br}} = \frac{1}{k_m} + \frac{1}{k_\tau} = 274\ kN/mm$$

H. Styles et al inter-module joint verification

Stiffness of the connection

An accurate estimation of the stiffness of the connection will be made for both stages of the bolt deformation, the initial slip stage and the shear deformation stage. The values are estimated for each bolt separately first and then combined to find the total stiffness of the joint. The schematization of the bolt stiffness is shown in Figure 107 below.



(**a**) (**b**)

Figure 107 Styles inter-module connection: (**a**) Top view, (**b**) Schematization of longitudinal bolt stiffness.

It can be seen that the stiffness of bolts 2 and 3 are parallel and together are in series with bolts 1 and 4. The resultant stiffness therefore is calculated as follows:

$$k = \left(\frac{1}{\frac{1}{k_{b1}}+\frac{1}{k_{b2}}}\right) + \left(\frac{1}{\frac{1}{k_{b3}}+\frac{1}{k_{b4}}}\right) + \left(\frac{1}{\frac{1}{k_{b5}}+\frac{1}{k_{b6}}}\right)$$

Slip stiffness single bolt

The slip capacity of a single bolt is calculated by dividing the slip force by the hole clearance.

$$k_{b,slip} = \frac{P_{slip}}{\Delta_{slip}} = \frac{1}{8.58} = 8.58\ kN/mm$$

Slip stiffness entire connection

Since all bolts are the same, the slip stiffness of the entire connection is equal to:

$$k_{slip} = \frac{1}{\frac{1}{k_{b1}}+\frac{1}{k_{b2}}+\frac{1}{k_{b3}}} + \frac{1}{\frac{1}{k_{b4}}+\frac{1}{k_{b5}}+\frac{1}{k_{b6}}} = 5.72\ kN/mm$$

Shear stiffness single bolt

$$k_\tau = \frac{G * A_s}{L}$$

A_s= Tensile stress area of bolt

G= Shear modulus of steel

$$k_\tau = \frac{80000 * 84.3}{31} = 218\ kN/mm$$

Shear stiffness entire connection

$$k_\tau = \frac{1}{\frac{1}{k_{b1}}+\frac{1}{k_{b2}}+\frac{1}{k_{b3}}} + \frac{1}{\frac{1}{k_{b4}}+\frac{1}{k_{b5}}+\frac{1}{k_{b6}}} = 145\ kN/mm$$

Tension stiffness

The stiffness of a single plate in the connection is calculated first, then combined in series for the overall stiffness.

$$k_m = A * E * d * e^{B*\left(\frac{d}{l}\right)} \qquad (Wileman\ et\ al\ (1991))$$

A, B = Numerical constants, for steel these constants are 0.787 and 0.629 respectively
d = diameter of bolt clearance hole (mm)
l = grip length (mm)

$$k_{m(25mm)} = 0.787 * 210000 * 14 * e^{0.629*\left(\frac{14}{25}\right)} = 3291\ kN/mm$$

Overall tension stiffness

$$k_m = \frac{1}{\frac{1}{k_{b1}} + \frac{1}{k_{b2}} + \frac{1}{k_{b3}}} + \frac{1}{\frac{1}{k_{b4}} + \frac{1}{k_{b5}} + \frac{1}{k_{b6}}} = 731\ kN/mm$$

Overall stiffness

$$\frac{1}{k_{br}} = \frac{1}{k_m} + \frac{1}{k_\tau} = 121\ kN/mm$$

I. Inter-module joint rotational stiffness classification

The rotational stiffness of the inter-module joint has been calculated to find out if it classifies as a pinned, semi-rigid or rigid joint. The stiffness coefficients ki of the bolted end-plates are calculated and used in the formula to calculate the rotational stiffness S_j.

Figure 108 has been used to calculate the values for m and l_{eff}.
Leff is smallest value from Table 6.6 in EN 1993-1-8 (2005).

- $m = 20\ mm$
- $l_{eff} = 2 * \pi * m = 125{,}7\ mm$

$$k_{5,left,right} = \frac{0{,}9 * l_{eff} * t_p^3}{m^3} = 210\ mm$$

Using M12 bolts and a large length L_b for each bolt due two the thickness of 26 mm plates and two 2 mm plates, stiffness coefficient k_{10} can be calculated.

- $A_s = 84{,}3\ mm^2$
- $L_b = 72\ mm$

$$k_{10} = \frac{1{,}6 * A_s}{L_b} = 1{,}9\ mm$$

Using:

- $E = 210{,}000\ N/mm^2$
- $z = 5{,}8\ mm$
- $\mu = 1, since\ M_{j,ed}\ \leq \frac{2}{3}\ M_{j,Rd}$
- $No\ more\ than\ 1\ bolt\ row\ in\ tension$

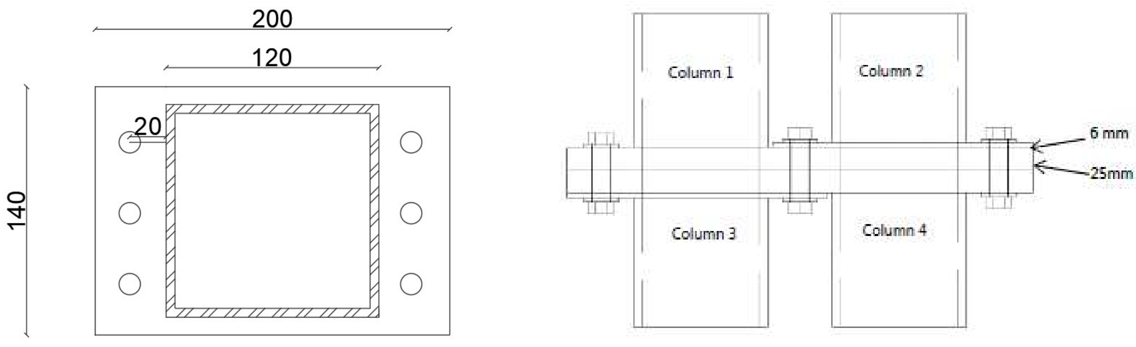
$$S_j\ = \frac{E * z^2}{\mu * \sum(\frac{1}{k_{5,left}} + \frac{1}{k_{5,right}} + \frac{1}{k_{10}})} * 10^{-6} = 1278\ kNm/rad$$

The value of the rotational stiffness S_j has been compared to the lower and upper boundaries of a semi-rigid joint to find out if it falls between these boundaries or that the joint is considered pinned or fully rigid.

$$\frac{0.5EI_c}{L_c} = 172\ kNm/rad, \qquad \frac{25EI_c}{L_c} = 8602\ kNm/rad$$

- $I_c = 5{,}1 * 10^{-6}\ m^4$
- $L_c = 5{,}1 * 10^{-6}\ m^4$

The value for S_j falls well between these two boundaries and the inter-module joint can therefore be considered semi-rigid.



(**a**) (**b**)

Figure 108. Inter-module joint schemes: (**a**) Top view, (**b**) Cross-section.

J. Pile foundation design

Foundation load

$$F_{ed,ULS} = \gamma_G * G_k + \gamma_{Q;1} * Q_{1;k} * + \sum \gamma_{Q;i} * \Psi_{0;i} * Q_{i;k}$$

Table 65. Calculation of foundation load.

Fu.C.	Design 2	Q1 = Wind			
	SLS Load	Load factor	Ψ	ULS Load	Unit
G,total	182	1,2		218	kN
Q,1,wind	70	1,5	1	105	kN
Q2,res	39	1,5	0,4	24	kN
Total load				391	kN

$$F_{ed} = (218 + 24) * 8/4 + 105 = 589\ kN$$

Strong ground pile capacity

The resistance of the soil profile is calculated based on the Koppejan method (Van Tol, 2006). A standard soil profile in The Hague, the Netherlands, has been used for this calculation. This soil profile can be found in the next appendix.

Table 66. Foundation pile properties.

l_{pile}	12	m
D_{round}	0,25	m
D_{eq}	0,28	m
A_{pile}	0,049	m^2

Tip of the pile resistance

Three values for q_c are calculated to determine the resistance of the tip of the pile.

- $q_{c;I;avg}$ is the lowest average resistance in the trajectory from the level of the tip of the pile up to a minimum depth of 0,7*D_{eq} and maximum depth of 4*D_{eq} below the pile.
- $q_{c;II;avg}$ is the average resistance from the bottom of $q_{c;I}$ up to the pile tip level.
- $q_{c;III;avg}$ is the average resistance from pile tip level and a level that is 8*D_{eq} above.

$$p_{r;max;tip} = \alpha_p * \beta * s * \frac{1}{2} * (\frac{1}{2} * (q_{c;I;avg} + \frac{1}{2} * q_{c;II;avg}) + q_{c;III;avg})$$

Based on the standard soil profile, the values for q_c are as follows:

- $q_{c;I;avg} = 16\ MPa$
- $q_{c;II;avg} = 15\ MPa$
- $q_{c;III;avg} = 15\ MPa$

α_p = Factor for pile class; 1.0 for driven piles
β = Factor for foot of pile; 1.0 for smooth piles
S = Factor for pile shape; 1.0 for square piles
$p_{r;max;tip} = 15{,}3\ MPa$
$F_{r;max;tip} = A_{tip} * p_{r;max;tip} = 0{,}06 * 15{,}3 = 0{,}95\ MN$

Shaft resistance

$p_{r;max;shaft} = \alpha_s * \min(q_c; 15)$
α_s= 0,01 for prefab piles
q_c = average resistance between the level of the tip of the pile and 1 meter above.

$q_c = 15\ MPa$
$p_{r;max;shaf} = 0{,}01 * \min(15; 15) = 0{,}15$
$F_{r;max;shaft} = A_{shaft} * p_{r;max;shaft} = 1 * 0{,}15 = 0{,}15\ MN$

Load-bearing capacity pile

$$F_{r;d} = \xi * \frac{F_{r;max;tip} + F_{r;max;shaft}}{\gamma_{mb}} = \frac{0{,}75 * (F_{r;max;tip} + F_{r;max;shaft})}{1{,}25} = 662\ kN$$

Unity check

$$UC = \frac{F_{ed}}{F_{rd}} = \frac{589}{662} = 0{,}89$$

Longitudinal horizontal displacement

Overturning moment

$$M_{k,wind} = \frac{1}{2} * q_{c,wind} * b_{module} * (h_{storey} * n_{storeys})^2$$
$$M_{ed,wind} = * \frac{1}{2} * 0{,}96 * 3{,}5 * 3{,}1 * 8)^2 = 1035\ kNm$$

Pile plan properties

Since there are four columns along the length of the module, there will also be four piles along its length. The distance to the centre of rotation a_i is calculated for all piles and is used to find the axial force leading to the rotation φ.

$A_p = D_{eq}{}^2 = 280\ x\ 280\ mm^2 * 10^{-6} = 0{,}08\ m^2$
$L_{pile} = 12\ m$
a_i = Distance between center of pile to the rotation center of the pile group
$a_1 = 2{,}14\ m$
$a_2 = 5{,}14\ m$
I_{p} = Moment of inertia pile group in 1 direction
$I_{\mathrm{p}} = \sum a_i{}^2 = 2 * 2{,}14^2 + 2 * 5{,}14^2 = 62\ m^2$

Displacement calculation

P_{n} = axial force in the pile due to M_{wind}
$P_{\mathrm{n}} = \frac{\mathrm{M} * a_i}{I_{\mathrm{p}}} = \frac{1035 * 5{,}14}{6232} = 85{,}8\ kN$
$\Delta \mathrm{l} = \frac{P_{\mathrm{n}} * \mathrm{L}}{D^2 * \mathrm{E}} = \frac{85800 * 12000}{280^2 * 20000} = 0{,}00068\ mm$
$\varphi = \frac{\Delta \mathrm{l}}{a_{max}} = \frac{0{,}00068}{5{,}14} = 0{,}00012\ mm/mm$
$u_{top} = \varphi * \mathrm{h}_{building} = 0{,}00012 \frac{\mathrm{mm}}{\mathrm{mm}} * 24800\ \mathrm{mm} = 2{,}88\ \mathrm{mm}$

Weak ground pile capacity

The same calculation method is used to calculate the rotation and displacement of a foundation on a weaker soil, in which it is necessary to use longer piles. The tip of the pile and shaft resistances are calculated based on a 19 m long pile, again using the given soil profile in The Hague. The dimensions of the pile are based on the required pile resistance.

Table 67. Foundation pile properties.

l_{pile}	19	m
D_{round}	0,28	m
D_{eq}	0,28	m
A_{pile}	0,049	m^2

$A_p = {D_{eq}}^2 = 320\ x\ 320\ mm^2 * 10^{-6} = 0{,}10\ m^2$

Displacement calculation

P_n = axial force in the pile due to M_{wind}

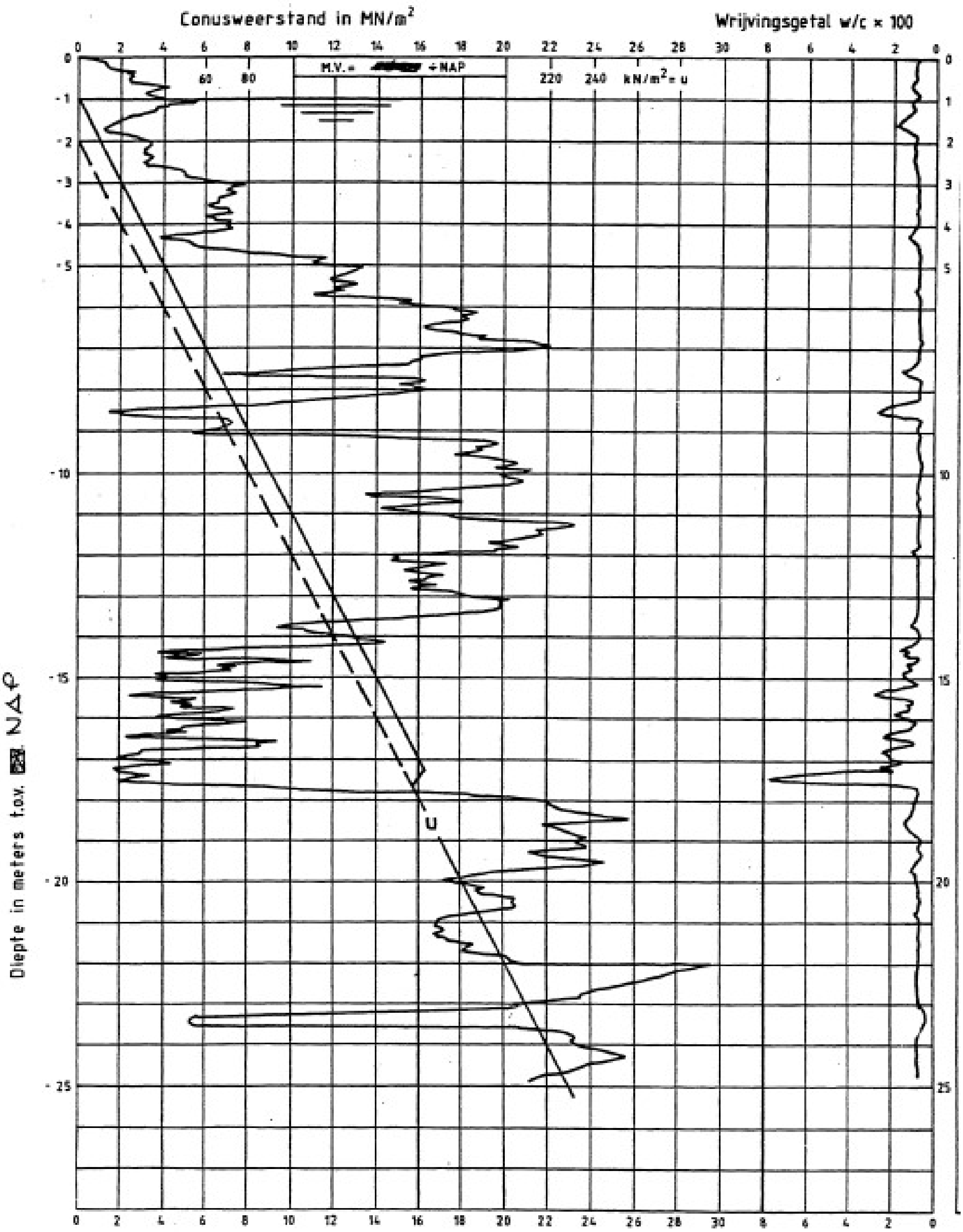
$P_n = \frac{M * a_i}{I_p} = \frac{1035 * 5{,}14}{6232} = 85{,}8\ kN$

$\Delta l = \frac{P_n * L}{D^2 * E} = \frac{85800 * 19000}{280^2 * 20000} = 0{,}00068\ mm$

$\varphi = \frac{\Delta l}{a_{max}} = \frac{0{,}00086}{5{,}14} = 0{,}00015\ mm/m$

$u_h = \varphi * h = 0{,}00015 * 24800 = 3{,}66\ mm$

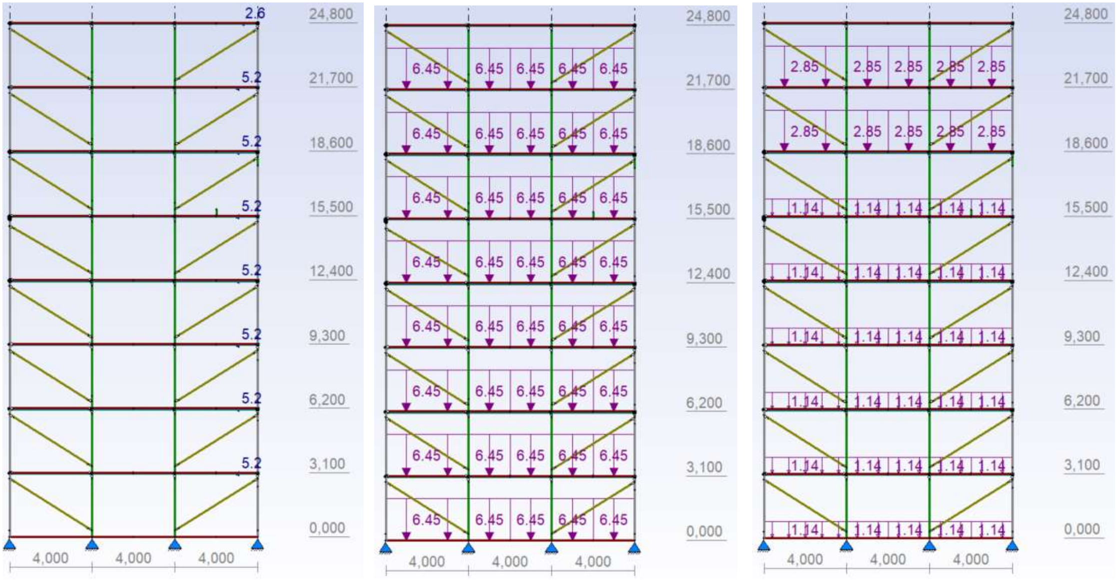
H7.10 Soil profile



Sondering te Den Haag 1981

Plein

K. Variant loads on braced frame

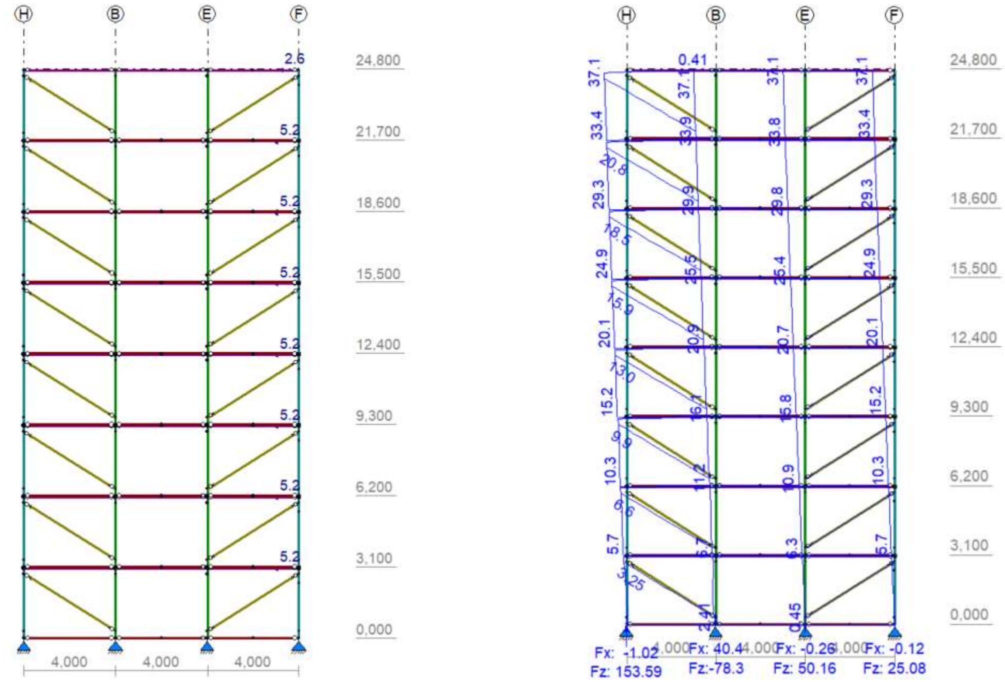


(**a**) (**b**) (**c**)

Figure 109. Variant model loads: (**a**) Wind load, (**b**) Permanent load, (**c**) Variable load.

L. Longitudinal Variant geometry and horizontal displacement

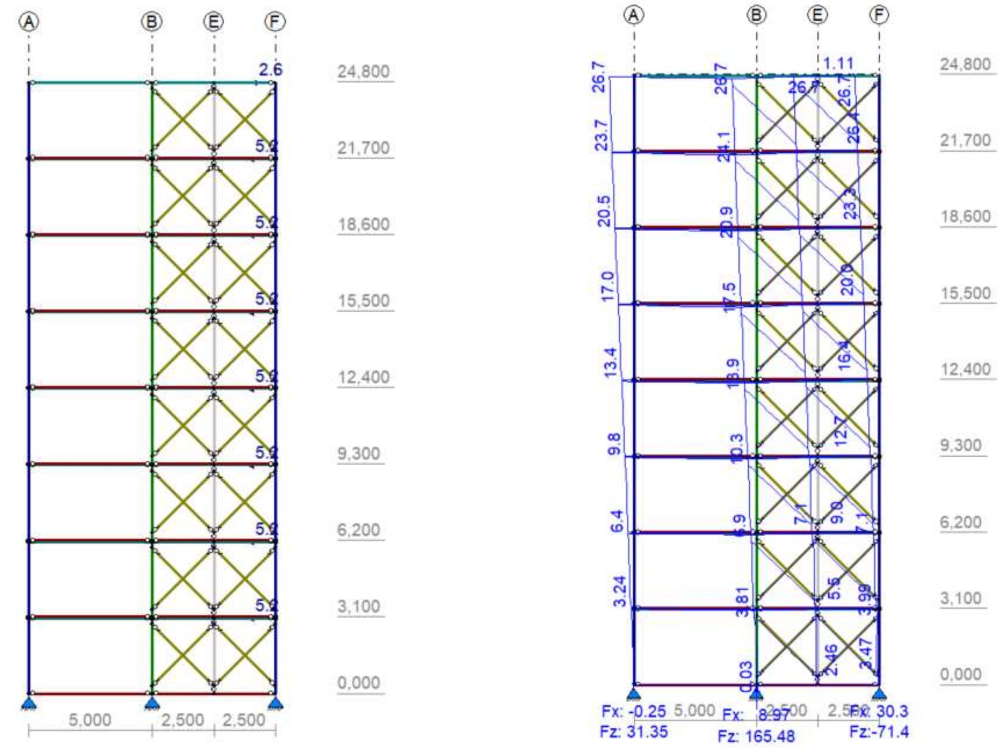
Variant S-O-S



(**a**) (**b**)

Figure 110. Variant 1 S-O-S: (**a**) Geometry and wind load, (**b**) Horizontal displacement on governing load combination.

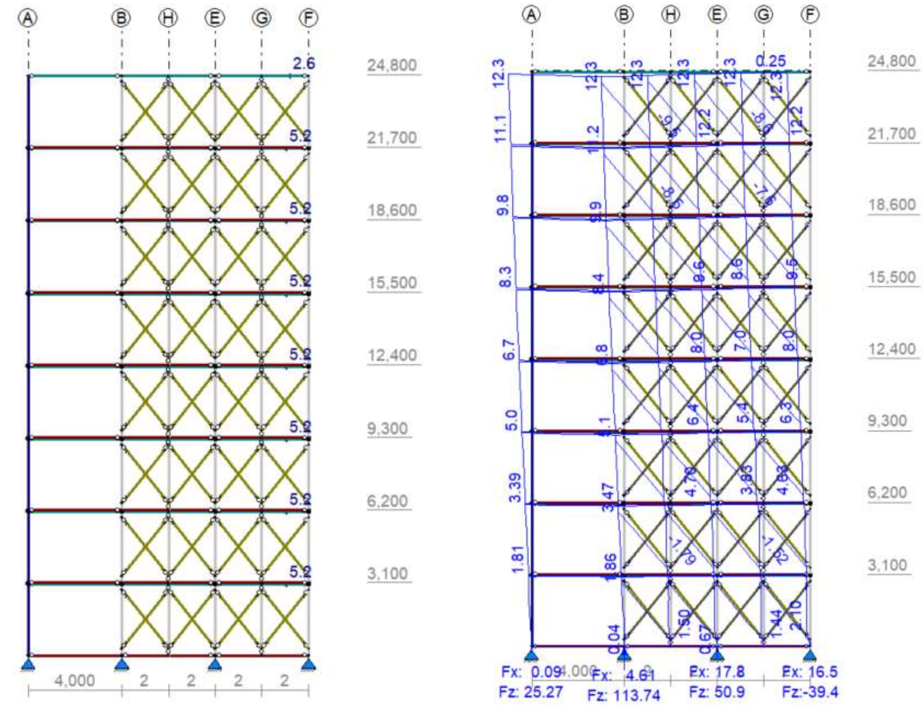
Variant O-F



(**a**) (**b**)

Figure 111. Variant 2 O-F: (**a**) Geometry and wind load, (**b**) Horizontal displacement on governing load combination.

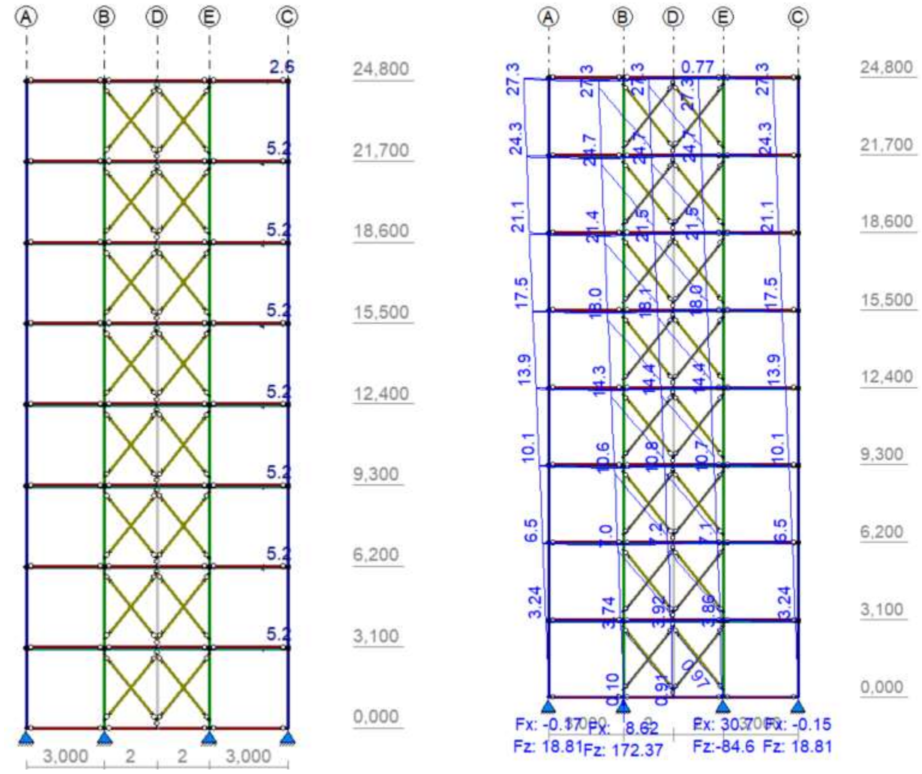
Variant O-F-F



(**a**) (**b**)

Figure 112. Variant 3 O-F-F: (**a**) Geometry and wind load, (**b**) Horizontal displacement on governing load combination.

Variant O-F-O

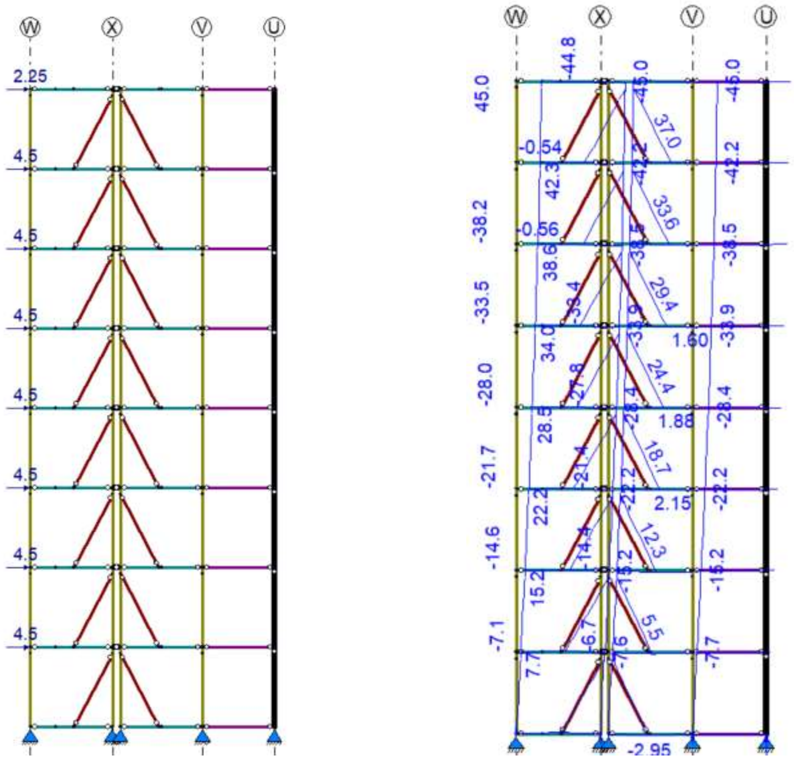


(**a**) (**b**)

Figure 113. Variant 4 O-F-O: (**a**) Geometry and wind load, (**b**) Horizontal displacement on governing load combination.

M.Transverse Variant geometry and horizontal displacement

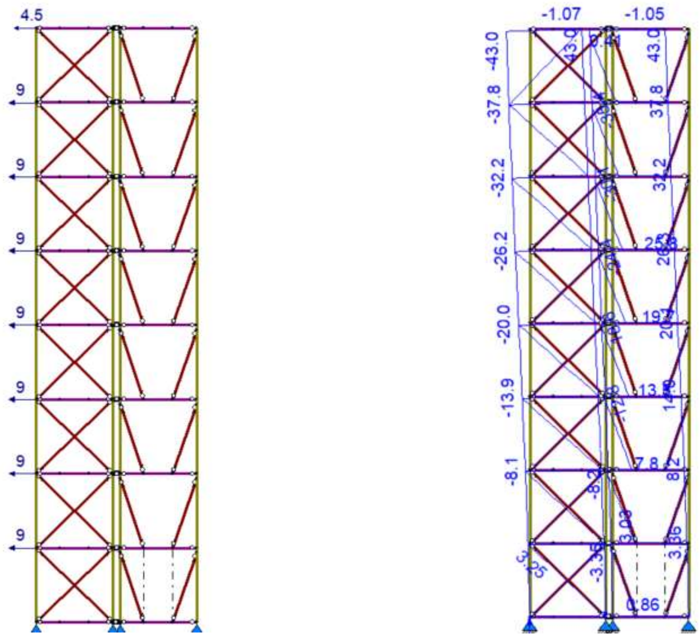
Variant 1.1 Box profiles



(**a**) (**b**)

Figure 114. Variant 1.1 Box profiles: (**a**) Geometry and wind load, (**b**) Horizontal displacement for governing load combination.

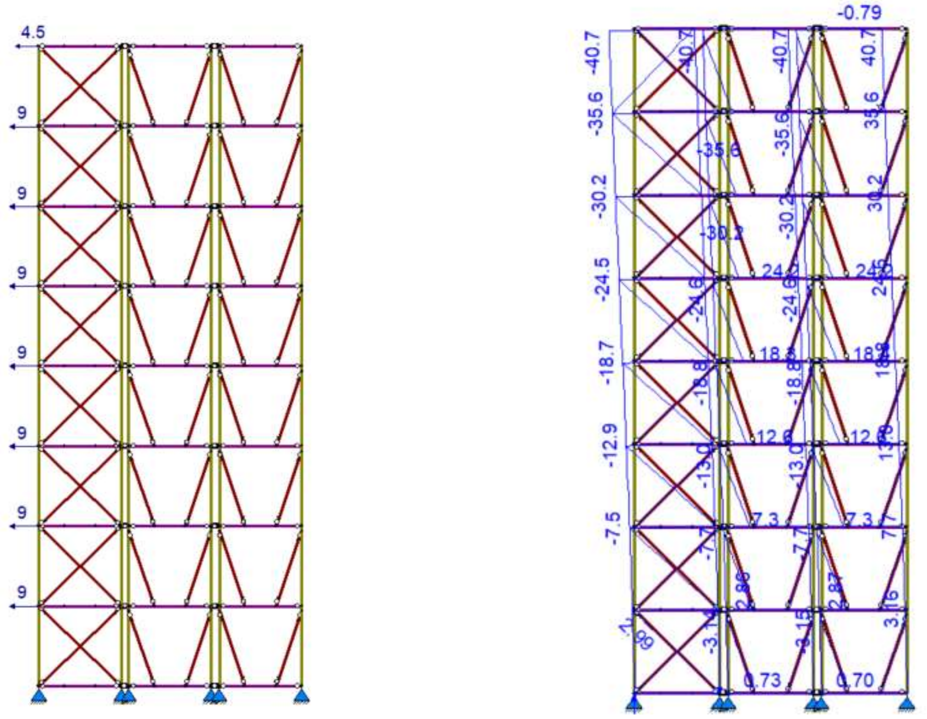
Variant 2.1 Strips



(**a**) (**b**)

Figure 115. Variant 2.1 Strips: (**a**) Geometry and wind load, (**b**) Horizontal displacement for governing load combination.

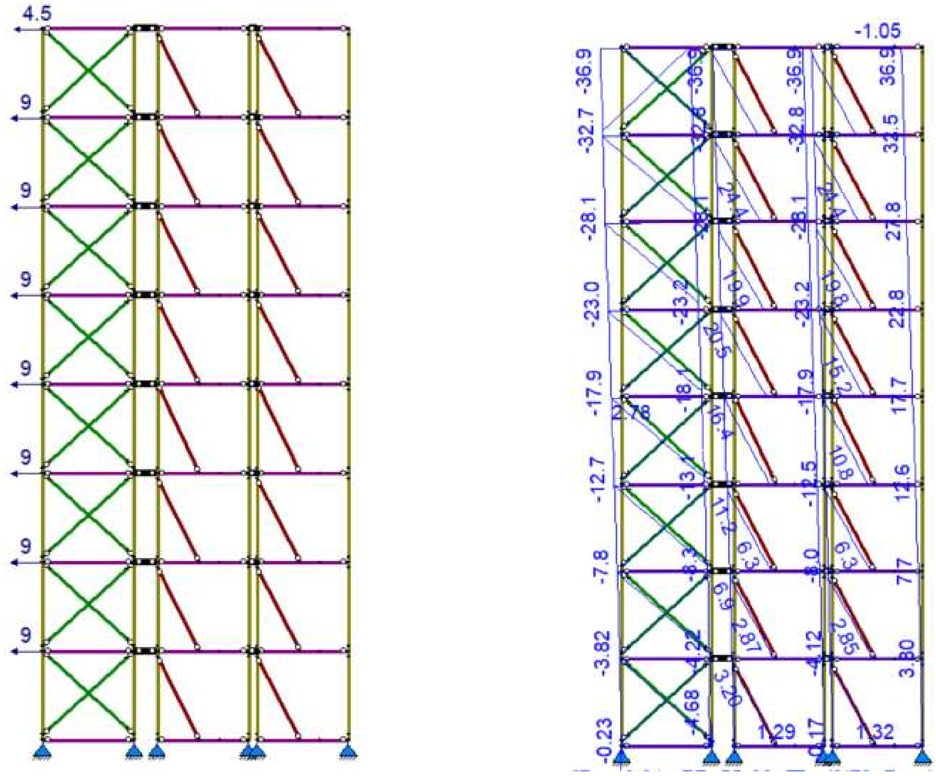
Variant 2.2 Strips



(**a**) (**b**)

Figure 116. Variant 2.2 Strips: (**a**) Geometry and wind load, (**b**) Horizontal displacement for governing load combination.

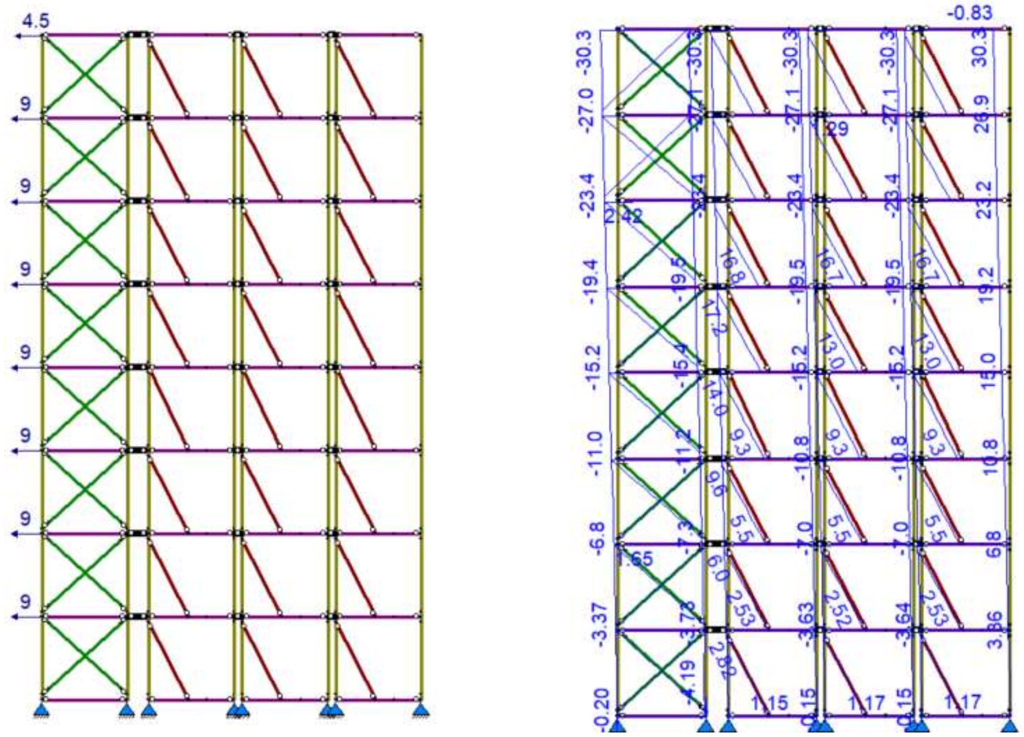
Variant 3.1 Mix



(**a**) (**b**)

Figure 117. Variant 3.1 Mix: (**a**) Geometry and wind load, (**b**) Horizontal displacement for governing load combination.

Variant 3.2 Mix



(**a**) (**b**)

Figure 118. Variant 3.2 Mix: (**a**) Geometry and wind load, (**b**) Horizontal displacement for governing load combination.

N. Excel Variant S-O-S calculations

Tension check, Building properties, Element sizes

Longitudinal tension check			**Longitudinal**			**Transverse**		
			Loaded length	4,00	m	*Loaded length*	2,00	m
q,wind,combi	0,96	kN/m2	F,k,Slab	21,00	kN	F,k,Slab	10,50	kN
M per brace, 8 floors	517,44	kNm	*F,var*	*4,50*	*kN*	*F,var*	*2,25*	*kN*
F,k,tension	129,36	kN	F,k,ceiling	2,69	kN	F,k,ceiling	1,34	kN
2nd order effect	1,15		F,k, long wall	3,20	kN	F,k, long wall	1,60	kN
F,ed,tension	223,15	kN	F,k, short wall			F,k, short wall	1,28	
Longitudinal column compression	-11,17	kN	F,k,column	0,65	kN	F,k,column	0,45	kN
			Corridor volume			Corridor volume		
			F,k,corridor			F,k,corridor		
H,k	5,22	kN	F,k,floor beam	0,51	kN	F,k,floor beam	0,25	kN
H,transverse	17,88	kN	Column compression force	234,32	kN	Column compression force	127,30	kN

Building Properties		
Number of floors	8	
Braced edge span	4,00	m
Unbraced internal span	4,00	m
Module length	12,00	m
Loaded module width	3,22	m
External width	3,50	m
Free floor height	2,60	m
Floor height	3,10	m
Total height	24,80	m
Internal columns		
Diagonal length	5,06	m
Door width	0,70	m
Window width	2,00	m
Element Sizes		
Weight longitudinal		
q,slab	5,25	kN/m
q,wall	0,80	kN/m
q,roof	0,67	kN/m
q,var	1,13	kN/m

Element Sizes					
Middle Steel column			**Edge Steel column**		
B	0,12	m	B	0,12	m
H	0,12	m	H	0,12	m
t	0,006	m	t	0,004	m
A	0,002664	m2	A	0,00184	m2
I,yy	5942592	mm4	I,yy	4167339	mm4
z	0,057	m	z	0,058	m
Longitudinal bracing					
Bracings/module	**2,00**				
t	0,010	m			
h	0,15	m	**Frame Ceiling joist**		
A	0,0015	m2	B	0,05	m
Frame Wall stud			H	0,14	m
B	0,03	m	t,f	0,0020	m
H	0,1	m	t,w	0,0014	m
t	0,0012	m	A	0,000388	m2
t,w	0,0012	m	I,yy	1437400	mm4
A	0,000189	m2	z	0,07	m
I,yy	280009	mm4	**Lower Edge beam - C**		
z	0,05	m	b	0,1	m
Frame edge beam - RHS			h	0,2	m
B	0,03	m	t	0,004	m
H	0,1	m	A	0,001600	m2
t	0,0012	m	I,yy	10360363	mm4
A	0,000309	m2	z	0,1	m

Partition structures

Wall buildup	Centres between studs		0,40	m	
	Thickness (m)	Height (m)	Vol.weight (kN/m3)	Weight kN/m length	
Plasterboard	0,030	2,600	5,00	0,39	
Mineral wool	0,100	2,600	0,45	0,12	
	Area (m2)	*Height (m)*	*Number/m*		
C-studs	0,00019	2,600	2,50	0,10	
Plasterboard	0,015	2,600	5,00	0,20	
Total	0,15			0,80	
Ceiling buildup	Centres between joists		0,60	m	
	Thickness (m)	Span (m)	Vol.weight (kN/m3)	Weight kN/m length	
Plasterboard	0,030	3,500	5,00	0,26	
Mineral wool	0,120	*3,500*	0,45	0,19	
Plasterboard	0,015	3,500	5,00	0,13	
	Area (m2)	*Span (m)*	*Number/m*		
C-studs	0,00039	3,500	1,67	0,0894	Equal to wall
Total				0,67	
Balcony					
	Thickness (m)	Cantilever (m)	Vol.weight (kN/m3)	Weight kN/m length wall	
Concrete slab	0,200	1,500	25,00	7,50	
Floor					
	Thickness (m)	Span (m)	Vol.weight (kN/m3)	Weight kN/m length wall	
Concrete slab	0,120	3,500	25,00	5,25	
Cover layer	0,00	3,50	20,00	0,00	
End Facade open (V)					
	Thickness (m)	Height (m)	Vol.weight (kN/m3)	Weight kN/m length wall	
Glass	0,010	*2,800*	25,00	0,70	
End Facade closed (V)					
	Thickness (m)	Height (m)	Vol.weight (kN/m3)	Weight kN/m length wall	
Mineral wool	0,100	*2,600*	0,45	0,12	
Weight from wall				0,80	
Total				0,92	

Element verification

Lower edge beam			
Properties			
A		0,00160	m2
E		210000000	kN/m2
I		0,000010	m4
S235		250,00	N/mm2
SLS Load			
G,slab		5,25	kN/m
G,selfweight		0,13	kN/m
G,wall		0,40	kN/m
q,G,sls		5,78	kN/m
q,Q,sls		2,81	kN/m
q,uls,ed		**11,15**	kN/m
q,sls,k		8,59	
Deflection SLS			
span		4,00	m
w,ed		13,16	mm
UC	✓	0,82	-
Bending Stress ULS			
Sigma,bending		215,26	N/mm2
UC Stress	✓	0,86	

Upper edge beam			
Properties			
A		0,00104	m2
E		210000000	kN/m2
I		0,000004	m4
S235		250,00	N/mm2
SLS Load			
G,selfweight		0,13	kN/m
G,weight ceiling joists		0,08	kN/m
G,wall		0,40	kN/m
G,roof		1,08	kN/m
q,G,sls		1,69	kN/m
q,Q,sls		1,61	kN/m
q,uls,ed		4,44	kN/m
Deflection SLS			
span		4,00	m
w,ed		13,33	mm
UC	✓	0,83	-
Bending Stress ULS			
Sigma,bending		180,85	N/mm2
UC Stress	✓	0,72	

Middle Column			
Properties			
A		0,00266	m2
E		210000000	kN/m2
I,c-section		0,0000059	m4
S235		250,00	N/mm2
SLS Load			
q,G,sls		6,97	kN/m
q,Q,sls		2,81	kN/m
G,selfweight		0,21	kN/m
Loaded width		4,00	m
F,sls,k		318,41	kN
F,k,storey during wind		33,05	kN
F,sls,k,storey		39,80	kN
ULS Load	Wind. Gov		
F,wind,uls,ed		129,36	kN
q,uls,ed		8,62	kN/m
F,self,uls		6,26	kN
F,uls,ed		411,43	kN
Verifications			
ULS Load	Residential		
F,wind,uls,ed		0,00	kN
q,uls,ed		**11,15**	kN/m
F,self,uls		6,26	kN/m
F,uls,ed		363,09	kN
Lk		**2,80**	m
Gov. Normal force		411,43	kN
Buckling force		1570,917	kN
UC Buckling		0,26	
Normal stress		154,44	N/mm2
UC Stress	✓	0,72	
N,b,Rd		446	kN
UC Normal force		0,92	

Edge Column			
Properties			
A		0,00184	m2
E		210000000,00	kN/m2
I,c-section		0,00000417	m4
S235		250,00	N/mm2
SLS Load			
q,G,sls		6,97	kN/m
q,Q,sls		2,81	kN/m
G,selfweight		0,15	kN/m
q,G,side wall,sls		1,23	kN
Loaded width		2,00	m
F,sls,k		161,42	kN
F,storey during wind		16,80	kN
F,sls,k,storey		20,18	kN
ULS Load	Wind. Gov		
F,wind,uls,ed		129,36	kN
q,uls,ed		8,62	kN/m
F,self,uls		4,33	kN
F,uls,ed		271,59	kN
Verifications			
Lk		2,80	m
Normal force		271,59	kN
Buckling force		1101,63	kN
UC Buckling		0,25	
Normal stress		147,60	N/mm2
UC Stress	✓	0,59	
N,b,Rd		290	kN
UC Normal force		0,94	

O. Maple Derivations

Deflection at length 'a' for a beam loaded by force F

```
restart;
#F:=40:
#EI:=16650:L:=3.5: a:=1:

ODE1 := EI·diff(w1(x), x$4) = 0 :
ODE2 := EI·diff(w2(x), x$4) = 0 :
sol := dsolve({ODE1, ODE2}, {w1(x), w2(x)}) : assign(sol) :
w1 := w1(x); w2 := w2(x);
```

$$w1 := \frac{1}{6}\_C5\,x^3 + \frac{1}{2}\_C6\,x^2 + \_C7\,x + \_C8$$

$$w2 := \frac{1}{6}\_C1\,x^3 + \frac{1}{2}\_C2\,x^2 + \_C3\,x + \_C4$$

```
phi1 := -diff(w1, x) : kappa1 := diff(phi1, x) : M1 := EI·kappa1 : V1 := diff(M1, x) :
phi2 := -diff(w2, x) : kappa2 := diff(phi2, x) : M2 := EI·kappa2 : V2 := diff(M2, x) :

x := 0 : eq1 := w1 = 0 : eq2 := M1 = 0 :
x := a : eq3 := M1 = M2 : eq4 := phi1 = phi2 : eq5 := w1 = w2 : eq6 := V1 − F − V2 = 0 :
x := L : eq7 := w2 = 0 : eq8 := M2 = 0 :
sol2 := solve({eq1, eq2, eq3, eq4, eq5, eq6, eq7, eq8}, {_C1, _C2, _C3, _C4, _C5, _C6, _C7, _C8}) : assign(sol2) :
AKE X A PARAMETER
x := a :

w1;
```

$$-\frac{F\,(L-a)\,a^3}{6\,EI\,L} + \frac{F\,a^2\,(2\,L^2 - 3\,L\,a + a^2)}{6\,EI\,L}$$

Deflection at length 'a' for a beam loaded by force q

```
restart;
#q:=6.3:a:=1:EI:=13500:L:=3.5 :

ODE1 := EI·diff(w1(x), x$4) = q :
w1 := rhs(dsolve(ODE1, w1(x)));
```

$$w1 := \frac{q\,x^4}{24\,EI} + \frac{\_C1\,x^3}{6} + \frac{\_C2\,x^2}{2} + \_C3\,x + \_C4$$

```
phi1 := -diff(w1, x) : kappa1 := diff(phi1, x) : M1 := EI·kappa1 : V1 := diff(M1, x) :

x := 0 : eq1 := w1 = 0 : eq2 := M1 = 0 :
x := L : eq3 := w1 = 0 : eq4 := M1 = 0 :
sol2 := solve({eq1, eq2, eq3, eq4}, {_C1, _C2, _C3, _C4}) : assign(sol2) :
AKE X A PARAMETER
x :='a':

w1;
```

$$\frac{q\,a^4}{24\,EI} - \frac{q\,L\,a^3}{12\,EI} + \frac{q\,L^3\,a}{24\,EI}$$

Fixed column node displacement

```
> restart; with(plots) :
  #EI:=700: l1:=1:l2:=2:F:=300:
> ODE1 := EI·diff(w1(x), x$4) = 0 :
> ODE2 := EI·diff(w2(x), x$4) = 0 :
> sol := dsolve({ODE1, ODE2}, {w1(x), w2(x)}) : assign(sol) :
> w1 := w1(x) : w2 := w2(x) :
  phi1 := -diff(w1, x) : kappa1 := diff(phi1, x) : M1 := EI·kappa1 : V1 := diff(M1, x) :
> phi2 := -diff(w2, x) : kappa2 := diff(phi2, x) : M2 := EI·kappa2 : V2 := diff(M2, x) :
> x := 0 : eq1 := w1 = 0 : eq2 := M1 = 0 :
> x := l1 : eq3 := w1 = w2 : eq4 := V1-F = V2 : eq5 := M1 = M2 : eq6 := phi1 = phi2 :
> x := l1 + l2 : eq7 := M2 = 0 : eq8 := w2 = 0 :
> sol2 := solve({eq1, eq2, eq3, eq4, eq5, eq6, eq7, eq8}, {_C1, _C2, _C3, _C4, _C5, _C6, _C7, _C8}) : assign(sol2) :

> x :='l1':
  w1;
  x :='l1 + l2':
```

$$-\frac{F\, l2\, l1^3}{6\, EI\, (l1 + l2)} + \frac{F\, l1^2\, l2\, (l1 + 2\, l2)}{6\, EI\, (l1 + l2)}$$

```
>
```

Unsupported column node displacement

```
> restart; with(plots) :
  #EI:=700: l1:=1:l2:=2:F:=300:
> ODE1 := EI·diff(w1(x), x$4) = 0 :
> ODE2 := EI·diff(w2(x), x$4) = 0 :
> sol := dsolve({ODE1, ODE2}, {w1(x), w2(x)}) : assign(sol) :
> w1 := w1(x) : w2 := w2(x) :
  phi1 := -diff(w1, x) : kappa1 := diff(phi1, x) : M1 := EI·kappa1 : V1 := diff(M1, x) :
> phi2 := -diff(w2, x) : kappa2 := diff(phi2, x) : M2 := EI·kappa2 : V2 := diff(M2, x) :
> x := 0 : eq1 := w1 = 0 : eq2 := M1 = 0 :
> x := l1 : eq3 := w1 = w2 : eq4 := V1-F = V2 : eq5 := M1 = M2 : eq6 := phi1 = phi2 :
> x := l1 + l2 : eq7 := M2 = 0 : eq8 := w2 = b :
> sol2 := solve({eq1, eq2, eq3, eq4, eq5, eq6, eq7, eq8}, {_C1, _C2, _C3, _C4, _C5, _C6, _C7, _C8}) : assign(sol2) :

> x :='l1':
  w1;
  x :='l1 + l2':
```

$$-\frac{F\, l2\, l1^3}{6\, EI\, (l1 + l2)} + \frac{(F\, l1^2\, l2 + 2\, F\, l1\, l2^2 + 6\, EI\, b)\, l1}{6\, EI\, (l1 + l2)}$$

Internal column node displacement

```
> restart; with(plots) :
  #EI:=700: l1:=0.3:l2:=1.7: l3:=1: F:=45:a:=0.0055:b:=0:
> ODE1 := EI·diff(w1(x), x$4) = 0 :
> ODE2 := EI·diff(w2(x), x$4) = 0 :
> ODE3 := EI·diff(w3(x), x$4) = 0 :
  sol := dsolve({ODE1, ODE2, ODE3}, {w1(x), w2(x), w3(x)}) : assign(sol) :
> w1 := w1(x) : w2 := w2(x) : w3 := w3(x) :
  phi1 := -diff(w1, x) : kappa1 := diff(phi1, x) : M1 := EI·kappa1 : V1 := diff(M1, x) :
> phi2 := -diff(w2, x) : kappa2 := diff(phi2, x) : M2 := EI·kappa2 : V2 := diff(M2, x) :
  phi3 := -diff(w3, x) : kappa3 := diff(phi3, x) : M3 := EI·kappa3 : V3 := diff(M3, x) :
> x := 0 : eq1 := w1 = 0 : eq2 := M1 = 0 :
> x := l1 : eq3 := w1 = w2 : eq4 := V1 - F = V2 : eq5 := M1 = M2 : eq6 := phi1 = phi2 :
  x := l1 + l2 : eq7 := w2 = a : eq8 := w2 = w3 : eq9 := V2 + F = V3 : eq10 := M2 = M3 :
> x := l1 + l2 + l3 : eq11 := M3 = 0 : eq12 := w3 = b :
> sol2 := solve({eq1, eq2, eq3, eq4, eq5, eq6, eq7, eq8, eq9, eq10, eq11, eq12}, {_C1, _C2, _C3, _C4, _C5, _C6, _C7, _C8, _C9, _C10, _C11, _C12}) : assign(sol2) :
> x :='l1':
  w1;
```

$$-\frac{F\, l2\, l1^3}{6\, EI\, (l1 + l2 + l3)} + \frac{(F\, l1^3\, l2 + 3\, F\, l1^2\, l2^2 + 2\, F\, l1\, l2^3 - F\, l2^3\, l3 + 6\, EI\, a\, l1 + 6\, EI\, a\, l2 + 6\, EI\, a\, l3)\, l1}{6\, EI\, (l1^2 + 2\, l1\, l2 + l1\, l3 + l2^2 + l2\, l3)}$$

P. Element strength verification

Material properties

Table 68. Material properties.

Concrete properties	Value	Unit
E,c30	30000000	kN/m^2
f,rd,c	16,00	N/mm^2
Volumetric weight (reinforced)	25,00	kN/m^3
Steel properties		
E,st	210000000	kN/m^2
f,rd,uls (S355)	322,73	N/mm^2
Volumetric weight	79,00	kN/m^3

Load factors

Table 69. Load factors.

		Load factor γ				
	Load (kN/m^2)	Design 1	Design 2	Ψ0	Ψ1	Ψ2
Variable loads						
Residential	1,75	0	1,5	0,4	0,5	0,3
Corridor	2	0	1,5	0,4	0,5	0,3
Snow	0,56	0	1,5	0	0,2	0
Wind	*varies*	0	1,5	0	0,2	0
Permanent load		1,35	1,2			

Load combinations SLS,ULS

- Fu.C.1 Permanent load governing
- Fu.C.2 Residential load governing
- Fu.C.3 Wind load governing

The resulting force is the load in kN/m that acts on the floor beam. The column load can be found by multiplying this load by the loaded span. Current values of the S-O-S variant are used in Table 70 below.

Table 70. Load combinations (Variant S-O-S Example).

Fu.C.1	Design 1	G Only			
	SLS Load	Load factor		ULS Load	Unit
G,total		1,35		9.29	kN/m
Q,i		0		0	kN
Total load				9.29	kN/m
Fu.C.2	**Design 2**	**Q1 Res**			
	SLS Load	Load factor	Ψ	ULS Load	Unit
G,total		1,2		8.26	kN/m
Q,1,res		1,5	1	4.22	kN/m
Q,2,snow		1,5	0		
Q,2,wind		1,5	0		
Total load				12.48	kN/m
Fu.C.3	**Design 2**	**Q1 Wind**			
	SLS Load	Load factor	Ψ	ULS Load	Unit
G,total		1,2		8.26	kN/m
Q,1,wind		1,5	1	0	kN
Q2,res		1,5	0,4	1.69	kN/m
Q2,snow		1,5	0		
Total load				9.95	kN/m

Floor slab

Element properties

Table 71. Reinforced concrete slab properties.

Slab		
$f_{rd,c}$	16	N/mm^2
h_{slab}	120	mm
$q_{uls,ed}$	6,23	kN/m
Reinforcement		
$f_{rd,s}$	323	N/mm^2
$c.t.c.(centres)$	200	mm
d_{bar}	10	mm
A_{bar}	79	mm^2
ρ_{bar}	0,44	%
$c_{nom} = d_{bar} + 10\ mm$	20	mm

Moment capacity

$d = h_{beam} - c_{nom} - d_{bar} = 90\ mm$

$$N_s = f_{rd,s} * A_{bar} * 10^{-3} = 27{,}9\ kN$$
$$x_u = \frac{N_s}{\alpha * centres * f_{rd,c}} = 11{,}6\ mm$$
$$M_{rd} = N_s * \frac{d - \beta * x_u}{centres} = 11{,}9\ kNm$$
$$M_{ed} = \max\left(\frac{1}{8} * q_{uls,Ed} * l_{slab}{}^2 ; \frac{1}{4} * Q_{uls} * l_{slab}\right) = 8{,}04\ kNm$$
$$UC = \frac{M_{Ed}}{M_{Rd}} = 0{,}68$$

Shear capacity

$$f_{ck} = 30\ N/mm2$$
$$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{\frac{1}{2}} = 0{,}54$$
$$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 = 2{,}0$$

$$V_{Rd,c} = \frac{C_{Rd,c}}{\gamma_c} * (100 * \rho_l * f_{ck})^{\frac{1}{3}} * b_w * d = 0{,}57\ N/mm2$$
$$V_{Ed,c} = \frac{1}{2} * q_{uls,Ed} * \frac{l_{slab}}{d} = 0{,}12\ N/mm2$$
$$UC = \frac{V_{Ed,c}}{V_{Rd,c}} = 0{,}20$$

Lower C-section edge beam

Element properties

- $b = 70\ mm$
- $h = 250\ mm$
- $t = 5\ mm$
- $A = 1950\ mm^2$
- $I_{yy} = 1{,}7 * 10^7\ mm^4$
- $l_{span} = 4\ m$
- $E = 210{,}000\ N/mm^2$

Load

$$q_{k,SLS} = G_k + Q_{res}$$
$$q_{ed,ULS} = \gamma_G * G_k + \gamma_{Q;1} * Q_{res}$$
$$\gamma_G = 1.2$$
$$\gamma_Q = 1.5$$

$$G_k = q_{slab} + q_{wall} + q_{self} = 5.80\ kN/m$$
$$q_{slab} = 5.25\ kN/m$$
$$q_{wall} = 0.40\ kN/m$$
$$q_{self} = 0.15\ kN/m$$

$$Q_{res} = \frac{q_{res} * w_{loaded}}{2} = 2{,}81\ kN/m$$
$$q_{res} = 1.75\ kN/m^2$$
$$w_{loaded} = 3.2\ m$$

$$q_{ed,SLS} = 5.80 + 2.81 = 8.61\ kN/m$$

$q_{ed,ULS} = 1.2 * 5.80 + 1.5 * 2.81 = 11.18\ kN/m$

Deflection

$$u_{Ed} = \frac{5}{384} * \frac{q_{k,SLS} * l^4}{E * I}$$

$$u_{Rd} = \frac{l_{span}}{250}$$

$$UC = \frac{u_{Ed}}{u_{Rd}} = \frac{7.84}{16.0} = 0.49$$

Stress

$$M_y = 1/8 * q_{ed,ULS} * l_{span}{}^2$$

$$\sigma_{y,ed} = \frac{M_y}{W_y}$$

$$\sigma_{rd} = \frac{f_{m;k}}{\gamma_m} = \frac{235}{1.1} = 214\ N/mm^2$$

$$UC = \frac{\sigma_{Ed}}{\sigma_{Rd}} = \frac{178}{214} = 0.84$$

Concrete edge beam

Element properties

Table 72. Reinforced concrete edge beam properties.

Beam		
$f_{rd,c}$	16	N/mm^2
h_{beam}	320	mm
d_{beam}	286	mm
$q_{uls,ed}$	8.8	kN/m
l_{beam}	4.0	m
Reinforcement		
$f_{rd,s}$	323	N/mm^2
$c.t.c.$	30	mm
d_{bar}	12	mm
A_{bar}	113	mm^2
ρ_{bar}	1,3	%
$c_{nom} = d_{bar} + 10\ mm$	22	mm

Moment capacity

$$d = h_{beam} - c_{nom} - d_{bar} = 286\ mm$$

$$N_s = f_{rd,s} * A_{bar} * 10^{-3} = 36.5\ kN$$

$$x_u = \frac{N_s}{\alpha * centres * f_{rd,c}} = 101\ mm$$

$$M_{rd} = N_s * (d - \beta * x_u) * n_{bars} = 27.0\ kNm$$

$$M_{ed} = \max\left(\frac{1}{8} * q_{uls,Ed} * l_{beam}{}^2; \frac{1}{4} * Q_{uls} * l_{beam}\right) = 24.2\ kNm$$

$$UC = \frac{M_{Ed}}{M_{Rd}} = 0.90$$

Shear capacity

$$f_{ck} = 30\ N/mm2$$

$$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{\frac{1}{2}} = 0.54$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 = 2.0$$

$$V_{Rd,c} = \frac{C_{Rd,c}}{\gamma_c} * (100 * \rho_l * f_{ck})^{\frac{1}{3}} * b_w * d = 0.83\ N/mm2$$

$$V_{Ed,c} = \frac{1}{2} * q_{uls,Ed} * \frac{l_{slab}}{d} = 0.08\ N/mm2$$

$$UC = \frac{V_{Ed,c}}{V_{Rd,c}} = 0.10$$

Column

Values for S-O-S variant are used.

Element properties

- *RHS section, cold formed*
- $b = 120\ mm$
- $h = 120\ mm$
- $t = 6\ mm$
- $A = 2664\ mm^2$
- $I_{yy} = 5{,}9 * 10^6\ mm^4$
- $z = 57\ mm$
- $E = 210{,}000\ N/mm^2$
- $\sigma_y = 355\ N/mm^2$
- $\alpha = 0.49$
- $L_k = 2.8\ m$

Cross-section Classification

$$r = 6\ mm$$

$$c = h - 2 * t_f - 2 * r = 96\ mm$$

$$\varepsilon_{S235} = 1{,}00$$

$$\frac{c}{t} = 16 \ \leq 33 * \varepsilon$$

The cross-section is therefore classified as class 1.
The formulas used are retrieved from the Design Manual for Steel Structures (Nijgh et al).

Flexural buckling resistance

$$N_{cr} \ = \pi^2 * \frac{EI}{L_k{}^2}$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}}$$

$$\Phi = 0{,}5 * (1 + \alpha * (\lambda - 0.2) + \lambda^2)$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}}$$

$N_{b,Rd} = \frac{\chi * A * f_y}{\gamma_{M1}}$
$N_{cr} = 1571\ kN$
$\lambda = 0.63$
$\chi = 0.77$
$\Phi = 0.80$
$N_{b,Rd} = 480\ kN$

Design load

Load combination

$q_{ed,ULS} = \gamma_G * G_k + \gamma_{Q;1} * Q_{1;k} * + \sum \gamma_{Q;i} * \Psi_{0;i} * Q_{i;k}$
$\gamma_G = 1.2$
$\gamma_Q = 1.5$
$Q_{1;k} = Q_{wind}$
$Q_{2;k} = Q_{res}$
$\Psi_{res} = 0.4$

Wind load

$Q_{1;k} = Q_{wind}$
$F_{ed,wind} = \frac{\frac{1}{2} * q_{wind} * (h_{storey} * n_{storey})^2}{l_{bracing}} * \frac{w_{module}}{2} * 1{,}10 * \gamma_{Q;1}$
$q_{wind} = 0.96\ kN/m^2$
$h_{storey} = 3.1\ m$
$w_{module} = 3.5\ m$
$l_{bracing} = 4\ m$
$n_{storeys} = 8$
$F_{ed,wind} = 213\ kN$

Permanent load

$G_k = q_{slab} + q_{wall} + q_{roof}$
$q_{slab} = 5.25\ kN/m$
$q_{wall} = 0.8\ kN/m$
$q_{roof} = 0.67\ kN/m$

Variable load

$Q_{res} = \frac{q_{res} * w_{loaded}}{2} = 2{,}81\ kN/m$
$q_{res} = 1.75\ kN/m^2$
$w_{loaded} = 3.2\ m$

$N_{Ed} = (1.2 * G_k + 1.5 * 0.4 * \gamma_1 * Q_{res}) * l_{loaded} * + F_{ed,wind}$
$l_{loaded} = 4\ m$

$N_{Ed} = 496\ kN$

Unity check

$\sigma_{ed} = \frac{N_{Ed}}{A} = 155\ N/mm^2$
$UC = \frac{\sigma_{ed}}{\sigma_{rd}} = 0.58$

$$UC = \frac{N_{Ed}}{N_{b,Rd}} = 0.82$$

Reduced cross-section resistance

One side is for 70% open to provide an access hole to the inter-module connection. The cross-section of the column is therefore reduced by 70% on one of the four sides.

$$A_{red} = (3 + 0.3) * (120 - 6) * 6 = 2257\ mm^2$$
$$N_{b,red,Rd} = \frac{\chi * A_{red} * f_y}{\gamma_{M1}} = 435\ kN$$
$$UC = \frac{N_{Ed}}{N_{b,red,Rd}} = 1.14$$

A thickness of 7 mm instead of 6 mm is required to satisfy the unity check.

Combined axial force and internal moment verification

The maximum moment M_{ed} is determined based on the maximum internal moments in the columns across the design variants in an 8 storey building.
The design values for normal force N_{ed} are calculated for the load combination in which the wind load is governing and in which the residential variable load is reduced, using ψ_{res} = 0,4.

$$N_{Ed} = (1.2 * G_k + 1.5 * 0.4 * \gamma_1 * Q_{res}) * l_{loaded} * + F_{ed,wind}$$

Using the geometry of each variant, the resulting normal force has been calculated in Excel. The acting moment M_{ed} is determined using the Technosoft model in which the lowest floor always has the largest moment. An example is shown in Figure 119 of variant 1 E-O-E.

Table 73. Values for calculation combined axial force and moment verification.

	Unit	Variant 1 S-O-S	Variant 2 O-F	Variant 3 O-F-F	Variant 4 O-F-O
f,k,s	N/ mm^2	355	355	355	355
A	mm^2	2664	2664	1824	2250
N,ed	kN	490	516	392	451
N,pl,ed	kN	946	946	648	799
M,ed	kNm	12	6,1	3,2	6,5
M,el,Rd	kNm	42	42	29	35
labda,y	-	0,74	0,74	0,56	0,78
Chi,y	-	0,70	0,70	0,81	0,68
C,my	-	0,95	0,95	0,95	0,95
C,mLT	-	0,95	0,95	0,95	0,95
k,yy	-	1,33	1,35	1,20	1,41
UC		1,12	0,98	0,88	1,09

An example of the calculation is shown for variant 1 S-O-S which has the largest value for both N_{ed} and M_{ed}.

$$M_{pl,Rd} = ((b * t) * \frac{h - t}{2} * 2 +* (h - 2 * t) * t * \frac{h - 2 * t}{4} * 4) * \sigma_y = 42\ kNm$$
$$N_{Rd,pl} = \frac{A_v * f_y}{\gamma_{M1}} = 946\ kN$$

$$C_{my}, C_{mLT} = 0{,}95 + 0{,}05 * \alpha_h = 0{,}95 + 0 = 0{,}95$$

$$k_{yy} = C_{my} * \left(1 + (\bar{\lambda}_y - 0{,}2) * \frac{N_{Ed}}{\chi_y * \frac{N_{Rk}}{\gamma_{M1}}} \right) = 1.33$$

Symmetrical column cross section, only M_{y} is present and χ_{LT} is equal to 1.
A steel strength of S355 is required in order to satisfy the verification of combined axial force and moment.

$$\frac{N_{Ed}}{\chi_y * \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} * \frac{M_{y,Ed}}{\chi_{LT} * \frac{M_{y,Rk}}{\gamma_{M1}}} = \frac{490}{0{,}78 * \frac{946}{1}} + 1{,}16 * \frac{12}{1 * \frac{42}{1}} = 1{,}12$$

Again, an increased thickness of 7 mm instead of 6 is required to satisfy the combined axial force and internal moment verification. For variant 4, the thickness needs to be increased as well from 5 mm to 6 mm.

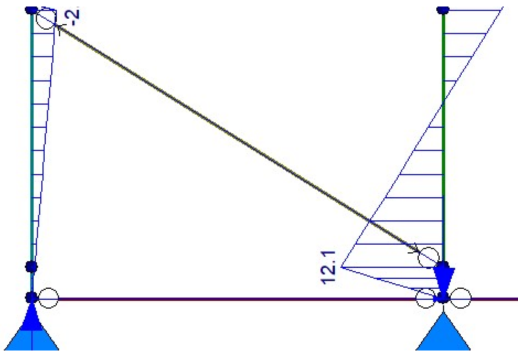


Figure 119. Variant 1 S-O-S Governing internal bending moment in 8 storey model (Technosoft).

Q. Internal connections verification

Indications of end, edge distances and spacing lengths

- $End\ distance\ e_1 = 1{,}2 * d_0$
- $Edge\ distance\ e_2 = 1{,}2 * d_0$
- $Spacing\ p_1 = 2{,}2 * d_0$
- $Spacing\ p_2 = 2{,}4 * d_0$

Fin plate resistance

a) Shear resistance of the bolts

$$V_{Rd,1} = \frac{n * F_{V,Rd}}{\sqrt{1 + \left(\frac{6 * e}{(n+1) * p_1}\right)^2}}$$

b) Bearing resistance at the fin plate

$$V_{Rd,2} = \frac{n}{\sqrt{\left(\frac{1 + n * \alpha}{F_{b,ver,Rd}}\right)^2 + \left(\frac{n * \beta}{F_{b,ho\ ,Rd}}\right)^2}}$$

$$\alpha = 0$$

$$\beta = 6 * \frac{z}{p_1 * n * (n+1)}$$

c) Resistance of the gross cross-section of the fin plate

$$V_{Rd,3} = \frac{h_p * t_p}{1.27} * \frac{f_{yp}}{\sqrt{3}\ * \gamma_{M0}}$$

d) Net area resistance of the fin plate

$$V_{Rd,4} = A_{v,net} * \frac{f_{up}}{\sqrt{3}\ * \gamma_{M0}}$$

$$A_{v,net} = \ t_p * (h_p - n_1 * d_0)$$

e) Block tearing resistance of the fin plate

$$V_{Rd,5} = F_{eff,,rd} = 0{,}5 * \frac{f_{up} * A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} * f_{up} * \frac{A_{nv}}{\gamma_{Mo}}$$

$$A_{nt} = \ t_p * (e_{2p} - \frac{d_0}{2})$$

$$A_{nv} = t_p * \left(h_p - e_1 - (n_1 - 0{,}5 *\right) * d_0)$$

f) Fin-plate in bending

$$V_{Rd,6} = \infty$$

g) Bearing resistance at the web

$$V_{Rd,8} = \frac{1}{\sqrt{\left(\frac{\frac{1}{n} + \alpha}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta}{F_{b,hor,Rd}}\right)^2}}$$

h) Beam web in shear: Gross section

$$V_{Rd,9} = A_{b,v} * \frac{f_{y,bw}}{\sqrt{3} \; * \gamma_{M0}}$$

i) Beam web in shear: Net section

$$V_{Rd,10} =* \frac{f_{u,bw}}{\sqrt{3} \; * \gamma_{M2}}$$
$$A_{b,v,net} = \; A_{b,v} - n_1 * \; d_0 * t_{bw}$$

j) Beam web in shear: Shear block

$$V_{Rd,11} = F_{eff,2,Rd} = 0{,}5 * \frac{f_{ubw} * A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} * f_{y,bw} * \frac{A_{nv}}{\gamma_{Mo}}$$
$$A_{nt} = \; t_{bw} * (e_{2b} - \frac{d_0}{2})$$
$$A_{nv} = t_{bw} * (e_{1b} + (n_1 - 1) * p_1 - (n_1 - 0{,}5) * d_0)$$

k) Shear resistance of the joint

$$V_{Rd} = min\{V_{Rd,i}\}$$

Gusset plate resistance

a) Shear resistance of the bolts

$$V_{Rd,1} = \frac{n * F_{V,Rd}}{\sqrt{1 + \left(\frac{6 * e}{(n+1) * p_1}\right)^2}}$$

b) Bearing resistance at the gusset plate

$$V_{Rd,2} = \frac{n}{\sqrt{\left(\frac{1 + n * \alpha}{F_{b,ver,Rd}}\right)^2 + \left(\frac{n * \beta}{F_{b,hor,Rd}}\right)^2}}$$
$$\alpha = 0$$
$$\beta = 6 * \frac{z}{p_1 * n * (n+1)}$$

c) Resistance of the gross cross-section of the gusset plate

$$V_{Rd,3} = \frac{h_p * t_p}{1.27} * \frac{f_{yp}}{\sqrt{3} \; * \gamma_{M0}}$$

d) Net area resistance of the gusset plate

$$V_{Rd,4} = A_{v,net} * \frac{f_{up}}{\sqrt{3} \; * \gamma_{M0}}$$
$$A_{v,net} = \; t_p * (h_p - n_1 * d_0)$$

e) Block tearing resistance of the gusset plate

$$V_{Rd,5} = F_{eff,,rd} = 0{,}5 * \frac{f_{up} * A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} * f_{up} * \frac{A_{nv}}{\gamma_{Mo}}$$
$$A_{nt} = \; t_p * (e_{2p} - \frac{d_0}{2})$$
$$A_{nv} = t_p * \left(h_p - e_1 - (n_1 - 0{,}5 \, *) * d_0\right)$$

f) Gusset-plate in bending

$$V_{Rd,6} = \infty$$

g) Bearing resistance at the bracing

$$V_{Rd,8} = \frac{1}{\sqrt{\left(\frac{\frac{1}{n}+\alpha}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta}{F_{b,ho\ ,Rd}}\right)^2}}$$

h) Bracing in tension: Gross section

$$V_{Rd,9} = A_{b,v} * \frac{f_{y,br}}{\sqrt{3}\ * \gamma_{M0}}$$

i) Bracing in tension: Net section

$$V_{Rd,10} =* \frac{f_{u,br}}{\sqrt{3}\ * \gamma_{M2}}$$

$$A_{b,v,net} = A_{b,v} - n_1 * d_0 * t_{br}$$

j) Bracing in tension: Block action

$$V_{Rd,11} = F_{eff,2,Rd} = 0{,}5 * \frac{f_{ubr} * A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} * f_{y,br} * \frac{A_{nv}}{\gamma_{Mo}}$$

$$A_{nt} = t_{br} * (e_{2b} - \frac{d_0}{2})$$

$$A_{nv} = t_{br} * (e_{1b} + (n_1 - 1) * p_1 - (n_1 - 0{,}5) * d_0)$$

k) Shear resistance of the joint

$$V_{Rd} = min\{V_{Rd,i}\}$$

Connection 1 Fin plate resistance

Element properties

Table 74. Properties of connection 1.

PCF Ceiling joist	**Bolted connection**	**Fin-plate**
$t = 2\ mm$ $h = 140\ mm$ $b = 50\ mm$	Bolts M12, 4.6 $n = 2$ $n_1 = 2$	$h_p = 70\ mm$ $b_p = 40\ mm$ $t_p = 5\ mm$
PCF Upper Edge beam		
$t = 4\ mm$ $h = 150\ mm$ $b = 50\ mm$ $f_{y,s} = 235\ N/mm^2$ $f_{u,s} = 350 N/mm^2$	$d_b = 12\ mm$ $d_0 = 14\ mm$ $f_{y,b} = 240/mm^2$ $f_{u,b} = 400\ N/mm^2$	$e_1 = 20\ mm$ $p_1 = 35\ mm$ $e_2 = 20\ mm$ $e_{2b} = 20\ mm$

Resistance Fin plate

Table 75. Fin plate resistance in connection 1.

Shear resistance of the bolts	$V_{Rd,1}$	$18\ kN$
Bearing resistance at the fin-plate	$V_{Rd,2}$	$27\ kN$
Resistance of the gross cross-section of the fin-plate	$V_{Rd,3}$	$40\ kN$
Resistance of the net cross-section of the fin-plate	$V_{Rd,4}$	$38\ kN$

Block tearing resistance of the fin-plate	$V_{Rd,5}$	$32\ kN$
Gross area resistance of the beam web	$V_{Rd,9}$	$81\ kN$
Net area resistance of the beam web	$V_{Rd,10}$	$110\ kN$
Beam web in shear: Shear block	$V_{Rd,11}$	$20\ kN$
Shear resistance of the joint	V_{Rd}	$18\ kN$

Unity check

$V_{Rd} = 18\ kN$
$V_{Ed} = 1.73\ kN$

$$UC = \frac{V_{Ed}}{V_{Rd}} = \frac{1.73}{18} = 0.10$$

Connection 2 Fin plate resistance

Element Properties

Table 76. Properties of connection 2.

Upper edge beam	Bolted connection	Fin-plate
$t = 4\ mm$ $h = 150\ mm$ $b = 50\ mm$	Bolts M12, 4.6 n=2 n1=2	h_p=75 mm b_p=40 mm t_p=5 mm
RHS Column		
$t = 6\ mm$ $h, b = 120\ mm$	d_b=12 mm d_0=14 mm	e_1=20 mm p_1=35 mm $e_2 = 20\ mm$ $e_{2b} = 20\ mm$

Resistance Fin plate verification

Table 77. Fin plate resistance in connection 2.

Shear resistance of the bolts	$V_{Rd,1}$	$18\ kN$
Bearing resistance at the fin-plate	$V_{Rd,2}$	$27\ kN$
Resistance of the gross cross-section of the fin-plate	$V_{Rd,3}$	$40\ kN$
Resistance of the net cross-section of the fin-plate	$V_{Rd,4}$	$38\ kN$
Block tearing resistance of the fin-plate	$V_{Rd,5}$	$32\ kN$
Gross area resistance of the beam web	$V_{Rd,9}$	$81\ kN$
Net area resistance of the beam web	$V_{Rd,10}$	$99\ kN$
Beam web in shear: Shear block	$V_{Rd,11}$	$39\ kN$
Shear resistance of the joint	V_{Rd}	$18\ kN$

Unity Check

$V_{Rd} = 18\ kN$
$V_{Ed} = 8.9\ kN$

$$UC = \frac{V_{Ed}}{V_{Rd}} = \frac{8.9}{18} = 0.50$$

Connection 3 Gusset plate resistance

Element properties

Table 78. Properties of connection 3.

Bracing	Bolted connection	Gusset plate
$t = 10\ mm$	Bolts M12, 8.8	$h_p = 160\ mm$
$l = 130\ mm$	$n = 3$	$b_p = 160\ mm$
$b = 100\ mm$	$n_1 = 3$	$t_p = 10\ mm$
$f_{y,s} = 235\ N/mm^2$	$d_b = 14\ mm$	$e_1 = 25\ mm$
$f_{u,s} = 350 N/mm^2$	$d_0 = 16\ mm$	$p_1 = 40\ mm$
	$f_{y,b} = 640/mm^2$	$e_2 = 50\ mm$
	$f_{u,b} = 800\ N/mm^2$	$e_{2b} = 50\ mm$

Resistance Gusset plate verification

Table 79. Fin plate resistance in connection 3.

Shear resistance of the bolts	$V_{Rd,1}$	$71\ kN$
Bearing resistance at the gusset plate	$V_{Rd,2}$	$151\ kN$
Bracing in tension: Gross section	$V_{Rd,3}$	$139\ kN$
Bracing in tension: Net section	$V_{Rd,4}$	$133\ kN$
Block tearing resistance of the gusset plate	$V_{Rd,5}$	$147\ kN$
Net area resistance of the gusset plate	$V_{Rd,9}$	$217\ kN$
Gross area resistance of the gusset plate	$V_{Rd,10}$	$226\ kN$
Gusset plate in shear: Shear block	$V_{Rd,11}$	$145\ kN$
Shear resistance of the joint	V_{Rd}	$71\ kN$

Unity check

$V_{Rd} = 71\ kN$
$V_{Ed} = 62\ kN$
$$UC = \frac{V_{Ed}}{V_{Rd}} = \frac{62}{71} = 0.88$$

Weld verification

Element properties

- $F_H = 0\ N$
- $F_V = 9{,}000\ N$
- $a = 3\ mm$
- $L = 75\ mm$

If $L < 150 * a$, then $\tau_{||}$ gets a factor 1,0 instead of 1,5

$$\tau_{||} = 1{,}5 * \frac{F_V}{2 * a * L}$$
$$\tau_{||} = 1 * \frac{9{,}000}{2 * 3 * 75} = 20\ N/mm^2$$

$$\sigma_{\perp} = \tau_{\perp} = \frac{\frac{1}{2} * \sqrt{2} * F_H}{2 * a * L}$$

$$\sigma_{w,ed} = \sqrt{\sigma_{\perp}{}^2 + 3 * (\tau_{\perp}{}^2 + \tau_{||}{}^2)} \leq f_{w,d}$$

$$f_{w,d} = \frac{f_u}{\beta * \gamma_{M2}}$$

$$f_{w,d} = \frac{350}{0,8 * 1,25} = 350\ N/mm^2$$

Unity check

$$\sigma_{w,ed} = \sqrt{3 * (20^2)} = 35\ N/mm^2$$

$$UC = \frac{f_{w,d}}{\sigma_{w,ed}} = 0,10$$

R. Internal connections Excel calculations

Connection 1: Ceiling Joist – Edge beam

Ceiling Joist - Edge beam connection

Ceiling joist				**Connection**				Size in mm	*Minimum size in mm*
t	2	mm		d,b	12	mm	h,plate	75	*64,4*
h	140	mm		d0	14	mm	b,plate	40	*33,6*
b	50	mm					z	20	
Edge beam							t,plate	5	
t	4	mm		**Bolts M12 4.6**					*Minimum*
h	150	mm		f,y,b	240	N/mm2	e1	20	*16,8*
b	80	mm		f,u,b	400	N/mm2	p1	35	*30,8*
				f,y,s	235	N/mm2	e1b	45	*35*
n,bolts	2			f,u,s	350	N/mm2	e2	20	*16,8*
n1	2						e2b	20	*16,8*

Verification

a) Shear resistance of bolts				**b) Bearing resistance at the fin-plate**				**c) Fin-plate in shear (Gross Section)**					
F,v,Rd	13	kN		F,b,Rd	19	kN		V,Rd,4	40	kN			
V,rd,1	18	kN		V,rd,2	27	kN							
a,v	0,5			k1, edge	2,3			gamma M0	1				
A,s (M12)	84	mm2		k1, inner	0,3								
gamma M2	1,25			1 row, alfa	0								
				beta	1,71								
d) Net-area of fin-plate				**e) Block tearing resistance of fin-plate**				**f) Fin-plate in bending**					
V,rd,5	38	kN		Eccentric loading	F,eff,2,Rd	32	kN	V,Rd,6	infinite				
A,v,net	235	mm2		A,nv	170	mm2							
				A,nt	65	mm2							
i) Beam web in shear: Gross section				**j) Beam web in shear: Net section**				**k) Beam web in shear: Shear block**				**i) Shear resistance**	
V,Rd,9	81	kN		V,Rd,10	110	kN		V,Rd,11	20	kN		18	kN
A,b,v	600	mm2		A,b,v,net	544	mm2		A,nt	26	mm2			
								A,nv	118	mm2			

Connection 2: Edge beam – Column connection

Edge beam - Column connection

Edge beam				**Connection**				Size (mm)	*Minimum (mm)*		**Fillet Weld**		
t	4	mm		d,b	12	mm	h,plate	75	*64,4*		a >/	2,3	mm
h	150	mm		d0	14	mm	b,plate	40	*33,6*		t,//	20	N/mm2
b	50	mm					z	20			Factor t,//	1	
Column							t,plate	5			a >/	3	mm
t	6	mm		**Bolts M12 4.6**							f,w,d	350	N/mm2
h	120	mm		f,y,b	240	N/mm2	e1	20	*16,8*		beta,w (S2	0,8	
b	120	mm		f,u,b	400	N/mm2	p1	35	*30,8*		sigma,w	35	N/mm2
				f,y,s	235	N/mm2	e1b	45	*40*		UC	0,10	
n,bolts	2			f,u,s	350	N/mm2	e2	20	*16,8*				
n1	2						e2b	20	*16,8*				

Verification

a) Shear resistance of bolts				**b) Bearing resistance at the fin-plate**				**c) Fin-plate in shear (Gross Section)**					
F,v,Rd	13	kN		F,b,Rd	19	kN		V,Rd,4	40	kN			
V,rd,1	18	kN		V,rd,2	27	kN							
a,v	0,5			k1, edge	2,3			gamma M0	1				
A,s (M12)	84	mm2		k1, inner	0,3								
gamma M2	1,25			1 row, alfa	0								
				beta	1,71								
d) Net-area of fin-plate				**e) Block tearing resistance of fin-plate**				**f) Fin-plate in bending**					
V,rd,5	38	kN		Eccentric loading	F,eff,2,Rd	32	kN	V,Rd,6	infinite				
A,v,net	235	mm2		A,nv	170	mm2							
				A,nt	65	mm2							
i) Beam web in shear: Gross section				**j) Beam web in shear: Net section**				**k) Beam web in shear: Shear block**				**l) Shear resistance**	
V,Rd,9	98	kN		V,Rd,10	123	kN		V,Rd,11	39	kN		18	kN
A,b,v	720	mm2		A,b,v,net	608	mm2		A,nt	52	mm2			
								A,nv	236	mm2			

Connection 3: Bracing – Beam, Column connection

Bracing - Beam, Column connection

Gusset plate			**Bracing**			Sizes in mm		*Minimum size in mm*
t	10	mm	d,b	14	mm	l,plate	130,0	*108,8*
h	160	mm	d0	16	mm	b,plate	100,0	*38,4*
Column						z	50,0	
t	6	mm				t,plate	10,0	
h	120	mm	**Bolts M14, 8.8**					
			f,y,b	640	N/mm2	e1	25,0	*19,2*
n,bolts	3		f,u,b	800	N/mm2	p1	40,0	*35,2*
n1	3		f,y,s	235	N/mm2	e1p	25,0	*19,2*
			f,u,s	350	N/mm2	e2	50,0	*19,2*
						e2b	50,0	*19,2*

Verification

a) Shear resistance of bolts			**b) Bearing resistance at the gusset plate**			**c) Bracing in shear (Gross Section)**		
F,v,Rd	50	kN	F,b,Rd	49	kN	V,Rd,4	139	kN
V,rd,1	71	kN	V,rd,2	151	kN	gamma M0	1,00	
F,b,hor,Rd			k1, edge	2,50				
a,v	0,50		k1, inner	2,50				
A,s (M16)	157	mm2	1 row, alfa	0,00				
gamma M2	1,25		beta	2,50				
s) Bracing in shear (Net Section)			**e) Block tearing resistance of fin-plate**			**g) Gusset plate in shear: Gross section**		
V,rd,5	133	kN	Eccentric loading F,eff,2,Rd	147	kN	V,Rd,9	217	kN
A,v,net	820	mm2	A,nv	650	mm2			
			A,nt	420	mm2	A,b,v	1600	mm2
h) Gusset plate in shear: Net section			**i) Gusset plate in shear: Shear block**			**j) Shear resistance**		
V,Rd,10	226	kN	V,Rd,11	147	kN		71	kN
A,b,v,net	1120	mm2	A,nt	420	mm2			
			A,nv	650	mm2			

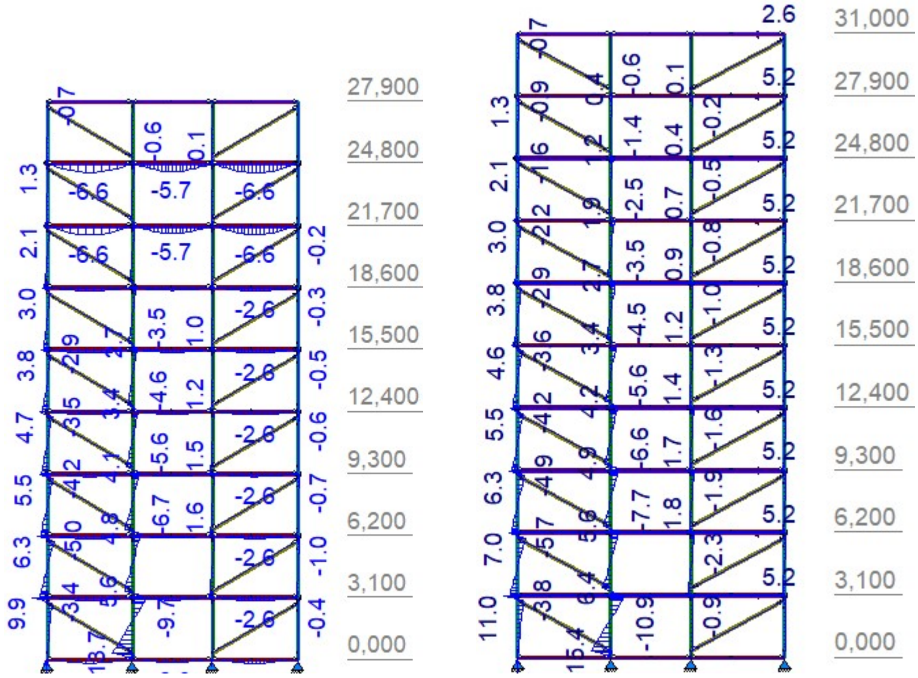
S. Structural assessment

Table 80. Stabilising column loads for 8 to 10 storeys using increased bracing length.

		Variant 1 S-O-S		Edge column	Variant 2 O-F		Internal column	Variant 4 O-F-O		Edge column
Storeys	q,wind (kN/m^2)	F,wind (kN)	F,weight (kN)	F,ed (kN)	F,wind (kN)	F,weight (kN)	F,ed (kN)	F,wind (kN)	F,weight (kN)	F,ed (kN)
1	0,82	3	29		2	23		3	26	
2	0,82	11	59		9	46		11	52	
3	0,82	25	88		20	69		25	77	
4	0,85	47	117		38	92		47	103	
5	0,87	76	147		61	115		76	129	
6	0,91	113	176		91	138		113	155	
7	0,93	159	205		127	161		159	180	
8	0,96	213	235	351	171	184	516	199	206	450
9	0,98	257	274	424	207	219	608	238	253	535
10	1,01	301	317	502	244	258	707	280	293	625

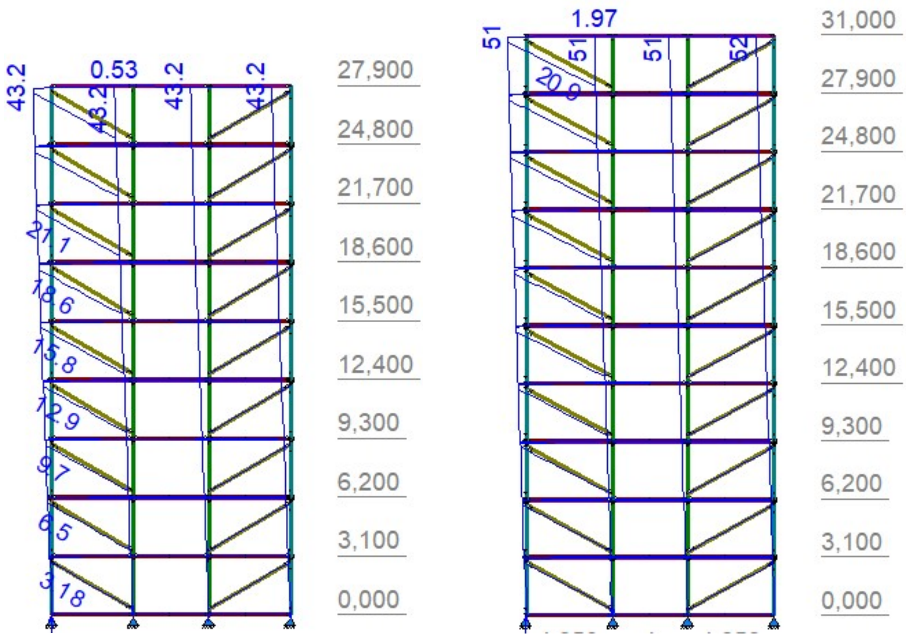
T. Technosoft models 9 and 10 storeys

Variant 1 S-O-S



(a) (b)

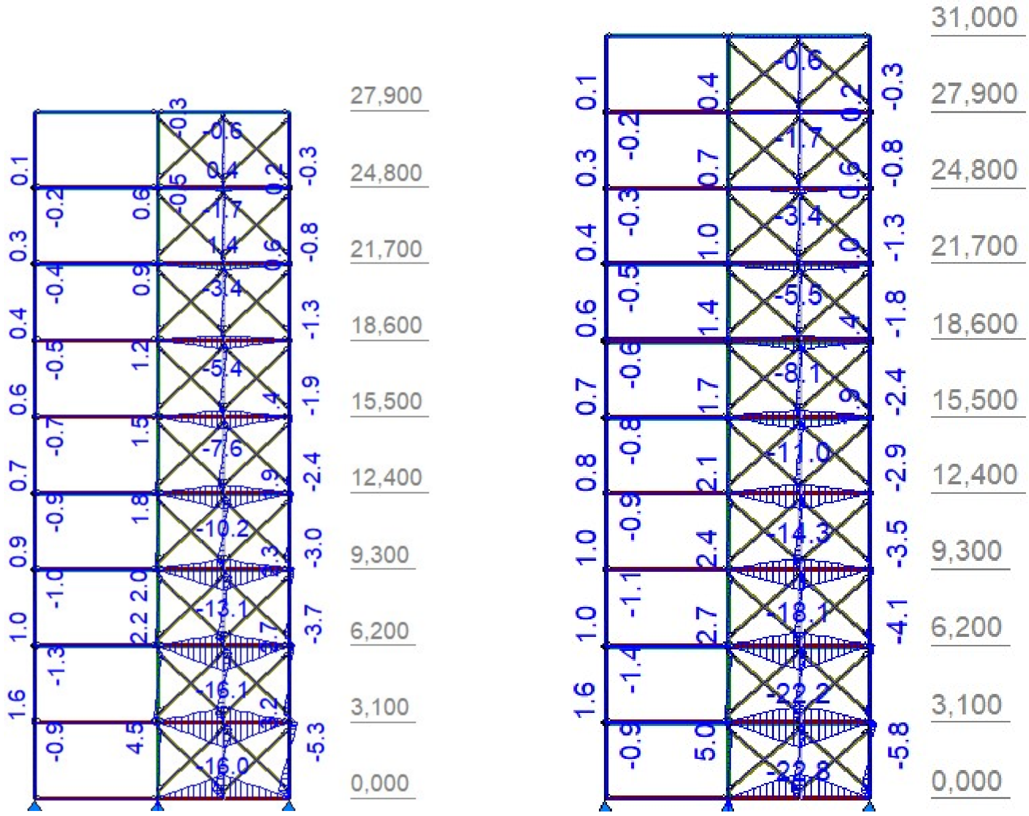
Figure 120. Internal moments Variant 1 S-O-S: (**a**) 9 storey model, (**b**) 10 storey model.



(a) (b)

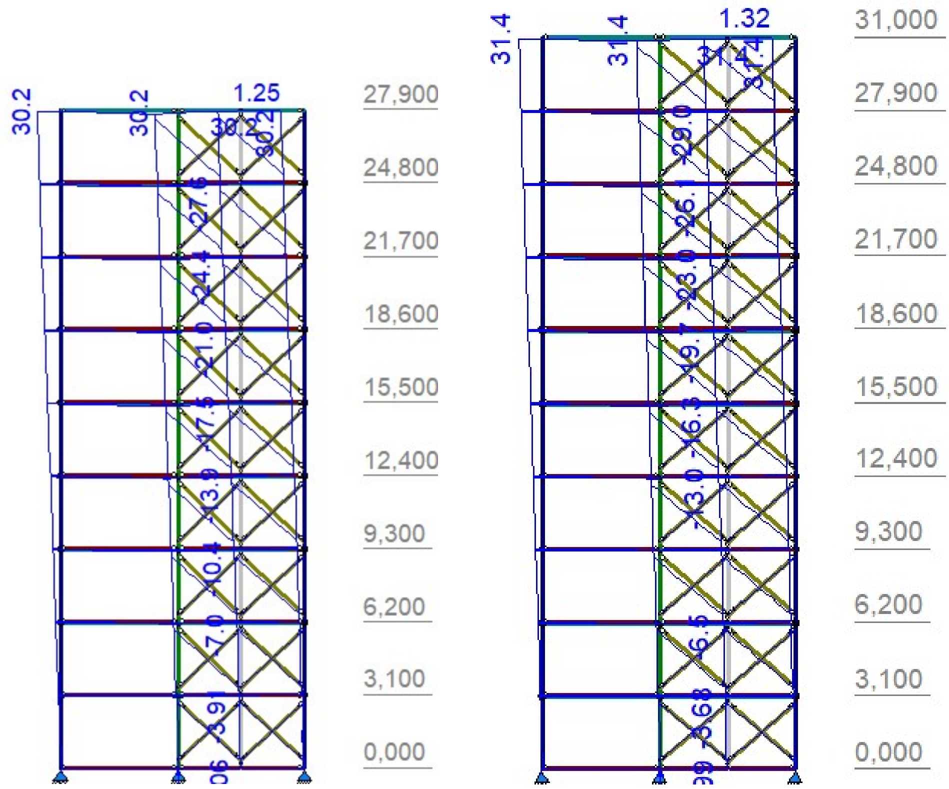
Figure 121. Horizontal displacement Variant 1 S-O-S: (**a**) 9 storey model, (**b**) 10 storey model.

Variant 2 O-F



(**a**) (**b**)

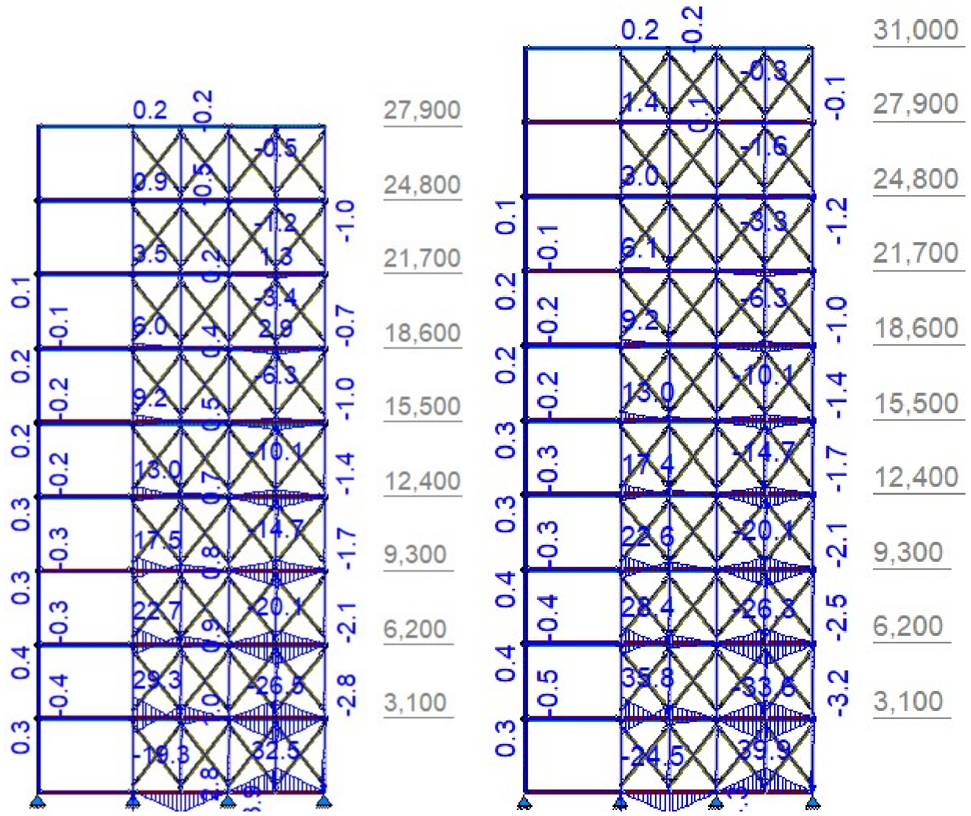
Figure 122. Internal moments Variant 2 O-F: (**a**) 9 storey model, (**b**) 10 storey model.



(**a**) (**b**)

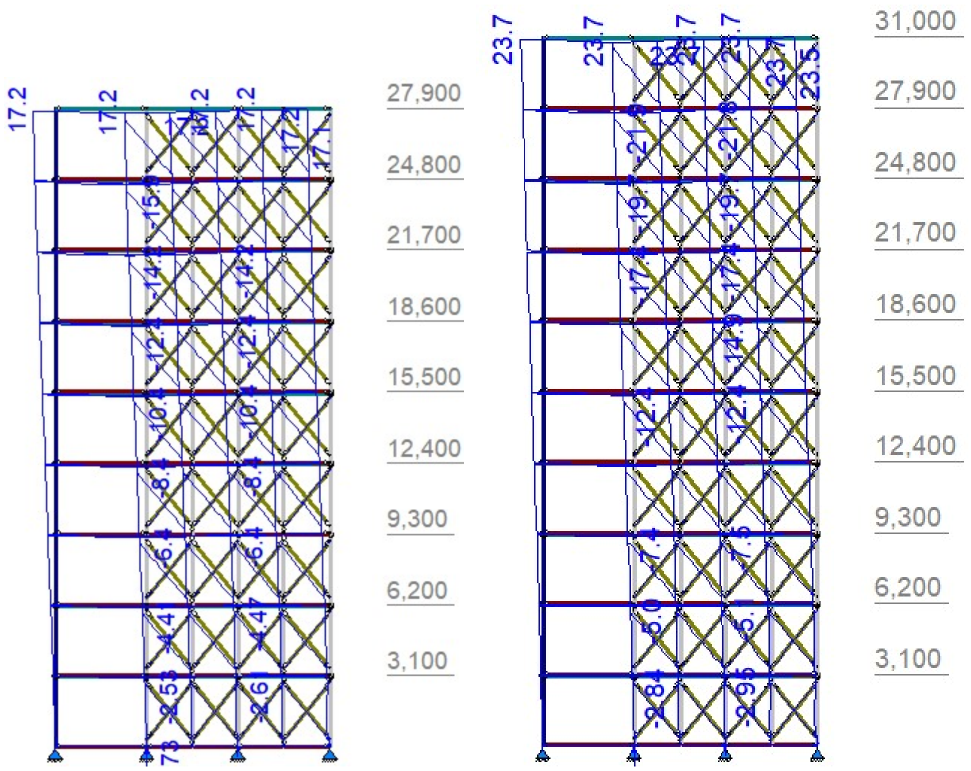
Figure 123. Horizontal displacements Variant 2 O-F: (**a**) 9 storey model, (**b**) 10 storey model.

Variant 3 O-F-F



(a) (b)

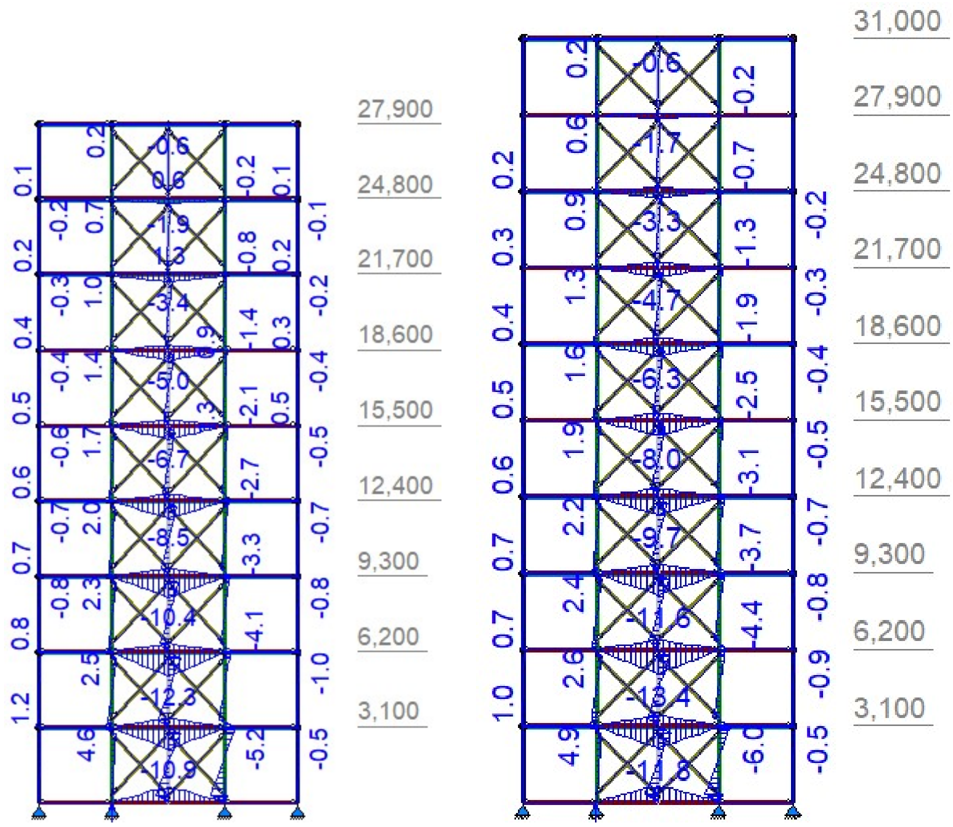
Figure 124. Internal moments Variant 3 O-F-F: (**a**) 9 storey model, (**b**) 10 storey model.



(a) (b)

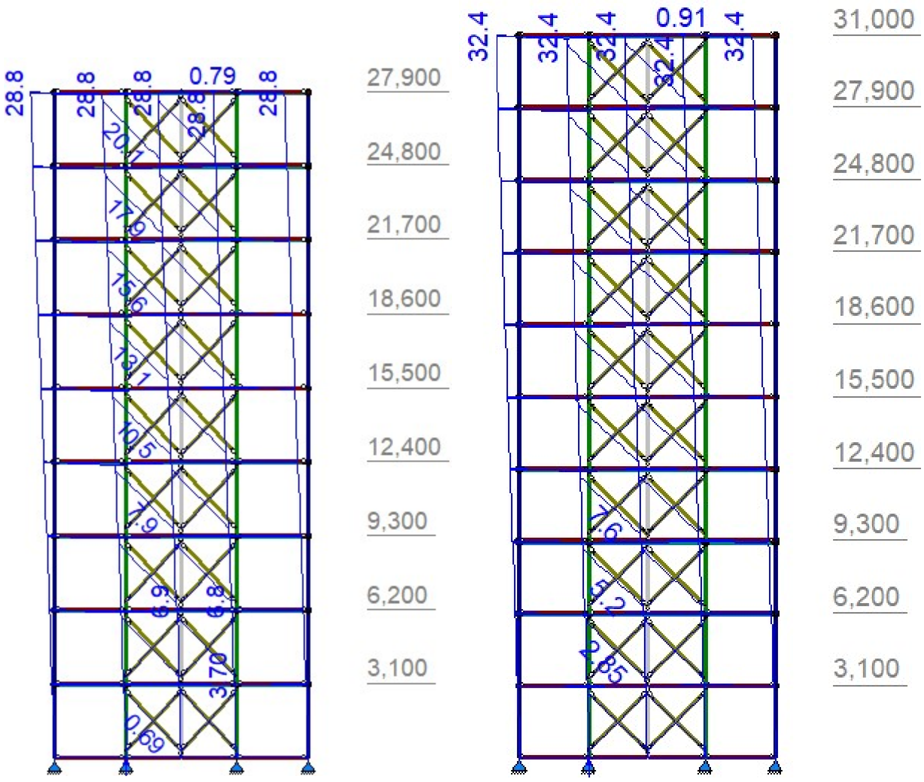
Figure 125. Horizontal displacement Variant 3 O-F-F: (**a**) 9 storey model, (**b**) 10 storey model.

Variant 4 O-F-O



(a) (b

Figure 126 Internal moments Variant 4 O-F-O: (**a**) 9 storey model, (**b**) 10 storey model.



(a) (b

Figure 127. Horizontal displacement Variant 4 O-F-O: (**a**) 9 storey model, (**b**) 10 storey model.

U. Functional and Environmental Assessment

Table 81. Functional properties.

Name	Open section of wall	Wall open (%)	Length (m)	Total area (m^2)	Internal walls	Wall width (mm)	Usable area (m^2)	Space efficiency factor	Wall-to-floor ratio
S-O-S	Middle	33,3	12,0	42,0	2	0,15	37,2	0,89	0,29
O-F	Single edge	50,0	10,0	35,0	1	0,17	31,0	0,88	0,35
O-F-F	Single edge	33,3	12,0	42,0	2	0,17	36,6	0,87	0,30
O-F-O	Both edges	60,0	10,0	35,0	1	0,05	33,8	0,96	0,32

Table 82. Element sizes.

Variant	Area (mm^2)				Extra material use (kN)				
Name	Edge column	Internal column	Lower edge beam	Upper edge beam	Edge column	Internal column	Lower edge beam	Bracing	Total
S-O-S	2664	1840	1600	1040	5,9	0,0	0,0	0,0	5,9
O-F	1856	2664	1960	1200	2,7	3,2	0,8	0,9	7,6
O-F-F	1404	1990	1600	1040	1,0	0,6	0,0	2,6	4,1
O-F-O	1155	2664	1600	1040	0,0	3,2	0,0	0,9	4,1

Table 83. Material use first part.

		Plasterboard		Insulation		Steel frame studs/joists		Frame edge beam	
Name	Partition structure	Thickness layer (m)	Subtotal (m^3)	Thickness layer (m)	Subtotal (m^3)	Area (m^2/m length)	Subtotal (m^3)	Area (m2/m length)	Subtotal (m^3)
S-O-S	Wall	0,045	3,45	0,10	6,00	0,00047	0,036	0,00020	0,006
O-F	Wall	0,045	2,63	0,10	5,00	0,00047	0,027	0,00020	0,005
O-F-F	Wall	0,045	3,45	0,10	6,00	0,00047	0,036	0,00020	0,006
O-F-O	Wall	0,045	2,64	0,10	5,00	0,00047	0,028	0,00020	0,005
S-O-S	Floor and Ceiling	0,045	1,81	0,12	4,20	0,00065	0,008	0	
O-F	Floor and Ceiling	0,045	1,50	0,12	5,04	0,00065	0,006	0	
O-F-F	Floor and Ceiling	0,045	1,80	0,12	4,20	0,00065	0,008	0	
O-F-O	Floor and Ceiling	0,045	1,55	0,12	0,00	0,00065	0,006	0	

Table 84. Material use second part.

		Structural edge beams		Columns	Steel reinforcement	Concrete	
Name	Partition structure	Area (m2/m length)	Subtotal (m^3)	Total (m^3)	Subtotal (m^3)	Thickness (m)	Subtotal (m^3)
S-O-S	Wall	0,0026	0,08	0,009	0	0	0
O-F	Wall	0,0032	0,09	0,009	0	0	0
O-F-F	Wall	0,0026	0,08	0,011	0	0	0
O-F-O	Wall	0,0026	0,07	0,010	0	0	0
S-O-S	Floor and Ceiling	0	0	0	0,0504	0,12	5,04
O-F	Floor and Ceiling	0	0	0	0,042	0,12	4,20
O-F-F	Floor and Ceiling	0	0	0	0,0504	0,12	5,04
O-F-O	Floor and Ceiling	0	0	0	0,042	0,12	4,20

Table 85. Environmental analysis.

		Unit	Plasterboard	Insulation	Wood	Steel	Concrete	Total
	Volumetric weight	kN/ m^3	8,50	0,50	6,00	79,00	25,00	
	Embodied Energy	MJ/kg	6,75	16,60	10,00	21,50	1,90	
	Embodied Carbon	$kgCO_2/kg$	0,39	1,28	0,46	1,53	0,22	
S-O-S	Volume	m^3	5,27	10,20	0	0,19	5,04	21
	Weight	kg	4476	510	0	1509	12600	19095
	Embodied energy	MJ	30216	8466	0	32437	23940	95059
	Embodied carbon	$kgCO_2$	1746	653	0	2308	2772	7479
O-F	Volume	m^3	4,13	10,04	0	0,18	4,20	19
	Weight	kg	3507	502	0	1386	10500	15895
	Embodied energy	MJ	23673	8333	0	29792	19950	81747
	Embodied carbon	$kgCO_2$	1368	643	0	2120	2310	6440
O-F-F	Volume	m^3	5,25	10,20	0	0,19	5,04	21
	Weight	kg	4464	510	0	1522	12600	19095
	Embodied energy	MJ	30129	8466	0	32716	23940	95251
	Embodied carbon	$kgCO_2$	1741	653	0	2328	2772	7494
O-F-O	Volume	m^3	4,19	5,00	0	0,16	4,20	14
	Weight	kg	3562	250	0	1288	10500	15600
	Embodied energy	MJ	24040	4150	0	27692	19950	75833
	Embodied carbon	$kgCO_2$	1389	320	0	1971	2310	5990

Table 86. Environmental result.

			Unit	Per m^2 of total area	Steel/ m^2	Concrete/ m^2	Module weight (tonnes)
S-O-S	Volume	21					
	Weight	19095	kg	455			19
	Embodied energy	95059	MJ	2263	772	570	
	Embodied carbon	7479	$kgCO_2$	178	55	66	
O-F	Volume	19					
	Weight	15895	kg	454			16
	Embodied energy	81747	MJ	2336	851	570	
	Embodied carbon	6440	$kgCO_2$	184	50	66	
O-F-F	Volume	21					
	Weight	19095	kg	455			19
	Embodied energy	95251	MJ	2268	779	570	
	Embodied carbon	7494	$kgCO_2$	178	55	66	
O-F-O	Volume	14					
	Weight	15600	kg	446			16
	Embodied energy	75833	MJ	2167	791	570	
	Embodied carbon	5990	$kgCO_2$	171	56	66	