Building light and comfortable

Concept development of a light-weight steel and timber building system regarding human induced vibration comfort

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by

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to obtain the degree of Master of Science
at the Delft University of Technology,
to be defended publicly on Thursday December 20, 2018 at 16:00 PM.

Student number:	4149912	
Project duration:	January 15, 2018 – December 20, 2018	
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Preface

After an intense journey that started in the field of Architecture, I present this thesis as the final chapter of my studies at the Delft University of Technology. In order to obtain the Master of Science degree in Civil Engineering this research shows the combination of a scientific and practical approach exploring new fields in the Building Engineering sector. This concluding thesis showed me the never-ending challenges that lie ahead in the engineering sector and convinced me to continue exploring these aspects in the next chapter of my career.

I would like to thank my graduation supervisors and committee members Michiel Visscher, Johan Kraus, Sander Pasterkamp, Roland Abspoel and Rob Nijsse for their guidance throughout my whole thesis. First, I want to thank Michiel Visscher from Royal HaskoningDHV for introducing me to the innovative approach of light-weight construction and its related challenges. Also for being the most solution minded supervisor I could wish for by answering all my questions and guiding me through my thesis when I faced dead-end trails. Furthermore, I would like to thank Johan Kraus from TNO, for his academic knowledge regarding vibrations that helped me during the start of the project. Also for his realistic approach into how to tackle complex problems. Next, I want to thank Sander Pasterkamp and Roland Abspoel for their critical questions and practical knowledge helping me to achieve my objectives. Lastly, I would like to thank Rob Nijsse for his advice, guidance and role as chairman of the graduation committee.

I want to thank my colleagues at Royal HaskoningDHV for creating a pleasant working environment and putting effort into helping me achieve the objectives of this research. Next, I would like to thank Level Acoustics & Vibrations for their cooperation and involvement by letting me use their tool. Furthermore, I want to thank the fellow students at the TU Delft who helped me with new insights and distraction from my thesis when I needed it the most.

Finally, I would like to thank my friends and family for their support and motivation during my entire studies.

R.V.M. Cobelens
Delft, December 2018

Abstract

Limitations in the modern housing market supply and the high demand for city centre living space ask for a robust urban densification way of building. This results in the exploration of innovative vertical extension projects, such as the 'De Karel Doorman' case in Rotterdam. However, demanding the construction method to be extreme light-weight revealed an unexpected normative serviceability phenomenon. The reduced mass did not dissipate enough vibrational energy induced by human activities such as walking, leading to an excessive and disturbing perception of vibrations. This caused nuisance for both the home situation where the motion takes place as for the neighbouring floor fields. The critical motion-related limit state has to be satisfied to create a comfortable living environment.

This thesis aims to further develop the concept of light-weight steel and timber building structures focussing on vibration comfort. This is done by exploring structural measures that can steer the vibrational floor response for both the induced situation as for the transmittance to adjacent fields. Broadening research shows the general impact of the damping, natural frequency and modal mass. From this starting point, new practical building tools are developed to affect and control the path and magnitude of vibrations positively. General guidelines are provided that show the demands for a structural assembly to create suitable apartments.

The proposed measures to steer the vibration comfort were researched using both the conventional hand-calculation method from the SBR-guideline and by more accurate finite element analyses from SoViST and Autodesk Robot. The resulting OS-RMS_{90} values indicate the response velocity of the floor and have to meet the limit criteria proposed for the specific function of a building. For the light-weight residential building concept, these criteria were set to 0,8 [-] and 0,2 [-] for respectively the home and neighbouring situation.

It was found that for light-weight building structures the implementation of a large amount of stiffness is inevitable in both the floor assembly as in the junction. The consequence of additional mass and height can be balanced by using efficiently shaped profiles and smart placement of the joists. For the supporting beams, these demands encourage the use of rectangular hollow structural sections whereas for the floor assembly I-joists are recommended. Additional transverse stiffness stretches the clustering of natural frequencies for orthotropic plates but is most effective for two-way span floors. A smaller span will result in improved comfort levels but will complicate the structural assembly by introducing more elements and connections.
Besides the overall performance enhancing measures, it was found that the limit criterion for neighbouring apartments is harder to achieve without additional interventions. Introducing more substantial obstacles for the vibrations to overcome along the path, will reflect the transmittance and hence steer the floor response towards improved comfort levels. The use of an alternating floor field can provide in this issue as it avoids the mode-coupling of natural frequencies from adjacent elements. One other recommendation is the differentiation of the home-separating and in-home junctions. This results in maintaining more vibration energy in the home situation and limits the nuisance caused from excitements in a neighbouring apartment.

The as-built 'De Karel Doorman' revealed the impact of additional stiff elements in the wall that substantially increase the bending and torsional stiffness in the junction. It was found that these elements mitigate the nuisance caused by footfalls to imperceptible values for adjacent floor fields. However, these elements do leave a mark on the flexibility of the floor plan.

Light-weight building structures face new challenges and acknowledge the shift from strength-design to serviceability-design criteria. Regarding vertical extension projects, not just the building engineering aspects but also the practical implementation was found to contribute in the consideration for structural assembly measures.

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Introduction

1.1. Relevance of research

For a long time the desire of home-seekers to live in an urban location has been growing. However, it has reached an extent to which the cities can no longer deliver the required capacity for inhabitants [28]. This social, geographical phenomenon in combination with a scarcity of building space demands communities to come up with new ideas to grant these people a place to live [8]. While demolishing is not considered sustainable and refurbishment does not meet the asked capacity [30], building over or through other buildings can be regarded as a good alternative for densifying the city [8]. In this way, functionality can be retained, and new properties can arise in the form of high-rise in an urban area.

In the city of Rotterdam, the 'De Karel Doorman' project by Royal HaskoningDHV (RHDHV) shows that urban densification can be achieved by activating the unused load bearing potential of existing heritage structures [12]. The ultra-light-weight building concept offered the possibility to carry a five times higher load, taking full advantage of the hidden potential of the structure. Furthermore, existing buildings and infrastructure could remain intact and in use during construction. RHDHV further developed this concept and made it feasible, inspiring the city for future applications [4]. The concept offers lots of potential for cities and metropolises around the globe and creates a new type of vertical urbanism.

The method of construction for this light-weight building system was unusual and unforeseen challenges during erection led to improvised engineering solutions. Also, the lack of a construction site and the small footprint that was available in the city centre of Rotterdam made it a logistic puzzle [4]. However, this also gave some opportunities since it made the steel construction suitable for flexibility and sustainable by using light-weight and reusable steel [4]. A blueprint for an integrated industrial, flexible and demountable building system was touched upon, leaving room for improvement for possible future applications.

Minimising the weight of the 'to be added' volume pushed the boundaries of building aspects regarding comfort within the structure. Where acoustics and vibration levels are generally considered to be taken care of by the mass of a building [8][37], this issue now had to be dealt with differently. Unfortunately, not all considerations turned out the way intended, an unforeseen risk that was taken when working on these kinds of innovative ideas [12]. Therefore interventions had to be made at a stage where there was little influence left on the design. Although the situation was not ideal, the result was still considered of high quality and an inspiration for further developments.

Parallel to constructing the 'De Karel Doorman', the Netherlands Organisation for Applied Scientific Research (TNO) in Delft researched the possibilities of light-weight building systems. By good fortune, they became involved with the case and generated new insights and established new principles to improve the critical comfort aspects. After finalising their research, they launched a website [33] and a tool was developed by Level Acoustics & Vibrations in which the archetypes of these systems can be used in an early stage to check the comfort demands.

Concluding, light-weight building systems seem to be an exciting opportunity regarding urban densification. By facing the critical vibration comfort design criteria in an early stage, it is possible to develop this concept even further. Researching this topic will create an understanding of involved factors, help to find limitations, but most importantly explore the potential benefits of performance-enhancing measures, suitable for future use.

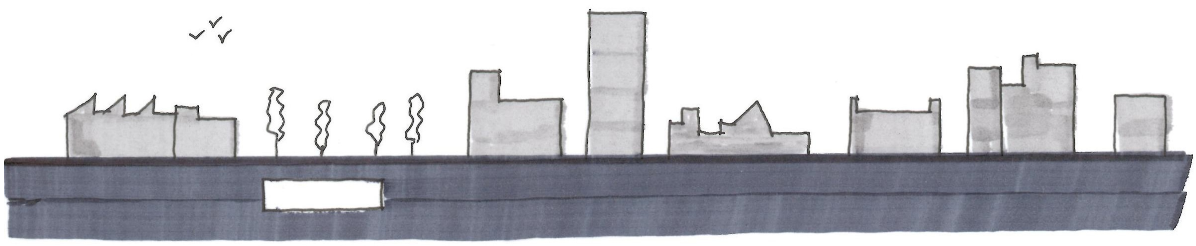


Figure 1.1: A typical densified urban area [12].

1.2. Problem statement

The 'De Karel Doorman' is an inspiring project covering a socially relevant topic of urban densification. Within this need, RHDHV found a way to come up with an innovative approach. However, this also led to new challenges including the vibration comfort of a light-weight building system.
During the construction phase, design adaptations to influence the floor vibration transfer had to be made. The most critical design measures to reach the target value for the vibration level were using a bidirectional beam system in the wooden floors, eliminating acoustic spring rubbers in the mid-beams in the apartments and applying (non-structural) slender steel columns between the steel beams, inside the separating walls between the apartments as can be seen in figure 1.2. These adjustments were made to create extra stiff reactions to vibrations and to reflect the vibration energy, preventing it from passing through to neighbouring apartments.
Pushing the boundaries of this extreme light-weight building system reveals that new challenges emerge when aiming for high-quality residential apartments. As was found during this project the current guidelines do not provide sufficient grip on the vibration control, especially regarding the transmittance to adjoining floor fields. The evaluation of vibration comfort proved to become the dominant design guideline for light-weight building systems.
To obtain comfortable apartment blocks, further research is required and improving design tools have to be presented — this way the conflicting issues regarding the limited weight of structures should be overcome, ultimately leading to a building system suitable for future designs.

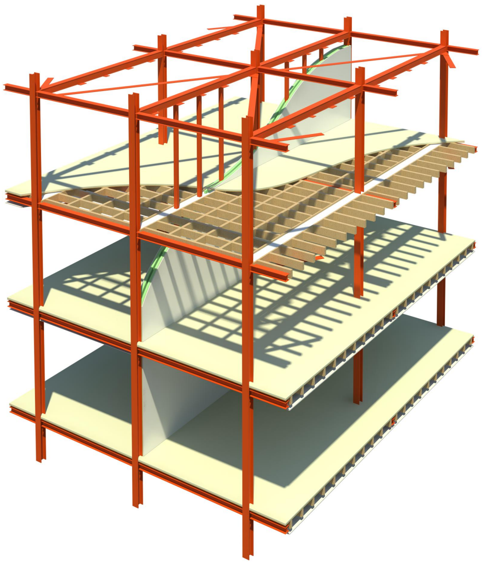


Figure 1.2: Segment of ultra-light building concept from De Karel Doorman [12].

1.3. Research question

How can the light-weight steel and timber building system, within the densified urban context, be further developed regarding acceptable vibration demands?

1.4. Objective

The objective of this research is to give a deeper understanding of light-weight steel and timber building systems and their behaviour regarding vibration comfort. To do so, the involved factors have to be found, and theoretical enhancements have to be translated into practical interventions. This will lead to measures that better control the comfort level in both the home situation and for the transmittance to neighbouring units. This research will be done as part of concept development of the light-weight building system aiming at generating better designs and guidelines for possible future applications within urban densified regions.

1.5. Methodology

The report outline is described below and indicates the topics covered per chapter. In here a method is provided for the approach of the objectives. Regarding each section, the relevant sub-questions will be answered.

Chapter 1 - Introduction
In chapter 1 the general introduction of the thesis is handled. Based on the challenges experienced with the 'De Karel Doorman' case the relevance of the research is portrayed. From the problem statement a research question is drafted. Combining the objective and methodology an outline is given for the report.

Chapter 2 - Background information
The purpose of this chapter is to create a clear understanding of the current state of knowledge regarding urban densification, vertical extensions of buildings and light-weight building systems and its demands by means of a literature review. The main focus will lie on the vibration performance. The case study that is used for designing and comparing the light-weight building systems in the following chapters will be elaborated in here as well. The following sub-questions are answered:

- 2.1 What is the need for urban densification and what are its possibilities and restrictions?
- 2.2 Why is vertical extension a suitable solution and in what typologies does it come?
- 2.3 Which building systems (i.e. structures and floors) are suitable for light-weight buildings and how do they come together in a connection?
- 2.4 What demands and requirements are necessary regarding light-weight building systems in the urban context?
- 2.5 How did 'De Karel Doorman' innovate the light-weight building market and where lie possibilities for improvements?

Chapter 3 - Literature research
The phenomenon vibration will be broken down into several parts, namely the source, path and receiver. For each, the theoretical background will be explained and its influence on the complete vibration comfort aspect. It is necessary to understand this theory to discover which practical measures are useful. The following sub-questions are answered:

- 3.1 What aspects are involved in vibration comfort?
- 3.2 How does the source of vibrations affect the level of comfort?
- 3.3 How does the path of vibrations affect the level of comfort?
- 3.4 How does the receiver of vibrations affect the level of comfort?

Chapter 4 - Theoretical building aspects
The underlying theory of light-weight building systems is analysed to get a grip on enhancing engineering choices. In this chapter, a theoretical approach regarding measures to steer the vibration response is made. Not only the vibration comfort for the home situation is considered but also concerning adjacent fields. On different detail levels, measures will be proposed and tested on a numerical level with suitable software. The goal is to come up with new concepts, integrating the conditions of the load-bearing structural system and the floors. The following sub-questions are answered:

- 4.1 Which structural properties influence the comfort level with regards to vibration induced by humans?
- 4.2 How do theoretical measures in the floor systems steer the vibration comfort?
- 4.3 How do theoretical measures at the structural junction steer the vibration comfort?
- 4.4 How do theoretical measures in alternating floor systems steer the vibration comfort?

Chapter 5 - Practical building aspects
In this chapter, the new concepts will be tested on their implementation after the theoretical knowledge is translated into practical measures. A model providing more insight into the dynamic footfall analyses will be used indicating the impact of the interventions on both the home and neighbouring situation. The following sub-questions are answered:

- 5.1 What practical measures steer the vibration comfort in the home situation?
- 5.2 What practical measures steer the vibration comfort in the neighbouring situation?
- 5.3 What are the optimised system configurations regarding the structure, floor and junction?

Chapter 6 - New building concepts
New structural assemblies are compared with the as-designed and as-built building system of the 'De Karel Doorman'. New guidelines to steer the vibration performance and improved concepts are provided for the future use of light-weight building systems. This chapter will show several new building concepts that provide the desired comfort level.

Chapter 7 - Conclusion and discussion
The results and findings obtained in the different parts of the research will contribute to the answer of the research question. Moreover, conclusions and recommendations for future consequences are given.

2

Background information

2.1. Urban densification

Large cities and especially their centres are very attractive to home seekers, causing severe pressure on the capacity available in urban areas [28]. Some of the reasons may be increased urban activity, better public transport or better amenities [8][25]. Not only does this demand come from individuals, but also the municipalities are encouraging a higher degree of densification to improve the regional development of economic productivity and the emergence of new jobs [25]. By creating a concentrated mix of functions at logistic nodes, long-distance traffic will be reduced. Additionally, this should lead to a more mixed community where different ethical and social-economic ranks come together [25].

2.1.1. Developments

Preventing cities from urban sprawl and creating soulless quarters full of new buildings, numerous solutions can be distinguished to generate more inner-city living space, see figure 2.1. Several researchers suggested the following options to solve this trending topic within urban planning [12][30]:

Building on empty sites
Creating new constructions in free space seems like the most natural way of supplying additional households. However densified urban cities often don't have a lot of open space left available for construction sites. These areas are mostly served as public spaces and do not allow developers to use it for new housing demands. Also, the limited free space that is primarily available does not allow enough for the required need. Almost every square meter in a densified urban area has already been sourced out.
Demolition
Demolition and construction of a new building is the most common construction practice, as it is the most effective regarding economic and technical grounds. At the same time, a higher or bigger building can replace the existing construction increasing urban densification with the same amount of building space. Nevertheless, this option creates noise disturbances, waste material and pollution and it is harmful to the environment. Also, it destroys existing building that may become the architectural heritage of the city and urban fabric in the future.
Refurbishment
Refurbishment is the most sustainable of construction as it re-uses existing structures, and can preserve old buildings for the future. However, this solution is not always financially feasible and does not provide a solution for the growing needs of extra space in city centres.
Extension
Extensions on an existing building can be an alternative solution method, as they combine the benefits of demolition and refurbishment. Extensions can provide social, sustainable and economic benefits and a solution for urban densification [2]. They can reduce the environmental impact of conventional construction methods, as they avoid demolition of existing structures. They have the potential to preserve existing architecture and make use of the residual or hidden capacity of the structures. There are multiple technical complexities related to the existing building structure that may difficult the structural design of this type of construction.

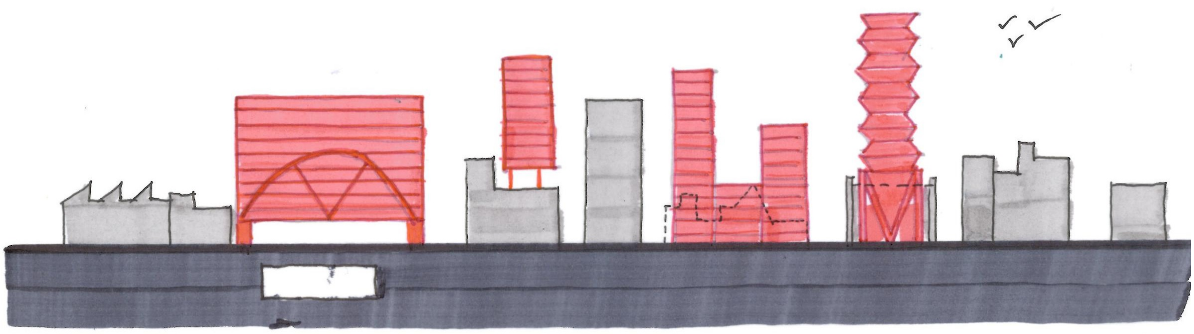


Figure 2.1: Urban densification possibilities [12].

2.1.2. Typologies

For vertical extension, several typologies can be distinguished to be used either separately or combined. Recent research [11] presented most of the following typologies that can be distinguished for small extensions (up to three levels) and large extensions. In figure 2.2 a schematic view can be seen.

Small extensions
A small extension on top of a building can often be build without major structural interventions and requires no demolition. Anchorage to the existing structure is easily integrated. A new *pop-up* achieves its structural stability and carries its load paths directly from and to the existing structure. This typology is only used for one or two additional levels.
When complete floor areas are extended, it is labelled as a *topping*. Small adjustments of the existing structure are required, which consists of refurbishments or demolition works. For ease of works, the existing structural grid is protruded to the new levels, keeping the same floor plans. This typology is mostly used for up to three additional levels.

Large extensions
When a high amount of floor levels is wanted, additional stability cores can be constructed. The floor plans can lend its stability to these new concrete cores. For unbraced frames this provides a situation in which the existing columns only have to take up the gravity loads and the lateral loads are directed to the core. Sometimes demolition works are necessary depending on the placement of the new core.
An *outrigger* structure connects the inner structure with the perimeter columns. This will create axial forces in the columns, generating an adverse moment compared to the moment from horizontal loads. Reduction of both the acting moment and of the deformations is the result. Demolition and refurbishment are consequences of using this typology. However, its increased construction levels are a benefit.
When using a *tube*-system, the whole building gets transformed to the core. The walls of the core coincide with the façade of the building. The building has become the core, and the core has become the building. Whether or not the existing perimeter is already capable of being strong and stiff to the desired extent, a new structure will arise.
Table-structures provide the possibility of designing a completely new floor plan without being limited by the modular sizes of the substructure. A transition structure will be placed between the old and new volume of the building, changing the grid sizes to the desired dimensions. A possible downside is that the loads will not be spread evenly amongst the existing building which could lead to differences in the settlement. Investing in such an intermediate structure is a case specific consideration, but the convenience in which it results for designing optimal floor plans is evident.

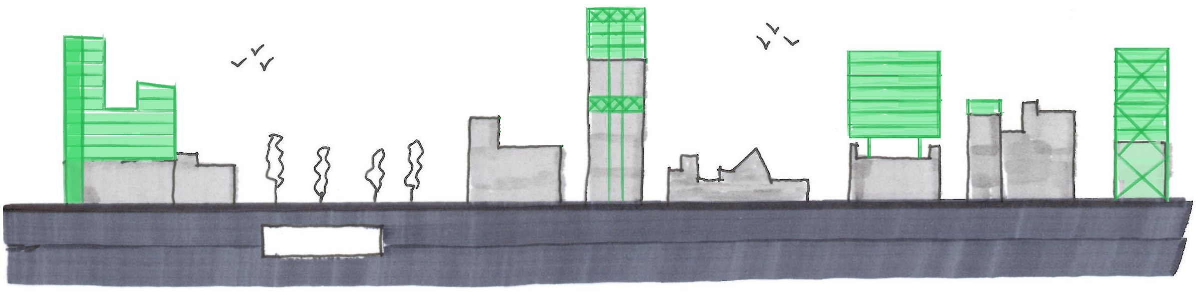


Figure 2.2: Urban densification typologies.

2.2. Vertical extension

The busiest part of an urban region is, as can be imagined, not the most suitable place for a new construction site. It is not just the amount of involved parties that make it difficult, such as the municipalities, residents and project developers, but also the regulations play an important role. This kind of projects often cover a high budget and a lot of value, making it complicated to come up with decisions benefiting every stakeholder. Strict boundary conditions regarding the construction management of such a project require good coordination. Aspects such as available space, timeframe restrictions, information traffic and logistics management should be integrated into the design from the start to reduce the nuisance for both local residents and bypassers. A three-phased assessment, as proposed for roof stacking [2], is the starting point when a vertical extension project is considered. This assessments is an on-going task that is repeated many times as design and construction proceeds. The following three phases describe the involved configurations.

1. Urban policy configurations
Strict policies and regulations from municipalities can counteract the need and potential for densification. Not only physical restraints are formulated but also cultural interventions have to be considered. Buildings in a city centre can become the heritage and limit the possibilities for interventions. These modifications should not interfere with the conservation of architectural qualities. The formulated restrictions come in the form of planning permissions, zoning laws and building code requirements and take into account maximum heights, daylight requirements, access to parking lots, etc [8]. With an ever-changing society that comes up with its own demands, flexibility in these policies becomes more and more important to adapt to future use.
2. Engineering configurations
Regarding vertical extension, the physical restraints of the underlying structure are of great importance. A structural analysis should provide insight into the residual or hidden capacity [2]. From this data, the potential of the stacking is revealed more deliberately. It will become apparent to what extent the structure is capable of bearing new elements. The information is retrieved either by calculations, if construction data is available, or by measurements. Investigations, of not only the building but also the soil and foundation, should give insight into how the conditions of the structure have changed over time. If needed, additional reinforcement has to be implemented.
3. Architectural configurations
A more detailed assessment will be made when architectural configurations are implemented. As soon as all disciplines from the different stakeholders come together, an integrated plan will be established. This leads to more real design options and gives more insight into the best principles and procedures that have to be used. The final approval for the vertical extension follows from these plans.

2.3. Light-weight building systems

The structural design of a building is the main bearing structure and links elements such as beams, columns, floors and walls. Its primary goal is to make sure the forces flow correctly to the foundation. Creating a light-weight building system requires the main permanent loads to be cut back as much as possible, without losing the structural integrity. These loads can predominantly be found in the substructure and the floors of a building. It is therefore clear that not all building systems are suitable for vertical extension kind of building systems without requiring too many interventions and adaptations in the existing structure. For example, concrete structures will add too much weight given their high mass properties and therefore limit the amount of levels to top the old building. In the coming sections the typical light-weight structural elements will be deliberated more into detail. This will range from the largest scale up to the detail level.

2.3.1. Substructures

In common vertical expansion projects several systems are used, however, most often *steel framing* is considered as most suitable [5]. This construction method uses linear steel elements such as beams, columns and bracing elements and is later on completed with finishing elements such as façades, floors and walls. By sep-

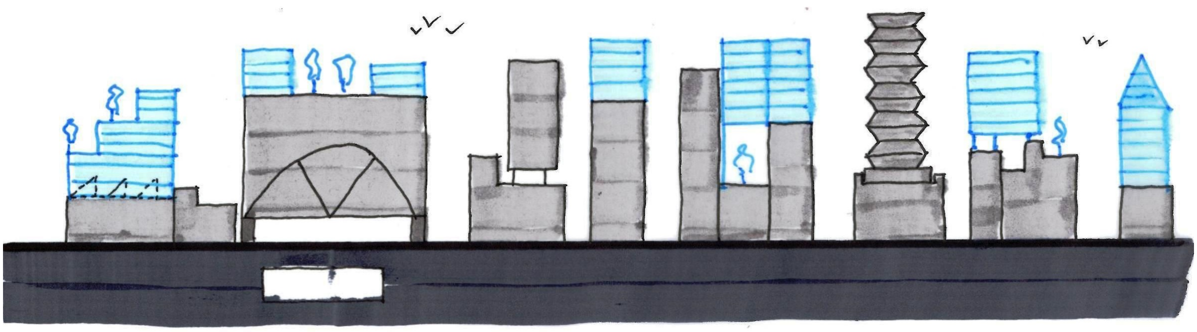


Figure 2.3: Urban densification vision [12].

arating the load bearing elements with the service elements, a high degree of flexibility is achieved, leaving room for future adaptations [1]. Because of the use of prefab elements, dimensional stability and the use of dry connections the speed of erection is very high. Steel framing makes use of its high strength capacity and its ductility in combination with its low self-weight. Susceptibility to fire is a big challenge when using steel structures. Good protection of the elements is necessary to prevent softening of the steel at high temperatures.
Other options for light-weight building systems are *wood light frame* construction and *light gauge steel* construction. These methods use plane like elements such as floors and walls to compose a construction frame. The prefab elements are commonly made in the measurements of the modular size of the frame. Before being transported to the construction site, the parts are already completed with isolation and a finishing layer in the workplace. Disadvantages of these systems are combustibility for the timber configuration and thermal bridging for the light gauge steel configuration due to the used cold-rolled steel profiles [1]. Both systems however also have reduced flexibility options since plate-like structures are used. So far only a few high-rise projects have been carried out using these configurations. However, more and more research is being done for future use [31].

2.3.2. Floors

Floor structures can be considered to be the central component for users of a building. Traditional floors were only designed to carry loads; therefore simple timber floors were common. With the developments of reinforced concrete new floor types became the standard. Additionally, the extra mass of the floors improved the acoustic insulation and vibration comfort. For light-weight configurations, this type of floor does not meet with the modern comfort requirements. Other floor systems have to be considered for these type of building designs:
Composite floors combine the advantages of both concrete and steel, creating composite action. It consists of cast-in-situ concrete with reinforcement, poured on corrugated steel sheets which are attached to steel beams using steel nails to achieve monolithic behaviour. The corrugated steel profiles enable a fast erection speed since multiple plates can be lifted at once [5]. Also for limited spans, no additional formwork is needed [5].
Typical *steel frame floors* are composed with cold-formed C or sigma profiles. The frames can be installed to the construction system just like in timber frame constructions. Often there is an isolation material between the profiles.
Nowadays there are new adaptations of these kinds of traditional floor systems. By combining positive aspects of different floor systems, innovative concepts can be created. Examples are the *Quantum floor* which behaves as a steel frame composite floor. Another alternative is the *Slimline floor* which consists of a prefabricated concrete plate on the bottom in which the flange of the steel profile is casted. The beams are provided with openings for cables and piping systems. The *Ides floor* is the most lightweight flooring system and combines this with minimal height. Within this construction height margin of the floor, all needed systems are integrated [5].
Timber floor systems are typically constructed with timber joists and wood-based sheathing. From a structural point of view, a timber floor can be treated as a two-dimensional thin plate structure semi-rigidly connected with a series of parallel joist members. The modern-day joists are either made from sawn timber or engineered wood products. Also, I-joists and open-web joist can be found on the market nowadays. Since the span to depth ratio is usually very large, the serviceability requirements are often more governing than the strength capacities.
People are in constant contact with floors, either moving or stationary. This makes the flooring system an essential element where comfort and structural integrity have to be combined. Occupants are most susceptible to annoyance caused by dynamic movements produced by human activity such as walking and running [24]. Small flaws in the design of the floors can cause the performance not to meet its standard.

2.3.3. Junctions

The intersection point of primary and secondary elements can produce a significant influence on the structural design. In here not only the flow of forces is transmitted but also comfort aspects such as acoustics and vibrations have to be taken into consideration. Since these junctions often enclose the boundary between separate rooms, it is an ideal place to deal with the strict regulations regarding energy transmittance to adjacent rooms.

The key element within the junction is the structural element, i.e. the supporting beam. This part takes care of all weight supported on top of it and has to remain strong and stiff enough for the whole lifetime of the building. Typical secondary structural elements such as the floors and walls are connected to this beam and carry out a certain dead weight. By adjusting the location and type of coupling, characteristics of the junction can be tweaked. Assembling the junctions with the correct boundary conditions is a careful job and may play a big role in the level of comfort for acoustics and vibrations [17].
Schematic representations of light-weight building junctions are composed by TNO [33], see figure 2.4. These most common used details in low rise vertical extensions show a graphic display of the combination of a substructure with several type of floors. The way of assembling the various elements will be further elaborated in the following chapters. In here it will be discussed how the type of beam, force flow and the rigidity of the connections influence the buildings behaviour.

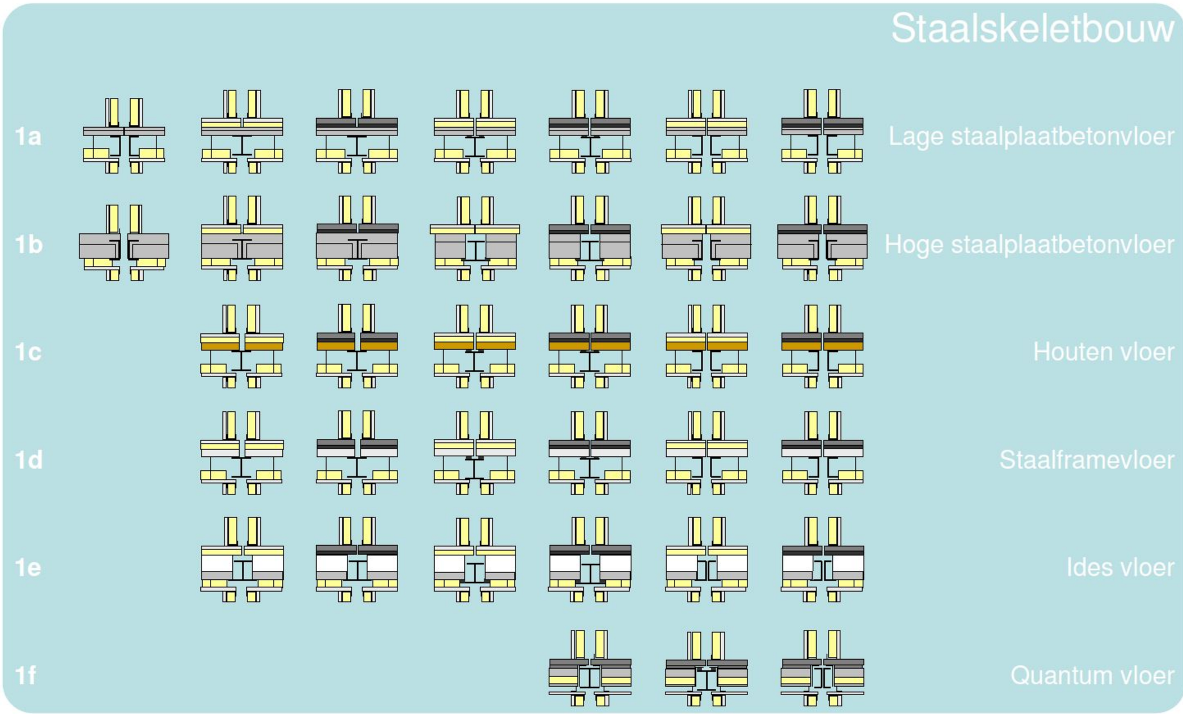


Figure 2.4: Steel framing with light-weight floors schematic junctions [33].

2.4. Demands and requirements

Vertical extension is a way of building that requires careful treatment. As mentioned before a structural integer building should be provided for the long term. This means not only engineering a safe construction but also acquiring the possibility to adapt to future market demands. The most critical aspects for these type of projects can be split up in the two following groups:

2.4.1. Building engineering aspects

Weight and mechanical properties
Since the potential for additional mass volumes strongly depends on the load-bearing capacity of the existing structure, weight should be minimised as much as possible. However, there is a clear correlation between the weight and mechanical properties of materials. Distributing the mass and its properties smartly can still create a high-quality system which is capable of fulfilling its demands avoiding a massive structure.
A combination of steel and timber materials is commonly used for the structure of vertical extension buildings. Even though steel has a high density, its ability to achieve light-weight together with high mechanical properties follows from its thoughtfully shaped cross-sections. This significantly reduces the weight of the product. The use of timber in subsystem components creates excellent advantages in reduction of mass and besides creates a more sustainable product while reducing carbon emission. It, however, does leave challenges for the acoustic and vibration comfort levels.

Fire resistance
Every building has to provide a minimum safety level concerning fire regulations. This performance is defined by the reaction of materials with extreme heat and fire. Even though steel is not flammable, its mechanical properties will decrease when exposed to these circumstances. As opposed to timber, where while increasing the combustion rate it does not lose its mechanical properties. Packing the elements is a solution to increase the resistance to fire and heat, but has to be done with care.
Acoustic performance
Lightweight materials tend to give little resistance to the transfer of sound pressures. The acoustic performance can be critical for internal and adjacent units both horizontal and vertical. Layered design of walls, including sound insulation and cavities, can reduce the sound transfer. Also, special flooring systems and ceiling designs can contribute to a more comfortable living environment.
Vibration comfort
A typical light-weight construction issue is the level of vibration comfort. While in traditional building systems mass was able to dissipate the energy produced by walking on floors, in light-weight buildings this is not the case. The mass, stiffness and damping levels of the construction have to be tweaked to tackle this problem. Just so the floor its behaviour does not correspond with the dynamic loads imposed by human activities.
Thermal performance
Thermal control of the indoor climate can be dealt with in different ways. Steel and timber have a poor thermal resistance value, letting through heat loss easily. Therefore additional insulation materials have to be applied to create a comfortable indoor environment. Also the thermal mass of the light weight building products creates new challenges. Active regulation of the temperature is mostly done by absorbing and storing heat, which is then used to regulate the temperature during day or night time. In general, heavier building materials are more suitable for this.

2.4.2. Practical aspects

Logistic
These specific vertical extension typologies have to be constructed in the hectic urban environment, that doesn't accept nuisance for too long. Speed in transport, lifting and assembling is therefore of high importance, but also the supply has to take place in a short amount of time and there is little room for storage. It is better to solve this logistic puzzle as soon as possible and adapt it in the design of a building system.
Industrial
By manufacturing a design that is not bounded to a single project, an improved typology can be created for roof stacking. This means standardized design guidelines that can be applicable to all likewise projects. In order to make sure the elements can easily be replaced or repaired a modular kind of design is desirable. By doing so it is possible to create a repeatable production process. The parts of such a system should be designed under controlled circumstances to achieve high quality. Ideally a minimum number of elements is used and assembly procedures should be kept simple.
Flexible
Freedom of design is becoming of more interest for both architects and engineers with the ever changing demands on the building market. Adaptability during the entire design and use process is becoming a key matter. Freedom of changing the function and layout of building requires engineers to come up with a new way of thinking. Incorporating all these elements should however lead to a building that can fulfil the demands for users on the short and the long term. A good example is the positioning of load bearing elements such as columns or separating walls in the floor plan, but also the incorporation of MEP in flooring systems. It makes it difficult that it is unknown how future developments will work out and influence the use of the building.
Demountable
A demountable structure has strong links with a flexible way of building. By (re)using demountable elements and materials from other buildings without alterations, a sustainable product can be designed. A typical construction method for this type of configuration is to use dry connections.

2.5. De Karel Doorman

In the early years of the 21st-century plans for increasing the housing supply in the form of vertical extension in the city centre of Rotterdam were made. The old *Ter Meulen* building, which was built shortly after the second world war in the destroyed Rotterdam area, was considered a viable option given its structural assembly of columns and beams, and its lack of structural walls. During the original design phase, plans for expanding the building with a single floor were already taken into account in the structural design. Modern techniques showed that it would be possible to extend the building with not just one or two floors, but with a total of sixteen new stories. This had to be done while keeping the substructure as untouched as possible, but optimising the use of the existing load-bearing system. The following topics were the main concepts used to achieve the extension [12]:

- Analysing the current building system and revealing hidden load bearing capacities
- Separating the horizontal and vertical loads
- The use of a light-weight vertical extension building system

Applying the techniques as mentioned above in combination with a demanded sixteen stories new building resulted in a maximum floor weight of 250 kg/m^2. Compared to standard Dutch concrete apartment buildings this is only 1/5th of the normal applied weight. To achieve this extreme low weight a steel substructure was used in combination with timber floors and plasterboard walls, see the image below.

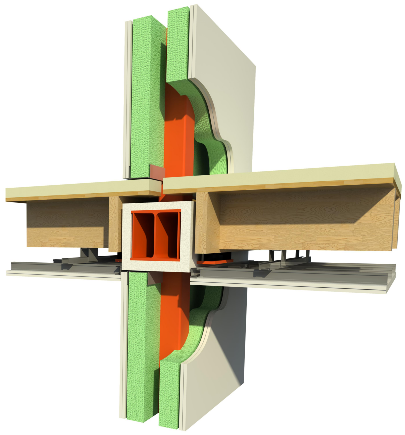


Figure 2.5: Segment of ultra-light building concept from De Karel Doorman [12].

By stretching the limits of weight within the building system, new challenges arose, as was experienced during the execution of the vertical extension. After constructing the first few stories, it was observed that the floors were easily vibrated just by walking across them. Although the construction was not finished yet, it was suspected that the perceptible vibrations would appear stronger than expected. Especially the transmittance to adjacent apartments went easier than expected. The question arose whether the vibration behaviour of the apartments would meet the expectations of the high-quality market.
Tests proved that the individual elements (timber floors, rubbers and steel construction) reached their calculated natural frequencies. However, when these different components were combined the behaviour did not meet its target any more, resulting in a vibration susceptible arrangement [35].
Enhancements to the vibration behaviour during an already started erection process led to improvised structural modifications that were unforeseen when the initial design was made. This meant using a bidirectional joist system in the floors, introducing more stiffness in both the span direction and transverse direction. Also, slender steel columns were inserted between home-separating beams in combination with welding extra steel plates to these beams. This was done to influence the bending and torsional behaviour of the joint and hence reduce the transmittance of the vibration to neighbouring units.

These and more adjustment made the building system meet its target values regarding vibration comfort and thus becoming an acceptable structure. Following this issue, guidelines regarding the transmittance for sending and receiving floors were drafted revealing the critical serviceability limitations for light-weight designs. A further developed structural concept is requested that prevents the need for on-site interventions and already encounters the normative vibration comfort demands during early design stages.

3
Literature research

3.1. Floor vibration

A general topic of interest when designing light-weight building systems, as also followed from the 'De Karel Doorman' case, is the level of vibration comfort. Where in common rules of thumb it is perceived that vibration is proportional with the movement of mass, it is now the challenge how to steer this vibration energy induced by human activities when the effect of mass is reduced.

The general procedure for determining the comfort level in buildings imposed by human dynamic loading is based on the three main components involved with vibrations, namely the source, the path and the receiver.
1. The loading induced by human activity can be displayed in a load-time function, that shows the load as a function of the time. The flow of this function depends on the type of activity and personal characteristics such as body weight and step frequency and is weighted according to its statistical demographic distribution. These characteristics define the *source*.
2. The *path* involves the structural elements which dissipate the energy produced by the dynamic loads. The response can be modelled in Single Degree of Freedom (SDOF) models for simple analysis and Multi Degree of Freedom (MDOF) systems for more complex structures. Each element within a structure has its significant damping, modal mass and natural frequency, which will prove to be of interest when the step forces are applied. Within the building system, it is possible to tweak with the properties and find out which are of importance when a certain level of vibration comfort is desirable. This makes the path the most critical variable in vibration comfort designing.
3. Vibration comfort is a subjective measurement. To make it measurable for general human perception several (heavy-weight floor design) guidelines exist to describe limit states [15][24][32][40]. In this thesis the One Step Root Mean Square (OS-RMS$_{90}$) is used from the SBR-guideline [36] as it can also describe a comfort level for neighbouring floor fields and is the most complete [22]. This OS-RMS$_{90}$ value indicates the level of acceptance from vibrations for the *receiver* for different types of buildings and their functions.

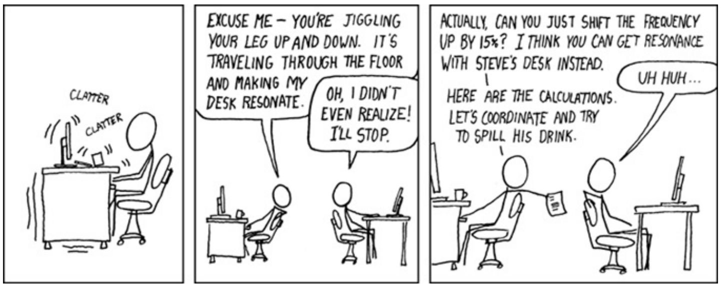


Figure 3.1: Abstraction of involved components during vibration [3].

For the fundamental abstraction of elements that participate in floor vibration, it is clear that the human interaction part cannot be influenced. It is impossible to ask people to walk faster or slower, or to make them less sensitive to the vibrations. Therefore the highest priority for optimising the comfort level lies within the scope of the path, i.e. the building structure. Since this path is still affected by the source and decides the outcome for the receiver, all three topics will be explained more into detail in the following sections.

3.2. Source

People can perform all kind of activities that will excite a floor. The most likely type of activity that induces dynamic loads within apartment buildings, however, is walking. The characteristics of walking functions will be discussed more in detail to ensure that new light-weight building systems adapt to these forces. It has to be noted that more intense activities such as jumping, running or aerobics will cause other dynamic loads. These will also be discussed in the upcoming subsections.

3.2.1. Walking

As mentioned before, walking can be considered the most important activity to take into account when designing residential buildings. In general, it can be stated that multiple persons rarely walk in the same phase. The vibrations induced by more than one person can thus both be stronger or weaker in comparison to a single person. Therefore only the vibrations caused by a single person will be taken into account when considering walking forces.

A velocity-time history of a person walking, including multiple steps, is shown in figure 3.2. Since the contact forces are quite periodic it is possible to only use the time history of the contact force of a single step and to describe this force-time in a normalised way.

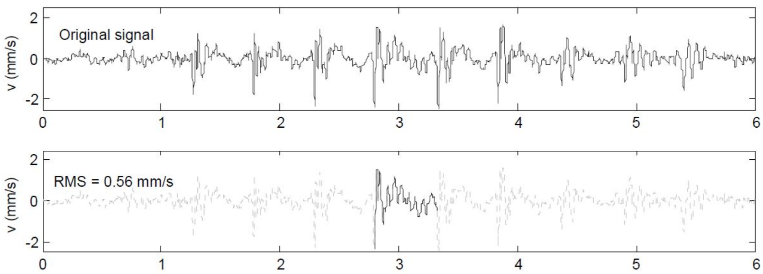


Figure 3.2: Velocity-time response of a floor imposed to walking loads for complete path (up) and one step (down) [9].

The way a floor is excited strongly depends on the type of person that walks over the structure. The most important characteristics that influence the level of excitement are the pace someone is walking in and their body weight. In figure 3.3 a typical normalised load-time function of a single step is illustrated for two different step frequencies.

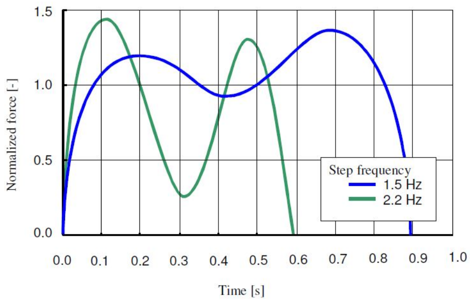


Figure 3.3: Load-time function of a single step for certain step frequencies [9].

This one step can be described by the polynomial function 3.1 [9]. In this function G is the mass of a single person (G = 40-125 kg). The coefficients K_1 - K_8 depend on the step frequency (f_s) according to table 3.1. The step load can be obtained by multiplying the normalised step load with the mass (G) of one person.

$$\frac{F(t)}{G} = K_1 t + K_2 t^2 + K_3 t^3 + K_4 t^4 + K_5 t^5 + K_6 t^6 + K_7 t^7 + K_8 t^8 \tag{3.1}$$

Table 3.1: Coefficients for determining the one step load [36].

	$f_s \leq 1,75 Hz$		$1,75 Hz < f_s < 2 Hz$		$f_s \geq 2 Hz$	
K_1	-8*f_s	+38	24*f_s	-18	75*f_s	-120,4
K_2	376*f_s	-844	-404*f_s	+521	-1720*f_s	+3153
K_3	-2804*f_s	+6025	4224*f_s	-6274	17055*f_s	-31936
K_4	6308*f_s	-16573	-29144*f_s	+45468	-94265*f_s	+175710
K_5	1732*f_s	+13619	109976*f_s	-175808	298940*f_s	-553736
K_6	-24648*f_s	+16045	-217424*f_s	+353403	-529390*f_s	+977335
K_7	31836*f_s	-33614	212776*f_s	-350259	481665*f_s	-880037
K_8	-12948*f_s	+15532	-81572*f_s	+135624	-174265*f_s	+321008

The load duration T_s of a single step is as follows:

$$T_s = 2,6606 - 1,757 f_s + 0,3844 f_s^2 \tag{3.2}$$

A different, but similar, way of expressing the step load function over time is by using a Fourier series, as in formula 3.3. In here a series of sine waves, each with its specific frequency, amplitude and phase shift composes the actual function. In this function G is the static load imposed by a single person. The dynamic load factor of the n-th harmonic is described by α_n, and the ϕ_n indicates the phase lag. The values for the first four harmonics can be found in table 3.2. The contribution of each harmonic is more clearly indicated, showing that not only the lowest frequency range is of importance, but also higher ones can contribute to the vibrational behaviour of a floor.

$$F(t) = G\Big(1 + \sum_{n=1}^{4} \alpha_n \sin(2\pi n f_s - \phi_n)\Big) \tag{3.3}$$

Table 3.2: Coefficients for determining the one step load [32].

activity		harmonic			
		n=1	n=2	n=3	n=4
walking	α_n	0,46	0,10	0,08	0,07
	ϕ_n	0	-$\pi/2$	π	$\pi/2$
	f_s	1,6-2,2Hz	3,2-4,4Hz	4,8-6,6Hz	6,4-8,8Hz

It can be seen that the maximum walking harmonic frequency is approximately 8,8 Hz. Higher contributions to walking load functions are unlikely to play a part in the vibration response of a floor. Engineering a floor with a lower natural frequency, a so-called 'low-frequency floor' however will make it likely that resonance occurs. This phenomenon will be explained more in detail in the following section.

3.2.2. Other activities

Different types of activities cause different kind of load time functions, as can be seen in figure 3.4. In here it shows that for example during walking there is a constant contact of the foot with the floor whereas during running there is not. Also, the normalised weight is higher for more intense activities. Therefore each activity has its specific definition. The peaks for a single step in most load-time functions correspond to the contact forces of a footstep. The first peak hits when the heel drops down on the floor whereas the second peak strikes when the foot is putting pressure on the floor to take off.

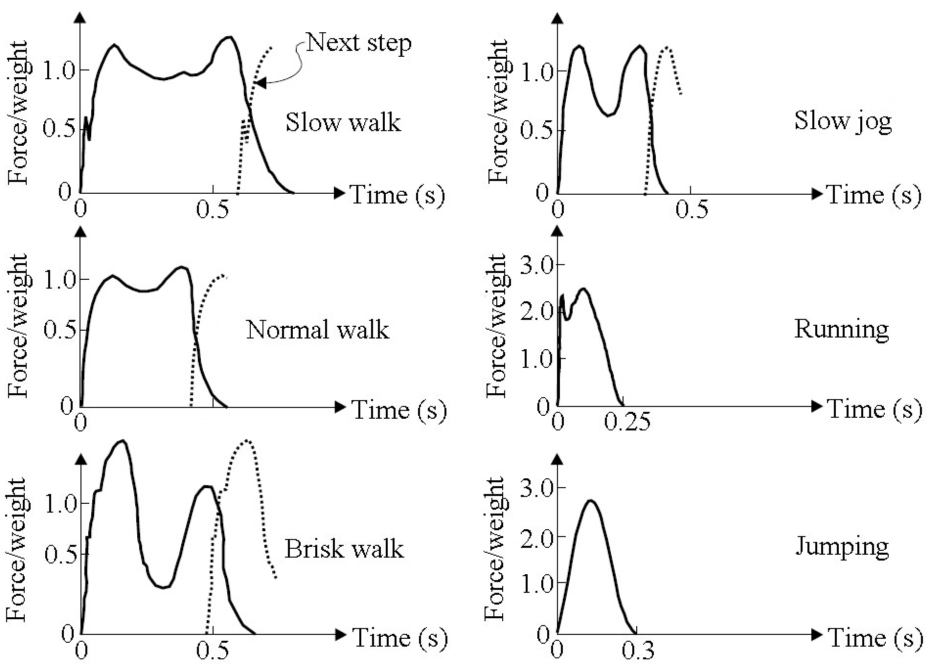


Figure 3.4: Load-time functions for different activities [38].

For loads imposed by aerobics and jumping people formula 3.4 can be used to obtain the load function [36]

$$F(t) = Q\Big(1 + \sum_{n=1}^{\infty} \alpha_n \sin(2\pi n f_s + \phi_n)\Big) \tag{3.4}$$

In this function, Q is the static load imposed by a crowded mass (Q = 0,8-1,2 kN/m^2 assuming one and a half person on one square meter). The dynamic load factor of the n-th harmonic is described by α_n and the ϕ_n indicates the phase lag. Both depend on the activity to which the floor is excited. These values for several harmonics can be found in table 3.3. Also, the step frequency f_s is dependent on the activity and size of the group of people. For individuals a frequency f_s of around 1,5-3,5 Hz can be found while for larger groups this frequency lies around 1,5-2,8 Hz [32].

Table 3.3: Coefficients for different activities [36].

activity		harmonic					
		n=1	n=2	n=3	n=4	n=5	n=6
low-impact aerobics	α_n	1,286	0,164	0,133	0,036	0,023	0,032
	ϕ_n	-π/6	π/6	-π/2	-π/6	π/6	-π/2
high-impact aerobics	α_n	1,570	0,667	0,000	0,133	0,000	0,057
	ϕ_n	0	-π/2	0	-π/2	0	-π/2
normal jumping	α_n	1,800	1,286	0,667	0,164	0,099	0,133
	ϕ_n	π/6	-π/6	-π/2	π/6	-π/6	-π/2
high jumping	α_n	1,866	1,571	1,132	0,667	0,269	0,000
	ϕ_n	π/4	0	-π/4	-π/2	π/4	0

The most important part of the load function is the natural frequency and the participating dynamic load. Whereas walking has its first harmonic around 2 Hz, the higher harmonics up to around 8 Hz can also contribute to the vibration impact. Therefore it is more convenient for residential buildings to produce floors with natural frequencies higher than 9 Hz. Cases in where the mass is reduced however, such as vertical extension projects, make it difficult to achieve this value.

3.3. Path

A typical building structure can be abstracted to a discrete system in where the concerned masses act independently. These systems can be modelled consisting of the following three elements: point masses with a mass m, springs with a stiffness k and dampers with a damping coefficient c. Simple systems include only one mass and can be solved more easily. This method is used to find the natural frequency for each mode of a continuous system. Interaction of multiple masses and springs increase the degrees of freedom, together with the complexity of finding the solution. Below the basics of both systems are explored to find the tools for improving the system.

3.3.1. Single Degree of Freedom

For understanding the dynamic response of a floor the basic principles of vibrations are described [17]. A Single Degree of Freedom (SDOF) model is used, which can be seen in figure 3.5, and simulates the response of a floor induced to human activities.

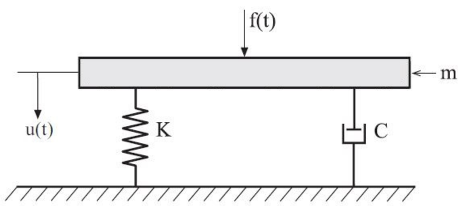


Figure 3.5: Mass-spring-damper SDOF system.

When a force is applied to a SDOF system, it will move from its equilibrium position and displace over time according to a harmonic vibration. Two types of vibrations are distinguished: free vibrations and forced vibrations. The first describes a system in which the mass is placed out of equilibrium without an additional force, and the latter contains a constant applied force. The SDOF system includes several forces that influence its behaviour, namely from the mass (F_m), from the spring stiffness (F_k) and from a damping element (F_c). Considering Hooke's Law ($F_k = ku(t)$) a restoring force, depending on the displacement $u(t)$ and stiffness k, will try to put the system back in equilibrium. This force will cause the system to accelerate, which from Newton's Second Law ($F_m = m\ddot{u}(t)$) is proportional to the mass m of the element. Eventually, the damping of the system c will cause the vibration to decrease over time, as a response to the velocity ($F_c = c\dot{u}(t)$). The magnitude of these elements can be enhanced by an external applied load $F(t)$. The sum of all these properties of a SDOF system can be described in the following equilibrium formulas:

$$F_m + F_c + F_k = F(t) \tag{3.5}$$

$$m\ddot{u} + c\dot{u} + ku = F(t) \tag{3.6}$$

As can be seen, this so-called equation of motion consists of three internal system forces and an applied external force $F(t)$ that need to be in equilibrium. F_m defines the inertial force and is proportional to the mass of the system put into motion by its acceleration. On the other hand, the damping force F_c depends on the velocity of the system. The force produced by the stiffness and displacement of the system F_k complete the internal forces. An external force equal to zero describes a free vibrating system.

Free vibration
The equilibrium for a free vibrating system can be derived from equation 3.5 and is expressed as follows:

$$m\ddot{u} + c\dot{u} + ku = 0 \tag{3.7}$$

From this equation it is possible to obtain more information about the influence of damping by solving it into the form of $u(t) = e^{rt}$. The partial solution that follows is described in the next formula:

$$r_{1,2} = \frac{-c \pm \sqrt{c^2 - 4km}}{2m} \tag{3.8}$$

The outcome of the discriminant affects the solution of the equation. Critical damping is found when the discriminant is put equal to zero ($c_{cr} = 2\sqrt{km}$) and means that the system will return to its equilibrium in the minimum amount of time. For determining the influence of damping within a system, it is essential to find out how the damping factor relates to this critical damping. For this thesis the damping ratio (see equation 3.9) of structural elements are assumed to be underdamped, giving a ζ factor smaller than one.

$$\zeta = \frac{c}{c_{cr}} \tag{3.9}$$

When solving the free vibration equation of motion a complex expression follows if the damping ratio is underdamped. The displacement u can, however, be rewritten into a real solution as a function over time.

$$u(t) = \underbrace{u_0 e^{-\zeta\sqrt{\frac{k}{m}}t}}_{damped\,amplitude} \underbrace{\sin\left(\underbrace{\sqrt{1-\zeta^2}\sqrt{\frac{k}{m}}}_{frequency} t - \phi\right)}_{vibration} \tag{3.10}$$

From here it can be seen that the deflection u is dependent on its initial conditions such as the initial displacement u_0 and initial phase ϕ. But also the system elements such as mass, stiffness and damping influence the deflection over time.
The natural frequency ω_0 of the system can also be found in formula 3.10. This is the frequency to which a system tends to oscillate when it is displaced from its equilibrium and subsequently released. Since most normal structures have a low damping ratio the natural frequency can be rewritten into the following equation:

$$\omega_0 = \sqrt{1-\zeta^2}\sqrt{\frac{k}{m}} \approx \sqrt{\frac{k}{m}} \tag{3.11}$$

Concluding, the natural frequency of a free vibrating system depends on its stiffness and mass. Since it is easier to work with frequencies expressed in Hertz as opposed to radians, the formula can be rewritten into the following one:

$$f_n = \frac{\omega_0}{2\pi} = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{3.12}$$

Forced vibration
When human-induced vibrations are considered, we are dealing with a continuously forced vibration. This external force $F(t)$ can be assumed to be as follows:

$$F(t) = F_0 \sin(\omega t) \tag{3.13}$$

First, it is possible to write the harmonic response of a forced vibration into the following form for deflection over time:

$$u(t) = \frac{F_0}{\sqrt{(k-\omega^2 m)^2 + (c\omega)^2}} \sin(\omega t - \phi) \tag{3.14}$$

Rewriting the equation, incorporating $\omega_0^2 = k/m$ and $c = 2k\zeta/\omega_0^2$, will result in the following formula:

$$u(t) = \underbrace{\frac{F_0}{k}}_{static\,deflection} \underbrace{\frac{1}{\sqrt{\left(1-\frac{f_s^2}{f_n^2}\right)^2 + \left(2\zeta\frac{f_s}{f_n}\right)^2}}}_{dynamic\,modification\,factor} \underbrace{\sin(\omega t - \phi)}_{vibration} \tag{3.15}$$

The formula shows some clear factors that influence the deflection over time. First of all the static deflection, defining the amplitude of the vibration, is affected by the magnitude of the force and its stiffness. A dynamic modification factor that consists of the damping ratio and the ratio of the forcing and natural frequency determines to what extent the amplitude will be increased. Finally, the period depends on the frequency.

3.3.2. Multiple Degree of Freedom

Since it is often the case a structure does not solely exists out of a floor but is a combination of multiple connected elements, a higher degree of freedom system has to be used. This so-called Multiple Degree of Freedom (MDOF) system, generates a more realistic view of a building system [26].

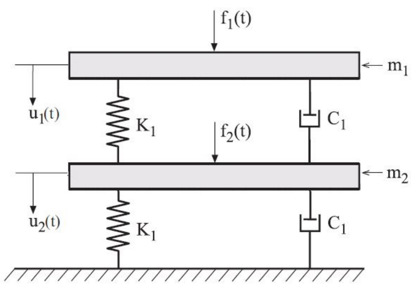


Figure 3.6: Mass-spring-damper MDOF system.

As goes for this system the equation of motion formula is computed in matrix form. This means that for each free body a separate equation can be established which will then be merged into a matrix. For the above depicted Two Degree of Freedom system this leads to the following equilibrium formulas:

$$m_1 \ddot{u}_1 + c_1 \dot{u}_1 + (k_1 + k_2) u_1 = F_1(t) \tag{3.16}$$

$$m_2 \ddot{u}_2 + c_2 \dot{u}_2 + k_2 u_2 = F_2(t) \tag{3.17}$$

Rewriting this in matrix form leads to the following equation of motion:

$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \begin{bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{bmatrix} + \begin{bmatrix} c_1 + c_2 & -c_2 \\ -c_2 & c_2 \end{bmatrix} \begin{bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{bmatrix} + \begin{bmatrix} k_1 + k_2 & -k_2 \\ -k_2 & k_2 \end{bmatrix} \begin{bmatrix} u_1 \\ u_2 \end{bmatrix} = \begin{bmatrix} f_1 \\ f_2 \end{bmatrix} \tag{3.18}$$

In more general form, for a multiple (n) degree of freedom system, the following formula can be proposed:

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = f(t) \tag{3.19}$$

In where M $(n \times n)$ stands for the mass matrix, C $(n \times n)$ for the damping matrix, K $(n \times n)$ for the stiffness matrix, f $(n\times1)$ is the external vector containing the dynamic external forces and u is the displacement vector $(n\times1)$.
Similar to the SDOF approach it is possible to analyse the natural frequencies of MDOF systems when a free vibration case ($f = 0$) is used for a system without damping ($C = 0$). The equation of motion for this free vibration MDOF system is:

$$M\ddot{u} + Ku = 0 \tag{3.20}$$

By setting the initial conditions of the displacement and acceleration to respectively $u(0) = u_0$ and $\dot{u}(0) = \dot{u}_0$, the movements of the masses can be found. These displacements will not follow a simple harmonic, and the deflected shape will change as the ratio of displacement of the individual masses varies over time.
There are specific initial displacements however that will result in harmonic vibrations, called mode shapes. The number of modes is equivalent to the number of degrees of freedom and each mode has a corresponding natural frequency. The lowest natural frequency is known as the fundamental natural frequency and is indicated as ω_1.
For describing the free dynamic deflection over time for a MDOF system, it is possible to formulate this with the following equation for a particular mode shape n:

$$u(t) = \phi_n(A_n \cos(\omega_n t) + B_n(\omega_n t)) \tag{3.21}$$

In this expression of the harmonic function, A_n and B_n are constants based on the initial conditions. The unknowns being the natural frequency ω_n and modes of vibration ϕ_n. Substituting equation 3.21 into the equation of motion (3.20) results in:

$$[-\omega_n^2 M\phi_n + K\phi_n]q_n(t) = 0 \tag{3.22}$$

By solving the matrix eigenvalue problem the natural frequencies and matching modes of vibration that satisfy this problem during movement can be found. If the stiffness matrix K and mass matrix M are known it is possible to find the corresponding values for ω_n^2 and ϕ_n by satisfying the following equation:

$$[K - \omega_n^2 M]\phi_n = 0 \tag{3.23}$$

The determinant of this equation is a polynomial of the n-th order (corresponding to the n number of degrees of freedom) with regard to ω_n^2. It is called the frequency equation and has n positive and real roots, given a symmetric and positive stiffness and mass matrix. Each of these roots corresponds to a natural frequency and for each of these values, a matching natural mode shape can be found.

It is found that for MDOF systems there is a relation between the participating elements. This means that the stiffness and mass of single elements can affect the vibration behaviour in a coupled system. For a higher degree of freedom system it is not that simple to find a quick solution that shows the harmonic response over time. Often finite element software is used that can perform such calculations. These programs are capable of finding the natural frequencies of the system and the deflection over time. Just as mentioned at the SDOF system the factors that influence the vibration response of a floor do not change, although now they are related to each other. From the formulas it follows that the following aspects will influence the outcome and thus the vibration comfort:

- Damping ratio
- Modal mass
- Natural frequency

In the following section these factors will be elaborated more into detail. For the damping, modal mass and natural frequency their contribution into structural components will be discussed.

3.3.3. Theoretical aspects

Discussing the actual matters that influence the floor systems will lead to design rules for improved performances. As stated before the damping, modal mass and natural frequency are of high importance when the comfort level needs to be increased. Finding out how these properties influence the frequencies and amplitudes of vibrations and subsequently the structural behaviour of systems should lead to more insight for further enhancements.

Damping

The damping capacity of a structure makes the induced vibrations reduce (and eventually stop) over time due to the dissipation of the energy and transfer of the vibration via joints to adjacent structures. Both the internal friction of a material and external friction to other elements can cause the damping of the system. For example, the furniture in a room will start vibrating themselves and remove energy. Damping is a complex mechanism within structural designs, and it is hard to predict what exactly causes damping and to what level. Although there are some analytic ways to calculate the level of damping, it is mostly determined based on actual measurements from tests [21].

As discussed in the theory section the level of damping has a strong influence on the magnification of the amplitudes of vibrations. The dynamic magnification factor (DMF), as defined in equation 3.15 is described as followed:

$$DMF = \frac{1}{\sqrt{\left(1-\frac{f_s^2}{f_n^2}\right)^2+\left(2\zeta\frac{f_s}{f_n}\right)^2}} \tag{3.24}$$

The ζ-factor indicates the percentage of critical damping. This is the amount of damping that is required to return the system to its equilibrium position without any oscillation in the minimum time. In the figure below the influence of several damping factors concerning the DMF are set out.

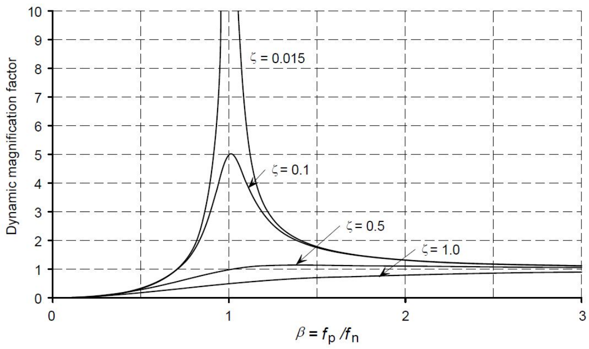


Figure 3.7: Dynamic magnification factor for different damping levels [32].

Each line represents a specific ζ-factor. It can be seen that especially around the area where the ratio between the load frequency (f_s) and natural frequency (f_n) is close to one, described as the β factor, which is more common for light-weight structures, the magnification of the vibration amplitude has a strong peak. Most structures have a relatively low ζ-factor, which is approximately 0.01 for steel and 0.06 for timber materials [13]. Additional elements that can contribute to a higher damping ratio are the furniture in a room and finishing elements in a room such as the ceiling or a floating screed. However, it is most unlikely that the total value of the damping ratio will reach far above 0.10 and will not magnify the vibrations.

Natural frequency

As can be seen in equation 3.24 the dynamic modification factor is not only dependent on the damping ratio but also on the ratio of the load frequency and the natural frequency. Since there is little to no possibility to influence the load frequency, modifications to the natural frequency of the structure are of high interest. This natural frequency is defined as a measure of the rate at which the system vibrates. In general terms the expression for the natural frequency can be described as followed:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{3.25}$$

From which it can be concluded that the stiffness, dependent from the boundary conditions, geometry and mechanical properties, and the mass of the structure are the key elements for determining this value. In more practical form this formula can be rewritten into equation 3.26. This expression holds for both beams and similar one way span structures such as floors [18]. In this formula the EI_L indicates the dynamic flexural rigidity, L the span, m the effective mass and κ_n a constant representing the support conditions for the n-th mode of vibration.

$$f_n = \frac{\kappa_n}{2\pi}\sqrt{\frac{EI_L}{mL^4}} \tag{3.26}$$

Since the stiffness of an element is conditional to its boundary conditions there will be a difference between hinged and fixed end conditions. Some standard values for different support conditions can be found in the table below.

Table 3.4: Coefficients for uniform beams [32].

	κ_n for mode n		
support conditions	n=1	n=2	n=3
pinned/pinned	π^2	$4\pi^2$	$9\pi^2$
fixed/fixed	22.4	61.7	121
fixed/free	3.52	22	61.7

For a floor system composed of multiple elements, such as primary beams, secondary beams and floor slabs the natural frequency can be found by using Dunkerly's approximation, see equation 3.27. In here the natural frequency of each element contributes to the complete system.

$$\frac{1}{f_{system}^2} = \frac{1}{f_{floor}^2} + \frac{1}{f_{beam}^2} + ... \tag{3.27}$$

For each natural frequency of a structure there is a matching mode shape, see figure 3.8. This shape will show the normalised displacement and can be expressed in a sinusoidal form for simply supported beams. Formula 3.28 shows the normalised amplitude at any position along the beam for each n-th mode multiplied with a time-varying amplitude function to give the actual displacement for each frequency at any time t. This results in the displacement u_n for varying time and locations. By superimposing all the mode shapes, it is possible to find the actual form of the complete system.

$$u_n(x,t) = \sin\left(\frac{n\pi x}{L}\right)\sin(2\pi f t) \tag{3.28}$$

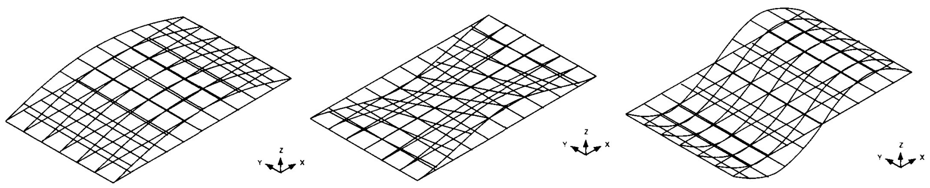


Figure 3.8: First three mode shapes for a two sided supported floor [24].

Modal mass

For each mode shape there is a certain amount of mass involved, the modal mass M. For exciting a high modal mass a lot of energy is needed and vice versa. It is clear that a high mass of a particular mode results in low participation to the overall vibration of the system. The following formula shows how the modal mass can be acquired for a floor supported on four sides [18]:

$$M_n = \iint_A m\phi^2 dA = \int_{x_{min}}^{x_{max}} \int_{y_{min}}^{y_{max}} \sin\left(\frac{n\pi x}{L}\right)\sin\left(\frac{n\pi y}{B}\right) m(x,y)\,dy\,dx \tag{3.29}$$

In this equation A holds for the area of the floor and m stands for the uniform mass per unit. In figure 3.9 it can be observed how the mode shape for orthotropic fields are combined for the x- and y-direction.

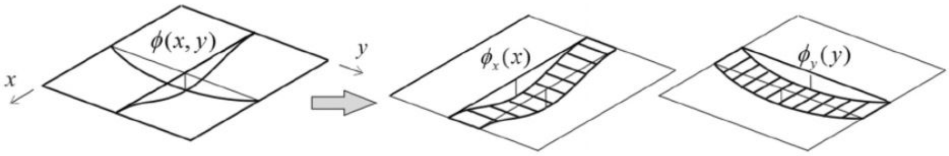


Figure 3.9: Approximated mode shape of orthotropic floor supported on four sides [18].

3.4. Receiver

There is a distinct relation between the load impacts induced by human activities and the response of these on structural systems. The reactions that walking people produce (footstep force) on a force scale are dominant in the low-frequency domain. Most common step frequencies lie around 2 Hz, with some minor contributions up to 8-10 Hz (see the left on figure 3.10). The mobility (velocity per force) is the transfer function of a typical structural system that indicates around which natural frequencies high response values are expected, as can be seen in the middle of figure 3.10. It can be defined as the dynamic flexibility that is most relevant regarding human sensitivity. The product of the force and mobility results in the actual vibration velocity as can be seen on the right in figure 3.10. This means that a higher vibration velocity will occur when the step frequency and eigenfrequency are closer to each other.

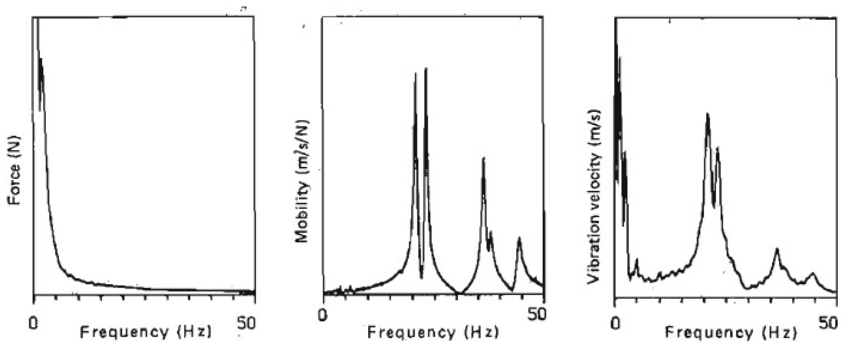


Figure 3.10: Footstep force, mobility of the structure and resulting vibrating velocity [29].

For generating an actual value that results in a comfort level the response of the floor due to walking, obtained from the step load function and the mobility of the floor, the One Step Root Mean Square is introduced [9]. This OS-RMS defines the root mean square over an interval between one step and the next of the frequency weighted velocity response at a certain point on the floor. Firstly, following from the time step analysis, for example figure 3.2, the velocity function is transformed from the time domain $\dot{u}(t)$ to the frequency domain $\dot{U}(f)$, using a fast Fourier transform (FFT). Next, the result has to be weighed according to human perception. This has to be done since the human perception of vibration varies with the frequency [9]. The following formula has to be used for this, where f_0 = 5,6 Hz and v_0 is the reference velocity set at 1.0 mm/s:

$$\dot{U}_B(f) = \frac{1}{v_0} \frac{1}{\sqrt{1+\left(\frac{f}{f_0}\right)}} \cdot \dot{U}(f) \tag{3.30}$$

After this weighting according to perception, the function is transferred from the frequency domain $\dot{U}_B(f)$ back into the time domain $\dot{u}_B(t)$ again. Since the maximum velocity of the vibration is not representative for the induced step force, a root mean square value has to be taken for the duration of a single step, the so-called OS-RMS.

$$OS-RMS_{n,m} = \sqrt{\frac{1}{T_s}\int_t^{t+T_s} \dot{u}_B^2(t)dt} \tag{3.31}$$

The result of this value only obtains the data from a single person with a specific step frequency and mass. To approve a structure for all varieties of people a combination of 35 different step frequencies times 20 different body masses are taken into account. In conjunction with a cumulative probability distribution factor (see Appendix A.3), a representative value of the population will follow for each of the 700 combinations, as is seen in 3.11. Finally, a 90%-fractile from the accumulated frequency distribution determines the representative velocity level, the OS-RMS$_{90}$. For generating the maximum value for the floor response it is only necessary to consider a fixed excitation point [23]. In other words, a walking path is not taken into account. At the place where the highest nuisance is expected the response point is chosen. For most common building systems this results in a point in the centre of a floor field.

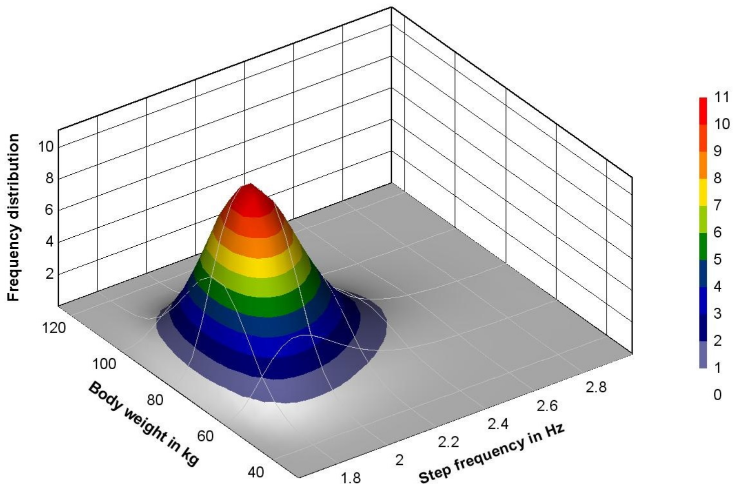


Figure 3.11: Frequency distribution for body weight and step frequency [9].

Vibration comfort is strongly dependent on the perceptibility of the people who are in a zone that is excited to vibrations. Even though guidelines are provided, a certain level of subjectivity is still involved. The OS-RMS$_{90}$ method comes with a table that globally indicates to what extent certain standards provide vibration comfort, see figure 3.12. An important aspect for these values is that the restrictions for neighbouring units are more strict than for the home situation. Self-induced deflections, or deflections caused by a source that is visible to the subjected person, are commonly regarded as less annoying since there is more control of the source [34]. However when an external force induces the vibrations a higher level of discomfort is experienced, meaning lower acceptance and higher demands for the OS-RMS$_{90}$. For different occupancies of the floor other threshold values hold as depicted in Appendix A.1.

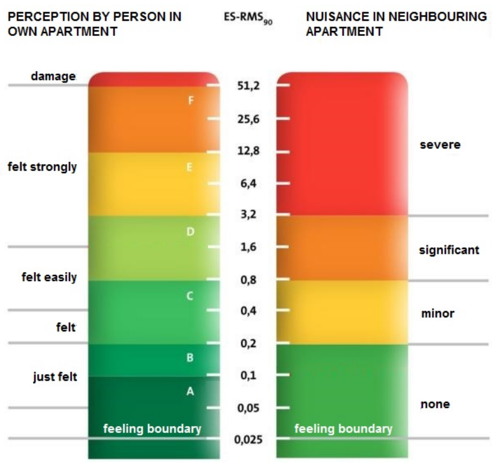


Figure 3.12: Target values OS-RMS$_{90}$ for vibration comfort in home situation (left) and neigbouring units (right) [12].

3.5. Summary

Human-induced vibrations are a common concern regarding residential light-weight buildings. For these type of structures, it is no longer a certainty that the participating mass during vibrational movement is sufficient to dissipate enough energy and maintain a comfortable indoor climate.

The vibration phenomenon can, conceptually, be broken down into the source, its path and the receiver. Since it is not possible to change the way people perform activities (source), nor their susceptibility to the oscillations from the floor (receiver), it is the path and thus the structural assembly that has to be considered for modifications to achieve an adequate level of comfort.

The path represents the structural building system and is dependent on the properties of the individual components such as the floors and beams (Single Degree of Freedom system) but also on how they behave when they are assembled (Multiple Degree of Freedom system). For both cases it follows that the contribution (or resistance) to vibrations is determined by the level of damping, natural frequencies and activated modal masses from the building system. This structure its characteristics can be expressed in a mobility-frequency function [m/s/N] showing the magnitude of susceptibility to vibrations for the natural frequencies.
Combined with the force-frequency function [N] of typical human activities it is possible to evaluate the response of a structure. For low-frequency floors, which light-weight timber floors can be accounted to, the natural frequencies coincide with those of the first few harmonics of human activities (2,2-8,8 Hz). If this is the case the occurrence of resonance is more likely which can cause severe vibration problems.
The level of comfort due to walking loads is expressed in a One-Step Root Mean Square velocity [m/s] value for a representative depiction of the population. The extent to which this OS-RMS_{90} value is interpreted does not only differ for the function of a room but also for the location of the receiver. It is considered more annoying when the source can not visually be controlled, making the level of acceptability more critical for vibrations transmitting from adjacent units than from the home situation.

Interventions in the structural assembly of steel and timber constructions are required to ensure a comfortable living space. It is necessary to either enhance standard light-weight building designs or to rearrange the structure to a format where the ratio of vibration comfort to weight is optimised. It is clear that designing for strength purposes is not sufficient any more and that for these type of light-weight structures the motion-related stiffness criteria become normative [14].

4

Theoretical building aspects

4.1. Single Degree of Freedom approach

To find engineering solutions for vibration comfort the assembly of the structure has to be investigated more in detail as followed from the theoretical approach of the floor vibrations. This building system responds to the dynamics footfall loads and results in a floor response whose vibration perceptibility has to be limited.
The discussed damping ratio, natural frequency and modal mass of a structural system are dependent on the material properties, geometry, boundary conditions, function and finishing of a structure. To be more specific it is necessary to define these values as individual structural properties of the composed floor-beam-column system and to show their possible contribution to comfort enhancements.

As goes for the standard approach of analysing floor fields, the system can be reduced to a single degree of freedom model. This implies that only a single element will vibrate (in this case the floor) and only the matching fundamental natural frequency will contribute to the level of vibrations. If this is the case, it is profound enough to use the formulas that are described in this section and hand calculations can be used to do a quick analyse of the structure. To get a grip on the structural properties that have an impact on vibration comfort these calculations will be the starting point for doing more elaborate design studies.
To check the results of the hand calculations with more accurate numerical analyses, a single floor field is modelled in appropriate software neglecting the influence of the boundary conditions. In other words, the floor acts as the only (single) member in the structure. The two approaches should correspond with respect to the theory and clarify the floor's behaviour. Also, these approaches are the benchmark for when later on multiple degree of freedom system are considered. It should become clear whether or not it is too conservative to use only SDOF models when designing for light-weight structures.

4.1.1. Theory

Natural frequency
First, the theoretical formulas for a floor system will be described. As goes for the natural frequency it is possible to fill in formula 3.25 with the stiffness and mass properties of an orthotropic plate dependent on its support conditions. A two-sided supported floor can be described as a wide beam whereas for a four-sided supported floor the transverse properties of the spanning field will contribute to the natural frequency. The general formula that follows from the rearrangements can be described as in formula 4.1 [29] and is subdivided into the following groups of interest: support conditions, geometry and material properties. For the four-sided supported floor an additional contribution factor is introduced.

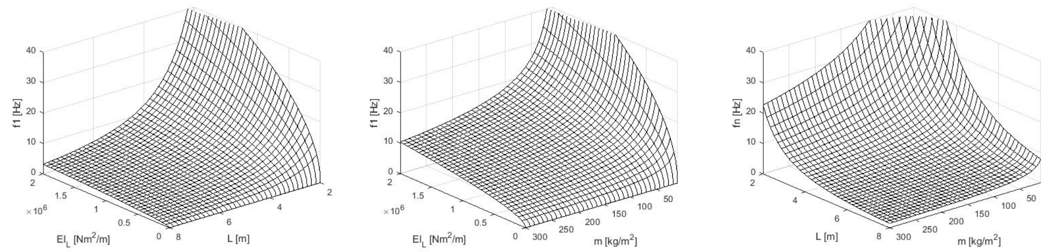
$$f_n = \frac{\kappa_n}{2\pi}\sqrt{\frac{EI_L}{mL^4}} \qquad \underbrace{\left(\sqrt{1+\left[2\left(\frac{L}{B}\right)^2+\left(\frac{L}{B}\right)^4\right]\left(\frac{EI_B}{EI_L}\right)}\right)}_{four-sided\,support\,factor} \tag{4.1}$$

with:

f_n	= natural frequency [Hz]
κ_n	= support condition factor [-]
EI_L	= flexural stiffness longitudinal [Nm^2/m]
EI_B	= flexural stiffness transverse [Nm^2/m]
m	= mass [kg/m^2]
L	= span [m]
B	= width [m]

From this equation it can be seen that some properties affect the natural frequency more than others. The *support conditions* can be described as either hinged or fixed, differing in value from π^2 to 22,4 respectively (see Appendix A.4 for other support conditions). The *geometry* and *material properties* that influence the natural frequency of the floor are its bending stiffness, mass and span. From the formula it follows that the influence of the span contributes to a significantly higher degree than the other aspects. Making it a valuable parameter to tweak the natural frequency.
When a plate is spanned in two directions in stead of one, the deflection becomes affected by both the properties in the longitudinal as the transverse direction. Therefore a *four-sided support factor* between 1,00 and 2,00 is applied when this type of spanning is used, which will lead to an increment of the natural frequency.

For the two-sided supported floor the geometry and material properties have been set out against each other in figure 4.1, showing their impact on the fundamental natural frequency of the floor. It is shown that more mass leads to a lower frequency, an effect that can cause resonance when it corresponds with the first few harmonics induced from walking loads. As more mass was perceived to lead to better vibration comfort, this shows that not solely the natural frequency but also the modal mass contributes to the floor response.
Another conclusion following from the parametric study, as was also discussed by Zegers [39], is the influence of the bending stiffness over different span lengths, see figure 4.1a. It seems that for larger spans the impact on changing the natural frequency becomes less effective, making it desirable to produce small span floors.



(a) Bending stiffness - Span (b) Bending stiffness - Mass (c) Span - Mass

Figure 4.1: Parametric study showing the effects of combining different individual parameters.

Modal mass

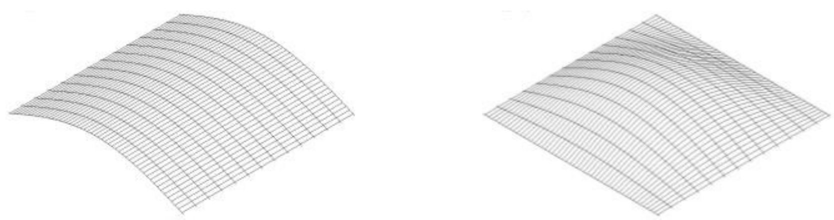
The participating modal mass follows from formula 3.29 and can be generalised for floors into the formula shown below (4.2)[36]. A factor β imposes the level of contribution of the total mass of the plate, following from its boundary condition. For floors supported with hinged connections this factor equals 0,64 whereas when fixed supports are used less mass will be oscillated. Therefore the contribution factor will only reach a value of 0,50.

$$M_n = \beta \, L \, B \, m \qquad \underbrace{\left(\frac{B_1}{B}\beta + \frac{B - B_1}{B} 1{,}00\right)}_{four-sided\ support\ factor} \qquad \text{with } B_1 = 2L\sqrt[4]{\frac{EI_B}{EI_L}} \tag{4.2}$$

with:

M_n	= modal mass [kg]
β	= support condition factor [-]
L	= span [m]
B	= width [m]
m	= mass [kg/m^2]
EI_L	= flexural stiffness [Nm^2/m]
EI_B	= flexural stiffness [Nm^2/m]

Similar to the formula of the natural frequency a distinction is made between two-sided and four-sided supported floors. For a two-way span the activated mass will reduce since there are no free edges, creating more constraints to oscillations. Figure 4.2 shows the first mode shape of both type of floors, illustrating the participating elements to a vibration.

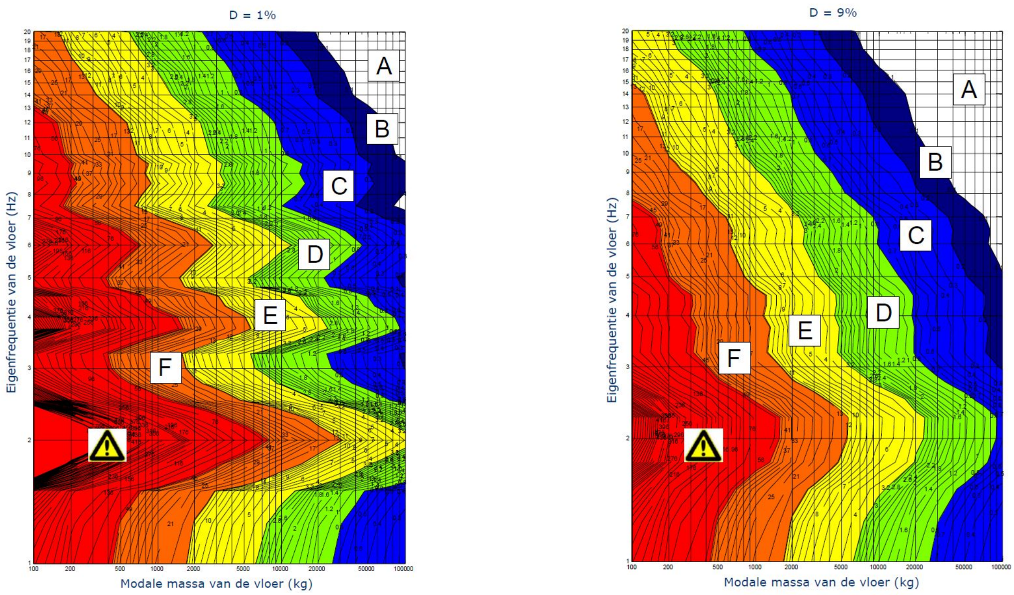


(a) One way span floor (b) Two way span floor

Figure 4.2: First mode shapes for two different floor systems.

Damping
The response of an oscillation will die out over time because of friction inside the materials and between elements. As discussed, the level of damping is a result of the used materials, possible finishings and function of the room (where the type and amount of furniture can result in more absorption of the vibrations). Since this damping level is hard to estimate and often follows from actual measurements, it is not easy to give an accurate assumption [20]. Some guidelines propose an approximation based on the previously summed characteristics, see Appendix A.2 [13]. However, it is better to approach the level of damping by using similar values of already build structures that resemble a new project design.

OS-RMS$_{90}$
It is possible to approach the OS-RMS$_{90}$ value for the home floor (where the excitation takes place), combining the three above described aspects for SDOF-systems. Figure 4.3 shows a graphical display of the vibration comfort level when the correct natural frequency of the floor (vertical axis) with the correct modal mass of the floor (horizontal axis) are combined for a certain percentage of the critical damping.
The figure shows two cases for extreme damping levels. When there is little damping contributing to the reduction of resonances, as is shown in figure 4.3a, strong peaks will become noticeable around the area of the first few harmonics of step loads (~2 Hz, ~4 Hz, ~6 Hz and ~8 Hz). This effect will reduce for higher levels of damping, as can be found when observing figure 4.3b. Making it beneficial to improve the level of damping for a building structure, which however is not as easy as it sounds.



(a) 1% damping

(b) 9% damping

Figure 4.3: OS-RMS$_{90}$ charts for SDOF floors for different damping levels [13].

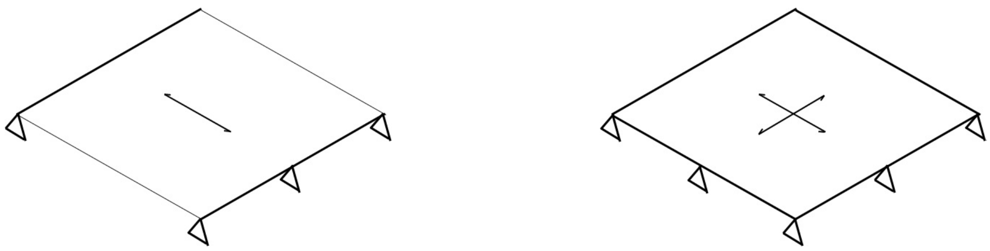
For approaching the OS-RMS$_{90}$ value with this SDOF approach, it can be concluded that the following theoretical values are of interest when aiming at enhancing the vibration comfort induced by human activities:

- The boundary conditions involve the type of connections, being either hinged of fixed and supported on two or four edges. For this assumption four cases can be discussed that will be studied more into detail.
- The plate of the floor is defined by its material properties. Meaning its bending stiffness in both directions (EI_L and EI_B) but also its mass (m) outline the quality of the floor.
- Last the dimensions of the floor will influence the performance due to the vibrations. This geometry is defined as the span L and the width B of the floor. Given the fact that orthotropic plates are being considered the ratio between properties in longitudinal and transverse direction will affect the outcome.

These properties will be studied in the following section to show their level of impact and to find out to what extent they can contribute to a better performance of the structure.

4.1.2. Cases and reference

It is possible to distinguish floors bases on their type of support conditions. Theoretically there are several options that could be applied for building systems. The combination of free, hinged and fixed supports together with a one-, two- or four-sided support indicate the various possibilities. For practical reasons the four most common support types will be investigated further. These cases will act as the frameworks to which the effects of changing the geometry and properties of the floor will be discussed. In figures 4.4 and 4.5 these cases are sketched together with their formulas for the fundamental natural frequency and modal mass.



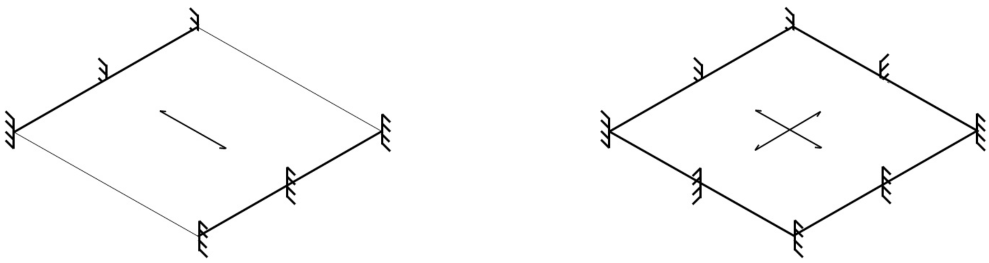
(a) Two-sided support

(b) Four-sided support

Figure 4.4: Plate with hinged connections.

$$f_n = \frac{\pi}{2}\sqrt{\frac{EI_L}{mL^4}} \qquad M_n = 0{,}64\,L\,B\,m$$

$$f_n = \frac{\pi}{2}\sqrt{\frac{EI_L}{mL^4}}\sqrt{1+\left[2\left(\frac{L}{B}\right)^2+\left(\frac{L}{B}\right)^4\right]\left(\frac{EI_B}{EI_L}\right)} \qquad M_n = 0{,}64\,L\,B\,m\left(1-0{,}72\frac{L}{B}\sqrt[4]{\frac{EI_B}{EI_L}}\right)$$



(a) Two-sided support

(b) Four-sided support

Figure 4.5: Plate with fixed connections.

$$f_n = \frac{22{,}4}{2\pi}\sqrt{\frac{EI_L}{mL^4}} \qquad M_n = 0{,}50\,L\,B\,m$$

$$f_n = \frac{22{,}4}{2\pi}\sqrt{\frac{EI_L}{mL^4}}\sqrt{1+\left[2\left(\frac{L}{B}\right)^2+\left(\frac{L}{B}\right)^4\right]\left(\frac{EI_B}{EI_L}\right)} \qquad M_n = 0{,}50\,L\,B\,m\left(1-\frac{L}{B}\sqrt[4]{\frac{EI_B}{EI_L}}\right)$$

The mentioned formulas indicate that there is an overlap in the level to which some properties contribute to the natural frequency and modal mass. It also shows that for the two-way spanned floors the ratio of the transverse stiffness to the longitudinal stiffness influences the outcome, as goes for the ratio of the width to span. For one-way spans these factors don't play a role.
Another observation is that plates with hinged connections tend to have a lower frequency but activate a higher mass in a vibration mode. Fixed connections on the other hand have a higher natural frequency but activate less mass during vibrations. Practical considerations for both type of supports are deliberated further on in this report.

To investigate the results within a realistic range of values for residential light-weight buildings the 'De Karel Doorman' project will be used as reference case. From this starting point the effects of changing properties and geometry to the four chosen frameworks will give more insight in the comfort outcome for different support conditions.

Case: De Karel Doorman
The 'De Karel Doorman' building in Rotterdam was one of the frontrunners regarding ultra-light weight building systems. For this reason it was also one of the first residence buildings which, due to the reduced self weight, revealed the issues of vibration comfort. In order to explore the possibilities of an improved building system the case 'De Karel Doorman' is taken as benchmark. In this section the description and translation from building system to discrete structural characteristics is elaborated in detail.

Due to the limitations for high-rise projects in combination with restrictions regarding self weight a steel frame with timber floors was proposed for 'De Karel Doorman'. For creating the required floor area of the new to be build apartments (i.e. ~100 m^2), grid dimensions of four by six meters were used. A single apartment, with certain exceptions, would exist of four (two by two) of these floor fields, being separated from neighbouring units by plasterboard walls.

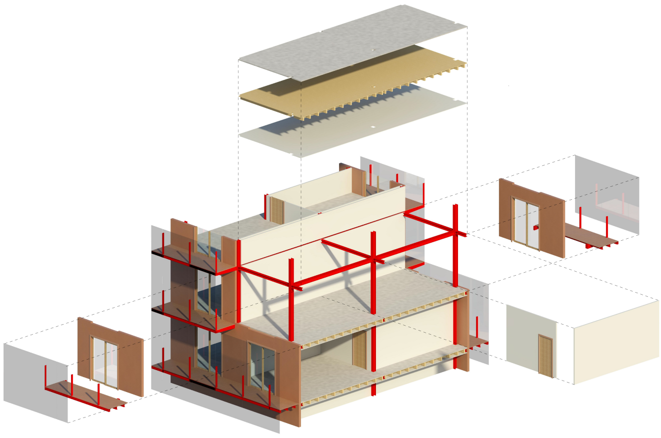


Figure 4.6: Building kit 'De Karel Doorman' [12].

For logistic and erection purposes the steel structure was composed of 3-storey high HE220B columns simply connected with HE220A beams in the transverse direction (supporting the one-way span timber floors) and HE180A beams in the longitudinal direction. This steel structure is connected to two concrete cores that puncture through the existing building. The grid transition from the old to the new part of the building was solved by a table system that was placed in between, making it possible for the new part to create the desired floor areas.
The timber floor is constructed of 45x225 mm Kerto S joists 600 mm centre-to-centre and an 18 mm thick plywood siding. On top of this a 55 mm anhydrite layer is applied. The cavity walls exist of 2x12,5 mm plasterboard with a 90 mm mineral wool insulation. The typical way of the connection node is shown in figure 4.7. It shows the floors resting on the bottom flanges of the beam being connected as hinges. Also, the blue lines indicate the kinematic coupling of the floors to the supporting beam. Adding up all the constructional elements an averaged minimum weight of the floor assembly is estimated to be roughly 2,04 kN/m^2.

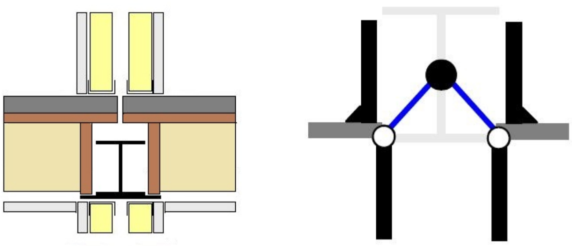


Figure 4.7: Schematic junction of 'De Karel Doorman' [33].

For investigating the performance of such a built up structure it is possible to abstract the floors and walls to a simple plate with a thickness of approximate one percent of the span [33]. For the case sketched above, this homogenisation results in the material properties for both the designed floors and walls as shown in table 4.1. Also the properties of the supporting beam are depicted in this table.

Table 4.1: Material properties of 'De Karel Doorman'.

	Floors	Walls	Beams (HE220A)	
E_x [N/m^2]	$2{,}140 \cdot 10^{11}$	$4{,}050 \cdot 10^{10}$	$2{,}000 \cdot 10^{11}$	E [N/m^2]
E_y [N/m^2]	$3{,}010 \cdot 10^{10}$	$3{,}100 \cdot 10^{9}$	$7{,}930 \cdot 10^{10}$	G [N/m^2]
G_{xy} [N/m^2]	$1{,}070 \cdot 10^{11}$	$2{,}025 \cdot 10^{10}$	$7{,}850 \cdot 10^{3}$	ρ [kg/m^3]
G_{yx} [N/m^2]	$1{,}505 \cdot 10^{10}$	$1{,}550 \cdot 10^{9}$		
G_{xz} [N/m^2]	$1{,}070 \cdot 10^{11}$	$1{,}550 \cdot 10^{9}$		
ρ [kg/m^3]	$5{,}100 \cdot 10^{3}$	$8{,}925 \cdot 10^{2}$		
h [m]	$4{,}000 \cdot 10^{-2}$	$2{,}500 \cdot 10^{-2}$		

For establishing the correct damping value to maintain during the research, reference projects have been checked to find a corresponding system. Although steel systems often appear to have a critical damping percentage of around 1-2%, this value increases when timber elements are used to approximately 5-6%. Also there is a distinction to be made in reference projects that are still considered as building shell or as finished structures [36].

For these reasons the level of damping is set to 5% during this phase of the research. This means that resonance is still likely to occur but the energy of the vibration will dissipate over time. While performing and analysing the sensitivity analysis of different properties, the damping level will be taken into account and changes in this value will be pointed out.

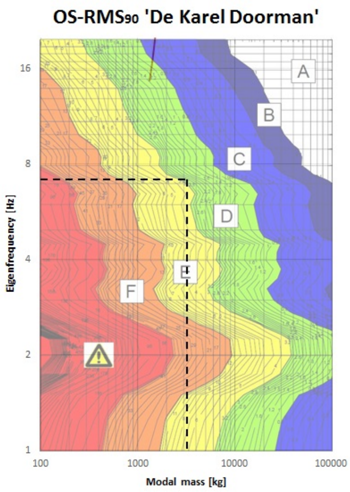


Figure 4.8: OS-RMS$_{90}$ chart 5% damping with 'De Karel Doorman' indicated.

For the case of the original 'De Karel Doorman', before enhancing adjustments were made, and having solely hinged connections with a floor span in one direction, the natural frequency and modal mass can be calculated. With a set 5% damping the OS-RMS$_{90}$ for a SDOF system can be found as depicted in figure 4.8. It shows that for the floor with a natural frequency of 7,35 Hz and a modal mass of around 3100 kg a resulting OS-RMS$_{90}$ value of approximately 3,20 is obtained. Located on the border of class D and E, this indicates severe vibrational discomfort.

Although the natural frequency is within the highest sensitivity range of human perception (between 4-8 Hz), but not too close to the first few harmonics of step loads, it is the combination with the low modal mass that makes this structure unsatisfactory. The figure shows how increasing the modal mass and natural frequency contribute to a better performing floor system. The two however will not always improve simultaneously, meaning that adjusting one building property may result in more beneficial results to the one but has an adverse impact on the other.

4.1.3. Structural tools

Stiffness

The stiffness of a structure is defined with the EI value. This value is the product of the Modulus of Elasticity (E) and the second moment of area (I). The E factor indicates the capacity to resist elastic deformations and is strongly dependent on the type of material. It can be expressed in N/m^2 or GPa. The I factor measures the efficiency of the cross sections resistance to bending and is purely based on the cross-section of a structural element. It is expressed in mm^4. In standard timber floor designs the vast majority of the stiffness is conditional to the dimensions and distribution of the joists. For the analyses the stiffness will be expressed per meter as Nm^2/m.

Increasing the bending stiffness of the floor system in the direction of the span (EI_L) will undeniably enhance the natural frequency of a floor system, but changes to the modal mass are theoretically limited. The graph below shows how for the changing stiffness the resulting OS-RMS$_{90}$ will move across the chart for each of the four cases described previously.

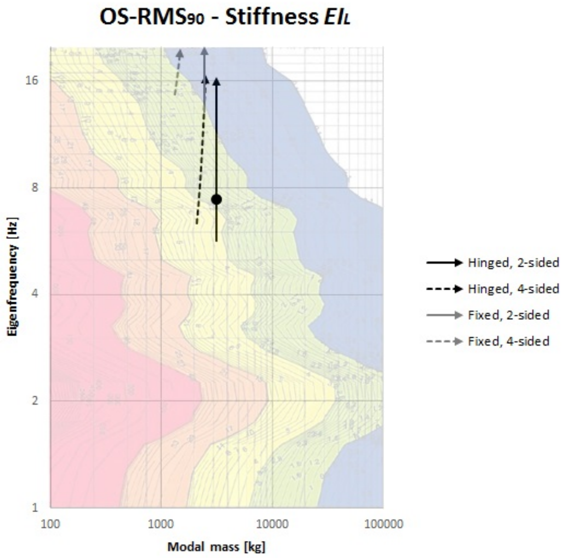


Figure 4.9: OS-RMS$_{90}$ chart with varying stiffness for different support conditions.

As logic reasoning would lead to the impression of stiffer structures having smaller deflections and hence resulting in less vibrations, the theory shows corresponding results. Especially the effect on the natural frequency is significant whereas an increase of stiffness only leads to a bit more modal mass in the case of two-way span floors.

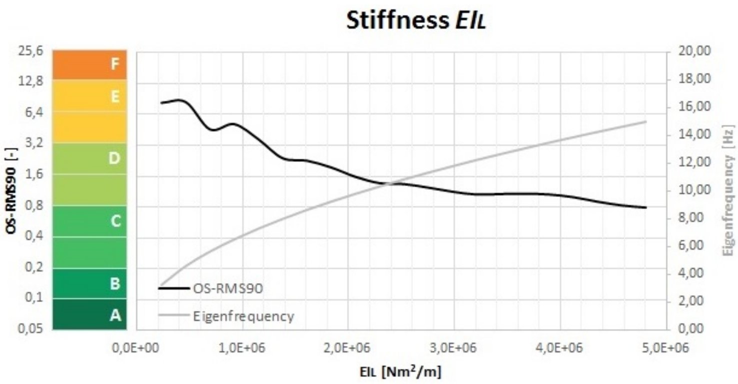


Figure 4.10: OS-RMS$_{90}$ and natural frequency for varying stiffness.

If the case of the two-sided hinged floor is investigated closer, the impact of the stiffness can be explained more elaborately. As figure 4.10 shows the numerical analysis, the coherence between the OS-RMS$_{90}$ value and the natural frequency is strong. More stiffness leads to higher natural frequencies which results in better vibration comfort values. The peaks in the OS-RMS$_{90}$ line indicate points of resonance with step frequencies. For these values the natural frequencies comply with the first harmonic of step loads, which as can be seen is around 2, 4, 6 and 8 Hz with decreasing impact, resulting in lower vibration comfort.

Span

The span of the floor is defined as the length in meters the floor has to cross between two opposed supports. As was found in the formula of the natural frequency (formula 4.1), it followed that this geometry factor contributes to the power four compared to other properties. From the modal mass formula (4.2), it showed that a larger span means that there is more area being oscillated and therefore enlarges the modal mass contribution.

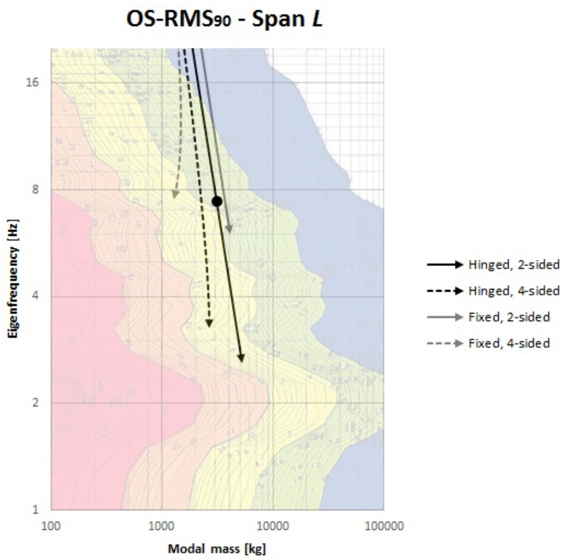


Figure 4.11: OS-RMS$_{90}$ chart with varying span for different support conditions.

Figure 4.11 shows how scaling up (and down) the span leads to a change in vibration comfort. Smaller spans lead to smaller deflections (for a constant stiffness) and thus improve the vibration comfort. On the contrary smaller spans also lead to less involved mass during vibrations. The chart shows that the impact of changes has a higher contribution to the natural frequency than the modal mass, thus making it favourable to use small spans. Modern practice designs however often want to create bigger open spaces, demanding larger spans without too many structural elements. A compromise has to be found in order to settle these conflicting interests.

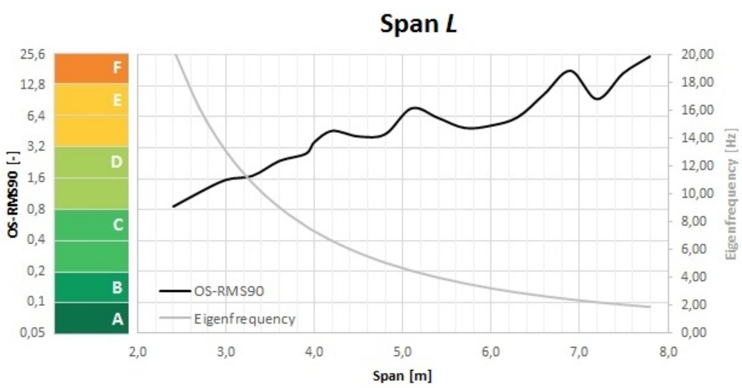


Figure 4.12: OS-RMS$_{90}$ and natural frequency for varying span.

Figure 4.12 shows the exponential change of the natural frequency due to the changing span. Also the matching OS-RMS$_{90}$ value that followed from numerical analyses for the two-sided support case with hinged boundary conditions can be seen. With spans larger than the original 4 meter, natural frequencies corresponding to the step load harmonics occur, resulting in resonance and thus worsen the vibration comfort. This phenomenon explains the peaks in the OS-RMS$_{90}$ line.

Mass
The mass of the floor is defined as all the self weight of the structure plus an additional load for permanent non-structural elements. In the 'De Karel Doorman' project a light weight floor of around 204 kg/m^2 was used. This was necessary in order to achieve a sixteen storey high vertical extension on top of an existing building. Compared to traditional floor systems this was an extremely low amount.

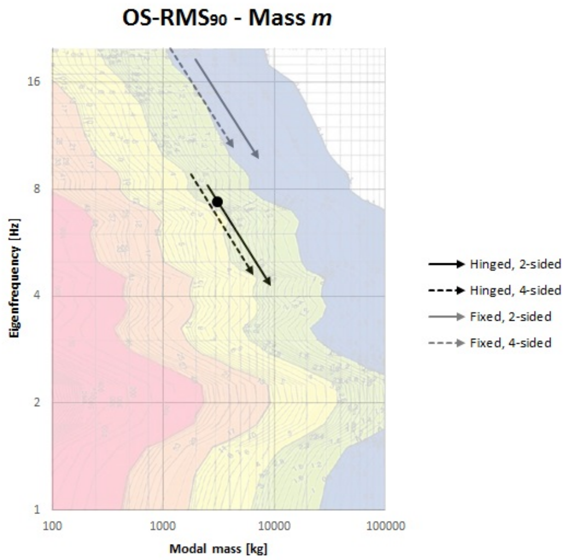


Figure 4.13: OS-RMS$_{90}$ chart with varying mass for different support conditions.

Figure 4.13 depicts how for the changing mass the natural frequency and modal mass are influenced. It becomes clear that the mass has contradicting effects regarding vibration comfort. More mass reduces the amplitude of the vibrations but also lowers the natural frequency, that might become critical when it is found in the range of step load harmonics. The chart shows that more mass does not necessarily lead to better comfort values.

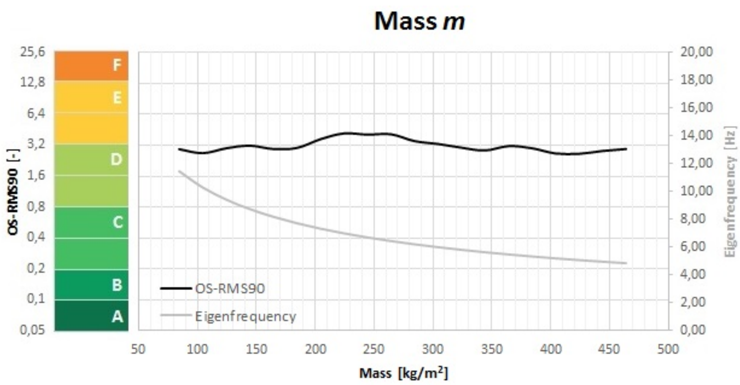
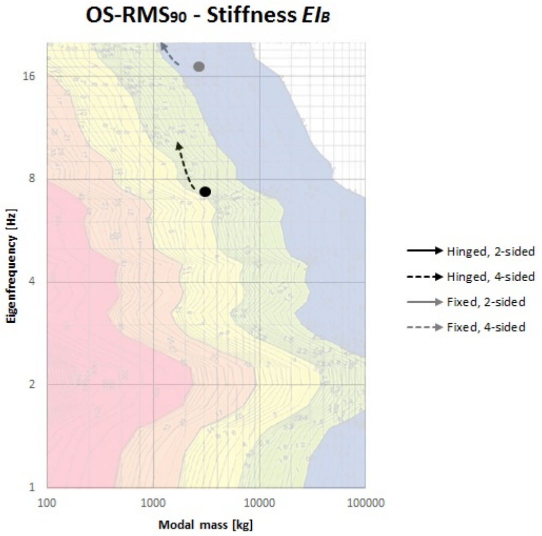


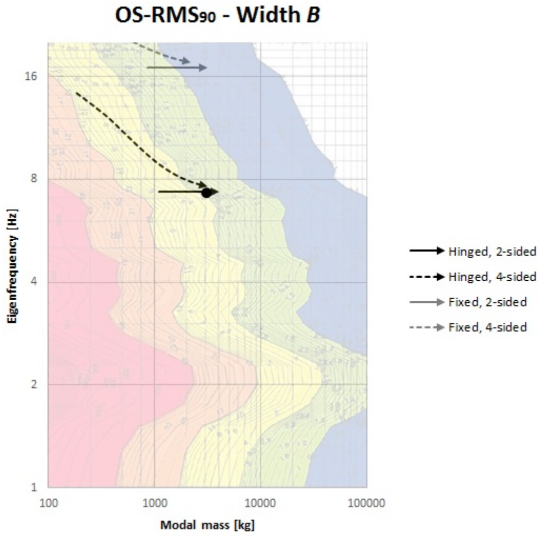
Figure 4.14: OS-RMS$_{90}$ and natural frequency for varying mass.

To check this statement, it is possible to plot the OS-RMS$_{90}$ for the different masses, as is done in figure 4.14. The OS-RMS$_{90}$ value remains constant around a certain value even though the natural frequency does change. Comparing this results with figure 4.13, the correspondence indicates indeed only small deviations. The coherence between natural frequency and modal mass is thus of importance when properties are changed.

Transverse properties
Where two sided supported floors are treated as wide beams, and are not affected by transverse properties, this is not the case for floors supported along all four edges. For these kind of floors the influence of the orthotropic properties and geometry can benefit the vibrational behaviour as was also indicated by Chui [6]. In figure 4.15 both the theoretical results for changing the transverse stiffness EI_B and width of the floor B are depicted.



(a) Transverse stiffness

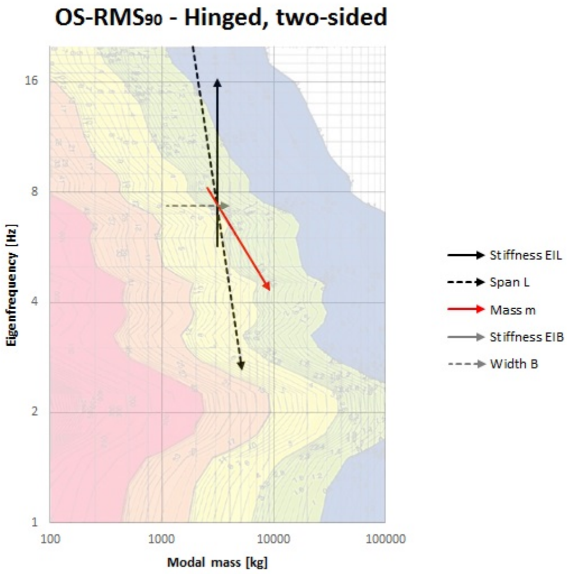


(b) Width

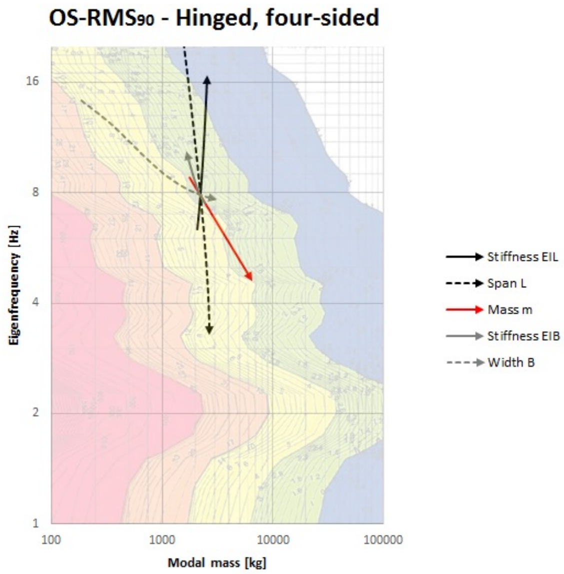
Figure 4.15: OS-RMS$_{90}$ chart with varying transverse properties for different support conditions.

For the changes in the transverse direction the results for the two-way span floors are of most interest. It is found that a higher ratio of transverse to longitudinal stiffness results in an increased natural frequency but lower modal mass. As was discussed in the theory the effects of the transverse stiffness becomes of more interest when a MDOF system is investigated and not just the fundamental natural frequency is considered of importance.
As goes for the width of a floor, which is often not considered as a useful parameter, the results do show an increase in modal mass for wider floors. This is accompanied with a lower natural frequency.

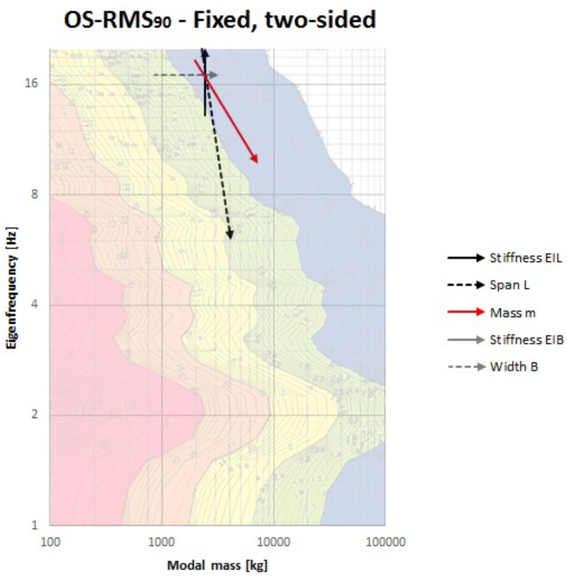
Combination
The effects of combining all the varying material properties and geometries is shown for each case in figure 4.16. As was described per element previously it shows that a higher stiffness and shorter span lead to improved OS-RMS$_{90}$ values. The effect of changing the mass, within the range of light-weight structures, will not lead to significant differences. It is only for the four-sided supported floors that the transverse properties affect the natural frequency and modal mass.
Reaching lower OS-RMS$_{90}$ values, and thus an improved vibration comfort, requires higher natural frequencies and more mass to participate during vibrations. The charts show that when these floor characteristics are not sufficient the OS-RMS$_{90}$ will shift to the red area, indicating strongly felt vibrations that could also lead to severe damage. The blue area on the opposite follows from high characteristics of the structure, resulting in lower OS-RMS$_{90}$ values creating a comfortable area regarding vibration sensitivity.
For building projects is it desirable to acquire an as low as possible OS-RMS$_{90}$ value, so the vibrations will not cause nuisance. Ideally this is below 0,8 for the home situation. Figure 4.16 shows that changes in the stiffness (EI_L) have the strongest tendency to lower figure OS-RMS$_{90}$ values, making it the theoretically most significant tool for improving the vibration comfort in SDOF systems, although the impact of the span also contributes to a great extent.



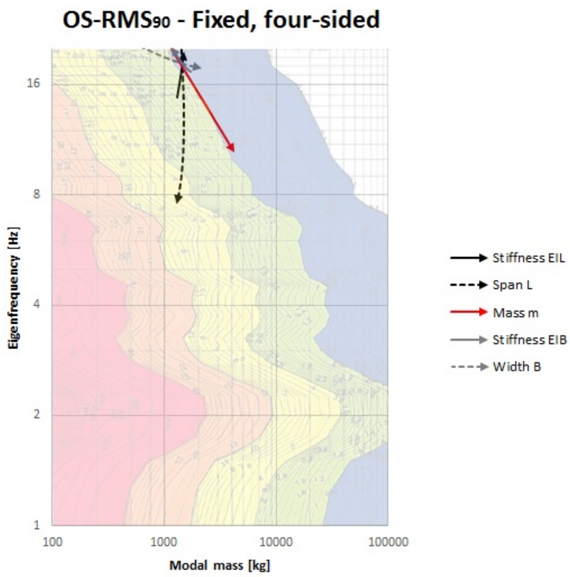
(a) Hinged, two-sided support



(b) Hinged, four-sided support



(c) Fixed, two-sided support



(d) Fixed, four-sided support

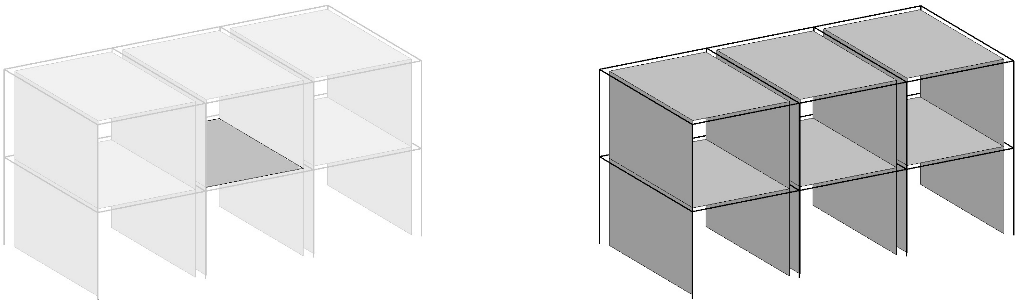
Figure 4.16: OS-RMS$_{90}$ chart with varying properties for different support conditions.

4.2. Multiple Degree of Freedom approach

During the design of the 'De Karel Doorman' it was found that the natural frequencies of the individual elements (i.e. beams and floors) were high enough to avoid nuisance from vibrations [35]. It wasn't until the elements were coupled together and the reality showed heavy, even visible, vibrations that the awareness arose that especially for these type of light-weight building systems a SDOF approach would not be sufficient enough.

4.2.1. Theory

To find out to what extent the findings from the previous section are still suitable, a comparison between the SDOF system (figure 4.17a) and MDOF (figure 4.17b) will be elaborated more in detail. This will be done to give more grip on the interventions in building systems that can actually enhance the vibration comfort.



(a) SDOF (b) MDOF

Figure 4.17: Type of used models for evaluating the vibration comfort.

As the theory shows that for MDOF systems it is the case that not only the first natural frequency contributes to the vibrations, a greater spectrum has to be considered including higher frequencies, see figure 4.18. For each specific natural frequency the structure has a related deformation. Each of these deformations contribute to the complete deflection the structure undergoes, with the one frequency having a higher impact than the other.

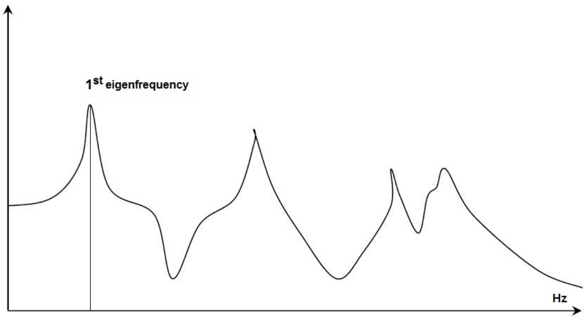


Figure 4.18: MDOF admittances spectrum in the frequency domain [19].

For approaching the fundamental natural frequency Dunkerleys equation can be used, see equation 3.27. This approximation combines the natural frequencies of the main elements, which in the case of the 'De Karel Doorman' are the floors and its supporting beams. For this type of structure the equation can be rewritten into the following form:

$$\frac{1}{f_{n,system}^2} = \frac{1}{f_{n,floor}^2} + \frac{1}{f_{n,beam}^2} + \ldots.. \tag{4.3}$$

What follows is that the natural frequency has an upper limit for the highest value of both. Only for cases where the natural frequency of either one is extremely high, the fundamental frequency will approach the value of the other element. Since this is often not the case and this equation only gives a rough estimation it is best to find the natural frequency with finite element analyses.

4.2.2. Cases and reference

Finite element software was used to calculate the response of a floor due to human induced vibrations for MDOF systems. A design programme appropriate for these kind of calculations is the SoViST tool, short for Sound and Vibration Steel Timber. This program was initiated by TNO and further developed by Level Tools, part of Level Acoustics & Vibration in Eindhoven. The tool was launched in 2017 after a result of long-lasting research in the field of sound and vibration transmittance in the construction joint of light-weight building systems.
Obtaining valid results from this tool means it first has to be checked if the calculated admittances comply with the measured admittances. To do so, measurements that were executed by TNO on the 'De Karel Doorman' building are compared to the results (under similar circumstances) from the SoViST tool. Since it will never be the case that the reality is fully compliant with a computer model a certain deviation is acceptable. In the table below both admittances are depicted.

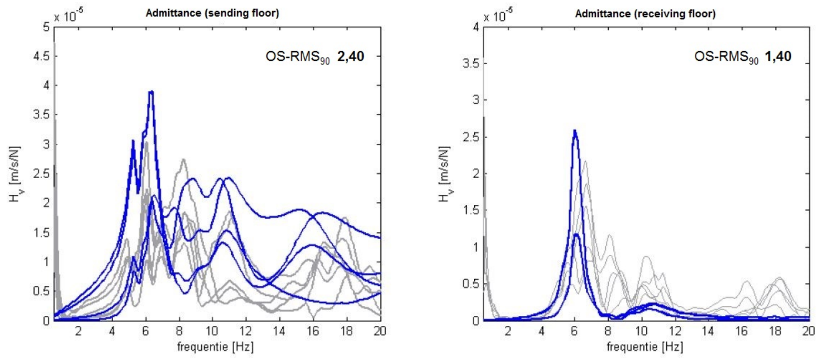


Figure 4.19: Admittances from measurements 'De Karel Doorman' [19].

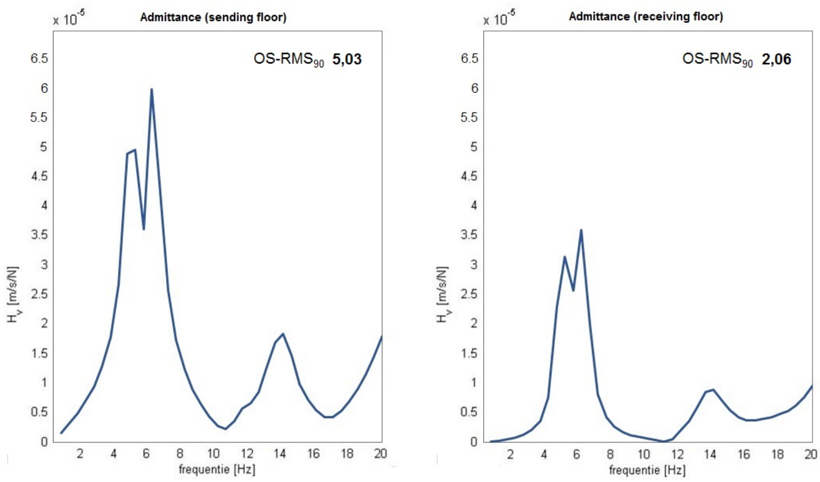
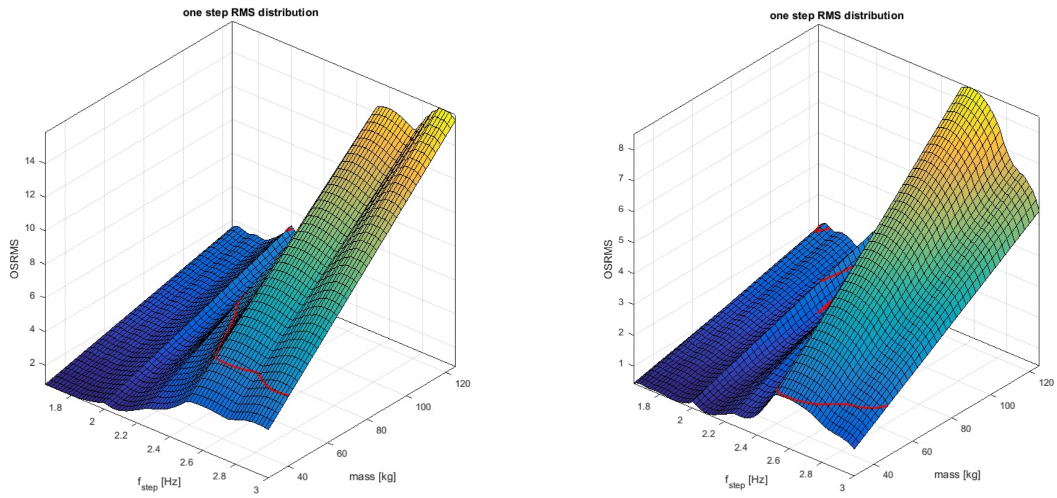


Figure 4.20: Admittances from calculations SoViST.

It can be observed that for the numerical analysis corresponding results are obtained. Both with respect to admittances in the home and the neighbouring floor the natural frequency is compliant (~6-8 Hz), however the mobility shows different magnitudes at these points. With respect to the OS-RMS$_{90}$ values the relative proportion between the home and neighbouring unit shows similarities. In absolute values the measured OS-RMS$_{90}$ are considerably lower. This can be explained by the level of damping.
For this comparison a damping level of 5% was presumed. It could be the case that in reality the level of damping is actually higher. To give comparable results this percentage value is maintained, keeping in mind the possible effects it can have on the comfort level for enhancements.

The figures on this page show the reasoning behind using the OS-RMS$_{90}$ value as comfort level. Figure 4.21 shows the mobility for both the floor that is induces by dynamic loads (a) and for the adjacent floor field (b). It can be observed that various step-frequencies and individuals their bodyweight combinations will result in different OS-RMS values. As was expected the higher step-frequencies, that are closer to the fundamental frequency of the system, result in the worst vibration comfort levels. Also a higher weight of a person inducing the step-load will give unfavourable values.
For neighbouring fields the OS-RMS values are significantly lower, but also the governing step-frequency changes. This indicates that different guidelines hold for the transmittance to other fields.

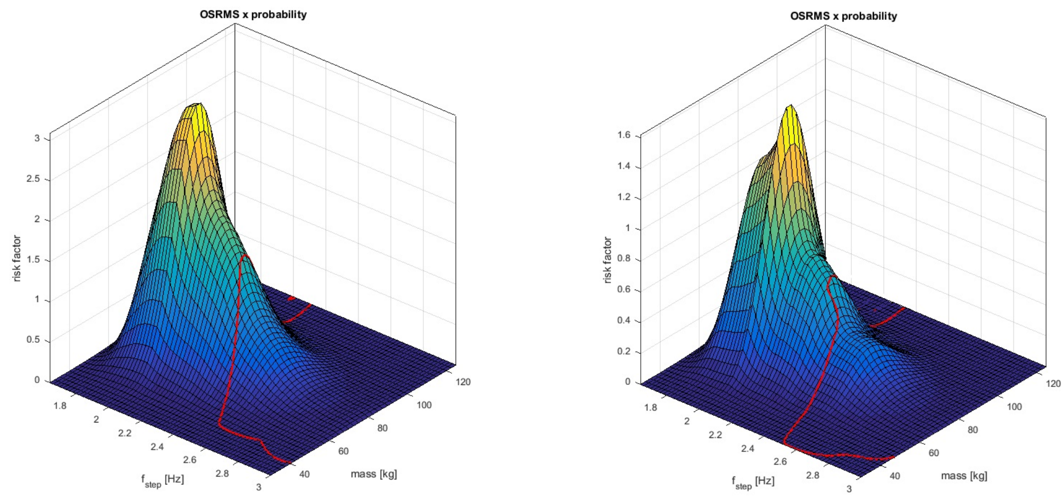


(a) Home floor

(b) Neighbouring floor

Figure 4.21: Mobility.

Figure 4.22 shows the risk factor for the above mentioned combinations. Where figure 4.21 showed that the highest OS-RMS values will follow from individuals who walk with a high pace and have above average body-weight, a weighting-factor for a representative depiction of the population will show more realistic values of the comfort level. This value is indicated with as the OS-RMS$_{90}$ and is shown with the red line in the figures. It follows that the average person walks with a step-frequency of 2 Hz and weights 75 kg. This person's OS-RMS values however will only result in a value of 3,09 [-] and 1,62 [-] for respectively the sending and receiving location. A considerably lower value than the 5,03 [-] and 2,06 [-] that follow for a representative population.



(a) Home floor

(b) Neighbouring floor

Figure 4.22: Risk.

4.2.3. Structural tools

The differences between the single and multiple degree of freedom systems have to be compared. Where the MDOF system combines the assembled structure, it also takes into account the transmittance to and through other elements than just the floor that is induced by activities. This leads to vibrations in not just the sending floor, but also the adjacent receiving floors will response to the dynamic load that is applied. The path of the vibrations will not stop at the boundaries of the floor, creating more involved elements in the transmittance and dissipation of the vibration. The studies in this section will show how variations in structural properties affect the OS-RMS$_{90}$ score. Also the effects of converging natural frequencies of individual elements will be evaluated regarding their influence on the transmittance of vibrations.
Theory already showed that for these neighbouring floors the perception criterion is more strict. Since there is less control over the source of the vibrations the threshold to acceptable OS-RMS$_{90}$ values is considerable lower. Due to damping along the path vibrations will already dissipate some of their energy. Whether or not this is enough has to be researched more deeply.

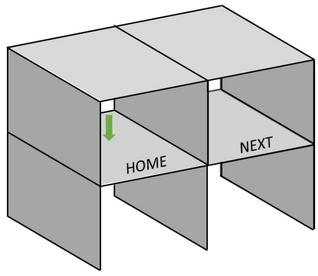


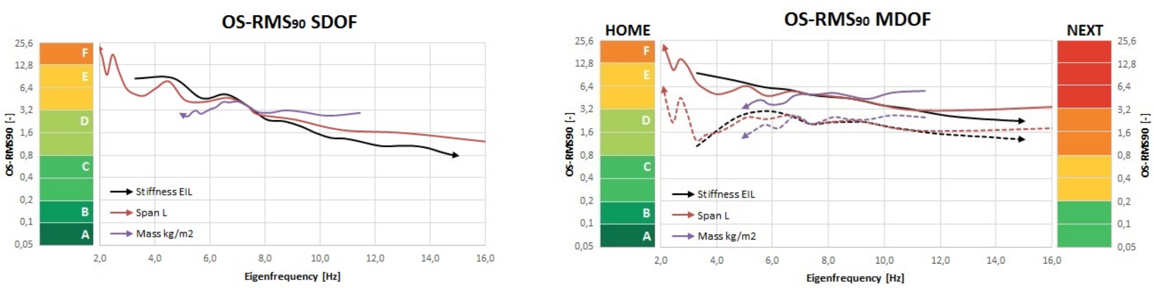
Figure 4.23: Schematic representation of sending and receiving floor fields.

To get a grip on the OS-RMS$_{90}$ for both the home and neighbouring units the path is divided into three parts that participate during human induced vibrations:

- Home floor (*HOME*)
- Junction
- Neighbouring floor (*NEXT*)

In the following sections it is checked what happens when the properties of all floors are changed evenly, what happens when the properties at the junctions change and what happens when the properties of every other floor is changed differently by creating an alternating floor system.

Flooring assembly
The change from a SDOF to a MDOF system (for a one-way span with hinged connections) with its varying properties and geometry is shown in figure 4.24. The OS-RMS$_{90}$ is set out against the natural frequency of the floor. What can be observed is that the trend lines for the single parameters still follow the same curve although the effects are less strong. Another major issue is the introduction of the vibrations in adjacent floor fields (indicated with the dotted lines and plotted on the right side). Since for this more realistic approach of the building system it follows that the OS-RMS$_{90}$ value is less favourable, it becomes even more essential to find measures to steer the vibration response. What follows is that the effects of the stiffness and span remain governing, whereas the small differences in mass still do not seem to affect the outcome significantly.

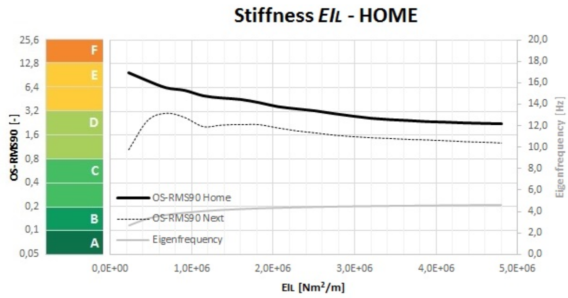


(a) SDOF

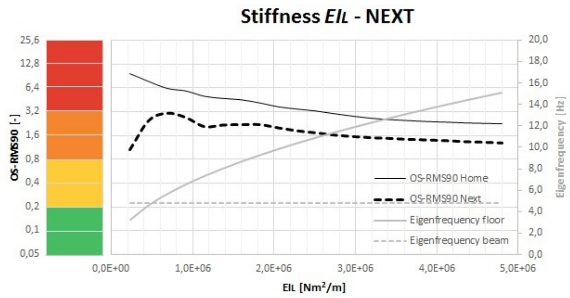
(b) MDOF

Figure 4.24: OS-RMS$_{90}$ and natural frequency for different degrees-of-freedom systems.

Where the introduced vibration in neighbouring floors show corresponding results as those of the excited floors, some distinctive values strike the eye. Where the natural frequencies of the floor and the supporting beam coincide the OS-RMS$_{90}$ at neighbouring units show higher values. This phenomenon is due to *mode-coupling* and creates the possibility for vibrations to transfer more easily from the one to the other elements. This holds since both parts in the structure tend to vibrate for the same frequency. By creating more distinction between these two values the barrier for transmitting the vibration energy becomes higher hence resulting in a better vibration comfort level in neighbouring units.

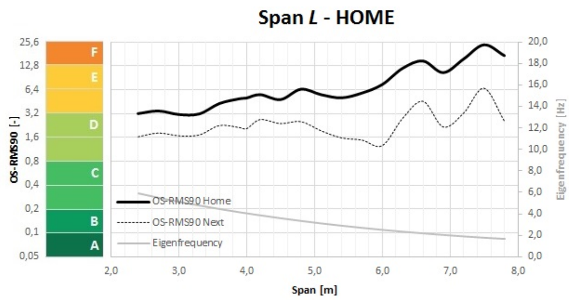


(a) Home floor

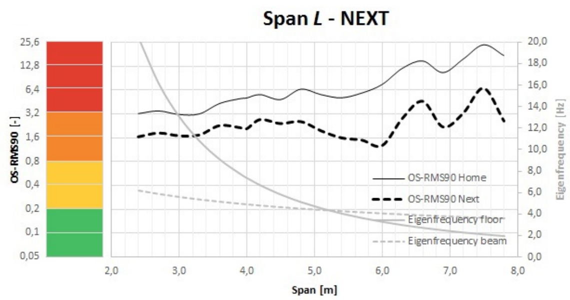


(b) Neighbouring floor

Figure 4.25: OS-RMS$_{90}$ and natural frequency for varying stiffness.

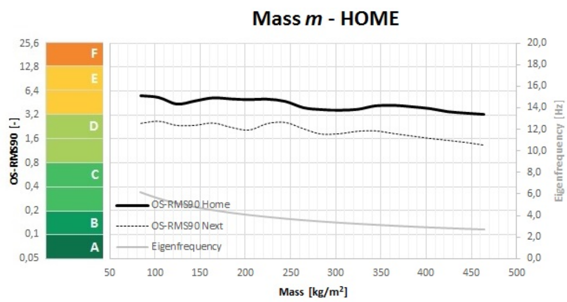


(a) Home floor

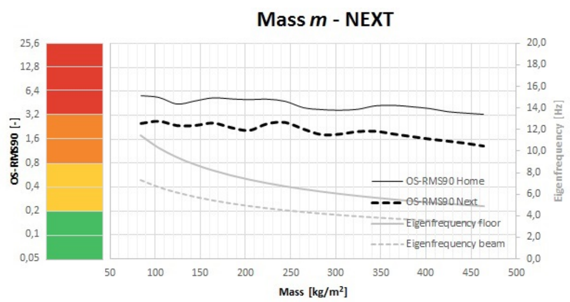


(b) Neighbouring floor

Figure 4.26: OS-RMS$_{90}$ and natural frequency for varying span.



(a) Home floor



(b) Neighbouring floor

Figure 4.27: OS-RMS$_{90}$ and natural frequency for varying mass.

The figures above show on the left hand side how the single parameters influence the OS-RMS$_{90}$ value for the induced situation (*HOME*) and how this corresponds to the natural frequency of the complete system. On the right hand side the same effects are highlighted for the neighbouring floors (*NEXT*). Here it is clearly observed that for the case of the varying stiffness EI_L (see figure 4.25b) a converging natural frequency of the floor and beam results in a higher OS-RMS$_{90}$ value and creates a less comfortable climate for adjacent rooms.

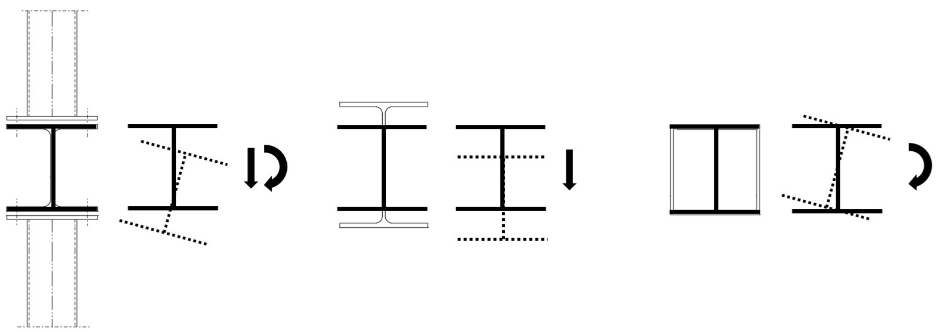
Structural junction
The structural beams are connected to the floor systems and will respond to both the static and dynamic loads that are imposed on them. The most characteristic property of the beam that can effect its resistance to vibrations is its stiffness, both bending and torsional related. Additional elements such as a wall (see figure 4.28a) can also introduce this stiffness. However, this is not ideal for the flexibility of a building system since it creates more obstacles in the floor field that can not be removed easily. Figure 4.29 shows the strong effects that stiff elements (in the form of a wall) can produce on the transmittance to neighbouring areas. If it is the case that adjustments to the floors or other structural elements do not result in a OS-RMS$_{90}$ value normative to the function of the rooms, this additional stiffness in the form of a wall is a practical solution.

The bending stiffness of a beam is a product of the cross-sectional shape of the profiles. Steel beams produce effective cross sections, distributing the area away from the centre of gravity. Figure 4.30 shows how for a varying second moment of area I_y the comfort level in both the home and neighbouring situation changes. What follows is that for the home situation a stiffer beam can result in strong improvements of the OS-RMS$_{90}$. More striking is the result in adjacent fields. For this case the trend line tends towards and imperceptible level of vibrations when the bending stiffness is increasing. Of course, the practical implementation of these kinds of values ask for extreme profiles sizes and have to be put in a realistic perspective. Higher I_y values come together with more area, hence more mass, and a higher distance from the central point resulting in higher profiles and thus a more prominent structure as can be seen in figure 4.28b.
As can be observed, the trend line for reducing the OS-RMS$_{90}$ in adjacent floor fields is interrupted at a certain point. This happens to be the case where the natural frequency of the beam and the floor coincide. This mode-coupling phenomenon, as mentioned before, increases the transmittance of vibrations and results in a worse comfort level.

Torsion is introduced to a beam when it is twisted due to an applied torque (T). By increasing the torsion stiffness of an element, it is possible to enhance the transmittance of vibrations to neighbouring fields. This type of stiffness is dependent on the modulus of rigidity (G) and length of the beam (l), together with its torsion constant (I_t), as is shown in equation 4.4. The torsion constant, same as the second moment of area, is a geometric property of the beam section and is expressed in mm^4. By increasing this property, it should be possible to reduce the angular twist (ϕ) of the beam (as illustrated in figure 4.28c) and hence reduce the vibration transfer. In general the I_t is equal to the I_y for circular cross-sections. However, for non-circular shapes warping occurs which reduces the effective torsion constant.

$$\phi = \frac{Tl}{GI_t} \tag{4.4}$$

Figure 4.31 shows the effect of increasing the torsion constant. It is observed that the change predominantly affects the OS-RMS$_{90}$ value at adjacent units, as was expected, although not as dominantly as was the case for bending stiffness. It has to be noted that this is the case for a hinged supported floor system, which means there is considerably less torque compared to when the floors are fixed to the beam.



(a) Additional wall (bending and torsion) (b) Larger profiles (bending stiffness) (c) Hollow sections (torsional stiffness)

Figure 4.28: Schematic representation of junction adjustments.

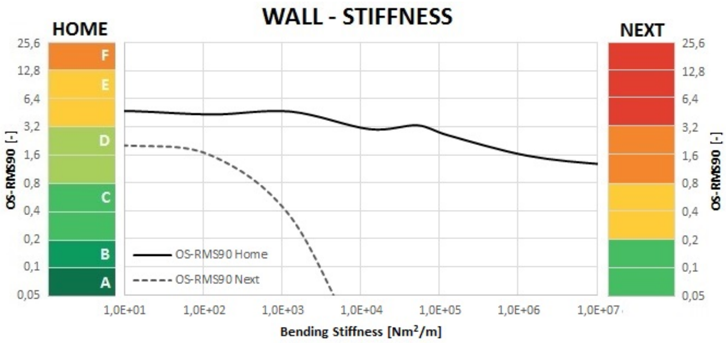


Figure 4.29: OS-RMS$_{90}$ for varying wall stiffness.

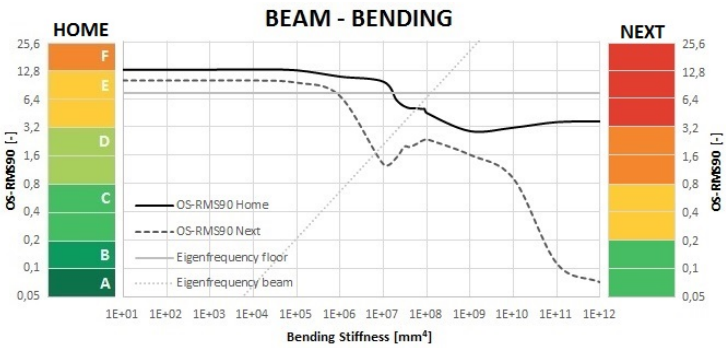


Figure 4.30: OS-RMS$_{90}$ for varying bending stiffness beam.

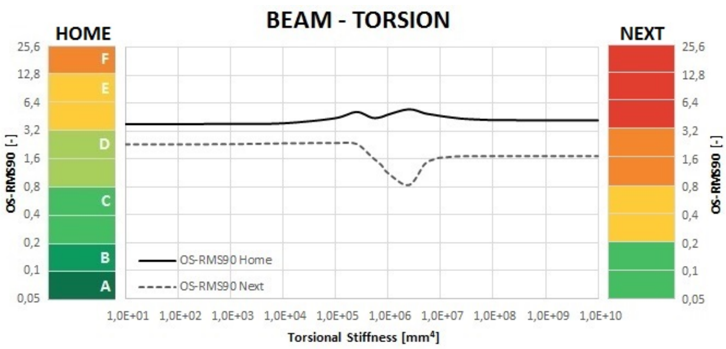
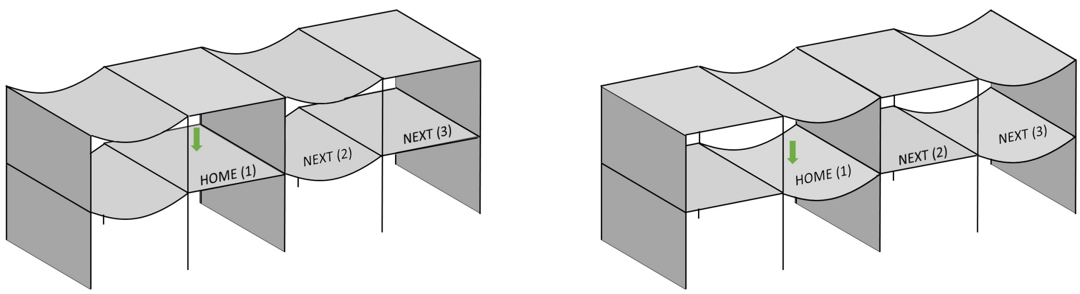


Figure 4.31: OS-RMS$_{90}$ for varying torsion stiffness beam.

Alternating floor systems
What follows from the previously discussed results is that mode-coupling is an undesired occurrence, especially regarding transmittance to receiving floors. Coinciding natural frequencies tend to transfer the vibrations more easily, creating a less comfortable climate. Avoiding this overlap of the individual structural properties is becoming a necessity in designing light-weight building systems. Not only the natural frequencies of the excited floor field and its supporting beams are of interest, but by creating an alternating system regarding different floor types it could also be possible to interfere with the transmittance. By varying the geometry and the material properties, it should be possible to create enough space between the fundamental natural frequencies to affect the flow of the vibration. The upcoming subsections describe the effects of alternating floor fields by changing either the stiffness, span or mass, the main components of the natural frequency.

Stiffness
By changing the stiffness of the individual floor fields, two situations are created. In situation 1 (figure 4.32a) the floor with the higher stiffness is loaded, whereas in situation 2 (figure 4.32b) the floor with the lower stiffness is loaded. The difference in stiffness is created to avoid mode-coupling and to reduce the transfer to neighbouring units. However, the governing values should be the maximum of both cases, simulating the worst results possible for both the home and neighbouring situation.
Figure 4.33 shows what happens in situation 1 when the stiffness of the loaded floor (1) is increased, whereas the neighbouring floor (2) maintains a constant rigidity and thus a constant natural frequency. The results for a third field (3) are also indicated since it can not be the case that this floor field will become governing as the adjacent unit.
Figure 4.34 indicated the results for situation 2 when the loaded floor maintain the same rigidity and the adjacent floor field is made stiffer. Again the results for the third floor field are added to make sure this field does not overrule the second field as governing neighbouring unit.
Both figures show results based on two events. Primarily it can be observed that for floors with increased stiffness also the level of vibration comfort improves. A seemingly logical occurrence since a stiffer structure tends to deflect less. In addition, it follows that for both cases the non-induced floor shows comparable results. A notable effect is the decrease in the third floor field, which, especially for situation 2, grows towards a governing value over the second floor field. Secondly, the effect of mode-coupling is visible around the spectrum of overlapping natural frequencies, indicated with the peaks in floor field (2) and (3). However, it is more likely that this is primarily happening due to the overlap of the properties of the floor and beam.
Figure 4.35 reveals that there is a slight improvement of the OS-RMS$_{90}$ value when the maximum values of both situations are combined. It is possible to conclude that for alternating floor systems the stiffness impairs a positive result to near floor fields whereas the home situation is barely changed. Avoidance of overlapping natural frequencies remains a critical point, showing the positive effect of changing these structural properties.



(a) Situation 1 (b) Situation 2

Figure 4.32: Alternating stiffness situations.

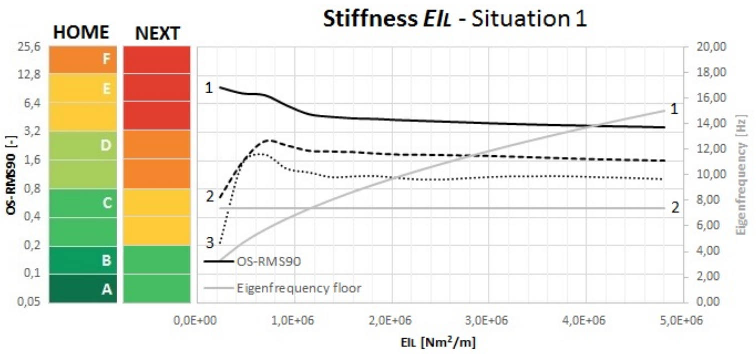


Figure 4.33: OS-RMS$_{90}$ and natural frequency for alternating floor stiffness (situation 1).

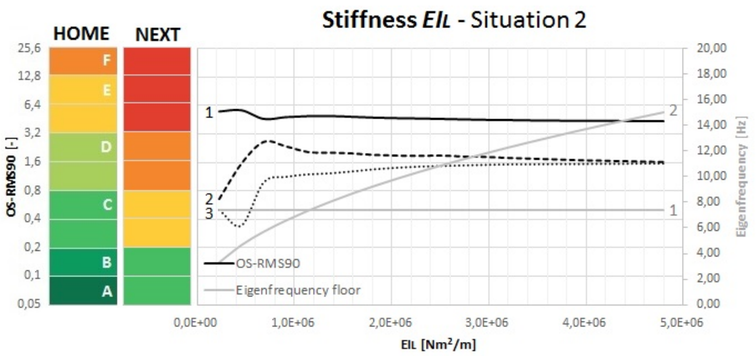


Figure 4.34: OS-RMS$_{90}$ and natural frequency for alternating floor stiffness (situation 2).

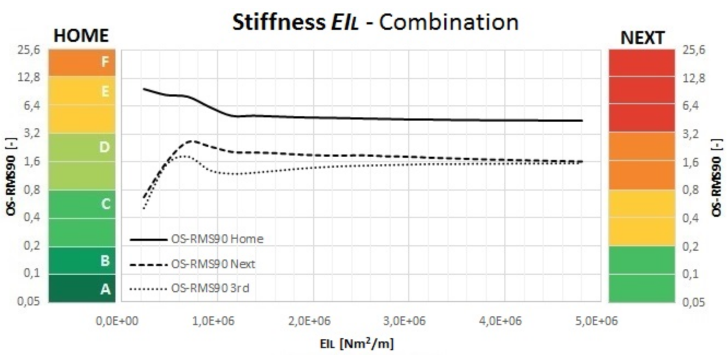


Figure 4.35: OS-RMS$_{90}$ for alternating floor stiffness (combined).

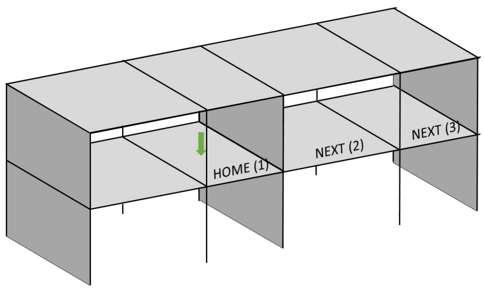
Span

Since the span works to the power four in the natural frequency equation (3.26) it seems like the most convenient aspect to converge the natural frequencies. However, the span is often difficult to change since it is bounded to the function and layout of a building system. A smaller span (for the same stiffness) tends to deflect less creating a more comfortable vibration climate. Smaller spans however also lead to more structural elements if the same floor area has to be maintained (beams, columns, connections, etc.). A good alternative is the alternating floor span, holding the same bay dimensions for the desired function but combined with uneven natural frequencies. The two cases for imposing a walking load on both the shorter and larger span are discussed below.

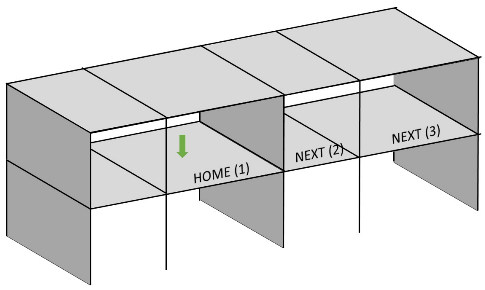
Figure 4.37 shows the results for situation 1, where the span of the induced floor field is varying from small (see figure 4.36a) to large (see figure 4.36b). It can be observed that for the home floor (1) a shorter span results in dropped OS-RMS$_{90}$ values resulting in a lower perceptibility of the vibrations and for larger spans vice versa. More striking is the effect of the alternating span on the adjacent floor field (2). It follows that the OS-RMS$_{90}$ drops to more acceptable values for both the cases where its span becomes larger or smaller. The alternating natural frequencies thus become a more substantial obstacle to overcome for the transmittance of the vibrations.

Situation 2, see figure 4.38, basically shows the mirrored results for situation 1, since the induced floor is now on the other side. Again it follows that now for larger spans a better OS-RMS$_{90}$ value is obtained whereas the alternating system shows improved results for both larger and smaller span cases.

The worst possible situations for both cases have to be taken as governing results. Figure 4.39 shows these combined values, implying an ever negative contribution to the OS-RMS$_{90}$ value in the home situation but an ever positive outcome for the OS-RMS$_{90}$ value in adjacent floor fields. Hence by avoiding mode-coupling, the transmittance of the vibrations is hindered creating a more comfortable environment for dynamic loads from the neighbours.



(a) Loaded small span



(b) Loaded large span

Figure 4.36: Alternating span situations.

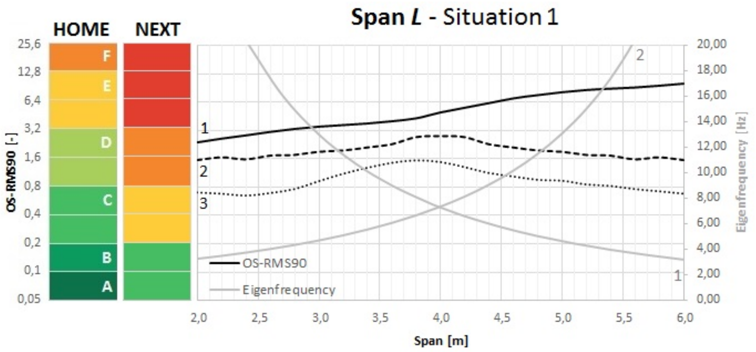


Figure 4.37: OS-RMS$_{90}$ and natural frequency for alternating floor spans (situation 1).

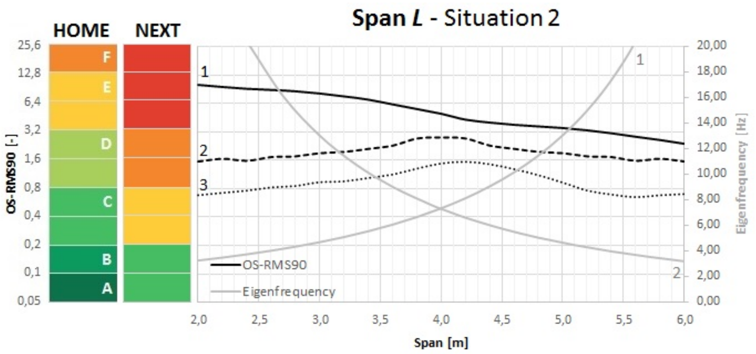


Figure 4.38: OS-RMS$_{90}$ and natural frequency for alternating floor spans (situation 2).

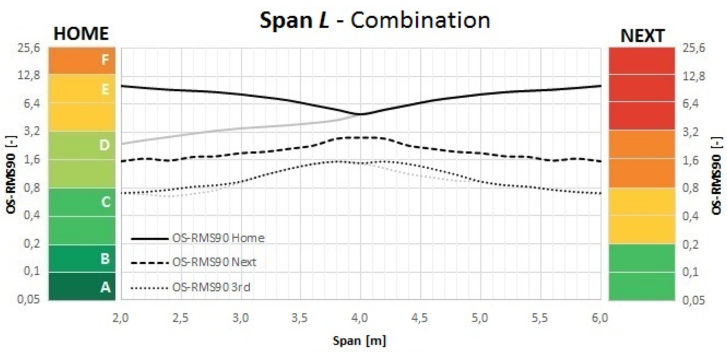
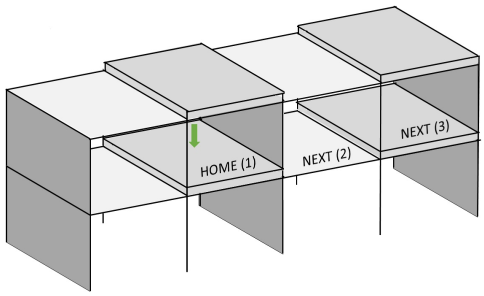


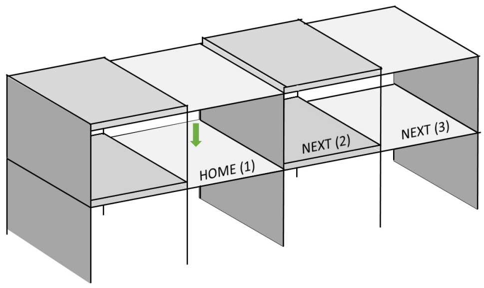
Figure 4.39: OS-RMS$_{90}$ for alternating floor spans (combined).

Mass

Mass is considered a limiting factor for vertical extension projects. However, it is a fundamental component of the natural frequency and modal mass of a structural element. Often mass is the result of structural adaptations that take part in the design of a building. By choosing a steel and timber construction method containing only one-fifth of traditional concrete building systems, small additions of the mass can produce inconvenience for the supporting structure.
The mass is, as mentioned, a result of changing the span or stiffness and hence the assembly of floor systems. Figure 4.41 shows the results for situation 1 (see figure 4.40a) in where the mass of the induced floor (1) is increasing. Figure 4.42 reveals the results for situation 2 in were the mass of the adjacent floor fields (2) is becoming larger (see figure 4.40b). What followed from the theory is that small changes in mass will not affect the OS-RMS$_{90}$ significantly, which is a positive consequence since mass is the result of other adjustments. The figures indeed show small differences for the home situation whereas due to the divergence of the natural frequencies of floors the OS-RMS$_{90}$ values tend to become more favourable when these properties move away from each other. Again avoiding mode-coupling is resulting in more comfortable living spaces when someone is walking on the other side of the wall.
Figure 4.43 reveals the combination of situation 1 and 2 proving the explained phenomenon of mode-coupling. Significant changes in the mass are not likely to occur however this will be deliberated more into detail in the following chapter.



(a) Situation 1



(b) Situation 2

Figure 4.40: Alternating mass situations.

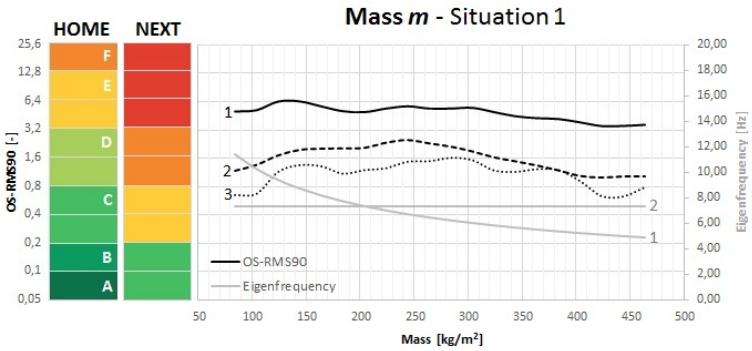


Figure 4.41: OS-RMS_{90} and natural frequency for alternating mass (situation 1).

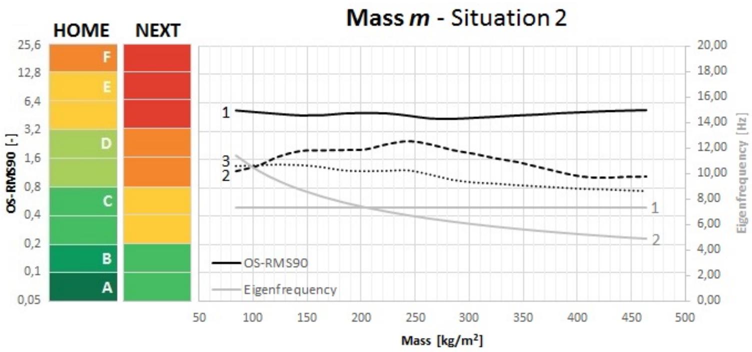


Figure 4.42: OS-RMS_{90} and natural frequency for alternating mass (situation 2).

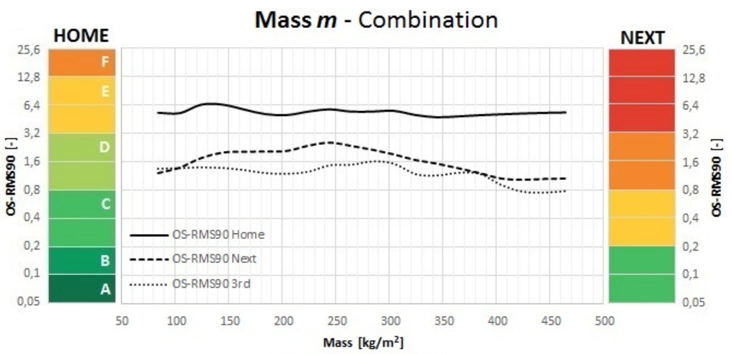


Figure 4.43: OS-RMS_{90} for alternating mass (combined).

4.3. Summary

As was conducted from the previous chapter the structural assembly of a light-weight steel and timber building system has to be modified to achieve vibration comfortable living areas. The theoretical values were translated into construction characteristics in the form of material properties, geometry and boundary conditions. What followed was a traditional but somewhat conservative approximation in the form of hand-calculations and a SDOF numerical analysis, showing the effects of variations of the offered tools on a floor its response to vibrations.

This SDOF approach revealed a substantial impact of the bending stiffness EI_L and the span L of a floor to the natural frequency and modal mass and hence to the OS-RMS$_{90}$. The influence of mass modifications is almost negligible to the comfort level but primarily has to stay within the limits of the bearing capacity of the substructure. A distinction was made between four common cases with different boundary conditions (hinged or fixed and spanning one-way or two-way). Especially for the four-sided supported floor fields, the transverse properties showed useful contributions for improved comfort levels.
A more accurate reflection of the floor response was revealed when the hinged, two-sided supported field was approached as a MDOF system in SoViST. A building system regarding multiple structural living units was modelled, showing the interaction of individual elements in a complete structure. Where the single components could ensure fulfilling natural frequencies, it was proved that the combination of loose elements would not consequently result in satisfying characteristics. This MDOF approach showed similar, however less strong effects for varying building tools and introduced a new phenomenon; mode-coupling.
It seemed to be of critical importance that for enhancing the OS-RMS$_{90}$ value at the receiving floors the natural frequencies of the loose elements should not coincide but should diverge to create more obstruction for the vibration transmittance. This can be done by either changing the properties of the intermediate beam or by creating an alternating floor field, where the characteristics of adjacent fields vary.
Not all adjustments tend to cooperate in a more comfortable living space in both the home and neighbouring situation. Adding stiffness and reducing spans (or changing support conditions) will predominantly result in more favourable OS-RMS$_{90}$ values in the home situation. Increasing the stiffness (both bending and torsional) of supporting beams or using an alternating floor system produce better OS-RMS$_{90}$ values for adjacent rooms.

Implementing the adjustments as mentioned above can theoretically continue infinitely. However, the practical execution will show limits regarding the attainable dimensions and profiles and will hence affect the assembly of the structure. This creates new limits within the height and mass of a building kit and is further explored in the coming chapter.

5

Practical building aspects

5.1. Practical description

The typical structure for light-weight building systems combines a steel framework with timber floors, as can be found in the floor plan of the 'De Karel Doorman' (see figure 5.1). In here it can be observed how the regularity of the floor spans appears, which is conventional for traditional building systems. A typical single apartment is indicated within the thick lines and exists of two by two floor fields. This reveals the presence of different type of transitions between floors. On the one side, a junction will separate the apartment from the neighbours whereas on the other side the junction appears within the home situation.

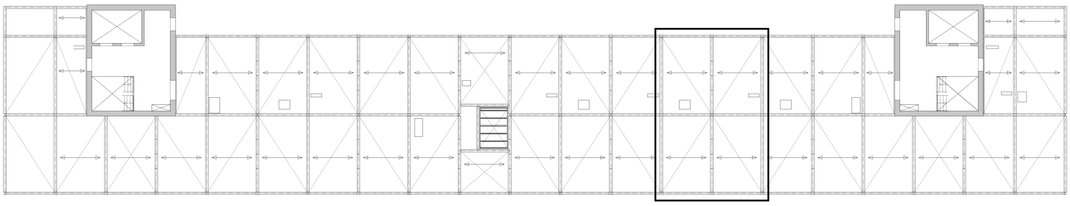
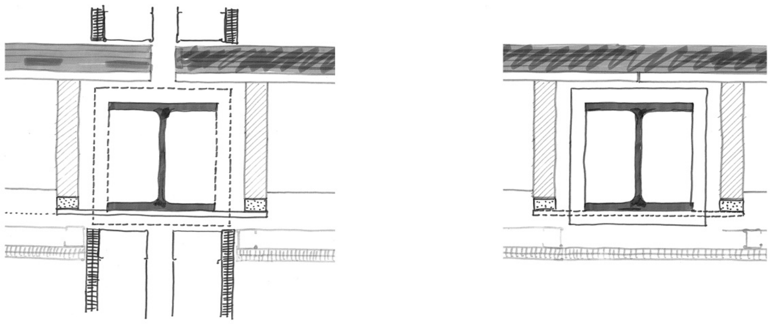


Figure 5.1: Floor plan 'De Karel Doorman'.

Taking a closer look at the schematic visualisation of these junctions and connections, as are shown in figure 5.2, some practical delimitations appear when alterations are considered. This is the case since the previously discussed theoretical values have to be translated to actual dimensions and available profiles, matching with the building regulations and limits of vertical extension projects.



(a) Home-separating junction (b) In-home junction

Figure 5.2: Schematic build-up steel and timber junctions.

The steel frame system is considered as most suitable for vertical extension projects by combining its strength to weight ratio. It makes use of optimised profile shapes of beams and columns together with a high level of flexibility and erection speed by using prefab elements and dry connections. For this building frame the main aspects that could have an impact on the level of vibrations are the distance between supporting beams (i.e. span) and the type of profiles used for these elements that carry the floors and its imposed loads.
The floor system has to produce a sufficient amount of strength and stiffness to guarantee the safety and serviceability of the structure within the boundaries of a maximum weight per area. In general, every type of floor has a different composition determining the detailing of the structural junctions. Where traditional heavy-weight floors are connected to the main supporting structure through the use of integrated beams, there is a distinction for light-weight floors. These floors are mostly fixed to the beams using simple supports after which these beams are connected to the main structure. This creates an essential difference in the transmittance of the vibrations compared to integrated heavy-weight floors.

The build-up of a standard timber floor is composed of joists with a specific width, height and spacing topped with a timber decking plate of a certain thickness and a screed layer. The joists take up the majority of the contribution to the bending stiffness of the plate [37], making it the most valuable parameter in floor designs to create a comfortable living space. The majority of the mass however, comes from the screed layer used to even the floor and accumulate for thermal and sound insulation.

The classification of the joints can be crucial regarding the structural integration of the complete system. The variation in rigidity, stretching from fixed to pinned connections, can produce severe differences in the transmittance of vibrations [17]. A balance in costs and execution-time between labour and material has to be found while providing the right amount of stiffness in the joint.
The effects of different classifications of the joint types can result in other required beam sections. The changed bending stiffness and rotational stiffness will then again bear higher or lower resistance to participation in vibration fluctuations.

A boundary to which vertical extension projects, within urban densified areas, have to act is the industrial aspect. By generating a robust typology, to be used for multiple cases, it is possible in an early stage to give the certainty of designing quality buildings. Also, the flexibility regarding new functions of a building has to be considered, demanding free open spaces without large obstacles. For environmental reasons a demountable structure requires dry connections, joining elements mainly with bolts. This also goes for the logistic aspect for vertical extension projects. It is unwanted to cause hindrance over a long period, requiring an fast on-site assembly method using prefabricated products. Especially for extension projects, the most critical limiting factor is the weight per area of the building assembly. A stiff enough floor assembly still has to be constructed but without the weight and height of the kit becoming disproportionate.

5.1.1. Cases and reference

In the following sections the practical building tools will be adapted working towards an optimal configuration. Firstly the individual parameters regarding joist profiles, span direction and lengths, beam profiles and type of connections will be shown both numerical for the input and visually for the vibration response of the floor fields. This should also lead into more insight in the actual flow of vibrations in floor fields.
To achieve a better interpretation of the vibrations a different software programme is used, namely Autodesk Robot. With this software it is possible to simulate footfall analyses for certain walking frequencies (Hz), a number of footsteps and the weight of a moving person (kg). A benefit from this tool is that the excitation and response are not bounded to the centre of a floor, showing the complete behaviour of the system. The disadvantage however, is that Autodesk Robot does not calculate the OS-RMS$_{90}$ value but only indicates the root mean square velocity (vRMS) for a single person with a set walking pace and bodyweight.

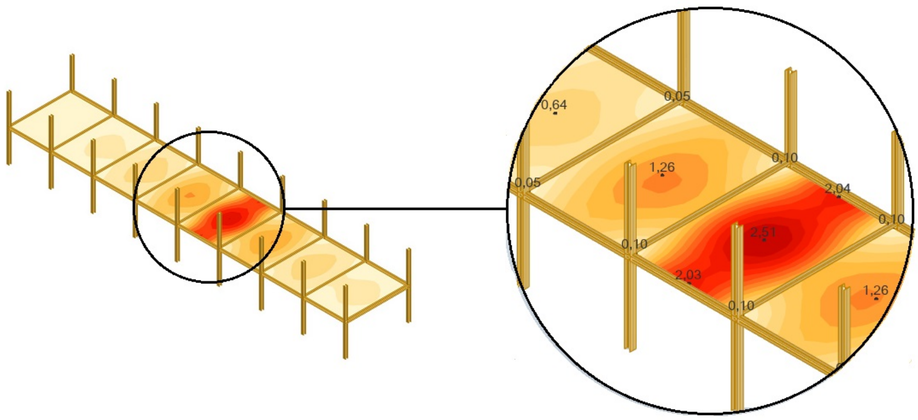


Figure 5.3: Footfall mapping on vRMS scores of 'De Karel Doorman' for an average person.

As reference project the 'De Karel Doorman', as initially designed, is used to indicate how practical alterations affect the comfort level. The input for the footfall analysis is based on the average person and will result in a mean OS-RMS value of the floor. This person has a walking pace of two steps per second (2 Hz) and a body mass of 75 kg. Figure 4.21, same as test results performed by TNO [19], show that for this mean person the single vRMS value tends to be too optimistic regarding the OS-RMS$_{90}$ score for a representative population. A multiplication factor of approximately 1.5-2.0 between this single vRMS value and the OS-RMS$_{90}$ score follows but still has to be questioned since it will be case sensitive for structural systems their natural frequencies. Hence, the results from Autodesk Robot have to be considered within a certain range of deviations to give an equivalent indication as the OS-RMS$_{90}$ does and tell something valuable about the actual comfort level.

Table 5.1 indicates the characteristic values for both the individual elements and the combined system of 'De Karel Doorman'. The governing limiting consequences are formulated as the average mass per square meter and the height of the floor assembly. The numerical results from calculations in Autodesk Robot (see figure 5.3 are shown with the vRMS velocity and from SoViST with the OS-RMS$_{90}$ value. These two values indicate the difference in interpreting results for a single input and population representative data. Also the fundamental natural frequencies are depicted for both the SDOF and MDOF systems.

Table 5.1: Characteristic values 'De Karel Doorman'.

	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS$_{90}$ [-]	'DKD'
sending floor	204	225	7,35	2,51	**5,03**	-
beam	13	210	4,89	-		
receiving floor	204	225	7,35	1,26	**2,06**	-
system	231		5,33			

In the following sections, this case will be taken as the reference point to which the variation in OS-RMS$_{90}$ scores for both the sending and receiving floor will be indicated in the 'DKD' column.

5.2. Practical implementation

The following sections will show carefully chosen practical considerations regarding the main factors of influence for vibration comfort. Both for the sending as the receiving floor the impact will be displayed visually for vRMS scores and numerical for the OS-RMS$_{90}$ scores. The individual methods are described by means of floor assembly characteristics, structural profiles and type of junction connections.

5.2.1. Longitudinal bending stiffness

The flexural rigidity of a floor is dependent on the combination of the Modulus of Elasticity (E) and the second moment of area (I). By increasing the efficiency of the cross sections resistance to bending, it is possible to enhance the stiffness. This can be done by changing the geometry of the cross sections of the floor elements. As mentioned before the highest contribution to flexural rigidity in timber floors comes from the joists.
For an arbitrary rectangular timber joist profile the height contributes to the power three over the width in determining the second moment of area. When establishing the bending stiffness (EI_L) of a floor system (expressed per stretching meter), the height, width, spacing and joist profile types are considered as critical factors.

Joists are fabricated in standard dimensions due to ease of manufacturing. In this section the effects of various sized joist profiles (see Appendix B.1) on the bending stiffness and mass of a floor are shown. This appendix also indicates typical joist heights and how wider beams and reduced spacings can contribute to more bending stiffness, but also more weight. The dimensions and capacities are based on the available Kerto LVL S-beam data retrieved from timber producer Metsa Wood.

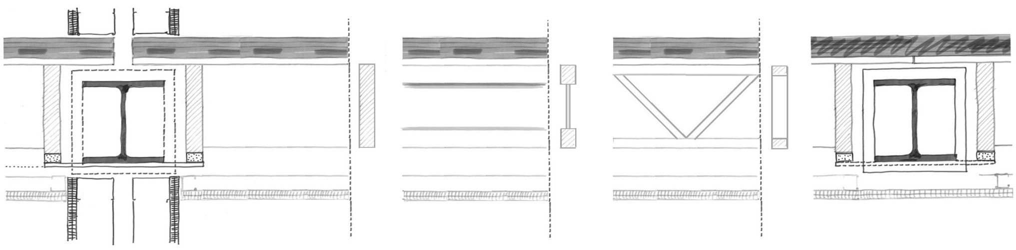


Figure 5.4: Schematic depiction of various joist profiles (resp. solid, I-joist and open-web).

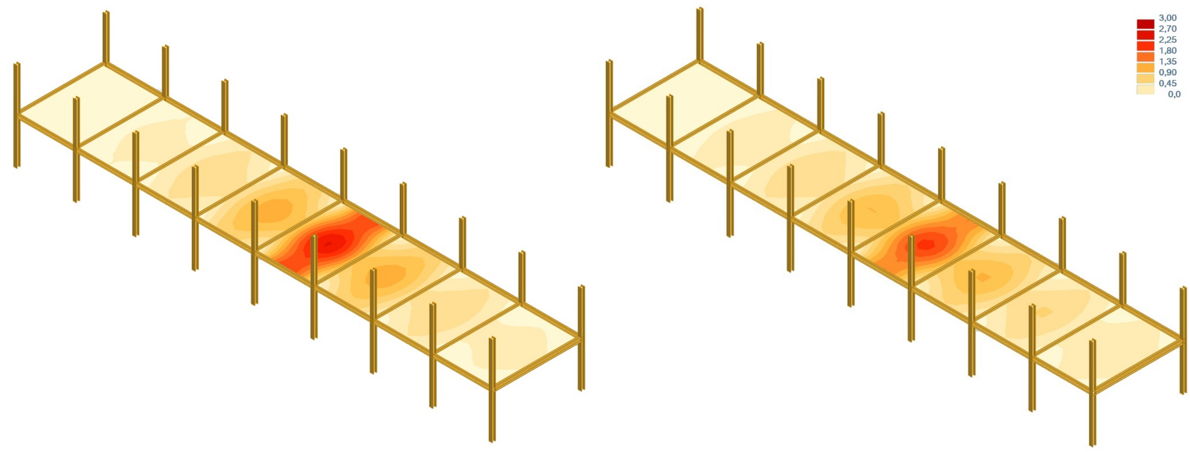
Since mass is a bounding parameter for the design of vertical extension projects, the positive stiffness effects due to changes of the floor assembly have to outweigh the disadvantage of additional mass. Using other types of joist profiles than the standard rectangular solid beam could be a smart alternative. Especially when the area of the cross-sections is spread more efficiently, i.e. more area away from the centroidal axis, the stiffness to mass ratio can grow. I-joists are an example of these type of beams to use in the assembly of a floor.

The balance between mass, height and stiffness of the floor has to be considered when choosing the right type of joist profile. Appendix B.1 shows the available profile dimensions and reveals the coherence between these factors for I-joists with a certain height and increased width and smaller spacings. It becomes clear that by using these type of joist profiles, beneficial results can be obtained compared to the rectangular joist profiles. To achieve the same bending stiffness less material and hence less mass is required. Converting this profit per square meter into a certain weight that has to be carried by a single column, multiplied by the number of stories, means that every small reduction can have significant effects.

Comparing the original design of ‘De Karel Doorman’ to an alternative with I-joists, reveals how smart and efficient material use leads to improved vibration comfort. Table 5.2 and figure 5.5 show how the two different type of joist profiles, with the same height and mass per area, differ in response. The results show that by efficiently shaping the sections of profiles a small benefit can be obtained without major modifications.

Table 5.2: Characteristic values I-joists ‘De Karel Doorman’.

	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS$_{90}$ [-]	'DKD'
sending floor	204	225	8,25	2,11	**4,39**	-12,7%
beam	13	210	4,89	-		
receiving floor	204	225	8,25	1,13	**1,96**	-4,9%
system	231		5,62			



(a) Rectangular joists, vRMS (2,51-1,26)

(b) I-joists, vRMS (2,11-1,13)

Figure 5.5: Footfall analysis for different type of floor joists.

Another alternative is the use of metal-web girders. By doing so a timber top and bottom rail are connected with metal webs joining them, creating a robust girder-like profile that can span wide areas. The disadvantage however, is that the level of damping for these profiles will decrease significantly [37]. The consideration between this effect and reduced mass has to be made.

Not just the type of the profile but also the positioning of the joists can lead to more control of the vibration comfort. By smartly placing the stiff elements the local static deflection can be reduced. As the footfall mapping of the original system shows, the largest deflections and vibrations can be observed at the centre and free edges of the plate. For one-way span floors additional or stiffer joists at these edges can result in the plate behaving towards a four-sided supported floor, creating more opportunities for transverse stiffness to participate in the vibration control.
Choosing deeper joist profiles to enhance the vibration comfort will have consequences on the assembly on the floor. It is preferable to place the joists closer together by using a smaller spacing. This can also reduce the local deflections that occur between the placed joists [37].

5.2.2. Transverse bending stiffness

Following from the numerical evaluations for vibrations, it is found that the first values for natural frequencies are not that far apart from each other. This clustering comes forward, as earlier described, due to the strong orthotropic properties of the floor. This clustering also has the effect of increasing the amplitude of vibrations and thus creating a less comfortable construction system [10].
By introducing bridging between the joists, either with solid blocking, cross-bridging or strapping (see figure **??**), a higher transverse stiffness will be created. This makes it possible to influence the spacing between the higher natural frequencies and thus the vibration response [37]. An additional benefit is the positive effect on the torsional rigidity of the joists, preventing torsional buckling of these elements [10]. In the theory it was found that the most practical method of bracing joists for light-weight floors is cross-bridging combined with strapping to the bottom of joists as it increases the natural frequency the most [16].

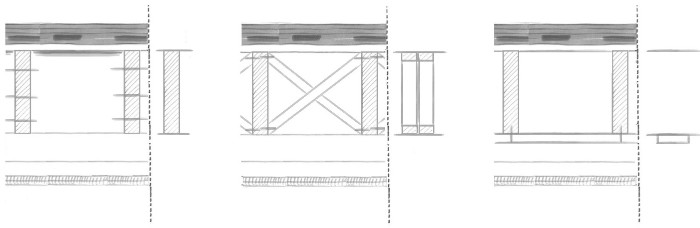


Figure 5.6: Schematic depiction of various transverse bridging (resp. solid blocking, cross-bridging and strapping).

The figures on the next page indicate how changes of the transverse stiffness affect the footfall response for different supporting conditions. Figure 5.7a shows the original system with an orthotropic floor assembly spanned in one direction. It can be observed that for this span type the orthotropic floor behaves as a wide beam with a somewhat constant level of vibration response over the width. At the ends of the floor, where it is not connected, the plate tends to oscillate more given the lack of resistance due to fixations. For an isotropic plate (figure 5.7b) the floor shows a more even spread of the vibration response levels, fading out towards the boundaries. This can be explained since the stiffness in both directions is evenly high. Table 5.3 indicates the characteristic values for this case using the numerical models from Autodesk Robot and SoViST. An isotropic floor requires additional volume and hence extra mass. However, an improvement in the vibration comfort is noticeable.

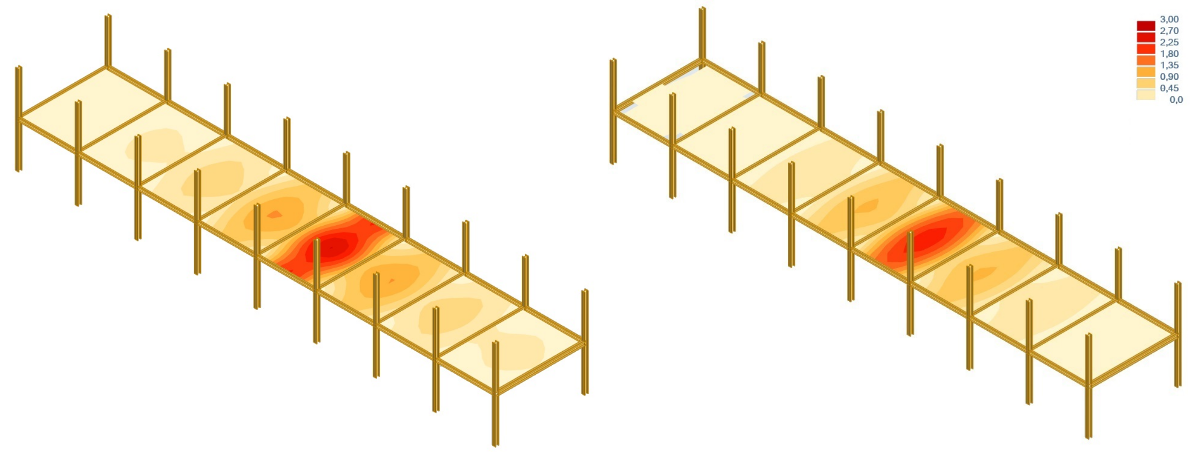
Table 5.3: Characteristic values isotropic floors 'De Karel Doorman'.

	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS$_{90}$ [-]	'DKD'
sending floor	212	225	7,35	2,16	**3,96**	-21,3%
beam	13	210	4,89	-		
receiving floor	212	225	7,35	1,02	**1,59**	-22,8%
system	239		5,57			

For less common two-way span floors the higher natural frequency and lower modal mass result in stronger vibration velocity values centring in the middle of the floor. As figure 5.8a shows, the vRMS slightly worsens from 2,51 mm/s for one-way spans to 2,77 for two-way span floor. In this case of a floor supported on four sides it can be observed that when the longitudinal to transverse stiffness ratio rises towards that of an isotropic floor, the floor velocity drops more significantly (with 30% from 2,77 to 1,93 mm/s) opposed to the two-sided supported case (with 14% from 2,51 to 2,16 mm/s).
Regarding the transfer of the vibration to neighbouring units the influence of transverse stiffening is comparable. Similar results as for the home situations follow making it an overall enhancing adjustment.

As is shown not just the amount but also the ratio of the transverse stiffness EI_B to the longitudinal stiffness EI_L influences the possibilities of steering the vibration levels. Same as for the longitudinal stiffness, it requires additional material and thus mass to achieve the desired stiffness values. Again a balance has to be found between the mass, height and stiffness of the structure.

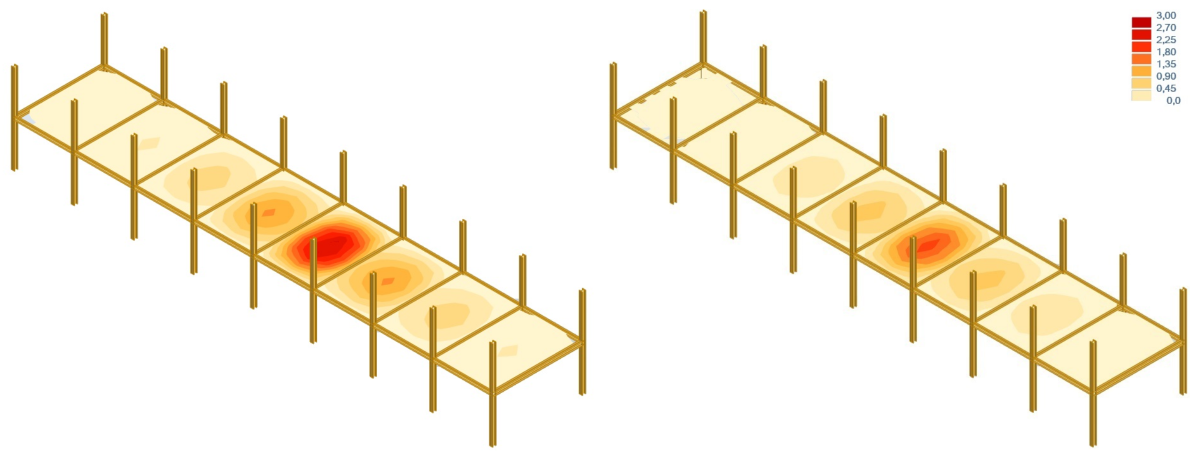
The clustering of the natural frequencies as mentioned can be stretched by increasing the transverse stiffness working towards an isotropic floor. By reducing the width of the floor this effect can also be achieved, however this is often not considered practical. An alternative, as was done in ‘De Karel Doorman’, is to construct the floor out of two elements with half the total width. A disadvantage is the reduced load sharing effect at the intersection point of the loose floors. Additional local stiffening can counteract this problem, just as the fact that a continuous screed layer will be applied on top of the floors.



(a) Orthotropic floor, vRMS (2,51-1,26)

(b) Isotropic floor, vRMS (2,16-1,02)

Figure 5.7: One-way span floors.



(a) Orthotropic floor, vRMS (2,77-1,33)

(b) Isotropic floor, vRMS (1,93-0,88)

Figure 5.8: Two-way span floors.

5.2.3. Floor span

The span is considered a valuable parameter when steering building system designs towards acceptable vibration comfort levels. It was found that the impact on the natural frequency can be significant, however, smaller spans also reduce the activated modal mass. Modern building codes involve the span as an important factor for serviceability design, both deformation-related and motion-related.

The general rule of thumb for deformation serviceability design is to limit the maximum deflection. This is done to avoid excessive sagging of floors or possible cracking of both structural as non-structural elements. For standard floors the following equation holds, in where the deflection can reach up to a specific factor of the floor span:

$$\delta_{max} = \frac{5}{384}\frac{ql^4}{EI} \leq \frac{l}{250} \tag{5.1}$$

Motion-related criteria tend to limit the discomfort of occupants by avoiding resonance of the excitations in the range of 1 to 8 Hz with the natural frequencies of building structures. An important parameter of this natural frequency is the span of a floor, as can be found in the general formula 4.1. The level of discomfort is indicated by the acceleration or velocity of the structure and has to remain below a certain threshold.
By taking into account a change of span according to deformation-related criteria, the required stiffness can change accordingly. Although the theoretical effect of the span is relatively high, the absolute implementation of the stiffness must not be neglected. For light-weight building systems it will prove to be of interest to not let the bending stiffness vary according to the deformation criteria.

The figures on the following page show the footfall analyses of several cases where the span, and corresponding deformation-related stiffness, is changed. First, it is checked what happens when the bay dimension is changed from two times a 4.0-meter span (total of 8.0 meters) to three times a 2.7-meter span (total of 8.1 meters). The results in table 5.4 and figure 5.9a indicate that even though the span is reduced the vibration comfort remains the same in the home situation and decreases for adjacent floor fields, probably because the path to this field is shorter and the vibration is dissipated less due to damping effects. Had the bending stiffness not been changed according to formula 5.1 but stayed constant to the original situation the OS-RMS$_{90}$ would have dropped with 36% to 3,21 for the excited floor and with 13% to 1,79 for the receiving floor. This proves that the span and stiffness are two individual properties of the floor that both affect the comfort level significantly and for serviceability criteria should not be considered as conditional factors to each other.

Table 5.4: Characteristic values 2.7 meter span 'De Karel Doorman'.

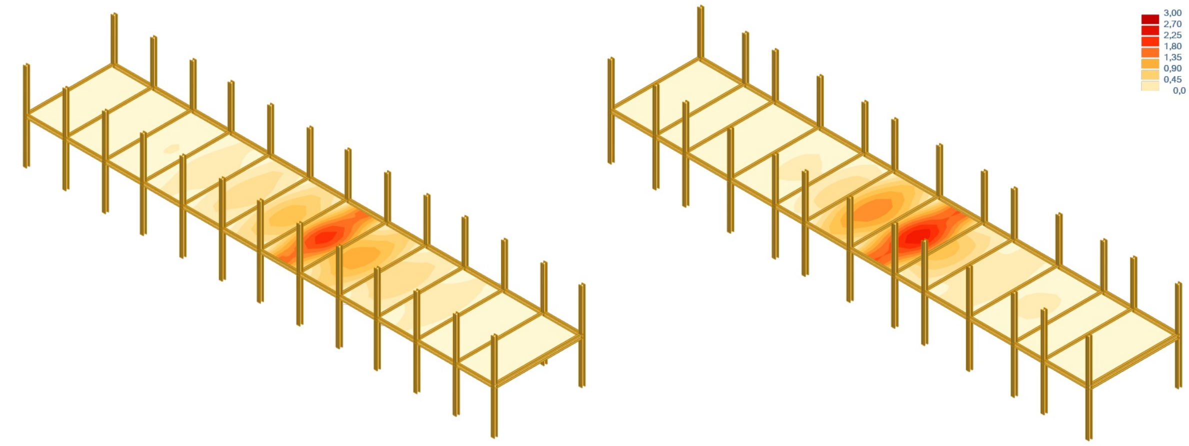
	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS$_{90}$ [-]	'DKD'
sending floor	200	200	9,33	2,18	**5,06**	+0,6%
beam	19	210	4,94	-		
receiving floor	200	200	9,33	1,29	**2,67**	+22,8%
system	238		6,57			

By introducing the alternating floor system, with different spans and corresponding stiffness, several layouts can be produced. Figure 5.9b shows what would happen when an alternating 3.0-2.0-3.0 meter span (total bay width of 8 meters) is introduced. Since the 3.0-meter span bays are still adjacent to each other for bordering apartments, mode coupling isn't completely avoided. This configuration doesn't show promising results regarding improvements to the vibration comfort.
More practical for creating an alternating floor system is by changing the position of the supporting beam in the centre of an apartment bay. This way it is possible to avoid mode-coupling across the entire floor plan of the building, including to neighbouring apartments. Figure 5.10 shows the effects of an alternating 5.0-3.0 meter span floor layout. The table below indicates the corresponding normative results.

Table 5.5: Characteristic values alternating span 'De Karel Doorman'.

	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS$_{90}$ [-]	'DKD'
sending floor	201	200	8,98	2,80	**-**	-
beam	19	210	4,84	-		
receiving floor	214	225	6,37	0,86	**-**	-
system	238		5,68			

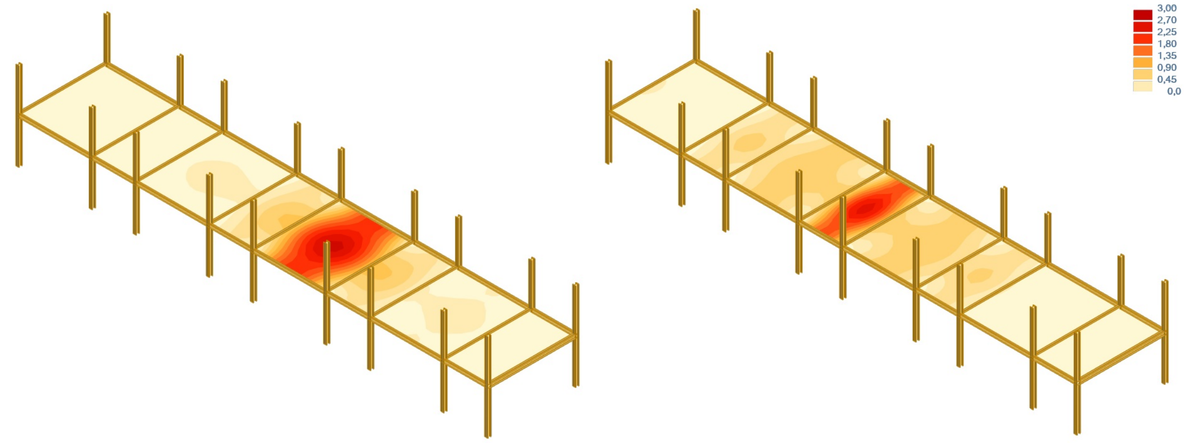
This type of arrangement will disturb the transmittance of vibrations to adjacent fields. The comfort level in the home situation will be less comfortable in the most unfavourable situation as the span is increased, but towards receiving floors the vRMS score drops with 32%. For these type of arrangements changing the stiffness of the floor not according to its deformation serviceability, as was also discussed by Hu [14], it is possible to enhance both the home and next door situation without requiring large adjustments.



(a) Triple 2.7 meter bay span, vRMS (2,18-1,29)

(b) Alternating 3.0-2.0-3.0 meter bay span, vRMS (2,45-1,55)

Figure 5.9: Small span floor systems footfall analysis.



(a) Alternating 5.0-3.0 meter bay span, vRMS (2,80-0,74)

(b) Alternating 5.0-3.0 meter bay span, vRMS (2,62-0,86)

Figure 5.10: Alternating floor systems footfall analysis.

5.2.4. Supporting beam profiles

By changing the cross-sectional shape of the floor supporting beams the mechanical properties will be affected. Efficient distribution of the material can lead to improved bending and rotation capacities. Results from the SoViST tool showed that especially the bending stiffness of a beam could produce significantly improved levels of vibration comfort in adjacent floor fields. The effects of torsion seemed to be a lot smaller, although the range of practical dimensions was not taken into account yet.

H-beams

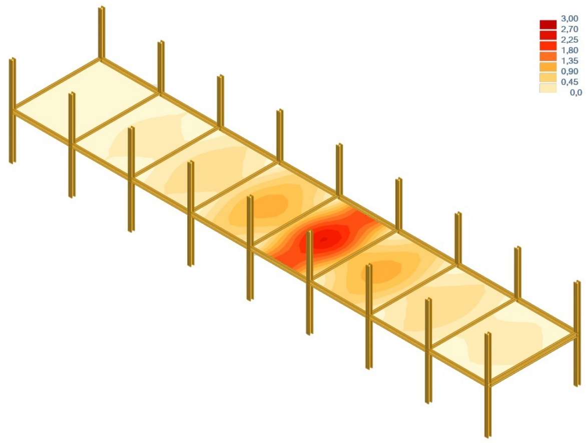
Appendix B.2 shows the effect of changing the type of H-shaped beam profiles for its bending and torsional stiffness capacities and the resulting additional weight of the structural assembly. To obtain higher levels of bending stiffness for these type of profiles, it is clear that larger sections have to be used. The lack of spreading in rotational capacities reveals that larger H-profiles do not lead to significantly better torsional stiffness. Besides the limiting factor of the additional weight, the larger profiles can also result in higher floor assemblies. Since the bending stiffness of a beam is considered as the most critical factor for interfering with the transmittance of vibrations, larger profiles will always contribute to better vibration control, providing that mode-coupling is avoided.

Figure 5.11: Schematic depiction of various H-beam profiles (resp. HEA220 and HEA300).

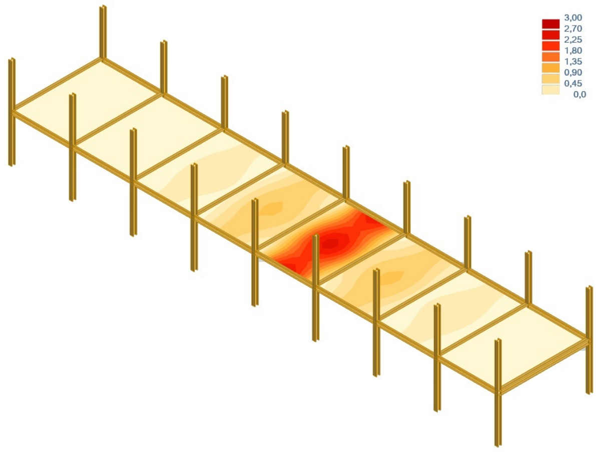
The figures on the following page indicate the footfall analyses for systems integrating the various type of HE beam conditions in the structural assembly. Figure 5.12a illustrates the original design with HE220A beam profiles and floors fixed as hinges to the beam. It can be observed that when using larger beam profiles, as is seen in figures 5.12b and 5.12c, the vibration comfort towards adjacent fields is positively affected. Table 5.6 indicates the characteristics of this structural assembly and reveals a substantial improvement of the vRMS score of 48% for receiving floors. As this value only shows the result for the case of a single person, here it becomes clear how for a representative population the OS-RMS$_{90}$ score gives a more weighted indication. This value only improves by 18%, which is still a considerable enhancement.

Table 5.6: Characteristic values HE300A beam ‘De Karel Doorman’.

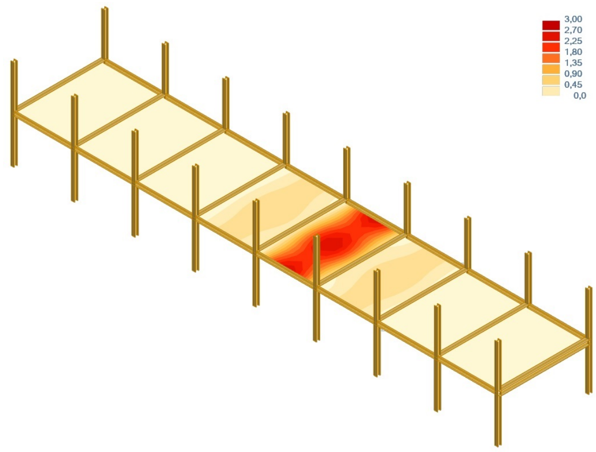
	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS$_{90}$ [-]	'DKD'
sending floor	204	225	7,35	2,69	**4,08**	-18,8%
beam	22	290	8,80	-		
receiving floor	204	225	7,35	0,65	**1,68**	-18,4%
system	241		6,48			



(a) HE220A, hinged, vRMS (2,51-1,26)



(b) HE260A, hinged, vRMS (2,55-0,98)



(c) HE300A, hinged, vRMS (2,69-0,65)

Figure 5.12: Footfall analysis, various HEA beam profiles.

Hollow sections

Hollow section beams, in general, have a higher torsion constant due to their continuously placed material away from the central point of gravity. Appendix B.2 shows a variation study for three type of hollow beams; the square hollow section (SHS), the rectangular hollow section (RHS) and the circular hollow section (CHS). In here the impact of the sectional shape on its stiffness capacities and weight is indicated. The differently shaped beams show varying values of bending stiffness and torsion stiffness. CHS-profiles, with similar profiles heights as SHS- and RHS-sections, have a lower bending resistance, but on the opposite show higher values for the torsional resistance.

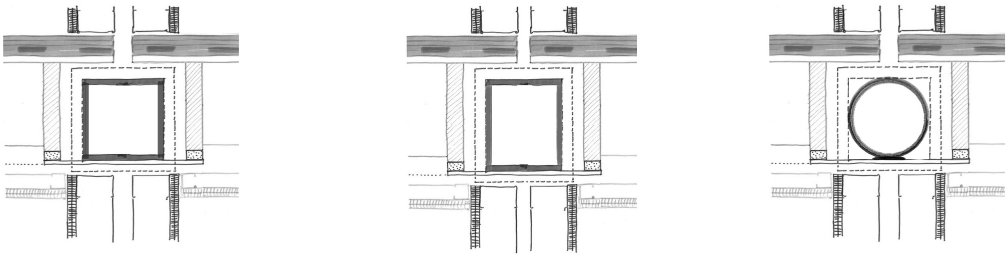
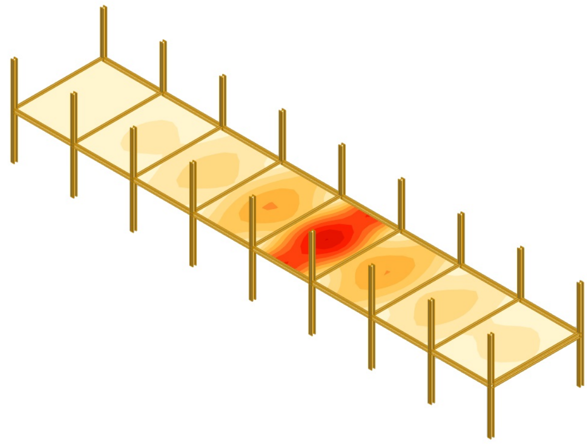


Figure 5.13: Schematic depiction of various hollow section profiles (resp. SHS, RHS and CHS).

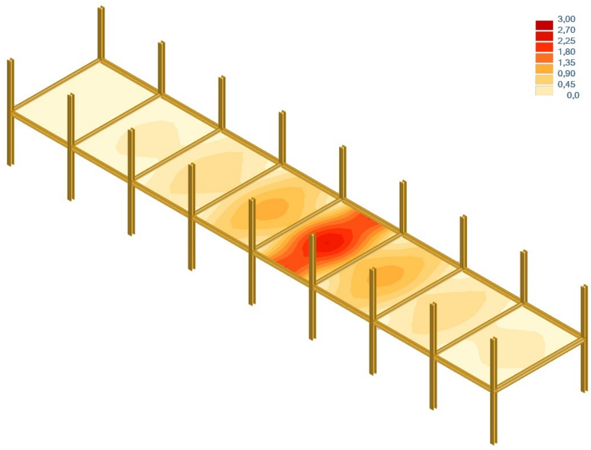
A comparison is made between the regular H-shaped beams and hollow section profiles. For these latter sections only small improvements to the vRMS scores are observed when increased torsional stiff profiles are used with a similar bending stiffness as the HEA profiles, see figure 5.14. However, regarding the OS-RMS_{90} scores improvements of around 15% for both the sending as receiving floor can be found. Compared to the HE300A beam configuration this indicates that by solely increasing the torsional stiffness similar improvements regarding hinder of the vibration transmittance are obtained. An example is indicated in table 5.7. In here the characteristics of a 200x200 SHS profile (with a thickness of 12,5 mm), that comply with the bending stiffness of a HE220A beam are revealed.

Table 5.7: Characteristic values SHS beam 'De Karel Doorman'.

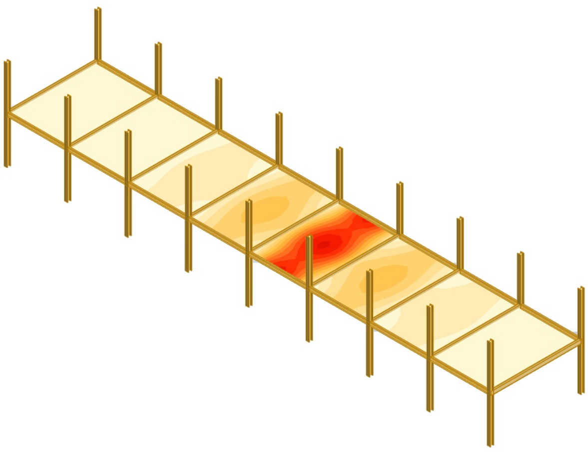
	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS_{90} [-]	'DKD'
sending floor	204	225	7,35	2,50	**4,28**	-14,9%
beam	18	200	4,80	-		
receiving floor	204	225	7,35	1,24	**1,71**	-17,0%
system	237		5,40			



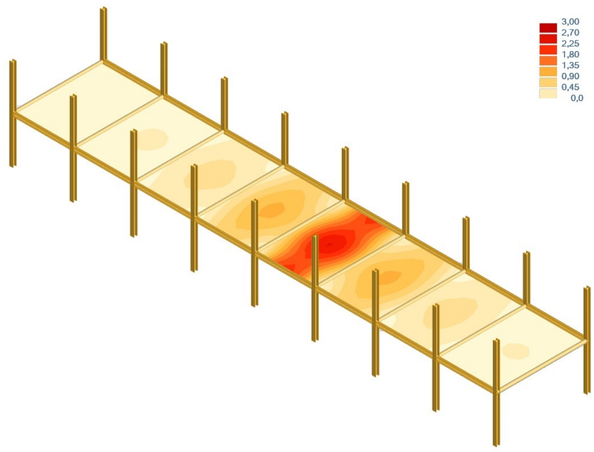
(a) HEA220, vRMS (2,51-1,26)



(b) SHS, vRMS (2,50-1,24)



(c) RHS, vRMS (2,56-1,04)



(d) CHS, vRMS (2,51-1,18)

Figure 5.14: Footfall analysis, various hollow section beam profiles.

5.2.5. Floor fixation

The type of fixation of floors to the supporting beams revealed to show strong deviations of vibration comfort levels. Especially the implementation of fixed supports opposed to hinged connections made the floors less susceptible to vibrations. The theoretical rigidity capacity varies between zero for hinged connections, and infinitely stiff for fixed links. However, practice will prove that the actual value will be somewhere in between.

Figures 5.16 and 5.17 show the change from hinged to fixed connections for both H- and CHS profiles. The footfall mapping indicates enhancements of the vRMS scores in the sending and receiving floors when fixed supports are used. Especially for profiles with higher torsional stiffness (i.e. hollow section profiles), the vibration velocity improves significantly with 41% in the home situation. Compared to the 11% decrease for the standard HEA beam profile this reveals that the section shape can contribute to a large extent in varying with the connection rigidity.

Table 5.8: Characteristic values fixed floor 'De Karel Doorman'.

	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	OS-RMS$_{90}$ [-]	'DKD'
sending floor	204	225	16,68	2,24	**2,72**	-45,9%
beam	13	210	4,89	-		
receiving floor	204	225	16,68	0,83	**0,33**	-83,9%
system	231		6,53			

Table 5.8 reveals the results of the original 'De Karel Doorman' building system with the exception that the floors are fixed with an infinite rigidity to the steel beams. Again it is revealed that the OS-RMS$_{90}$ scores indicate significantly different effects compared to the single person vRMS value. The reduction is found to be extremely positive for both the sending and receiving floor situation, caused by the doubling of the natural frequency.

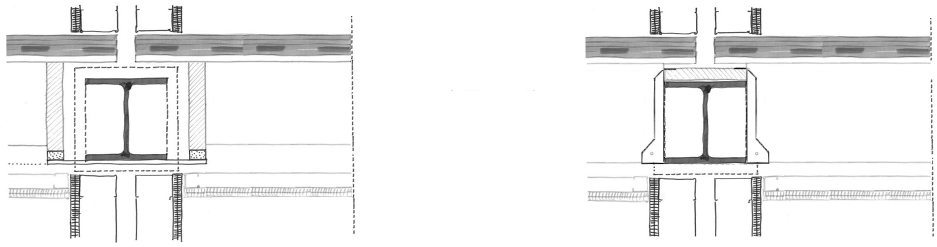
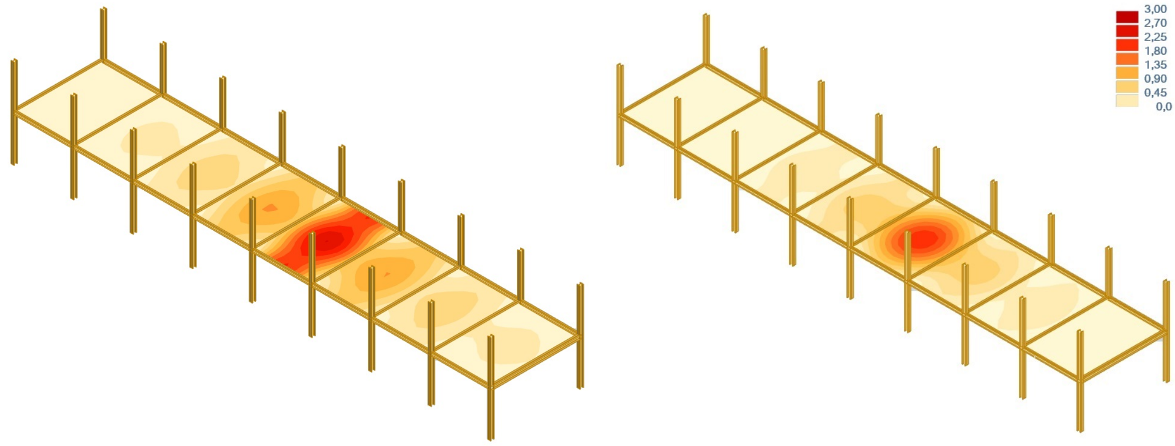


Figure 5.15: Schematic depiction of joist to beam fixation (resp. without and with bearings).

In practice, it is often found that the connections of timber floors to steel beams are designed as either hinged or near-hinged for ease of assembly purposes. The fixation can be performed in two methods. Either the joists of the floor are connected to the beam with bearing hangers, or the joists are placed on top of on of the flanges of the beam. Either way, the level of rigidity is not to such an extent that it can be considered as fixed. For the near-hinged connections it was found that the additional rotational stiffness does not affect the vibration performance of timber floor structures [17].

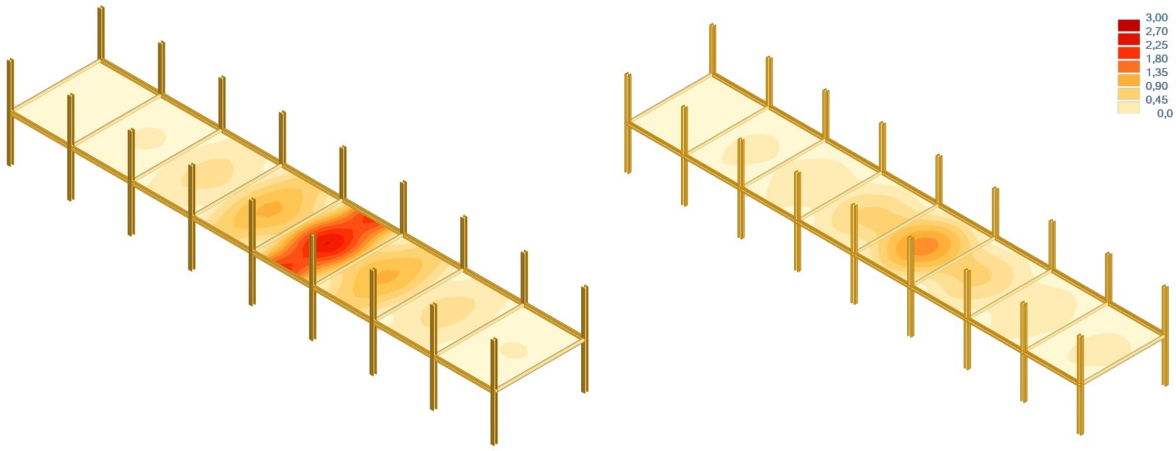
From a vertical extension point of view, it is found to be less time consuming to prefabricate the floor and place it on top of one of the flanges. By using slim floor beams, the support can take place on the bottom flange and a more integrated structure can be achieved regarding floor heights. This can add up to a substantial benefit if a large amount of additional topping levels for the extension is required.



(a) HEA hinged floor, vRMS (2,51-1,26)

(b) HEA fixed floor, vRMS (2,24-0,83)

Figure 5.16: Footfall analysis, HEA beams with different floor fixations.



(a) CHS hinged floor, vRMS (2,51-1,18)

(b) CHS fixed floor, vRMS (1,48-0,75)

Figure 5.17: Footfall analysis, CHS beams with different floor fixations.

5.2.6. Walls

Adding walls prove to be of significant efficiency when reduction of vibrations to neighbouring units is of importance. By implementing stiff elements between the supporting beams situated above one another, it is possible to affect the bending and rotational capacities of the junction. The downside of adding structural elements in the wall however, is the impact on the flexibility of the floor plan.
Table 5.9 shows the effect of adding slender steel columns where the wall is located to improve the stiffness at the junction. Three small 120x120x5 SHS profiles are connected between the supporting beams at a heart to heart distance of 1.5 meters contributing to the dynamic stiffness of the structure. The table below indicates the resulting floor response for the case where every other junction is enhanced with the slender steel profiles. This is done because not every node will allow for the placement of these elements as they can be within the floor area of a single apartment unit. The *OS-RMS$_{90}$ values are based on a situation where the additional beams are placed between every junction as it wasn't optional to distinguish different junction types across a floor plan in the SoViST tool. Therefore, the resulting values will be a bit more progressive that reality proves to be.

Table 5.9: Characteristic values stiff walls 'De Karel Doorman'.

	mass [kg/m^2]	height [mm]	f_n [Hz]	vRMS [mm/s]	*OS-RMS$_{90}$ [-]	'DKD'
sending floor	204	225	7,35	2,33	**1,79**	-64,4%
beam	13	210	4,89	-		
receiving floor	204	225	7,35	0,02	**0,03**	-98,5%
system	238		6,03			

What follows from this structural assembly is that these small interventions have a massive contribution to the transmittance to neighbouring floor fields. This can also be observed in the footfall mapping displayed in figure 5.18. The practice shows that this extremely favourable OS-RMS$_{90}$ value can be obtained, however does require large obstructions in the floor plan. Had only one slender column been placed in the centre of the beams an *OS-RMS$_{90}$ value of 0,09 would follow. The negligible additional mass makes it the most convenient method to interfere with the vibration transmittance.

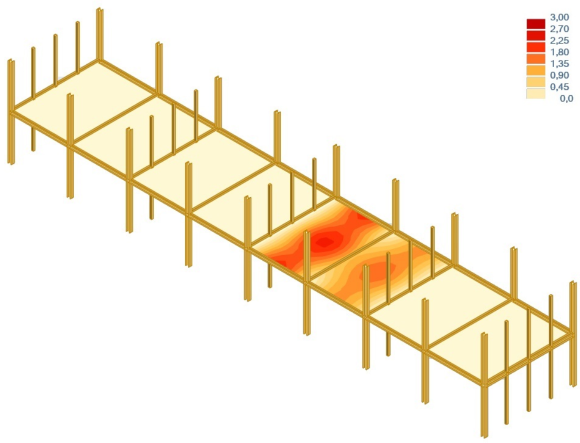


Figure 5.18: Footfall analysis, added stiffness in walls.

5.3. Summary

The previous section showed the possible practical implementations and their effects on the system its mass, height and natural frequency characteristics. More importantly, the results of the modifications revealed to what order the OS-RMS$_{90}$ scores for home and neighbouring situations could be steered. As was found, not all interventions contribute to the same extent or the same part of the construction. To make sure comfortable enough criteria are met in both the sending as receiving floor field a combination of several enhancements is considered. First, vibration comfort enhancing structural interventions are discussed for the individual cases.

Home situation
The most obvious structural intervention to achieve a better comfort level in the induced field is by reducing the maximum deflections of the floor. There are several ways to do this; increasing the bending stiffness of the floor, decreasing the span or by connecting the floors more rigidly to the supporting beams.
Using more efficiently shaped joist profiles can reduce the additional mass for an equivalent stiffness. I-joists are a typical example that is strongly encouraged to be used. Also the positioning of the joists can contribute to steering the vibration behaviour of plates. For example, inserting additional or stiffer joists along the free edges of one-way span plates can make the floor behave more as a two-way span floor. This way it also becomes more beneficial to use transverse stiffening, as it is predominantly effective when a floor is spanned in two directions.
The span of a floor is a dominant factor in both the deformation- and motion-related serviceability criteria. For light-weight building designs satisfying the maximum deflection or reaching a specific natural frequency is not profound enough any more to comply with the vibration comfort experience of occupants. The span and bending stiffness should not be changed proportional to these matching equations but have to be considered as individual parameters. A limiting factor for the span is the bay dimension of the function of a room. Therefore it is advised to alternate the internal dimensions for multiple spans within a 'home' situation, as this will not lead to more structural elements such as columns and beams and hence to more connections.
Standard timber floors are supported to steel beams using hinged or near-hinged fixation conditions. For extension projects fast assemblies are beneficial, making it desirable to use these simple support types. Research showed that the near-hinged supports already contain a certain amount of rotational stiffness, making the hinged approach a conservative but realistic one. Also the stiffness capacity of the supporting beam can enhance the vibration comfort in the home situation.

Neighbouring situation
The practical interventions that influence the home situation in general also hold for the adjacent floor field. The question is if these adjustments are sufficient enough to reach the OS-RMS$_{90}$ criteria for both cases. If not, other modifications to the building structure have to be done.
The structural element separating two adjacent floors is the most influential location to steer the vibration transmittance. By adding bending and to a lesser extent torsional stiffness in the junction, a severe drop in the OS-RMS$_{90}$ level can follow. Again, efficiently shaped cross-sections give the best results, introducing the recommendation of hollow section profiles. Integrating these sections as slim floor beams can minimise the additionally required height of the structural assembly.
By avoiding mode-coupling of the floors and beams, it is possible to interfere with the ease of transfer of energy from the one to the other part of the building. Alternating floor assemblies can lead to these improved floor responses for neighbouring fields. The alternations can be done by either changing the span, stiffness or mass and hence the natural frequency of adjacent elements.
A final solution could be inserting stiffness in the walls and increasing the bending and torsion capacity in the junction. This measure is utter efficient as it reduces the OS-RMS$_{90}$ value to almost imperceptible responses to dynamic loads for receiving floors. The downside is the permanent placement of obstacles in the free floor plan, removing the flexibility of the layout.

A combination of the interventions as mentioned above should lead to a new building concept for light-weight vertical extension projects. Since there are multiple possibilities to assemble the floor and structural configuration a distinction is made for the most practical cases. In the following chapter several cases are illustrated that satisfy the vibration perceptibility criteria with a OS-RMS$_{90}$ value of 0,8 for the home case and 0,2 for neighbouring fields. These cases are distinguished based on their minimal building interferences and are evaluated accordingly.

6

Building concepts

6.1. Original concepts

After describing the theoretical and practical vibration performance steering measures, a deliberate and suitable assembly of the interventions has to be made. The aim is to deliver insight into the approaches on how to achieve a comfortable light-weight building concept. As a comparison, the as-designed 'De Karel Doorman' is taken as a reference case, however this time using a higher damping level to match the measured data and give an accurate value-judgment of the comfort levels. Table 6.1 shows the corresponding characteristics and indicates the classification of both the sending and receiving floor fields regarding vibration comfort. To achieve a high-quality apartment building, a limiting OS-RMS$_{90}$ value is proposed to be 0,8 [-] (class C, perceptible) for the home situation and 0,2 [-] (no nuisance) for the neighbouring floor fields.

Figure 6.1: Desired OS-RMS$_{90}$-values for home and adjacent apartments.

As table 6.1 reveals, the target values were not met for the original design. On-site adjustments as described previously eventually led to sufficient control of the vibrations induced by walking (and other activities [19]) within the restrictions of light-weight vertical extension construction. The vibrations in the home system were predominantly solved by doubling joist profiles and hence improving the flexural rigidity of the floors. Since the structure is dealing with dynamic loads, additional transverse joists were also installed to counteract the first mode shape. By welding thin vertical plates to the ends of the HE220A flanges extra torsional stiffness was included. An additional benefit was the increase of local stiffness of the flanges on the spots where the timber beams would rest.

Table 6.1: 'De Karel Doorman' as-designed characteristics.

	System	
mass	231	kg/m^2
f_n	5,33	Hz

	HOME	NEXT
OS-RMS$_{90}$	4,38	1,58
Class	E	

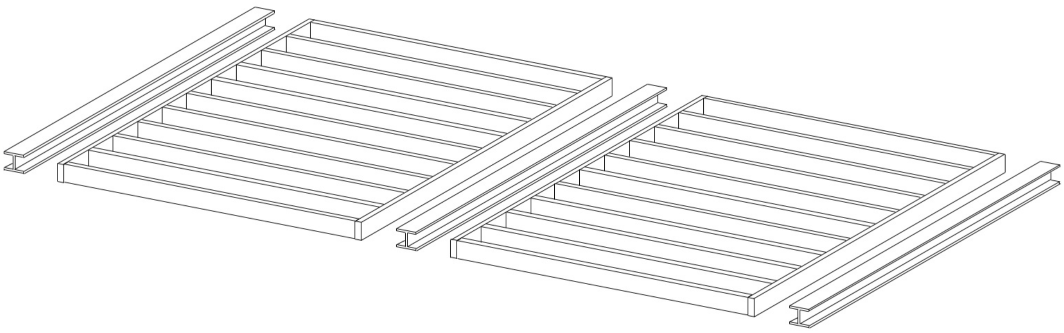


Figure 6.2: Schematic build-up 'De Karel Doorman' as-designed.

Results from TNO showed the impact of adding or removing rubbers between the floors and supporting beams. These rubbers are placed to reduce the transmittance of sound to the neighbours but showed beneficial results when they were eliminated for non-home separating beams. By doing so, the flexibility of the floors would reduce leading to smaller amplitudes in the mentioned mode shape. This intervention is not modelled resulting in the following '*' scores of the as-built system depicted in table 6.2. Had this intervention been implemented the OS-RMS$_{90}$ score for the home situation would drop to a value below 0,90 revealing the enormous dynamic impact of rubbers on a floor response.

Table 6.2: 'De Karel Doorman' as-built characteristics.

	System	
mass	242	kg/m^2
f_n	7,05	Hz

	HOME	NEXT
OS-RMS$_{90}$	1,40*	0,01*
Class	D	

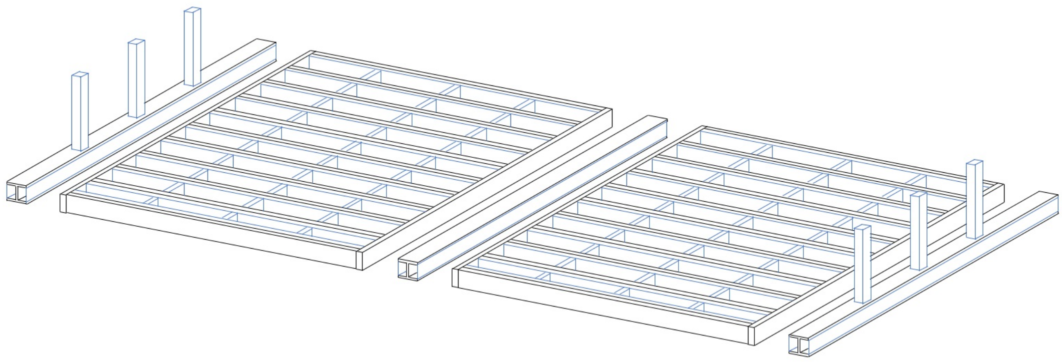


Figure 6.3: Schematic build-up 'De Karel Doorman' as-built.

The following sections show alternative design concepts for light-weight building systems (<250 kg/m^2) that meet the target criteria regarding vibration comfort. With the given tools it is possible to steer the vibration control towards these values. However multiple approaches lead to quality design. Based on the building engineering aspects (weight and mechanical properties, fire resistance, acoustic performance, vibration comfort and thermal performance) together with the practical aspects such as the logistics and an industrial-, flexible-, and demountable-design, guidelines can be produced for future light-weight projects.

6.2. New concepts

To steer new building concept towards acceptable vibration comfort design levels, two criteria have to be met; the perceptions in the home situation and the nuisance for neighbouring apartments. The latter one is found to be harder to achieve for light-weight building structures as it also follows stricter regulations. By using the given measures in a systematic approach it should become possible to obtain high-quality apartments.
Apart from the general vibration comfort enhancing measures such as additional stiffness in the floors and beams, there are multiple typologies (besides the as-built 'De Karel Doorman' design) that can produce a structural design that meets the building criteria. The following concepts describe the major measures to make this possible and can be distinguished based on their span type.

- Short span
- Two-way span
- Alternating span

These concepts will be described more in detail and show if and to what extent the considered implementation of structural adjustments affect the vibration comfort. This will lead to applicable building designs suitable for future within the light-weight vertical extension market.

6.2.1. Concept short span

The comfort level in the home situation is strongly dependent on the natural frequency and modal mass. From previous findings, it was concluded that a smaller span would positively affect its combined result in the form of the vibration velocity induced by walking people. By dividing the standard bay dimensions of a single apartment into three instead of two equally spaced fields, the improvements should become noticeable. The characteristics of this new configuration combined with the general measures of increased stiffness in the floors and the supporting beams are further described in this concept.

With the minimal interventions of increasing the flexural rigidity of the floor and applying a significant amount of torsional stiffness to the supporting beams promising OS-RMS$_{90}$ values can be obtained for short span configurations. For the floor assembly however, this does mean that the joist profiles have to be performed as one of the largest sections (96 mm wide by 240 mm high) and with a relatively close spacing (300 mm). This leads to a compact floor kit as is illustrated in figure 6.4. To obtain the required stiffness in the floors, a careful consideration for efficiently distributing the material is made. This implies finding a balance between additional weight and increased assembly heights.
The use of RHS beam profiles (250x150, thickness 12,5 mm) introduces a high torsional constant combined with a proportionate amount of bending stiffness compared to standard H-shaped beams. The combination of these configurations leads to the resulting characteristics as revealed in table 6.3.

Table 6.3: Short span concept characteristics.

	System	
mass	258	kg/m^2
f_n	12,50	Hz

	HOME	NEXT
OS-RMS$_{90}$	0,63	0,31
Class	C	

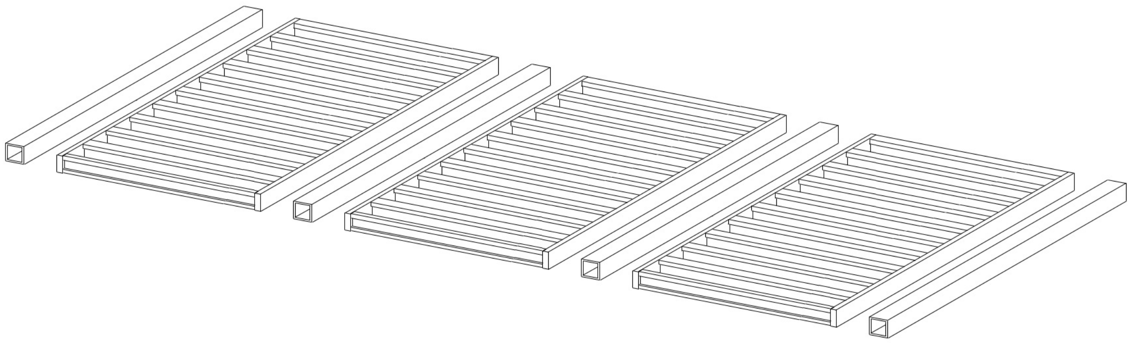


Figure 6.4: Schematic build-up short span concept.

As indicated, the home situation reveals a positive OS-RMS$_{90}$ score, even below the desired criteria of 0,8 [-]. The receiving floor, on the contrary, is close to the acceptable vibration criteria of 0,2 [-], however still requires additional enhancements for a comfortable apartment. This can be achieved with even larger joist or beam profiles, that does introduce extra mass and need an increased assembly height.

Regarding the building engineering aspects and practical demands this concept predominantly has an impact on the number of elements required for the assembly and hence the number of connections. The effect of more beams can also be found in the mass of the system, as it reaches above the 250 kg/m^2.
The introduction of RHS section profiles increases the difficulty of connecting the floors to the beams. Using slim floor beams that contain a wider bottom flange can solve this issue by integrating the beam in the height of the floor assembly.
An alternative approach would be to vary with the type of beams used at home-separating and in-home junctions. For this latter junction, beams that require less stiffness could be used to maintain the vibration energy in the home situation and reduce the transmittance to neighbouring floors. This results in a lower OS-RMS$_{90}$ score for the neighbours but a higher score for the home situation. Especially for the one-way span floor concepts this alternation can produce the final small contribution that is required, as there is a margin for the home situation.

6.2.2. Concept two-way span

Not just the longitudinal but also the transverse properties of a floor contribute to the dynamic stiffness. The previous building concept showed the two-sided support configuration that required shorter (triple) spans to complete the bay dimensions of an apartment building. This concept, on the other hand, reveals the possibilities of the floors supported on four sides for a larger (double) span configuration. For this two-way spanning the bending stiffness in the transverse direction contributes to a greater extent as it effectively reduces the deflection of a floor.

Introducing the same longitudinal flexural rigidity as the previous concept (large I-joists spaced closely) with the addition of equal transverse stiffness will result in an isotropic floor with the characteristics as described in table 6.4. The structural arrangement already takes into account vibration performance enhancing beams with additional bending and torsional stiffness, in the form of an RHS profile. Figure 6.5 shows the building assembly for this concept.
The resulting *OS-RMS scores reveal that for larger spans it is possible to obtain similar results as for shorter spans by utilising the transverse capacities of a floor. As this configuration was not suitable to model in the SoViST tool, an equivalent score to the OS-RMS$_{90}$ was approximated by taking into account twenty different (extreme) step-frequencies and body weights measured from footfall analyses in Autodesk Robot.

Table 6.4: Two-way span concept characteristics.

	System	
mass	253	kg/m^2
f_n	10,20	Hz

	HOME	NEXT
*OS-RMS	0,59	0,30
Class	C	

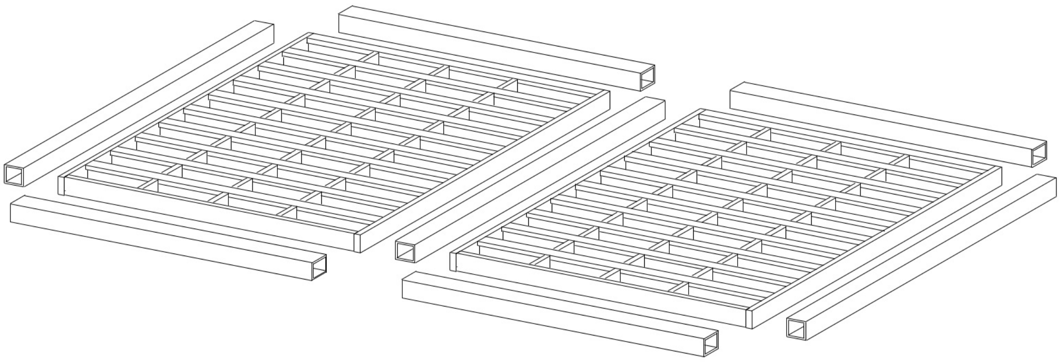


Figure 6.5: Schematic build-up two-way span concept.

Again it follows that the serviceability criteria for the home situation are met quite simple, however for the adjacent fields there is still progress to be made. The introduction of even more stiffness in the floor or to the supporting beams (or both) can eventually steer the vibration levels to the desired values.

From an engineering perspective the two-way span floors introduce both advantages and disadvantages. By optimising the transverse stiffness capacities, it is possible to produce large open floor plans. However, the required material to reach the given serviceability criteria leads to additional mass above the 250 kg/m^2. If the floors have to be carried in two directions, this also affects the number of connections required to fix the floors.

6.2.3. Concept alternating span

The implementation of the previous concepts will both eventually result in acceptable vibration comfort levels. The OS-RMS$_{90}$ criterion for the home situation is predominantly achieved by additional stiffness in the floors and beams and the introduction of shorter or two-way spans. However, to obtain a favourable OS-RMS$_{90}$ score for the neighbouring fields, overcapacity of the stiffness is required. This leads to inefficient material use as the transmittance of the vibrations can also be impeded by avoiding the mode-coupling of floors. This final concept reveals the implementation of an alternating floor span and its effects on vibration control.

By shifting the beam that is not home-separating away from the central axis of the bay, two unequal floor fields will be generated. This will result in two different natural frequencies of the floor that will interfere with the transmittance of vibrations and will hence improve the floor response at neighbouring apartments. The vibration energy will reflect at the junction creating better conditions in the receiving floor but will increase the perceptibility in the floor where the activity takes place. For this concept a repetitive 4.0-meter span will be changed to an alternating 5.0 and 3.0-meter span.
The single floor fields will bear the same flexural rigidity and are connected to RHS beam profiles as proposed in the previous concepts. For this configuration the characteristics as described in table 6.5 will follow. The two cases indicate the different location of applying the walking load. For *Case 1* this implies a person walking in the centre of the larger span whereas for *Case 2* the point of excitement is in the centre of the smaller span.

Table 6.5: Alternating span concept characteristics.

	System	
mass	245	kg/m^2
f_n	7,43	Hz

Case 1	HOME	NEXT
*OS-RMS$_{90}$	1,55	0,21
Class	D	

Case 2	HOME	NEXT
*OS-RMS$_{90}$	0,30	0,19
Class	C	

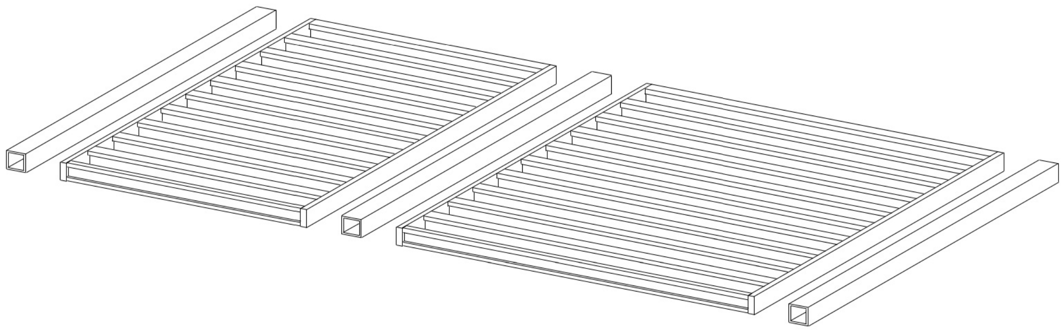


Figure 6.6: Schematic build-up alternating span concept.

The results show positive values for the receiving floors. It is observed that the alternating floor system strongly reduces the level of vibration for neighbouring fields. For this configuration it has to be noted the normative case will be when the larger span is excited. As expected the *OS-RMS$_{90}$ for the home field in this case will show unfavourable results. This can be explained since the span has increased with a considerable amount, without a corresponding more substantial stiffness. Also for this case, the resulting values are obtained measuring extreme data in Autodesk Robot, as explained in the previous concept.

With the use of an alternating floor field, the transmittance to neighbouring fields can strongly be affected, as this concept revealed. It has to be remarked that smaller alternations, than the proposed 5.0 by 3.0 meter, can also produce the positive effects and will require less additional stiffness for the larger span. A balance for the correct implementation is dependent on the project specifications.

6.3. Buildings aspects

Regarding light-weight vertical extension projects, several demands and requirements were proposed. The measures that have to be applied for creating vibration comfortable living spaces have to meet the building engineering and practical aspects as described in section 2.4.

Due to the additional stiffness, the proposed measures will meet both the deformation- and critical motion-related serviceability criteria. The extra weight following from these interventions could lead to new governing strength criteria. As larger beam sections are suggested it is unlikely that the structure will not fulfil its mechanical robustness.
Regarding fire resistance, it is recommended to limit the amount of additional structural elements. For the as-built 'De Karel Doorman' more interruptions in the structural assembly led to further investigations. Because of the height and function of the building, a 120 minutes resistance classification for the load-bearing structure has to be satisfied. The use of straight elements suits the required installation of packing, giving room for the implementation of RHS-beam profiles. Thorough investigations for the flanges (where the beams meet the floor unprotected) have to be done to make sure the requirements are met.
To meet the high demands for acoustics in light-weight building structures the floors used in 'De Karel Doorman' were supported on springs. Unfortunately, this led to a building system sensible to the transfer of vibrations [12]. Removing rubbers or adding extra stiffness to them, steered the vibration behaviour. Further research into the effects of rubbers for both acoustics and vibrational behaviour is recommended.

The practical aspects require a fast and easy assembly procedure with flexibility from the design through the service phase. The use of prefab elements is therefore advised, as was implemented in the design concepts. The proposed building systems all produced hinged connections to limit the on-site labour. From an economic point of view, it is recommended to use identical elements. However, the alternating span concept encourages varying parts. As discussed the as-built design of 'De Karel Doorman' implemented stiff elements in the wall, removing the flexibility of the floor plan. For this reason, the proposed building concepts show measures that only interfere in the horizontal parts of the structure.

6.4. Summary

The described concepts give insight into how to steer the vibration behaviour of a steel and timber building system. Several proposed practical interventions are described, and the impact of these measures on its response to dynamic loads are shown. If the conditional inputs are set out against each other in OS-RMS$_{90}$ scores for both the home and the neighbouring situations the results as depicted in figure 6.7 can be produced. In here the new concepts are balanced against the original 'De Karel Doorman' as-designed and as-built structural assembly.

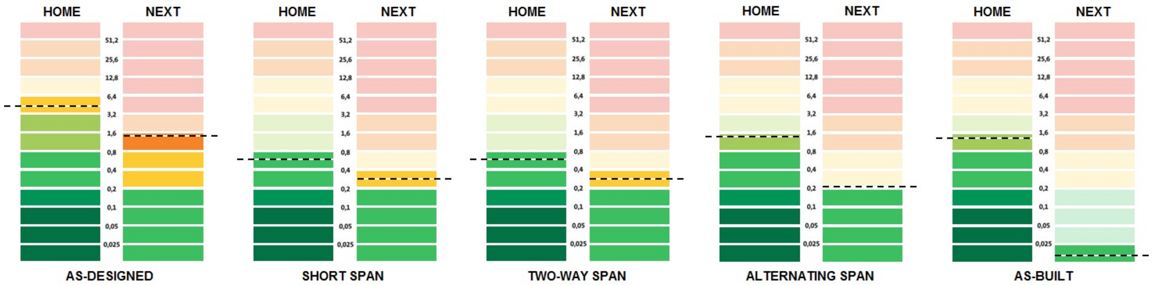


Figure 6.7: Resulting OS-RMS$_{90}$-values for different concepts.

It can be observed that between the as-designed and as-built configuration there is a significant improvement. As mentioned before the required interventions were applied during erection and would preferably have been avoided. Driven by innovation the new concepts produce alternative designs that meet the comfort criteria for light-weight extension projects.

The aimed for objective of an OS-RMS$_{90}$ criteria level of 0,8 [-] for the home situation, classified in category C as barely perceptible, and an OS-RMS$_{90}$ value of 0,2 [-] for adjacent floor fields, causing no hindrance for neighbours, prove to be feasible applying specific procedures.
The implementation of additional flexural rigidity in the floor is inevitable. The same holds for the desired higher bending and torsional capacity of the supporting beams. Stiffness can be applied to a certain extent until the limit criteria are met, however other measures will create high-quality apartments more efficiently. Both the reduced span and the two-way span will approach the comfort OS-RMS$_{90}$ criteria. In fact, for the home situation the concepts reach the target quite spacious, however for the neighbouring apartment additional measures are required. It was revealed that this criterion is harder to achieve without the use of additional elements. The alternating floor span will benefit this latter problem and is found to be a useful measure in interfering with the transmittance to adjacent floor fields.

The use of rigid floor and beam elements is required to meet the motion-related vibration criteria. It is encouraged to use an alternating floor system as this will stimulate the reflection of vibration energy. For a final design, project-specific dimensions have to be chosen. Had the alternating span design been enhanced with additional transverse stiffness in the larger floor field also the perceptibility in the home situation would have dropped to more acceptable values. This also holds for placing additional or more rigid joists along the free edges to simulate a two-way span floor.

The right balance between the desired structural configuration is project specific and has to be considered for individual cases. The handed measures for steering vibration comfort indicate the matter of effect, and their applicability is appointed. For future light-weight building structures this creates more freedom in the design choices and leads to an integrated engineering and practical vision.

7
Conclusion & discussion

7.1. Conclusion

Modern housing demands put pressure on the available market capacity. Extending buildings vertical with light-weight structures is considered a viable option to address this issue. Adjustments to the existing substructure can provide toppings to reach a considerable number of additional levels, but the weight factor of the assembly is defined within strict boundaries. This innovative building design pushes the engineering boundaries and emerges new challenges.
The vertical extension project 'De Karel Doorman' in Rotterdam from Royal HaskoningDHV put forward a new phenomenon; vibration comfort in light-weight building structures. Even though a qualitative strong enough building was designed, unforeseen serviceability criteria appeared to be crucial for the design. Vibrations induced by people walking became noticeable during erection and seemed to transfer easily across the floor plan. The reduced mass could not produce enough energy dissipation to create acceptable perceptibility measurements, leading to on-site interventions to ensure the quality of the light-weight building system.

This research aimed to provide insight into the dynamic behaviour induced by footfall to a light-weight steel and timber building structure and to come up with improved design measures for possible future use. The residential building 'De Karel Doorman' in Rotterdam was used as reference case being one of the frontrunners of this innovative but challenging construction method. To check the quality of structural interventions footfall analyses were performed and new concepts were proposed based on their feasibility. The designs included measures to enhance the response of dynamic footfalls in the home situation but also for the transmittance to adjacent apartments.

The dynamic analyses were performed using the SoViST tool that was built by TNO after research into light-weight construction methods and with Autodesk Robot that provided additional understanding in the path of vibrations. The comfort level is expressed in an OS-RMS$_{90}$ value that has to meet criteria levels based on the location and susceptibility of human perception to vibration. For home situations this level was set to 0,8 [-] and for adjacent floor fields to 0,2 [-].

7.1.1. Main findings

The main findings of this research are separated in theoretical and practical aspects. Below these findings are clustered and summarised:

Vibration comfort

- Vibration comfort is a governing motion-related serviceability criterion that has to be normative in light-weight building systems for both home and neighbouring floors. OS-RMS$_{90}$ [-] limit criteria are defined to indicate acceptable standards.
- Vibrations can be broken down into the source, path and receiver. Since it is impossible to change the walking behaviour of individuals or their perceptibility to vibrations, solely variation to the structural assembly of a building can influence the extent of nuisance from dynamic loads.
- Vibrations are influenced by the damping, mass and natural frequency of structural elements. Low mass structures require less energy to excite a floor resulting in stronger vibrations. The damping ratio indicates the matter of reduction of vibrations over time. By avoiding overlap of the natural frequency of structural elements (f_n) to the step frequency up to the first few harmonics (f_s), resonance can be avoided resulting in a reduced magnification of vibration amplitudes.

Vibration theory

- For footfall analysis of floor systems the boundary conditions, material properties and geometry of a plate are governing factors to influence the level of vibration. These characteristics can be subdivided into more structural properties such as the type of fixation, flexural rigidity, span, mass, etc.
- Flexural rigidity over the span of a floor (EI_L) shows to be the most influential parameter for SDOF analyses by increasing the bending stiffness without losing participating modal mass.
- For a MDOF footfall analysis the same parametric factors hold over the vibration comfort, however with a smaller impact. The OS-RMS$_{90}$ score is not solely dependent on the characteristics of the floor.

- MDOF analyses introduce the vibration to other masses, in this case neighbouring floor fields. The intermediate junction its stiffness capacity is governing for the resistance in the transmittance of the vibration energy.
- Mode-coupling is the correspondence of natural frequencies from individual elements. If these frequencies coincide, the vibration is more easily transferred. A variation is desired. This is the case for both the supporting beams as the adjacent floor fields.
- Alternating floor systems avoid mode coupling and show solely positive results to neighbouring floors. The normative situation for the home situation, however, will always show degraded comfort values.
- Vibration comfort for light-weight building systems requires higher capacities than defined by standard deformation-related and motion-related serviceability criteria. Especially the transmittance to neighbouring floor fields proves to be a critical design factor.

Vibration practice *Home*

- Flexural rigidity can be obtained by using more efficiently shaped floor joists, such as I-joists. This increases the stiffness to mass ratio compared to solid timber blocks. The placement of the joists can steer the vibration response behaviour by reducing the spacing or locating these at the large deflection points.
- Adding transverse stiffness will stretch the clustering of natural frequencies common for orthotropic floors. For four-sided supported floors this effect is best noticeable. Variations up to an isotropic floor show improved OS-RMS$_{90}$ scores but also add much weight to the system.
- Smaller spans strongly affect the vibration comfort, however, floors should not reduce their stiffness accordingly. The downside of reducing the span is the additional structural elements and connections that are needed when the same floor area wants to be designed.
- Additional bending stiffness in the junctions impedes the deflection at the boundaries. This improves the floor response for the home situation and lowers the OS-RMS$_{90}$ score.
- Increased rigidity at the floor to beam connection improves the floor response by reducing the maximum deformations. The practical implementation shows only little rigidity, however produces more promising results than the conservative purely hinged analyses.

Vibration practice *Next*

- Varying the stiffness or span of all floor fields equally results in the same degree of enhancement to neighbouring fields as for home fields. However, this latter level of comfort is often found to be further from acceptable limits, demanding additional interventions.
- Adding dynamic stiffness in the form of steel columns between supporting beams reduces the vibration transmittance significantly to imperceptible values but decreases the flexibility of the floor layout.
- Bending and torsional stiffness improve the hindrance of vibration energy to adjacent floor fields. Rectangular hollow section profiles are found to be the most suitable and practical for implementation.
- By fixing supporting beams rigidly to the columns, compared to pinned connections, will lower the next field level of vibrations.
- Varying the home-separating and in-home junction beams can make the home situation maintain more vibration energy and reflect the transmittance to adjacent fields.
- The overall response of alternating floor configurations will always improve the neighbouring OS-RMS$_{90}$ scores. For the larger span additional stiffening measures have to be taken.

7.1.2. Conclusions

The previously mentioned findings together can give a conclusion, within the limits of the research, and answer the research question:

How can the light-weight steel and timber building system, within the densified urban context, be further developed regarding acceptable vibration demands?

It was found that for light-weight building systems the governing design criteria is motion-related serviceability. This introduces a shift from the traditional dominating strength design criteria. To achieve the occupant's comfort for various situations the heavy-weight serviceability rules of thumb are not sufficient. To guarantee acceptable vibration response apartments, especially regarding transmittance to adjacent units, a different limit state than a maximum deflection or minimum natural frequency has to be considered. The SBR-guideline provides a building criterion, the OS-RMS_{90} score, that when achieved results in quality apartments. This criterion, however, has to be maintained for both the home as the neighbouring situation. It is found that this latter threshold produces the biggest challenge.

To answer the research question it has to be split up into two parts; steering comfort levels in the home situation and the neighbouring situation.

Light-weight building structures inevitably require large stiffness in both the floors and the junctions. These overall enhancing measures will result in acceptable OS-RMS_{90} values when enough of it is applied, however do lead to undesirable large or heavy structural assemblies. Introducing shorter spans or floors spanned in two directions can limit these consequences, but will require additional connections and structural members. This will ultimately lead to a more complex structural assembly.
It was shown that the comfort criterion in the home situation is attained by implementing the previous measures. However, the nuisance for neighbours would remain present. To control the transmittance to neighbouring floor fields, alternating floor fields are favourable. By doing so, mode-coupling is avoided, and the vibration energy will experience more resistance along its path.

For the home situation these measures mean using efficiently shaped joist profiles, spaced closely and located at points of large deformations, such as the centre or along the free edges of a floor. At the junctions, beams with considerable bending and torsional stiffness capacities have to be used. RHS profiles fit the requirements but additional plates at the bottom flange, or slim floor beams, have to be applied to integrate the floors in the junction design. Extra transverse joists in the floor will contribute to the response by stretching the clustering of natural frequencies for orthotropic floors, however will show stronger effects when a two-way span is used.
For the neighbouring situation repetitive floor fields are discouraged. Varying the span has the most noticeable contribution to deviations in the natural frequency and hence the prevention of mode-coupling. Producing different junction typologies for the home-separating and in-home beams can maintain the vibration energy in the home field, reducing the nuisance for neighbours.
The as-built 'De Karel Doorman' revealed the impact of additional stiff elements in the wall that increase the bending and torsional stiffness in the junction. It was found that these elements mitigate the nuisance from vibrations to imperceptible values. They do however affect the layout of the floor plan by introducing large obstacles.

Following the practical demands for building within urban densified areas, a fast and industrial assembly is of high importance. Therefore it is advised to limit the number of connections and to prefabricate as much as possible. The vibration criterion at neighbouring floor fields however, requires the implementation of more variety in structural elements.

7.2. Discussion

The results of this research can be applied to general steel and timber building systems with mass densities up to 250 kg/m^2. The following topics can influence the outcome of the results and restrict the field of implementations.

Modelling and calculation models
The used software for the footfall analyses both had their restrictions. The SoViST tool designed by TNO provided different easy adaptable design tools to research various standard building systems and showed a final OS-RMS_{90} value for both the home and neighbouring floor fields. The disadvantage of this tool was the limitation in studying the options for fixed connections, four-sided supported plates and alternating spans. Also SoViST only showed results for, although often most governing, centre field vibration excitations and responses. For this reason, the Autodesk Robot software was used, which required to model the building system from scratch. The downside of this programme was that it only showed the vibration levels for a single person with mass X and step frequency Y. This way a representative vRMS score could not always be compared to that of the OS-RMS_{90}.

Level of damping
The level of damping was based both on results conducted by TNO during tests on the 'De Karel Doorman' building during erection and on reference projects. Damping is often considered as a factor that is hard to predict [20]. Therefore it was chosen to set the damping on 5%, a mixture between set values for timber and steel projects. Comparing the mobility of the models in SoViST, however, showed that the damping could be increased to 8% to correspond with measured data.

Floors as plate
For ease of use it was chosen to model the floors as plates and homogenise the floor assembly of timber joists, decking plate and screed layer to a plate with a certain thickness. All capacities were translated to corresponding values, however slight deviations could occur since mass and area is distributed non-equal over the length and width of a floor. Research showed that the assumption of homogenizing the panel is reasonable within the frequency range that is of interest for human induced vibrations [27].

Connection rigidity
The fixation of structural elements was modelled as either pinned or rigid and its results were shown for these conditions. However, the reality indicates that there is a specific capacity region to which actual connections can be classified, taking away the idealisation of the chosen joint types. The moment resistance or rotational capacities will never show infinite or zero stiffness values.

Generalisations
Calculated models show abstracted results based on generalisations from the factors that influence floor vibration. It is not always fully compliant with the reality to, for example, model the human-induced footfall at a fixed position on the floor or to only consider the effects of a single person [23]. However, it will indicate fair approximations that are sufficient enough for a preliminary design phase. Reality will need physical models to show the actual level of correspondence.

7.3. Recommendation

The enhancement of light-weight building systems is an integral research that extends the limits of this thesis. Other areas that ask for more research and discoveries are depicted below.

Other materials
Besides the standard steel construction frame and timber floors there are other materials in upcoming that could prove to be of interest when designing for vibration comfort. Instead of the timber joists, cold formed thin steel profiles that have a different E-modulus could provide the right stiffness with minimal mass. An example of this is the IDES-floor. The use of FRP material, which is currently being used more and more in light-weight bridge designs, could also be worth investigating for its relatively high strength to mass ratio.
The possibility of creating an entire light-weight building out of timber is currently ongoing and even finished a project Canada recently. The reduction of vibration was dealt with by the adding of a concrete topping on the CLT floors to increase the weight and by the addition of a carpet tile and resilient flooring to reduce the floor hardness.

Damping
The stiffness of rubbers can significantly contribute to the dissipation of vibrational energy as was already shown by TNO [19]. The effects of adding or removing the rubbers at the supports of floors requires more attention when constructing light-weight buildings.
Another efficient way of varying the damping level is by adding non structural damping systems. This can be done in the form of active or passive dampers such as tuned mass dampers [15][7].
The matter of damping is dependent on the internal friction in the materials and the structural damping caused by friction between components. Both contribute almost equally to the total damping [21]. More research into the connections of the joists and floor plates could reveal more insight into these factors and how to increase them.

Alternating floor systems
Physical tests should indicate the level of contribution of deactivating mode-coupling and to what extent it is practical. An optimum in variation has not been found yet, although numerical analyses do show the effects of the phenomenon.

SoViST
The parametric design tool SoViST gives a practical and fast view of vibration comfort for light-weight building structures. Some additional features are recommended to it for more understanding and control over resulting OS-RMS$_{90}$ values:
• Include variations in span and support types
• Include alternating floor fields in the form of geometry and material properties
• Map the OS-RMS$_{90}$ scores over the entire floor field

Other activities
Focussed on residential functions the main dynamic load that causes a response in floors is the footfall from walking. Other activities such as jumping or forms of aerobics were described in the literature study. Since vibration response is a comfort measurement and not a safety criterion it is sufficient enough for this research to focus on the dynamic load imposed by a walking activity. For more rhythmic activities the strength and fatigue might also be worth investigating.

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A

Appendix A - Footfall response characteristics

A.1. Allocation of classes of perception

Figure A.3 indicates the allocation of classes of perception A to F to threshold values of OS-RMS$_{90}$ values and relation of occupancies of floors to comfort limits [13].

Table A.1: Allocation of classes of perception.

Class		A	B	C	D	E	F
OS-RMS$_{90}$	Lower limit	0.0	0.1	0.2	0.8	3.2	12.8
	Upper limit	0.1	0.2	0.8	3.2	12.8	51.2
Function	Critical areas						
	Hospitals, surgeries						
	Schools, training centers						
	Residential buildings						
	Meeting rooms						
	Senior residential buildings						
	Hotels						
	Industrial workshops						
	Sports facilities						

In this table the green boxes indicate recommended values, the yellow boxes indicate critical values and the red boxes indicate values that are not recommended.

A.2. Determination of damping

Damping values for vibration systems can be determined using table A.2 for different construction materials, furniture and finishing in the condition of use. The system damping is obtained by summing up the appropriate values for D_1, D_2 and D_3 [13].

Table A.2: Determination of damping.

Type	Damping (%)
Structural Damping D_1	
Wood	6%
Concrete	2%
Steel	1%
Composite	1%
Damping due to furniture D_2	
Traditional office	2%
Paperless office	0%
Open plan office	1%
Library	1%
Houses	1%
Schools	0%
Gymnastic	0%
Damping due to finishings D_3	
Ceiling under the floor	1%
Free floating floor	0%
Swimming screed	1%
Total damping $D = D_1 + D_2 + D_3$	

A.3. Cumulative probability distribution

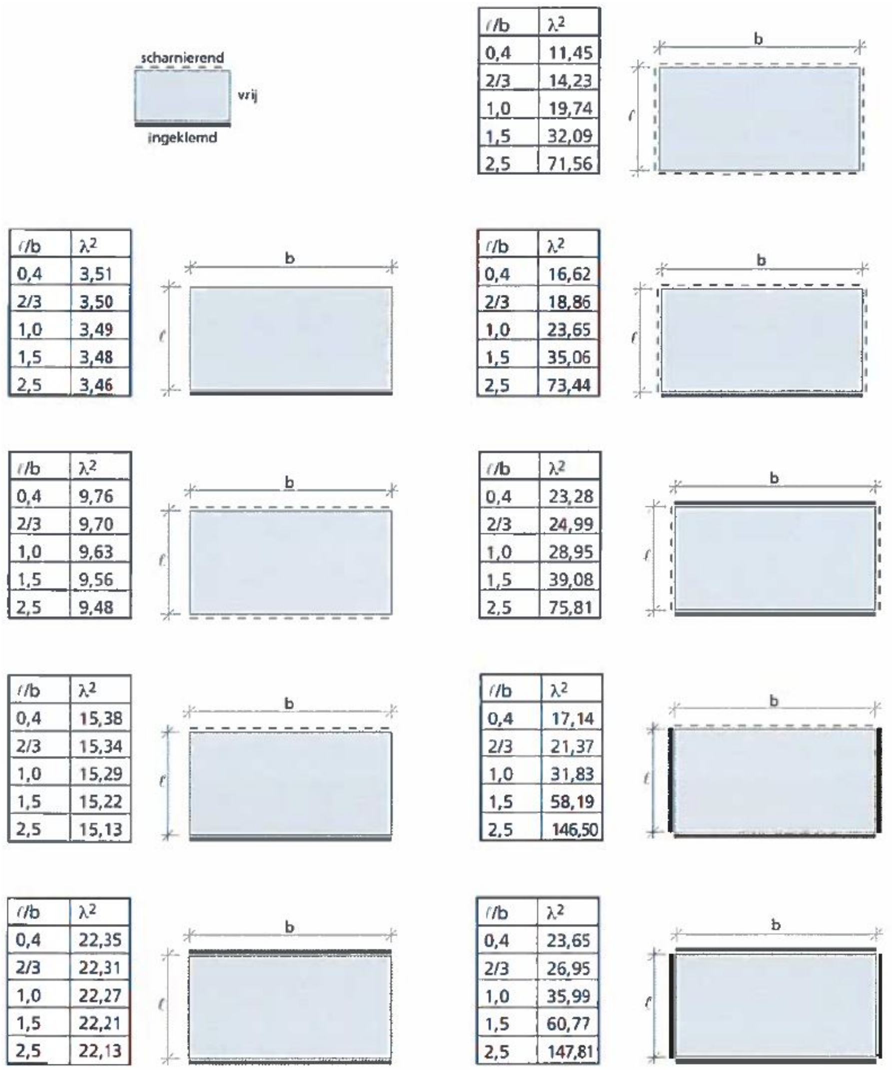
Table A.3 indicates the cumulative probability distribution functions for step frequency and body mass associated by figure 3.11.

Table A.3: Cumulative probability distribution functions for step frequency and body mass.

cumulative probability	stepfrequency	cumulative probability	mass
0,0003	1,64	0,0000	30
0,0035	1,68	0,0002	35
0,0164	1,72	0,0011	40
0,0474	1,76	0,0043	45
0,1016	1,80	0,0146	50
0,1776	1,84	0,0407	55
0,2691	1,88	0,0950	60
0,3679	1,92	0,1882	65
0,4663	1,96	0,3210	70
0,5585	2,00	0,4797	75
0,6410	2,04	0,6402	80
0,7122	2,08	0,7786	85
0,7719	2,12	0,8804	90
0,8209	2,16	0,9440	95
0,8604	2,20	0,9776	100
0,8919	2,24	0,9924	105
0,9167	2,28	0,9978	110
0,9360	2,32	0,9995	115
0,9510	2,36	0,9999	120
0,9625	2,40	1,0000	125
0,9714	2,44		
0,9782	2,48		
0,9834	2,52		
0,9873	2,56		
0,9903	2,60		
0,9926	2,64		
0,9944	2,68		
0,9957	2,72		
0,9967	2,76		
0,9975	2,80		
0,9981	2,84		
0,9985	2,88		
0,9988	2,92		
0,9991	2,96		
0,9993	3,00		

A.4. Eigenfrequency values floor supports

Figure A.1 indicates the values for λ^2 for various support conditions. This factor can be used in equation 4.1.



ℓ/b	λ^2
0,4	11,45
2/3	14,23
1,0	19,74
1,5	32,09
2,5	71,56

ℓ/b	λ^2
0,4	3,51
2/3	3,50
1,0	3,49
1,5	3,48
2,5	3,46

ℓ/b	λ^2
0,4	16,62
2/3	18,86
1,0	23,65
1,5	35,06
2,5	73,44

ℓ/b	λ^2
0,4	9,76
2/3	9,70
1,0	9,63
1,5	9,56
2,5	9,48

ℓ/b	λ^2
0,4	23,28
2/3	24,99
1,0	28,95
1,5	39,08
2,5	75,81

ℓ/b	λ^2
0,4	15,38
2/3	15,34
1,0	15,29
1,5	15,22
2,5	15,13

ℓ/b	λ^2
0,4	17,14
2/3	21,37
1,0	31,83
1,5	58,19
2,5	146,50

ℓ/b	λ^2
0,4	22,35
2/3	22,31
1,0	22,27
1,5	22,21
2,5	22,13

ℓ/b	λ^2
0,4	23,65
2/3	26,95
1,0	35,99
1,5	60,77
2,5	147,81

Figure A.1: Eigenfrequencies of floor plates [36].

B

Appendix B - Practical characteristics

B.1. Joist profiles

Table B.1 and table B.2 indicate the available profile dimensions for respectively rectangular joist profiles and I-joist profiles. On the following pages the stiffness values for different joist assemblies is indicated.

Table B.1: Available Kerto-S joist profiles (width [mm] by height [mm])

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	•	•							
33	•	•	•						
39	•	•	•	•					
45	•	•	•	•	•				
51	•	•	•	•	•	•			
57	•	•	•	•	•	•	•		
63	•	•	•	•	•	•	•	•	
75	•	•	•	•	•	•	•	•	•

Table B.2: Available Kerto Finnjoist I-profiles (width [mm] by height [mm])

	200	**220**	**240**	**300**	**360**	**400**
FJI38-39		•	•	•		
FJI45-36			•	•		
FJI45-39	•	•	•	•	•	•
FJI53-36			•	•		
FJI58-39	•	•	•	•	•	•
FJI69-36			•	•		
FJI89-39	•	•	•	•	•	•
FJI96-39			•	•		

B.1.1. Kerto-S joist profiles

Table B.3: Stiffness [Nm^2/m] for spacing 600 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	$5{,}75 \cdot 10^5$	$7{,}50 \cdot 10^5$							
33	$6{,}67 \cdot 10^5$	$8{,}81 \cdot 10^5$	$1{,}27 \cdot 10^6$						
39	$7{,}59 \cdot 10^5$	$1{,}01 \cdot 10^6$	$1{,}47 \cdot 10^6$	$2{,}18 \cdot 10^6$					
45	$8{,}51 \cdot 10^5$	$\mathbf{1{,}14} \cdot 10^6$	$1{,}68 \cdot 10^6$	$2{,}49 \cdot 10^6$	$4{,}18 \cdot 10^6$				
51	$9{,}43 \cdot 10^5$	$1{,}27 \cdot 10^6$	$1{,}88 \cdot 10^6$	$2{,}80 \cdot 10^6$	$4{,}72 \cdot 10^6$	$6{,}42 \cdot 10^6$			
57	$1{,}03 \cdot 10^6$	$1{,}40 \cdot 10^6$	$2{,}08 \cdot 10^6$	$3{,}11 \cdot 10^6$	$5{,}26 \cdot 10^6$	$7{,}15 \cdot 10^6$	$1{,}01 \cdot 10^7$		
63	$1{,}13 \cdot 10^6$	$1{,}54 \cdot 10^6$	$2{,}28 \cdot 10^6$	$3{,}42 \cdot 10^6$	$5{,}79 \cdot 10^6$	$7{,}89 \cdot 10^6$	$1{,}12 \cdot 10^7$	$1{,}53 \cdot 10^7$	
75	$1{,}31 \cdot 10^6$	$1{,}80 \cdot 10^6$	$2{,}69 \cdot 10^6$	$4{,}04 \cdot 10^6$	$6{,}87 \cdot 10^6$	$9{,}36 \cdot 10^6$	$1{,}33 \cdot 10^7$	$1{,}81 \cdot 10^7$	$3{,}12 \cdot 10^7$

Table B.4: Stiffness [Nm^2/m] for spacing 500 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	$6{,}57 \cdot 10^5$	$8{,}68 \cdot 10^5$							
33	$7{,}68 \cdot 10^5$	$1{,}03 \cdot 10^6$	$1{,}49 \cdot 10^6$						
39	$8{,}78 \cdot 10^5$	$1{,}18 \cdot 10^6$	$1{,}74 \cdot 10^6$	$2{,}58 \cdot 10^6$					
45	$9{,}89 \cdot 10^5$	$1{,}34 \cdot 10^6$	$1{,}98 \cdot 10^6$	$2{,}96 \cdot 10^6$	$4{,}99 \cdot 10^6$				
51	$1{,}10 \cdot 10^6$	$1{,}50 \cdot 10^6$	$2{,}22 \cdot 10^6$	$3{,}33 \cdot 10^6$	$5{,}63 \cdot 10^6$	$7{,}67 \cdot 10^6$			
57	$1{,}21 \cdot 10^6$	$1{,}65 \cdot 10^6$	$2{,}46 \cdot 10^6$	$3{,}70 \cdot 10^6$	$6{,}28 \cdot 10^6$	$8{,}55 \cdot 10^6$	$1{,}21 \cdot 10^7$		
63	$1{,}32 \cdot 10^6$	$1{,}81 \cdot 10^6$	$2{,}71 \cdot 10^6$	$4{,}07 \cdot 10^6$	$6{,}92 \cdot 10^6$	$9{,}43 \cdot 10^6$	$1{,}34 \cdot 10^7$	$1{,}83 \cdot 10^7$	
75	$1{,}54 \cdot 10^6$	$2{,}13 \cdot 10^6$	$3{,}19 \cdot 10^6$	$4{,}82 \cdot 10^6$	$8{,}21 \cdot 10^6$	$1{,}12 \cdot 10^7$	$1{,}59 \cdot 10^7$	$2{,}17 \cdot 10^7$	$3{,}74 \cdot 10^7$

Table B.5: Stiffness [Nm^2/m] for spacing 400 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	$7{,}82 \cdot 10^5$	$1{,}04 \cdot 10^6$							
33	$9{,}20 \cdot 10^5$	$1{,}24 \cdot 10^6$	$1{,}83 \cdot 10^6$						
39	$1{,}06 \cdot 10^6$	$1{,}44 \cdot 10^6$	$2{,}13 \cdot 10^6$	$3{,}19 \cdot 10^6$					
45	$1{,}20 \cdot 10^5$	$1{,}63 \cdot 10^6$	$2{,}43 \cdot 10^6$	$3{,}65 \cdot 10^6$	$6{,}20 \cdot 10^6$				
51	$1{,}33 \cdot 10^6$	$1{,}83 \cdot 10^6$	$2{,}74 \cdot 10^6$	$4{,}12 \cdot 10^6$	$7{,}00 \cdot 10^6$	$9{,}54 \cdot 10^6$			
57	$1{,}47 \cdot 10^6$	$2{,}03 \cdot 10^6$	$3{,}04 \cdot 10^6$	$4{,}59 \cdot 10^6$	$7{,}81 \cdot 10^6$	$1{,}06 \cdot 10^7$	$1{,}51 \cdot 10^7$		
63	$1{,}61 \cdot 10^6$	$2{,}22 \cdot 10^6$	$3{,}34 \cdot 10^6$	$5{,}05 \cdot 10^6$	$8{,}61 \cdot 10^6$	$1{,}18 \cdot 10^7$	$1{,}67 \cdot 10^7$	$2{,}28 \cdot 10^7$	
75	$1{,}89 \cdot 10^6$	$2{,}62 \cdot 10^6$	$3{,}95 \cdot 10^6$	$5{,}98 \cdot 10^6$	$1{,}02 \cdot 10^7$	$1{,}40 \cdot 10^7$	$1{,}98 \cdot 10^7$	$2{,}71 \cdot 10^7$	$4{,}67 \cdot 10^7$

Table B.6: Stiffness [Nm^2/m] for spacing 300 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	$9{,}89 \cdot 10^5$	$1{,}34 \cdot 10^6$							
33	$1{,}17 \cdot 10^6$	$1{,}60 \cdot 10^6$	$2{,}38 \cdot 10^6$						
39	$1{,}36 \cdot 10^6$	$1{,}86 \cdot 10^6$	$2{,}79 \cdot 10^6$	$4{,}20 \cdot 10^6$					
45	$1{,}54 \cdot 10^6$	$2{,}13 \cdot 10^6$	$3{,}19 \cdot 10^6$	$4{,}82 \cdot 10^6$	$8{,}21 \cdot 10^6$				
51	$1{,}72 \cdot 10^6$	$2{,}39 \cdot 10^6$	$3{,}60 \cdot 10^6$	$5{,}44 \cdot 10^6$	$9{,}28 \cdot 10^6$	$1{,}27 \cdot 10^7$			
57	$1{,}91 \cdot 10^6$	$2{,}65 \cdot 10^6$	$4{,}00 \cdot 10^6$	$6{,}06 \cdot 10^6$	$1{,}04 \cdot 10^7$	$1{,}41 \cdot 10^7$	$2{,}01 \cdot 10^7$		
63	$2{,}09 \cdot 10^6$	$2{,}91 \cdot 10^6$	$4{,}41 \cdot 10^6$	$6{,}68 \cdot 10^6$	$1{,}14 \cdot 10^7$	$1{,}56 \cdot 10^7$	$2{,}22 \cdot 10^7$	$3{,}03 \cdot 10^7$	
75	$2{,}46 \cdot 10^6$	$3{,}44 \cdot 10^6$	$5{,}21 \cdot 10^6$	$7{,}92 \cdot 10^6$	$1{,}36 \cdot 10^7$	$1{,}86 \cdot 10^7$	$2{,}64 \cdot 10^7$	$3{,}61 \cdot 10^7$	$6{,}23 \cdot 10^7$

Table B.7: Mass [kg^2/m] for spacing 600 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	201	201							
33	202	202	203						
39	203	203	204	206					
45	204	**204**	206	207	210				
51	205	206	207	209	211	213			
57	206	207	208	210	213	215	217		
63	207	208	210	212	215	217	220	222	
75	209	210	212	215	219	221	224	227	234

Table B.8: Mass [kg^2/m] for spacing 500 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	201	202							
33	203	203	205						
39	204	205	206	208					
45	205	206	208	210	212				
51	206	207	209	211	214	216			
57	207	209	211	213	217	219	222		
63	209	210	212	215	219	221	224	228	
75	211	213	216	219	223	226	230	234	241

Table B.9: Mass [kg^2/m] for spacing 400 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	203	204							
33	204	205	207						
39	206	207	209	211					
45	207	209	211	213	216				
51	209	210	213	215	219	222			
57	210	212	215	217	222	225	228		
63	212	214	216	220	224	228	231	235	
75	215	217	220	224	230	234	238	243	252

Table B.10: Mass [kg^2/m] for spacing 300 mm

	200	**225**	**260**	**300**	**360**	**400**	**450**	**500**	**600**
27	205	206							
33	207	208	210						
39	209	211	213	216					
45	211	213	216	219	223				
51	213	215	218	222	227	230			
57	215	217	221	225	230	234	239		
63	217	220	223	228	234	238	243	249	
75	221	224	229	234	241	246	252	259	271

B.1.2. Kerto Finnjoist I-profiles

Table B.11: Stiffness [Nm^2/m] for spacing 600 mm

	200	220	240	300	360	400
FJI38-39		$7{,}12 \cdot 10^5$	$8{,}41 \cdot 10^5$	$1{,}31 \cdot 10^6$		
FJI45-36			$9{,}34 \cdot 10^5$	$1{,}47 \cdot 10^6$		
FJI45-39	$6{,}81 \cdot 10^5$	$8{,}16 \cdot 10^5$	$9{,}69 \cdot 10^5$	$1{,}53 \cdot 10^6$	$2{,}25 \cdot 10^6$	$2{,}81 \cdot 10^6$
FJI53-36			$1{,}07 \cdot 10^6$	$1{,}70 \cdot 10^6$		
FJI58-39	$8{,}34 \cdot 10^5$	$1{,}01 \cdot 10^6$	$1{,}21 \cdot 10^6$	$1{,}93 \cdot 10^6$	$2{,}85 \cdot 10^6$	$3{,}58 \cdot 10^6$
FJI69-36			$1{,}35 \cdot 10^6$	$2{,}16 \cdot 10^6$		
FJI89-39	$1{,}20 \cdot 10^6$	$1{,}47 \cdot 10^6$	$1{,}78 \cdot 10^6$	$2{,}88 \cdot 10^6$	$4{,}29 \cdot 10^6$	$5{,}40 \cdot 10^6$
FJI96-39			$1{,}90 \cdot 10^6$	$3{,}10 \cdot 10^6$		

Table B.12: Stiffness [Nm^2/m] for spacing 500 mm

	200	220	240	300	360	400
FJI38-39		$8{,}23 \cdot 10^5$	$9{,}77 \cdot 10^5$	$1{,}54 \cdot 10^6$		
FJI45-36			$1{,}09 \cdot 10^6$	$1{,}73 \cdot 10^6$		
FJI45-39	$7{,}85 \cdot 10^5$	$9{,}47 \cdot 10^5$	$1{,}13 \cdot 10^6$	$1{,}80 \cdot 10^6$	$2{,}66 \cdot 10^6$	$3{,}34 \cdot 10^6$
FJI53-36			$1{,}26 \cdot 10^6$	$2{,}00 \cdot 10^6$		
FJI58-39	$9{,}69 \cdot 10^5$	$1{,}18 \cdot 10^6$	$1{,}42 \cdot 10^6$	$2{,}28 \cdot 10^6$	$3{,}39 \cdot 10^6$	$4{,}26 \cdot 10^6$
FJI69-36			$1{,}59 \cdot 10^6$	$2{,}56 \cdot 10^6$		
FJI89-39	$1{,}41 \cdot 10^6$	$1{,}73 \cdot 10^6$	$2{,}10 \cdot 10^6$	$3{,}43 \cdot 10^6$	$5{,}11 \cdot 10^6$	$6{,}44 \cdot 10^6$
FJI96-39			$2{,}25 \cdot 10^6$	$3{,}69 \cdot 10^6$		

Table B.13: Stiffness [Nm^2/m] for spacing 400 mm

	200	220	240	300	360	400
FJI38-39		$9{,}88 \cdot 10^5$	$1{,}18 \cdot 10^6$	$1{,}89 \cdot 10^6$		
FJI45-36			$1{,}32 \cdot 10^6$	$2{,}12 \cdot 10^6$		
FJI45-39	$9{,}41 \cdot 10^5$	$1{,}14 \cdot 10^6$	$1{,}37 \cdot 10^6$	$2{,}21 \cdot 10^6$	$3{,}29 \cdot 10^6$	$4{,}14 \cdot 10^6$
FJI53-36			$1{,}53 \cdot 10^6$	$2{,}47 \cdot 10^6$		
FJI58-39	$1{,}17 \cdot 10^6$	$1{,}44 \cdot 10^6$	$1{,}73 \cdot 10^6$	$2{,}81 \cdot 10^6$	$4{,}19 \cdot 10^6$	$5{,}29 \cdot 10^6$
FJI69-36			$1{,}95 \cdot 10^6$	$3{,}16 \cdot 10^6$		
FJI89-39	$1{,}72 \cdot 10^6$	$2{,}13 \cdot 10^6$	$2{,}58 \cdot 10^6$	$4{,}24 \cdot 10^6$	$6{,}35 \cdot 10^6$	$8{,}01 \cdot 10^6$
FJI96-39			$2{,}78 \cdot 10^6$	$4{,}57 \cdot 10^6$		

Table B.14: Stiffness [Nm^2/m] for spacing 300 mm

	200	220	240	300	360	400
FJI38-39		$1{,}26 \cdot 10^6$	$1{,}52 \cdot 10^6$	$2{,}46 \cdot 10^6$		
FJI45-36			$1{,}71 \cdot 10^6$	$2{,}77 \cdot 10^6$		
FJI45-39	$1{,}20 \cdot 10^6$	$1{,}47 \cdot 10^6$	$1{,}78 \cdot 10^6$	$2{,}90 \cdot 10^6$	$4{,}33 \cdot 10^6$	$5{,}47 \cdot 10^6$
FJI53-36			$1{,}99 \cdot 10^6$	$3{,}23 \cdot 10^6$		
FJI58-39	$1{,}51 \cdot 10^6$	$1{,}86 \cdot 10^6$	$2{,}25 \cdot 10^6$	$3{,}70 \cdot 10^6$	$5{,}54 \cdot 10^6$	$6{,}99 \cdot 10^6$
FJI69-36			$2{,}54 \cdot 10^6$	$4{,}16 \cdot 10^6$		
FJI89-39	$2{,}24 \cdot 10^6$	$2{,}78 \cdot 10^6$	$3{,}39 \cdot 10^6$	$5{,}60 \cdot 10^6$	$8{,}42 \cdot 10^6$	$1{,}06 \cdot 10^7$
FJI96-39			$3{,}65 \cdot 10^6$	$6{,}04 \cdot 10^6$		

Table B.15: Mass [kg^2/m] for spacing 600 mm

	200	**220**	**240**	**300**	**360**	**400**
FJI38-39		200	200	201		
FJI45-36			201	201		
FJI45-39	200	201	201	201	202	202
FJI53-36			201	202		
FJI58-39	201	201	202	202	203	203
FJI69-36			202	203		
FJI89-39	203	203	204	204	205	205
FJI96-39			204	205		

Table B.16: Mass [kg^2/m] for spacing 500 mm

	200	**220**	**240**	**300**	**360**	**400**
FJI38-39		201	201	202		
FJI45-36			201	202		
FJI45-39	201	201	202	202	203	204
FJI53-36			202	203		
FJI58-39	202	202	203	203	204	205
FJI69-36			203	204		
FJI89-39	205	205	205	206	207	207
FJI96-39			206	206		

Table B.17: Mass [kg^2/m] for spacing 400 mm

	200	**220**	**240**	**300**	**360**	**400**
FJI38-39		202	202	203		
FJI45-36			203	204		
FJI45-39	202	203	203	204	205	206
FJI53-36			204	205		
FJI58-39	204	204	204	205	206	207
FJI69-36			205	206		
FJI89-39	207	207	207	208	209	210
FJI96-39			208	209		

Table B.18: Mass [kg^2/m] for spacing 300 mm

	200	**220**	**240**	**300**	**360**	**400**
FJI38-39		204	205	206		
FJI45-36			205	206		
FJI45-39	205	205	205	207	208	209
FJI53-36			206	207		
FJI58-39	206	207	207	208	210	211
FJI69-36			208	209		
FJI89-39	210	211	211	212	214	215
FJI96-39			212	213		

B.1.3. Comparison

Figure B.1 and figure B.2 show for two different joist profiles the coherence between the stiffness and mass. It becomes evident that the use of I-joists more efficiently uses the material. Therefore, the stiffness to mass ratio is lower and more beneficial for extension projects.

Figure B.1: Stiffness to mass ratio for various rectangular joist profiles.

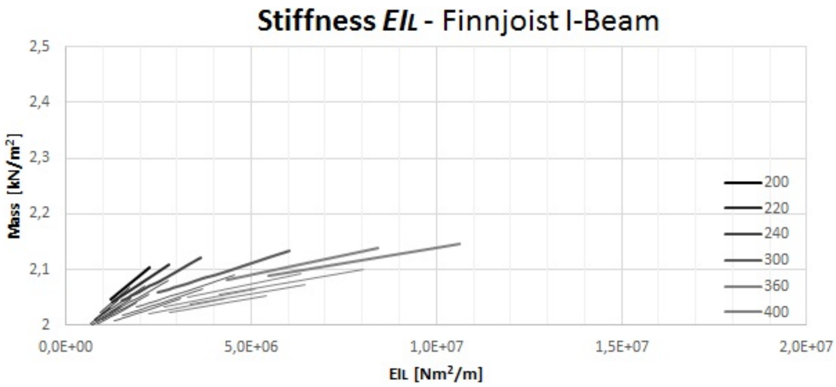


Figure B.2: Stiffness to mass ratio for various I-joist profiles.

B.2. Beam profiles

This section will show the characteristic values for different type of beam profiles to be used in the junction of a building system.

B.2.1. H-beams

Table B.19: HE beam characteristics.

		G [kg/m2]	h [mm]	Iy [mm^4]	Iz [mm^4]	It [mm^4]
HE160	**AA**	6,0	148	$1,28 \cdot 10^7$	$4,79 \cdot 10^6$	$6,33 \cdot 10^4$
	A	7,6	152	$1,67 \cdot 10^7$	$6,16 \cdot 10^6$	$1,22 \cdot 10^5$
	B	10,7	160	$2,49 \cdot 10^7$	$8,89 \cdot 10^6$	$3,12 \cdot 10^5$
	M	19,1	180	$5,10 \cdot 10^7$	$1,76 \cdot 10^7$	$1,62 \cdot 10^6$
HE180	**AA**	7,2	167	$1,97 \cdot 10^7$	$7,30 \cdot 10^6$	$8,33 \cdot 10^4$
	A	8,9	171	$2,51 \cdot 10^7$	$9,25 \cdot 10^6$	$1,48 \cdot 10^5$
	B	12,8	180	$3,83 \cdot 10^7$	$1,36 \cdot 10^7$	$4,22 \cdot 10^5$
	M	22,2	200	$7,48 \cdot 10^7$	$2,58 \cdot 10^7$	$2,03 \cdot 10^6$
HE200	**AA**	8,7	186	$2,94 \cdot 10^7$	$1,07 \cdot 10^7$	$1,27 \cdot 10^5$
	A	10,6	190	$3,69 \cdot 10^7$	$1,34 \cdot 10^7$	$2,10 \cdot 10^5$
	B	15,3	200	$5,70 \cdot 10^7$	$2,00 \cdot 10^7$	$5,93 \cdot 10^5$
	M	25,8	220	$1,06 \cdot 10^8$	$3,65 \cdot 10^7$	$2,59 \cdot 10^6$
HE220	**AA**	10,1	205	$4,17 \cdot 10^7$	$1,51 \cdot 10^7$	$1,59 \cdot 10^5$
	A	12,6	210	$5,41 \cdot 10^7$	$1,96 \cdot 10^7$	$2,85 \cdot 10^5$
	B	17,9	220	$8,09 \cdot 10^7$	$2,84 \cdot 10^7$	$7,66 \cdot 10^5$
	M	29,3	240	$1,46 \cdot 10^8$	$5,01 \cdot 10^7$	$3,15 \cdot 10^6$
HE240	**AA**	11,9	224	$5,84 \cdot 10^7$	$2,08 \cdot 10^7$	$2,30 \cdot 10^5$
	A	15,1	230	$7,76 \cdot 10^7$	$2,77 \cdot 10^7$	$4,16 \cdot 10^5$
	B	20,8	240	$1,13 \cdot 10^8$	$3,92 \cdot 10^7$	$1,03 \cdot 10^6$
	M	39,2	270	$2,43 \cdot 10^8$	$8,15 \cdot 10^7$	$6,28 \cdot 10^6$
HE260	**AA**	13,5	244	$7,98 \cdot 10^7$	$2,79 \cdot 10^7$	$3,03 \cdot 10^5$
	A	17,1	250	$1,05 \cdot 10^8$	$3,67 \cdot 10^7$	$5,24 \cdot 10^5$
	B	23,3	260	$1,49 \cdot 10^8$	$5,14 \cdot 10^7$	$1,24 \cdot 10^6$
	M	43,1	290	$3,13 \cdot 10^8$	$1,05 \cdot 10^8$	$7,19 \cdot 10^6$
HE280	**AA**	15,3	264	$1,06 \cdot 10^8$	$3,66 \cdot 10^7$	$3,62 \cdot 10^5$
	A	19,1	270	$1,37 \cdot 10^8$	$4,76 \cdot 10^7$	$6,21 \cdot 10^5$
	B	25,8	280	$1,93 \cdot 10^8$	$6,60 \cdot 10^7$	$1,44 \cdot 10^6$
	M	47,1	310	$3,96 \cdot 10^8$	$1,32 \cdot 10^8$	$8,07 \cdot 10^6$
HE300	**AA**	17,5	283	$1,38 \cdot 10^8$	$4,73 \cdot 10^7$	$4,94 \cdot 10^5$
	A	22,1	290	$1,83 \cdot 10^8$	$6,31 \cdot 10^7$	$8,52 \cdot 10^5$
	B	29,3	300	$2,52 \cdot 10^8$	$8,56 \cdot 10^7$	$1,85 \cdot 10^6$
	M	59,5	340	$5,92 \cdot 10^8$	$1,94 \cdot 10^8$	$1,41 \cdot 10^7$

B.2.2. Comparison

The following figures show the effect of changing the type of H-shaped beam profiles. The bending stiffness to weight ratio seem to be proportional, however do require more height when higher values of the I_y have to be obtained. For these HE-beams the torsional stiffness and mass also tend to work linear. Again bigger profiles have to be used to obtain higher characteristics.

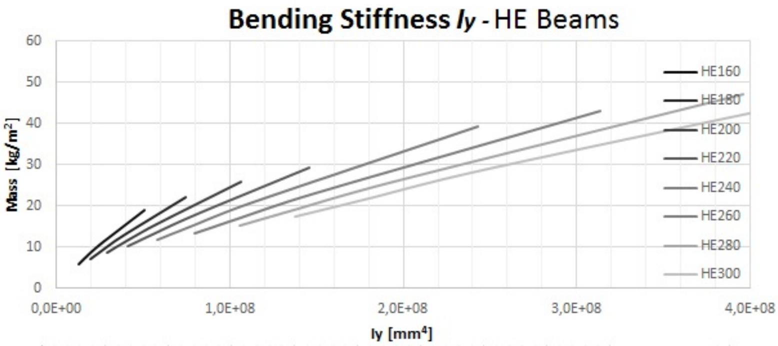


Figure B.3: Bending stiffness to mass ratio for various HE beam profiles.

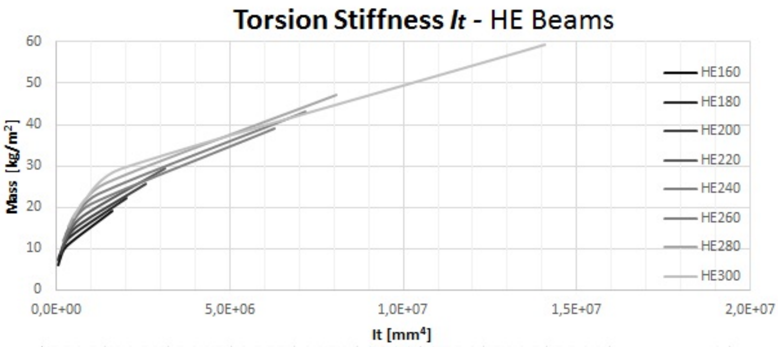


Figure B.4: Rotation stiffness to mass ratio for various HE beam profiles.

B.2.3. Hollow sections

Table B.20: Hollow section beam characteristics.

SHS		G [kg/m2]	Iy [mm4]	Iz [mm4]	It [mm4]
200x200	**5**	7,6	$2{,}45 \cdot 10^7$	$2{,}45 \cdot 10^7$	$3{,}76 \cdot 10^7$
	6,3	9,5	$3{,}01 \cdot 10^7$	$3{,}01 \cdot 10^7$	$4{,}65 \cdot 10^7$
	8	11,9	$3{,}71 \cdot 10^7$	$3{,}71 \cdot 10^7$	$5{,}78 \cdot 10^7$
	10	14,7	$4{,}47 \cdot 10^7$	$4{,}47 \cdot 10^7$	$7{,}03 \cdot 10^7$
	12,5	18,1	$5{,}34 \cdot 10^7$	$5{,}34 \cdot 10^7$	$8{,}49 \cdot 10^7$
	16	22,6	$6{,}39 \cdot 10^7$	$6{,}39 \cdot 10^7$	$1{,}03 \cdot 10^8$
250x250	**6,3**	12,0	$6{,}01 \cdot 10^7$	$6{,}01 \cdot 10^7$	$9{,}24 \cdot 10^7$
	8	15,1	$7{,}46 \cdot 10^7$	$7{,}46 \cdot 10^7$	$1{,}15 \cdot 10^8$
	10	18,6	$9{,}06 \cdot 10^7$	$9{,}06 \cdot 10^7$	$1{,}41 \cdot 10^8$
	12,5	23,0	$1{,}09 \cdot 10^8$	$1{,}09 \cdot 10^8$	$1{,}72 \cdot 10^8$
	16	28,8	$1{,}33 \cdot 10^8$	$1{,}33 \cdot 10^8$	$2{,}11 \cdot 10^8$
300x300	**6,3**	14,5	$1{,}05 \cdot 10^8$	$1{,}05 \cdot 10^8$	$1{,}61 \cdot 10^8$
	8	18,2	$1{,}31 \cdot 10^8$	$1{,}31 \cdot 10^8$	$2{,}02 \cdot 10^8$
	10	22,6	$1{,}60 \cdot 10^8$	$1{,}60 \cdot 10^8$	$2{,}48 \cdot 10^8$
	12,5	28,0	$1{,}94 \cdot 10^8$	$1{,}94 \cdot 10^8$	$3{,}03 \cdot 10^8$

RHS		G [kg/m2]	Iy [mm4]	Iz [mm4]	It [mm4]
200x120	**5**	6,0	$1{,}69 \cdot 10^7$	$7{,}62 \cdot 10^6$	$1{,}65 \cdot 10^7$
	6,3	7,5	$2{,}07 \cdot 10^7$	$9{,}29 \cdot 10^6$	$2{,}03 \cdot 10^7$
	8	9,4	$2{,}53 \cdot 10^7$	$1{,}13 \cdot 10^7$	$2{,}50 \cdot 10^7$
	10	11,6	$3{,}03 \cdot 10^7$	$1{,}34 \cdot 10^7$	$3{,}00 \cdot 10^7$
250x150	**5**	7,6	$3{,}36 \cdot 10^7$	$1{,}53 \cdot 10^7$	$3{,}28 \cdot 10^7$
	6,3	9,5	$4{,}14 \cdot 10^7$	$1{,}87 \cdot 10^7$	$4{,}05 \cdot 10^7$
	8	11,9	$5{,}11 \cdot 10^7$	$2{,}30 \cdot 10^7$	$5{,}02 \cdot 10^7$
	10	14,7	$6{,}17 \cdot 10^7$	$2{,}76 \cdot 10^7$	$6{,}09 \cdot 10^7$
	12,5	18,1	$7{,}39 \cdot 10^7$	$3{,}27 \cdot 10^7$	$7{,}33 \cdot 10^7$
	16	22,6	$8{,}88 \cdot 10^7$	$3{,}87 \cdot 10^7$	$8{,}87 \cdot 10^7$
300x200	**6,3**	12,0	$7{,}83 \cdot 10^7$	$4{,}19 \cdot 10^7$	$8{,}48 \cdot 10^7$
	8	15,1	$9{,}72 \cdot 10^7$	$5{,}18 \cdot 10^7$	$1{,}06 \cdot 10^8$
	10	18,6	$1{,}18 \cdot 10^8$	$6{,}28 \cdot 10^7$	$1{,}29 \cdot 10^8$
	12,5	23,0	$1{,}43 \cdot 10^8$	$7{,}54 \cdot 10^7$	$1{,}57 \cdot 10^8$

CHS		G [kg/m2]	Iy [mm4]	Iz [mm4]	It [mm4]
193.7	**5**	5,8	$1{,}32 \cdot 10^7$	$1{,}32 \cdot 10^7$	$2{,}64 \cdot 10^7$
	6,3	7,3	$1{,}63 \cdot 10^7$	$1{,}63 \cdot 10^7$	$3{,}26 \cdot 10^7$
	8	9,2	$2{,}02 \cdot 10^7$	$2{,}02 \cdot 10^7$	$4{,}03 \cdot 10^7$
	10	11,3	$2{,}44 \cdot 10^7$	$2{,}44 \cdot 10^7$	$4{,}88 \cdot 10^7$
	12,5	14,0	$2{,}93 \cdot 10^7$	$2{,}93 \cdot 10^7$	$5{,}87 \cdot 10^7$
219.1	**5**	6,6	$1{,}93 \cdot 10^7$	$1{,}93 \cdot 10^7$	$3{,}86 \cdot 10^7$
	6,3	8,3	$2{,}39 \cdot 10^7$	$2{,}39 \cdot 10^7$	$4{,}77 \cdot 10^7$
	8	10,4	$2{,}96 \cdot 10^7$	$2{,}96 \cdot 10^7$	$5{,}92 \cdot 10^7$
	10	12,9	$3{,}60 \cdot 10^7$	$3{,}60 \cdot 10^7$	$7{,}20 \cdot 10^7$
	12,5	15,9	$4{,}34 \cdot 10^7$	$4{,}34 \cdot 10^7$	$8{,}69 \cdot 10^7$
	16	20,0	$5{,}30 \cdot 10^7$	$5{,}30 \cdot 10^7$	$1{,}06 \cdot 10^8$
273.0	**5**	8,3	$3{,}78 \cdot 10^7$	$3{,}78 \cdot 10^7$	$7{,}56 \cdot 10^7$
	6,3	10,4	$4{,}70 \cdot 10^7$	$4{,}70 \cdot 10^7$	$9{,}39 \cdot 10^7$
	8	13,1	$5{,}85 \cdot 10^7$	$5{,}85 \cdot 10^7$	$1{,}17 \cdot 10^8$
	10	16,2	$7{,}15 \cdot 10^7$	$7{,}15 \cdot 10^7$	$1{,}43 \cdot 10^8$
	12,5	20,1	$8{,}70 \cdot 10^7$	$8{,}70 \cdot 10^7$	$1{,}74 \cdot 10^8$

B.2.4. Comparison

The following figures show the effect of changing the type of hollow section profiles. The different shapes beams show different corresponding values of bending stiffness and torsional stiffness. CHS-profiles with similar heights as SHS- and RHS-section have a lower bending resistance, but an the contrary show higher values for the torsional resistance.

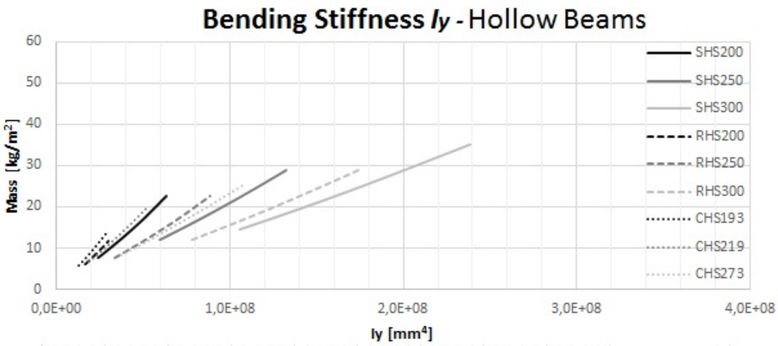


Figure B.5: Bending stiffness to mass ratio for various hollow beam profiles.

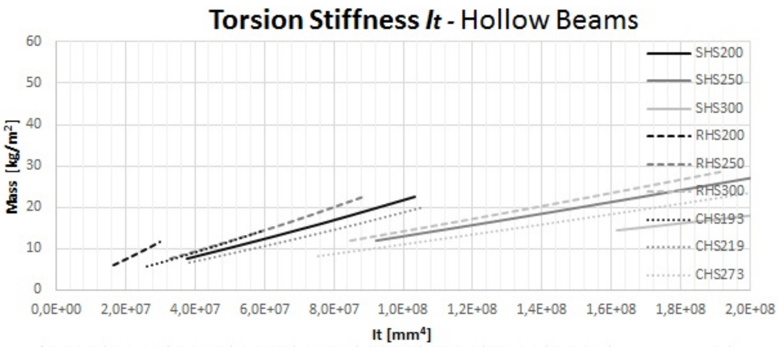


Figure B.6: Rotational stiffness to mass ratio for various hollow beam profiles.