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Department of Civil Engineering

MAUN SCIENCE PARK

Master Thesis

Development of a sustainable structural concept for the Maun Science Park

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STATUTORY DECLARATION

I hereby declare in lieu of an oath that I have independently prepared this master thesis and have fully indicated the aids used as well as the persons and institutions consulted.

Konstanz, 31. March 2021 [redacted]

ABSTRACT

Keywords: Maun Science Park, tall building structures, sustainable construction, life cycle assessment, concrete core structure, steel shear frame structure, rammed earth shear wall structure, wooden diagrid structure

In Maun, Botswana, a self-sufficient, sustainable and future-oriented district will be created, the Maun Science Park. Within this project, several 5-8 storey smart homes shall be built in sustainable construction. The aim of this thesis is to develop a sustainable structural concept for those homes of the Maun Science Park. In a first step, the general basics for tall building structures and sustainable construction were established. Based on those fundamentals, criteria for the structural requirements, the ecological as well as the social sustainability of a structural design could be defined. Subsequently, four structural systems were drafted: a concrete core structure, a steel shear frame structure, a rammed earth shear wall structure and a wooden diagrid structure. In addition to the pre-dimensioning of the systems, a life cycle assessment was set up to evaluate the ecological sustainability of the designs. With the help of a utility value analysis, the wooden diagrid structure was determined as the preferred variant. The comparison of the designs also allows to draw general conclusions for the development of sustainable tall building structures. The results of the life cycle assessment show the advantage of wood as an ecological building material over industrially manufactured building materials, such as steel and concrete. Whereas rammed earth, a likewise ecological building material, is not convincing due to its low strength. In general, a balance is created in the life cycle assessment between ecological and industrially manufactured products in regard of strength and environmental impact. In terms of social sustainability, the design of the structure system can significantly influence the flexibility and use of local resources. However, due to the diversity of sustainable construction, the development of a structural system should be linked to an overarching sustainability concept that takes architecture and stakeholders into account.

ABSTRACT

Schlüsselwörter: Maun Science Park, Hochhaustragwerke, Nachhaltiges Bauen, Ökobilanz, Kerntragwerk aus Beton, ausgesteifte Stahlrahmen Konstruktion, Schubwand Konstruktion aus Stampflehm, Diagrid Tragwerk aus Holz

In Maun, Botswana soll ein autarker, nachhaltiger und zukunftsorientierter Stadtteil entstehen, der Maun Science Park. Im Rahmen dieses Projektes sollen mehreren 5-8 stöckige smart homes in nachhaltiger Bauweise errichtet werden. Ziel der vorliegenden Arbeit ist es, ein nachhaltigen Tragwerkskonzept für die Gebäude des Maun Science Park zu entwickeln. Um dies zu erreichen wurden zu Beginn die allgemeinen Grundlagen für Hochhaustragwerke und zum Thema Nachhaltiges Bauen ermittelt. Basierend auf diesen Grundlagen konnten Kriterien für die strukturellen Anforderungen, die ökologische sowie die soziale Nachhaltigkeit des Tragwerkentwurfes definiert werden. Anschließend wurden vier Tragwerkssysteme entwickelt: eine Kerntragwerk aus Beton, eine ausgesteifte Stahlrahmen Konstruktion, eine Schubwand Konstruktion aus Stampflehm und eine Röhrentragwerk aus Holz. Hierbei wurde neben der Vorbemessung für die Tragwerke, eine Ökobilanz zur Bewertung der ökologischen Nachhaltigkeit der Entwürfe aufgestellt. Mithilfe einer Nutzwertanalyse konnte das Röhrentragwerk aus Holz als Vorzugsvariante ermittelt werden. Der Vergleich der Entwürfe lässt darüber hinaus allgemeine Schlüsse für die Entwicklung nachhaltiger Hochhaustragwerke zu. Die Ergebnisse der Ökobilanz zeigen den Vorteil von Holz als ökologisches Baumaterial im Gegensatz zu industriell gefertigten Bauprodukten wie Stahl und Beton. Stampflehm, ebenfalls ein ökologisches Baumaterial, kann hingegen aufgrund seiner geringen Festigkeit nicht überzeugen. Generell entsteht in der Ökobilanz ein Ausgleich zwischen ökologischen und industriell gefertigten Produkten in Bezug auf Festigkeit und Umweltauswirkungen. Im Hinblick auf die soziale Nachhaltigkeit kann die Gestaltung des Tragwerkes maßgeblich die Flexibilität und den Einsatz von lokalen Ressourcen beeinflussen. Aufgrund der Vielfältigkeit des Nachhaltigen Bauens sollte sich die Entwicklung eines Tragwerks an einem übergeordneten Nachhaltigkeitskonzept, welches Architektur und Zielgruppen berücksichtigt, orientieren.

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List of Abbreviations

AP ..... Acidification potential
BNB ..... Bewertungssystem Nachhaltiges Bauen für Bundesgebäude
CLT ..... Cross laminated timber
DGNB ..... Deutsches Gütesiegel für Nachhaltiges Bauen
EP ..... Eutrophication potential
glulam ..... Glued laminated timber
GWP ..... Global warming potential
LCA ..... Life cycle assessment
LCI ..... Life cycle inventory
LCIA ..... Life cycle impact assessment
MSP ..... Maun Science Park
ODP ..... Ozone depletion potential
PENRE ..... Non-renewable primary energy as energy source
PENRM ..... Non-renewable primary energy for material use
PENRT ..... Total non-renewable primary energy
PERE ..... Renewable primary energy as energy source
PERM ..... Renewable primary energy for material use
PERT ..... Total renewable primary energy
SLS ..... Serviceability limit state
ULS ..... Ultimate limit state

1 Introduction

Sustainable development is one of the greatest challenges facing humanity at the beginning of this century. The increasing world population and various global crises have made us aware of how sensitive our home planet is and that resources are limited. Many of the technologies for sustainable living are therefore concerned with preserving the lives of human populations in other habitats outside of Earth. In Botswana, however, a project is emerging with the aim of refocusing technological progress to enable sustainable living on this earth, the Maun Science Park (MSP)[1] project.

Within this project a self-sufficient, sustainable and future-oriented urban district shall be created in Maun, Botswana, where exactly these technologies will be researched and worked on.[2] For this purpose, in addition to a school and a business incubator, a settlement with 25 smart homes is to be built as a living lab. For a home like this the first design shows a green and modular, 5-8 storey building, the Tree of Life.

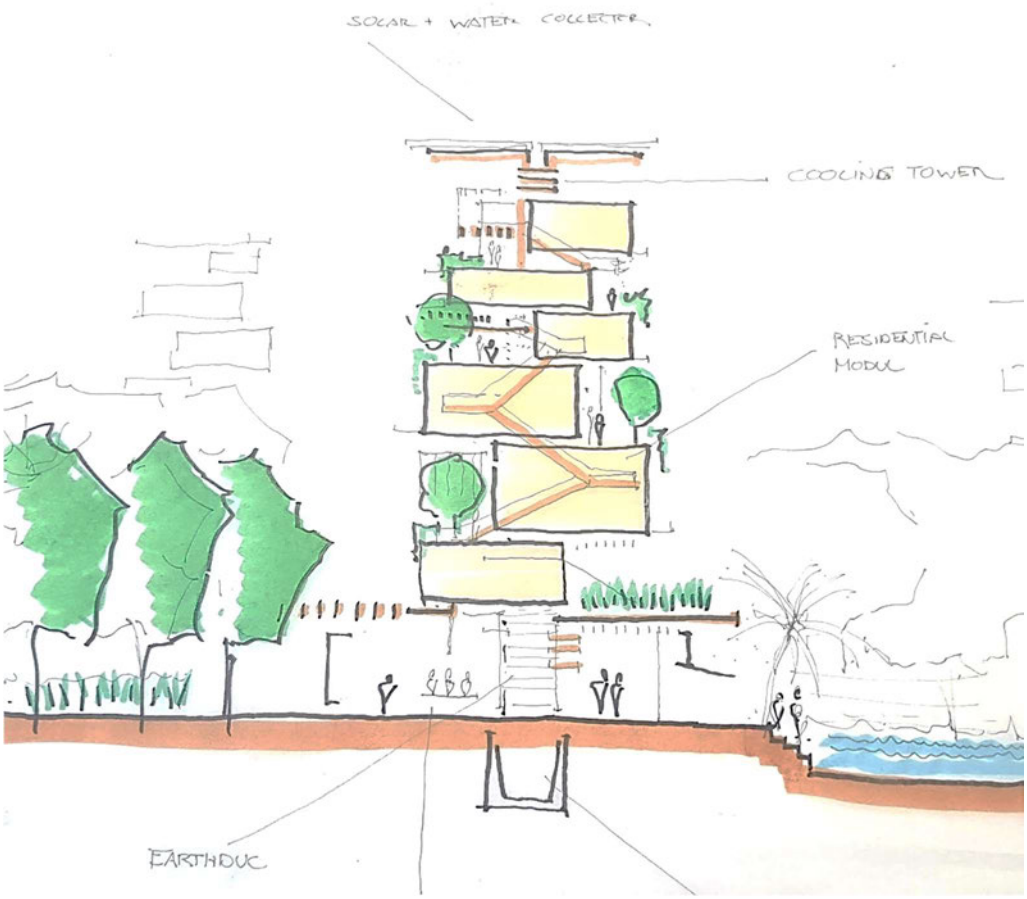


Figure 1 - First Design: Tree of Life[3]

[1] Maun Science Park 2021
[2] Refer to Bühler et al. 2020, pp. 36-45
[3] haascookzemmrich STUDIO 2050, Freie Architekten PartG mbB 2020

1.1 Objectives

Sustainable development does not stop at the building industry. Far from it, construction offers great potential for sustainability. Accordingly, the Tree of Life is intended to be a sustainable, self-sufficient building that meets the needs of the people in Botswana Within the concept of a Living Laboratory, it is not only designed for residential purposes, but also for various uses. This requires an intelligent construction with flexible space management. A modular, shoe-box design will allow free use of the floors and permit conversion options throughout the life cycle. In terms of sustainability, local, durable and recyclable materials should be used to create a construction that is as CO_2-neutral as possible. Furthermore, smart energy and water management as well as the possibility for urban farming should be considered for a self-sufficient building. The application of local construction methods and architecture enables the integration of the building into its surroundings. And by involving the local population into the construction process, a house "for Botswana" and "made in Botswana" can be realised. Moreover, the Tree of Life as a sustainable, self-sufficient building should become a pilot project that can be transferred to other projects throughout Africa.

The aim of this thesis is to develop a sustainable structural concept for the smart homes of the MSP, based on the first design. Therefore, all the requirements of the Tree of Life and the MSP project should be met.

1.2 Context

The MSP project was launched in the summer of 2019. As already mentioned, the vision of the project is to develop a self-sufficient, sustainable and future-oriented urban district in Botswana. The project was inspired by Vasilis Koulolias, professor at Stockholm University. Instead of developing technologies for space colonies, Koulolias aims to develop sustainable solutions for life on earth. After approval by the Botswana cabinet, a worldwide network of universities and scientists is now working together to realize the project.

Natural disasters, climate change and urbanization endanger the future human life on earth. This brings up challenges like food security, water conflicts, education, energy and waste management and many more. An African school of design and engineering (a.school) as well as a business incubator (smart living) will be established as research and start-up centre to find technical solutions for those problems. Furthermore, the interaction between man and environment shall be analysed and improved. For this purpose, a Living Laboratory (L.Lab) is designed with living space for humans and animals. The intelligent residential complexes will be linked with modern Internet of Things technologies, so that all units function and work together. This will guarantee a self-sufficient energy, water, food and waste management.

Thanks to the vision of sustainable life on earth and modern state-of-the-art technologies, the outcome of the MSP project will become a blueprint for whole of Africa, in terms of a symbiotic coexistence of humans, animals and the environment.[4]

1.3 Methodology and structure

At the very beginning of the paper, the general principles for the construction of tall building structures are determined. This is followed by the second cornerstone of the thesis, the definition of the sustainable construction. Whereby references to the Maun Science Park project is made. Based on the principles outlined, various structural designs for the Tree of Life can subsequently be drawn. In order to evaluate the designs, requirements and criteria must be defined, which are derived from the fundamentals and the project description. The best design is then determined with the help of a utility analysis. Finally, the feasibility of the preferred design is proven by means of a structural analysis.

[4] Refer to Bühler 2020

2 Basics of tall building structures

To create a design for the Tree of Life, the basics of tall building structures will be analysed. Even though structures of tall buildings were developed to support skyscrapers up to over 100 storeys, the construction methods will also help to build a stable and economic structure for the Tree of Life.

All over the world tall buildings are being built to meet urbanization and architectural needs. However, there is no unique definition of the term "tall building". A common definition was published by the Council on Tall Buildings and Urban Habitat (CTBUH)[5]. According to them, a tall building can be described by three characteristics. A tall building is either significantly higher than the surroundings, it appears like a tall building due to its slenderness, or it uses special techniques due to the structural height.

Nevertheless, all tall buildings have in common that their shape is determined by the structural system.[6] The increasing number of floors generate both, a higher dead weight as well as a bigger live load. At the same time, the wind loads increase with greater height and seismic loads become more dangerous. The structure must resist these lateral loads without losing its vertical load-carrying capability. Figure 2 shows the main effects due to increased horizontal loads: bending, overturning, shearing and torsion.

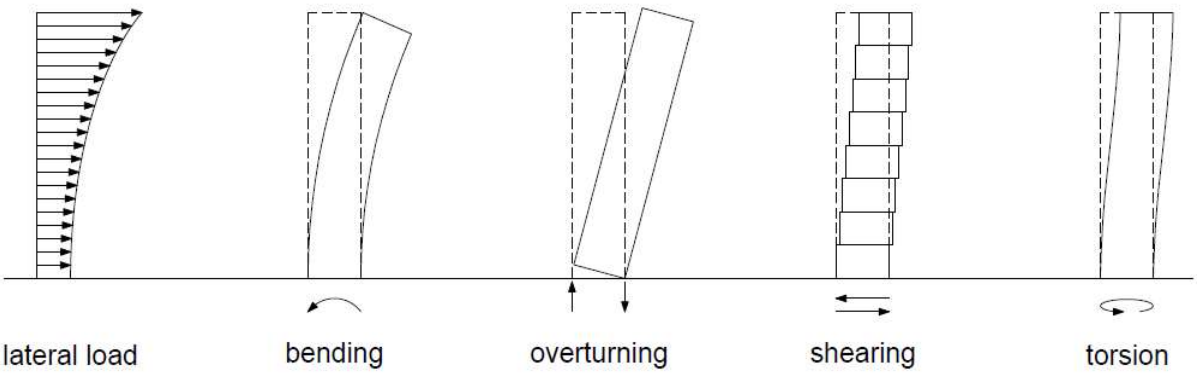


Figure 2 - Stress due lateral loads[7]

As a result, the load bearing structure of a tall building is determined by the horizontal loads with growing height. The lateral loads force a growing clamping moment, thus the demand on stiffening increases. To create the additional stiffness in a representing rigid-frame system, the lower columns and girders must be greater. Therefore, the material to resist lateral loads increases dramatically. At the same time, the material requirements for vertical load transfer

5 Council on Tall Buildings and Urban Habitat 2020
6 Refer to Phocas 2005, p.1
7 Adapted from Eisele 2014, p. 139

only increase linearly with the number of floors. This is due to the fact, that the material requirements for the slabs and beams remain the same because of constant spans. Only the columns have to carry the additional loads completely. Fazlur Khan was the first to recognize this effect, shown in Figure 3, and named it premium for height.[8] An investigation of Khan shows that the stiffness, rather than strength, becomes the dominant factor at a height beyond 10 storeys.

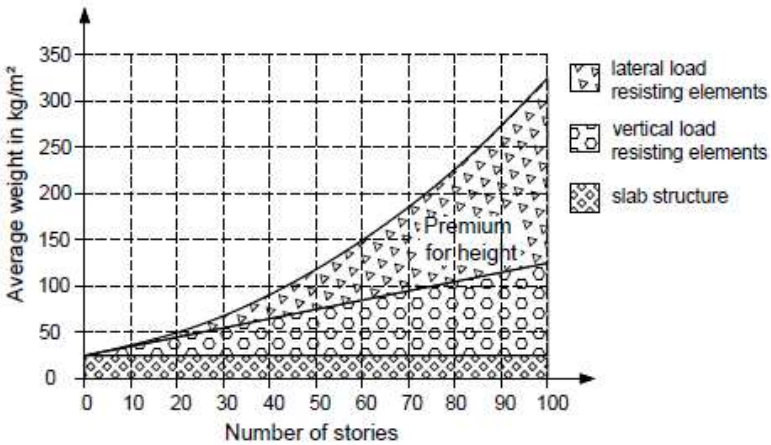


Figure 3 - Premium for height effect[9]

Special attention must therefore be paid to the stiffening elements in a tall building. They do not only affect the load bearing but also the deformation behaviour. The stiffening can be realized either using two-dimensional elements with diaphragm action or in a three-dimensional structure. Basic elements are rigid frames, braced frames and shear walls. Within the supporting structure, the slabs take a special role, as they have a load-distributing function for both vertical and horizontal loads. Last but not least, the foundation of the building plays a central role. The heavy loads of the upper structure must be transferred into the ground. In addition, the foundation system is essential for the building-ground interaction and thus has a direct impact on the building deformations.[10]

Based on the different support functions and the premium for height effect, a variety of structural systems have been developed. They enable economic high-rise solutions, depending on the height of the building.

2.1 Vertical load resisting elements

The vertical load transfer is supported by the commonly known construction components, walls and columns. Columns are mainly used in skeleton construction. They transfer the load vertically to the ground, while the beams provide the horizontal distribution towards them.

8 Refer to Gupta and Gupta 2017, pp. 6-7
9 Adapted from Gupta and Gupta 2017, p. 7
10 Refer to Phocas 2005, pp.1-2

The resulting frame structure inevitably causes high load concentrations. Due to the compressive forces, buckling of the columns, in both directions, must be taken into account. The major advantage of the skeleton construction is the separation of load-bearing function and room separation. This enables greater freedom of design.[11] In contrast, the bearing wall construction forms a combination of load-bearing function and room separation. This restricts the use of the free space, but at the same time allows the integration of the facade. A wall has great load bearing capacity along its plane, but is much weaker orthogonal to it. This leads to a buckling risk especially under concentrated loads. However, due to the diaphragm action, the loads are evenly distributed over the component. In order to maintain that effect, wall openings should be avoided when possible. A big issue of the bearing wall construction is the heavy dead weight. It makes the construction method rather unsuitable for high rise construction. An exception is the construction of cores for stiffening.[12]

The skeleton and the wall-bearing construction transfer the vertical loads into the ground, without interruption. Instead of a direct support system an indirect one can also be chosen. An interception, a cantilever or a hanging system allow a greater use of free space, due to the reduction of columns and walls.[13]

2.2 Horizontal load resisting elements

As mentioned above, bracing elements in tall buildings should be given a great attention. They must derive the lateral loads to the ground without causing large horizontal deformations. Therefore, they have to satisfy requirements of strength and rigidity. Both, horizontal and vertical stiffening elements are required to stabilize a building.

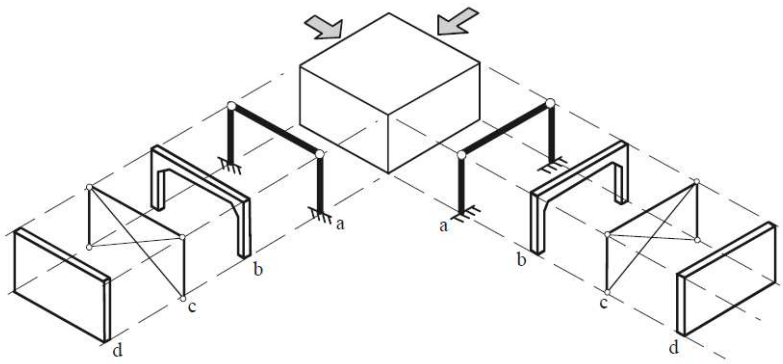


Figure 4 - Vertical stiffening elements: a) clamped column, b) rigid frame, c) braced frame, d) shear wall[14]

[11] Refer to Moro 2020, p. 290
[12] Refer to Moro 2020, pp. 288-289
[13] Refer to Eisele 2014, p. 144
[14] Widjaja 2012, p. 151

The horizontal stiffening is provided by the slabs, as you can see in chapter 2.3. For vertical stiffening elements clamped columns, rigid frames, braced frames or shear walls as shown in Figure 4 can be used. Clamped columns however are just suitable for one- or two-storey constructions, due to their lack of stiffness.[15]

2.2.1 Positioning of bracing elements

Regarding the static, at least three vertical bracing elements are required in the floor plan. But their lines of action must not intersect at the same point or run parallel. This is the only way to ensure a stable structure that prevents any displacement and rotation. Figure 5 shows simple examples of unstable and stable arrangements. By the use of additional stiffening elements, a statically undetermined system is created which has a higher redundancy and torsional stiffness. When torsion occurs, in addition to the horizontal loads, the resulting moment must also be removed. A torsion-free load transfer is only possible, if the centre of stiffness coincides with the centre of mass for all vertical elements. To minimize the additional load, eccentricity should be kept as low as possible, to reduce the torsion moment. At the same time, the inner lever arm can be increased to reduce the internal forces on the stiffening elements, caused by the torsion moment.[16]

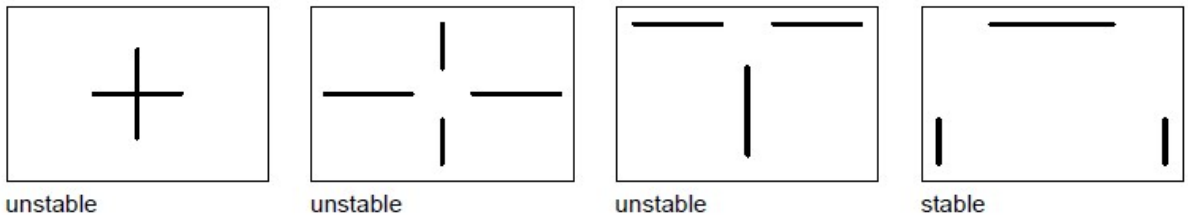


Figure 5 - Positioning of bracing elements in the floor plan[17]

The stiffening elements have to be arranged separately on each floor in this manner, otherwise the bracing effect of the entire building is lost. For this purpose, the elements should be arranged one above the other without an offset. So, the loads are simply and directly transferred to the ground without redirection. This has also a positive effect on the load-bearing behaviour, since the ballast can overcome any tensile forces from the tilting moment. In order to reduce the load from the overturning moment, it is helpful to increase the inner lever arm and to relocate the load-bearing elements in the facade. Stiffening elements should therefore always be used for horizontal and vertical load transfer.[18]

[15] Refer to Widjaja 2012, pp. 149-151
[16] Refer to Phocas 2005, p. 83
[17] Adapted from Eisele 2014, p. 140
[18] Refer to Moro 2020, pp. 366-367

2.2.2 Rigid frame

Basically, the frames shown in Figure 6 can be used: three-hinged frames, two-hinged frames or clamped frames. Whereas the clamped frame shows the best stiffness, but the construction of the clamping is quite complex.

Figure 6 - Rigid frames: three-hinged frame, two-hinged frame and clamped frame[19]

The load bearing capacity of the frames is based on the bending stiffness of the beam-to-column connection. The connection should have enough rigidity to maintain the original angle between the connected components.[20] This depends, among other things, on the flexural stiffness and thus the dimensions of the frame members. Furthermore, a wide span of the columns has a negative influence on the stiffness. The job of the rigid joint is to transfer the moments between the column and the beam. In this context, both horizontal and vertical loads are removed via bending tension and bending pressure. However, the redirection of force causes high stress peaks and therefore special attention should be paid to the connection details.[21] The biggest advantage of frame systems is the free space between the supports, which allows a variable use of the floor, especially in tall buildings.

2.2.3 Braced frame

Braced frames are an efficient system to resist lateral loads. With their higher stiffness, they are an improvement compared to the rigid frame. In addition to the vertical supporting elements, stiffeners are added so that the structure behaves like a vertical truss with dominant bending deformations. It should be noted, that a wide truss causes smaller internal forces and deformations compared to a narrow one. There are several ways to brace a frame, they can be summarized into four groups, see Figure 7: X-bracing, diagonal-bracing, K-bracing or knee-bracing. With the X-bracing, the components are only stressed in the axial direction due to hinged joints. This implies, that beams and columns can be dimensioned much more economically. Otherwise, the tension-loaded bracings have to be pre-stressed or alternatively, designed as compression struts. Accordingly, diagonal stiffeners must also be constructed to support both, pressure and tension. K-bracings also reduce the bending moment in the beam while having a shorter buckling length than diagonal bracings. For seismic endangered buildings, knee-bracings are usually used. Their eccentric positioning enables the controlled

[19] Widjaja 2012, p. 152
[20] According to Gunel and Ilgin 2017, p. 2668
[21] Refer to Phocas 2005, p. 85

creation of plastic joints for energy dissipation.[22] Apart from that, the general selection and arrangement of the bracings is usually based on the requirements for openings.[23]

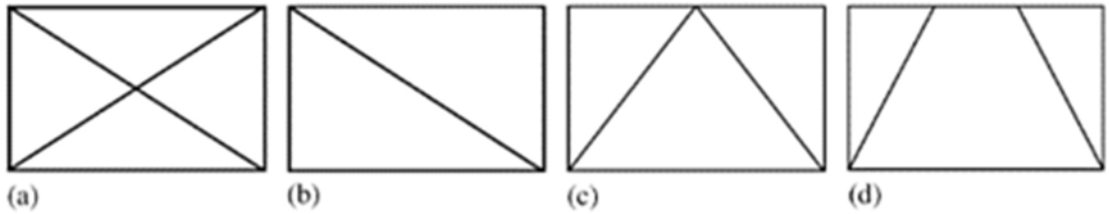


Figure 7 - Types of Braces: (a) X-bracing, (b) diagonal-bracing, (c) K-bracing, (d) knee-bracing[24]

2.2.4 Shear wall

Shear walls are very well suited for stiffening buildings. They behave like vertical cantilevers with the load bearing effect and stiffness in their plane. The thinner the wall, the greater the deformation at the end of the building. While narrow walls show bending deformation, compact walls tend to have shear deformation. In general, the walls can be made quite thin as long as they can withstand the shear stress. However, attention must be paid to their stability. Therefore, the walls are often arranged around the core to achieve a mutual transverse stiffening. In order to maintain the plain stress state, openings should be avoided or at least minimized. Multiple shear walls can be coupled either shear-stiff, shear-elastic or freely, to achieve greater stiffness.[25]

2.3 Slab structures

Slab support structures take a special role in the complete structural system. They distribute both, vertical and lateral loads to the load-bearing components (see Figure 8). Other important factors of the ceiling, apart from its load-bearing capacity, are its mass and its space-enclosing properties. On the one hand, the mass mainly defines the vertical loads that occur. And on the other hand, it influences the dynamic behaviour of the building. As a room-enclosing component, the ceiling has a significant impact on fire and noise protection. The choice of the slab system is generally based on the static requirements, the geometry of the floor plan and the construction possibilities.

As a bending active construction component, the slab distributes the vertical surface loads towards the ceiling beams or the walls, compare to Figure 8. The beams are usually constructed as girder grids with main and secondary beams. Depending on the geometry, an uniaxial or biaxial load transfer takes place. The girder grid passes the loads on to the supports. A flat

[22] Refer to Phocas 2005, pp. 87-89
[23] Refer to Günel and Ilgin 2017, p. 2669
[24] Günel and Ilgin 2017, p. 2668
[25] Refer to Phocas 2005, pp. 90-92

plate system without beams can also be used. However, this is most often limited to reinforced concrete. Apart from the vertical loads, the shear stressed slab also transfers the horizontal loads onto the vertical stiffening elements, due to the diaphragm action (Figure 8). Therefore, the slab must be constructed shear resistant. Usually, the shear deformations of the slabs can be neglected, which is equivalent to an unlimited stiffness. As with the shear walls, openings should be minimized or examined particularly. Dissolved floor plans also have a negative effect on the stiffness, but can be made possible by horizontal gaps. In addition, constraint loads must also be taken into account. They can be caused by different settlements, shrinkage, creep or temperature effects. [26]

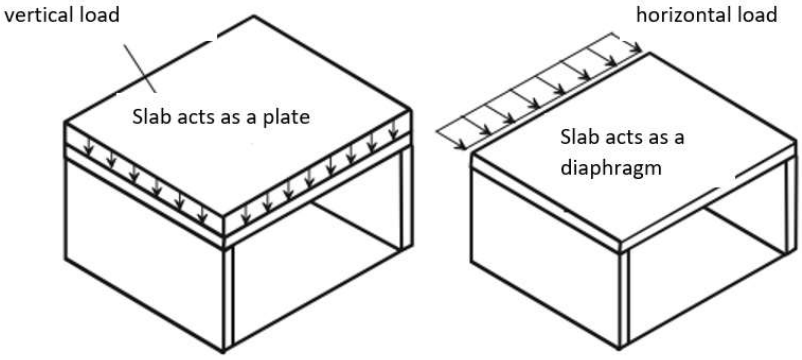


Figure 8 - Slab system load bearing effect[27]

Nowadays, ceilings are mainly made of reinforced concrete or in steel-concrete composite construction, although this is not mandatory. Due to technical developments, lightweight wooden ceilings with sufficient shear stiffness are now also possible. This is particularly advantageous since one of the main objectives for the ceiling construction of tall buildings is to reduce the building mass.

2.4 Foundation

The foundation transfers the loads from the structure to the load-bearing soil. In this process, global stability must always be ensured, especially in form of tilting and ground failure of the building. As already mentioned above, the foundation system is also of crucial importance for the building-soil interaction. In particular, the variable live, wind or earthquake loads can lead to differences in settlement that damage the structure above. For a long-term and unrestricted usability, therefore, a usable foundation system is needed. The structure and subsoil deformations can only be controlled if the overall system of the structure (supporting structure and foundation) and the subsoil (geology and groundwater) are taken into account during planning and construction. All known foundation engineering methods are available, from

[26] Refer to Phocas 2005, pp. 59-64
[27] Widjaja 2012, p. 149

flat foundations to deep foundations. Regardless of the design, these are usually made of reinforced concrete.[28]

Flat foundations are generally the simplest and most economical solution. However, a load-bearing soil is a precondition. Flat foundations transfer forces to the ground without clamping via the horizontal base surface. For the vertical forces, the distribution of the base pressure is decisive; it results from the elastic interaction between the foundation and the structure. The horizontal forces are transferred via the friction of the base surface. Problems with flat foundations, in particular with slab foundations of high-rise buildings, arise mainly in the case of point load removal and anchoring of tensile forces. In both cases, clever constructive solutions must be found. Settlement differences under shallow foundations or monolithic floor slabs can be prevented with expansion joints.[29]

If the subsoil in the upper layers does not have sufficient load-bearing capacity, deep foundations must be used. The main elements here are piles, either displacement piles or bored piles. The load-bearing capacity of piles is based on peak pressure and on surface friction. Both principles are used for the external bearing capacity of piles, which is determined by the strength properties of the foundation soil and is intended to ensure the absorption of the load within the permissible settlements. The internal load bearing capacity for transport and load introduction must also be verified with the help of the building material properties and dimensions. Several piles are usually connected with a pile grid plate. As a very rigid element, the thick plate ensures an even load distribution over several piles. If the plate is also involved in the load transfer by means of a bottom compression stress, this is referred to as a combined pile-plate foundation.[30]

2.5 Structural systems

Based on the vertical and horizontal construction elements, different structural systems for tall buildings were developed progressively. While at the beginning of the 20th century the structural elements were mainly used for vertical load transfer, this changed with the invention of the premium for height effect. Due to advanced developments in materials, technologies and computing, there are now many more systems than just the traditional rigid frame system. The structural systems for tall buildings can be classified by their structural behaviour under lateral loads. For buildings up to 40 floors, systems like rigid frame, shear wall or core systems are suitable. For skyscrapers over 40 storeys, the requirements for economical and efficient structural safety as well as the serviceability limit the structural

[28] Refer to Phocas 2005, p. 139
[29] Refer to Phocas 2005, pp. 139-140
[30] Refer to Phocas 2005, pp. 144-145 and 151-153

systems. For this reason, shear frame systems or tube systems can be used. Another alternative are outrigger systems or mega structures.[31]

Another classification is based on the arrangement of the lateral load resisting system. In an internal structure, all stiffening elements are located on the inside of the building, such as in the rigid frame system. In an external structure, in contrast, the major part of the lateral load-resisting system is placed within the building envelope. This includes mainly tube structures. Hybrid structures are in turn advanced systems which do not meet the criteria for internal or external structures, such as diagrid, outrigger or mega structures.[32]

Usually, tall building structures are realised in steel, reinforced concrete or composite construction. But nowadays the construction is no longer limited to this material, as for example an increasing number of wooden tall buildings show.[33]

2.5.1 Rigid frame systems

The most basic system is based on the stacking of rigid frames as presented in chapter 2.2. Figure 9 shows the model of a rigid frame system. It supports vertical and lateral loads with columns and beams. The columns are usually arranged according to architectural aspects in order to allow a maximum free use of the floor, which is a great advantage. At the same time, the span as well as the depth of the beams influence the stiffness of the system, which limits the arrangement. By focusing on a few extremely rigid frames, the remaining supports can be designed as pin-ended columns, which enable further architectural freedom. An additional frame field increases the stiffness even more, in comparison to a single one. For each additional field added, the extra stiffness is much lower.[34] Due to their high ductility, frame systems are particularly suitable for seismic areas.[35] The controlled formation of plastic joints leads to great energy dissipation. However, the disadvantage of these systems is their large lateral deformation. This disturbs the serviceability and can lead to damage in non-structural elements. The bending deformation of the cantilever is only one part of those deformations. The second deformation part is caused by the shear deformation due to the bending of individual construction elements. Not only the total deformation, but also the deformation of the storey must be limited. If not, the P-Δ-effect would endanger the stability of the building.[36] Frames are therefore only suitable for high-rise buildings up to 25 storeys.[37] A famous example

[31] Refer to Günel and Ilgin 2014, pp. 20-21
[32] Refer to Gupta and Gupta 2017, pp. 7-9
[33] Refer to Green and Taggart 2017, p. 20
[34] Refer to Phocas 2005, p. 86
[35] Refer to Günel and Ilgin 2014, p. 22
[36] Refer to Phocas 2005, pp. 85-86
[37] Refer to Günel and Ilgin 2014, p. 23

is the 12 storey, 55m high Home Insurance building in Chicago, which is recognised as being the first skyscraper.

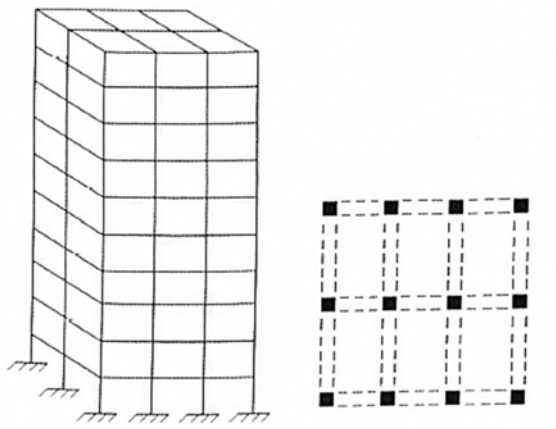


Figure 9 - Rigid frame system[38]

2.5.2 Shear wall systems

In a shear wall system, as the name suggests, the entire building is stiffened by shear walls. The superimposed wall panels and the height of the building consequently lead to a slender wall. Therefore, the behaviour of the system can be described as a vertical cantilever. The bending deformation in these systems is consequently dominant over the shear deformation.[39] For this reason, the deformation in the upper floors is much greater than in the lower floors. That is why the system is not suitable for super tall buildings with more than 35 floors.[40] Single shear walls are well suited in combination with flat slab systems. This way, a maximum floor height can be achieved while at the same time providing lateral-load resistance. In tall buildings however, several wall panels are usually coupled.[41] This increases the stiffness, while also allowing to break up the monolithic architecture with openings. With shear-stiff connections the load bearing behaviour remains the same. While with elastic ones, it depends on the stiffness of the coupling. For example, truss stiffener behaves like a monolithic connection, whereas beam connections show a significant shear deformation. A shear-free connection is of course also possible. The walls can be distributed over the floor plan as desired, as long as the static requirements are considered. Anyway, they are often arranged in the building core rather than in the facade, due to their room separating effect. In Figure 10 (a) a shear wall system is shown.

[38] Günel and Ilgin 2014, p. 22
[39] Refer to Phocas 2005, p. 90
[40] Refer to Günel and Ilgin 2014, p. 27
[41] Refer to Phocas 2005, p. 91

2.5.3 Core systems

Core systems are popular structures for tall buildings. Because the cores can be used both, as a supporting element as well as for the infrastructure, e.g. elevators and building services. Due to the tubular structure, in addition to their flexural rigidity, cores have a high torsional stiffness. Thus, they can withstand not only vertical loads but also lateral loads.[42] However, openings should be minimized in order to avoid a significant reduction in stiffness. The construction method is a typical example of an internal structure with the arrangement of the cores playing a significant role. A central arrangement minimizes torsional stress, see 2.2.1, and at the same time achieves pressure preload by the ceiling loads, which has a positive effect on the bending stress.[43] When several cores are used, the resulting constraining stresses which are caused by indirect action, must be taken into account.[44] The problem can be solved with expansion joints, but this means that each part of the building has to be stabilized separately. Another problem is the total stability of the core structures. They tend to tilt and must therefore be anchored constructively. The stiffness and tilt resistance are significantly influenced by the structural depth of the core. Since this is limited, core structures are only suitable for tall buildings up to 35 storeys. [45]

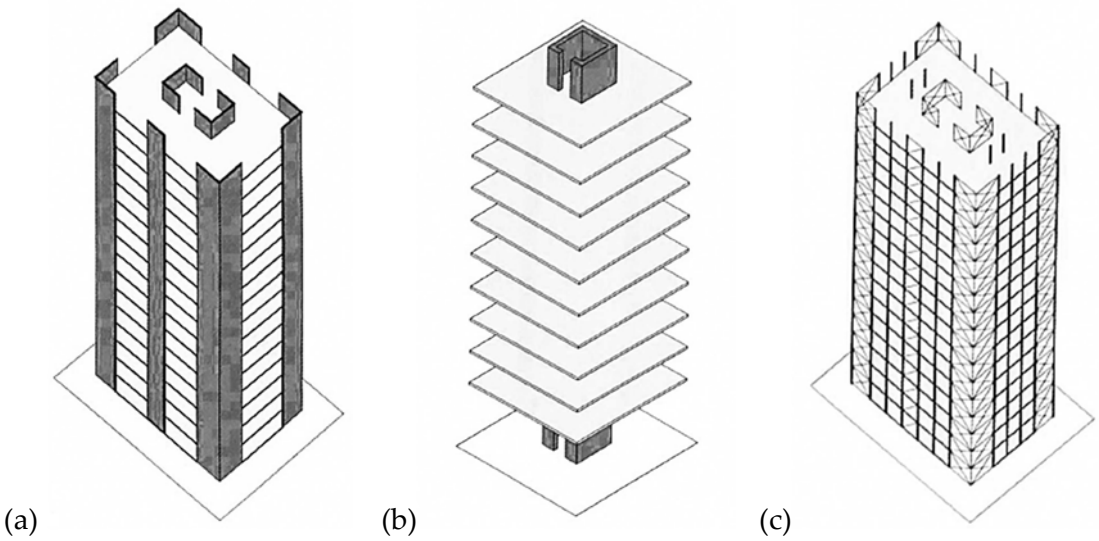


Figure 10 - Shear wall system (a), core system (b) and shear frame system (c)[46]

Core support structures can be designed as cantilever beams with clamped slabs, as shown in Figure 10 (b). However, the economic efficiency is low due to the clamping effect of the slabs. In combination with a double-jointed frame this can be improved, as the continuous perimeter columns and the core share the vertical load. At the same time the horizontal load is still

[42] Refer to Günel and Ilgin 2014, p. 25
[43] Refer to Phocas 2005, p. 95
[44] Refer to Widjaja 2012, p. 159
[45] Refer to Phocas 2005, p. 98
[46] Günel and Ilgin 2014, pp. 26-28

carried by the core only, due to hinged beams. Several floors can also be grouped into modules and then be intercepted or suspended. For interception systems, very rigid space trusses are used. These enable, for example, a freely usable first floor. Suspension systems do not have a stability problem of columns, but they do have a strain problem of the suspension.[47] Braced frames or shear walls are used for the core itself. Rigid frames have too little stiffness to be used just for the core. Braced frames are particularly popular in the USA for steel constructions. The type of bracing depends on the requirement of stiffness and usage. Cores from shear walls are usually build as partially closed cores.[48] The coupling is realised by floor beams. Even though, a closed core would be better, this way architecture and load bearing behaviour can be combined.

2.5.4 Shear frame systems

To increase the stiffness of frame systems, rigid frames can be combined with braced frames or shear walls, see Figure 10 (c). This way a shear frame system is created. These interactive systems have a higher stiffness and therefore allow greater heights compared to a stand-alone system. The reason for the better resistance is the different deformation behaviour of the subsystems, compare to Figure 11.

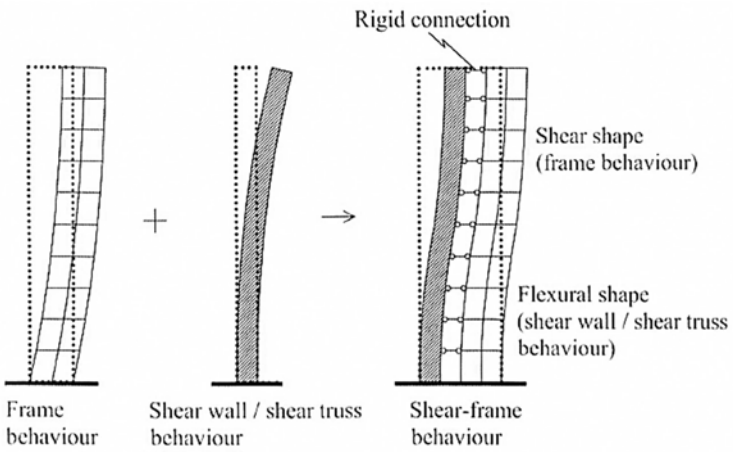


Figure 11 - Behaviour of the shear frame system under lateral loads[49]

While rigid frames show a distinctive shear behaviour, bending deformations dominate within shear walls and braced frames. Due to the coupling of the almost stiff slabs, the deformations are compensated by each other and the loads are redistributed.[50] Walls and bracing are often arranged as a partially closed building core because of their space-enclosing character. Slabs or beams are then used as connecting elements to approximate the behaviour of a closed core. In addition, the arrangement of the elements tries to avoid torsional stresses. Such systems can

[47] Refer to Phocas 2005, p. 96
[48] Refer to Günel and Ilgin 2014, pp. 25
[49] Günel and Ilgin 2014, p. 29
[50] Refer to Günel and Ilgin 2014, p. 29

respectively be called shear trussed frame (braced frame) system and shear walled frame system. They are well suited for tall buildings between 40 and 100 storeys. A world-famous example of a shear trussed frame system is the Empire State Building in New York (102 floors, 381m).[51]

2.5.5 Tube systems

The supporting principle of the tube system is based on a hollow box section cantilevering from the ground. With the complete stiffening in the perimeter of the building, a maximum diameter and thus inner lever arm is created. For load distribution and additional horizontal stiffening, the slabs are used. The resulting three-dimensional support structure offers ideal resistance to vertical and horizontal loads as well as greater space inside the building.[52] The structure is therefore not limited to a rectangular floor plan, it can also be shaped as a triangle, circle or even as a free-form. Columns can be added inside to support the vertical load transfer. Tube systems can be designed as framed-tube or as trussed-tube. Shear-walled tubes are usually not realized besides core structures, due to their architectural limitations. To further increase the stiffness and thus the height of the building, tube-in-tube or bundled-tube systems can also be realized.[53]

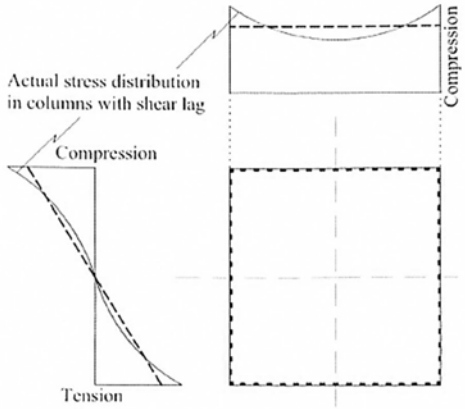


Figure 12 - Shear lag effect in a framed tube system[54]

The frame-tube system, shown in Figure 13 (a), was developed by Kahn from the rigid frame systems.[55] As with the frame, the stiffness is directly influenced by the span and dimensions of the elements. Consequently, the tube systems consist of closely spaced perimeter columns which are rigidly connected to deep spandrel beams at floor level. Due to the shear and bending flexibility of the components, the complete stiffness of a hollow box section cannot be achieved. The stress distribution in the frames is therefore not linear. The edge columns attract

[51] Refer to Günel and Ilgin 2014, p. 32
[52] Refer to Phocas 2005, p. 108
[53] Refer to Günel and Ilgin 2014, p. 72
[54] Günel and Ilgin 2014, p. 73
[55] Refer to Günel and Ilgin 2014, pp. 73-74

a much greater load than the middle ones, which is called shear lag effect, see Figure 12. This effect has a negative impact on the load-bearing behaviour and should therefore be kept to a minimum. The World Trade Center Twin Towers in New York were a well-known example for a framed tube system. With 110 floors, they reached a height of 415m and 417m.

The diagrid-tube is an adaptation to the frame tube.[56] It uses diagonal bracing instead of vertical columns, see Figure 13 (b). This allows an exclusively axial stressing of the elements, which makes them much more efficient. Together with the spandrel beams or slab structures, they form the triangles required for a truss structure. A beautiful example is the 30 St Mary Axe building in London.

To increase the span of the columns an external truss can be attached. In this way a trussed-tube system is constructed(Figure 13 (c)), whereby, in comparison to a framed tube, the stiffness is improved.[57] Several diagonal, multi-storey bracings are connected to the columns and some spandrel beams. Therefore, they participate in vertical and horizontal load transfer. With a reduced shear lag effect, the load bearing behaviour is almost the same to a vertical cantilever.

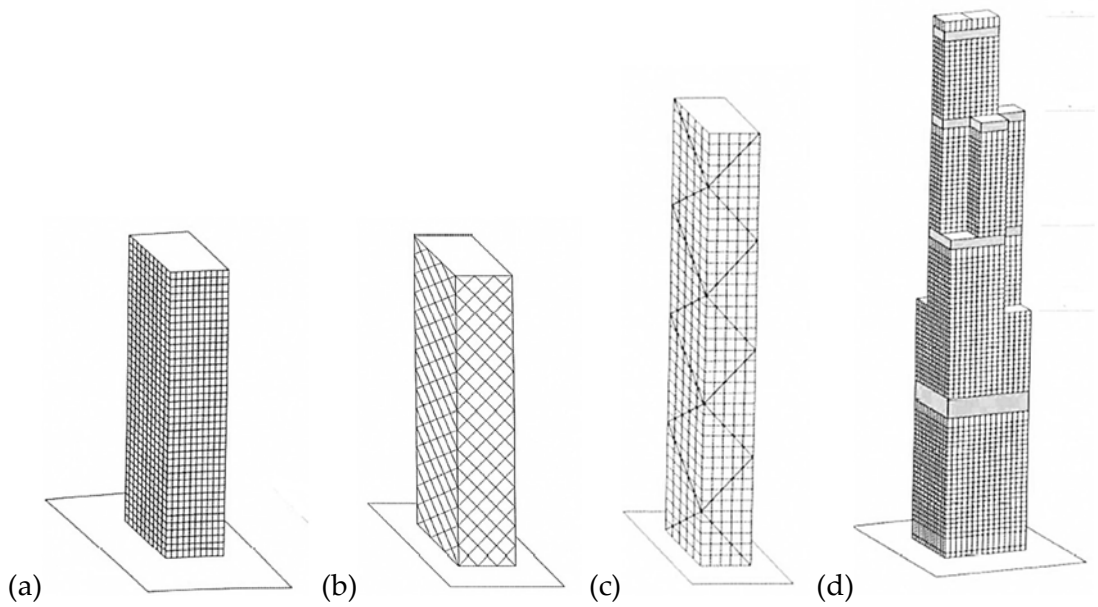


Figure 13 - Tube systems: framed tube (a), diagrid tube (b), trussed tube (c), bundled tube (d)[58]

Up to 80 floors can be reached with a tube-in-tube system.[59] An inner core and the outer tube-structure are connected with shear-resistant slabs or outrigger systems. The coupling ensures

[56] Refer to Günel and Ilgin 2014, p. 77
[57] Refer to Günel and Ilgin 2014, p. 82
[58] Günel and Ilgin 2014, pp. 72, 77, 84, 88
[59] Refer to Phocas 2005, pp. 126-127

a uniform deformation and thus an addition of stiffness. Consequently, the load resistance and stability are improved as well as the shear lag effect is reduced.

Bundled tube systems consist of several connected tubes.[60] Due to the different heights and shapes of the individual tubes, a great architectural variety can be achieved. From a static point of view, the tubes behave like a single tube system with a great stiffness and reduced shear-lag effect. Furthermore, with setbacks it is possible to increase the cross-section at the base and therefore control the slenderness of the building, as shown in Figure 13 (d). The Willis Tower in Chicago (108 floors, 442m) is a perfect example.

2.5.6 Outrigger systems

The outrigger system is a modification of the shear frame systems with core.[61] Some perimeter columns are connected to the core by means of a stiff outrigger. An outrigger consists of a horizontal shear-truss or shear-wall, which is usually rigid connected to the core and hinged to the column. This increases the effective flexural depth of the system and therefore the stiffness in bending direction. The moment, caused by lateral loads, is now supported by a pair of forces through the perimeter columns. The resulting inner moment acts across the outrigger as a relieving moment onto the core. In this way the tubular behaviour of the structure can be ensured, and the horizontal drift minimized. To generate sufficient stiffness, outriggers are at least one floor high and form an I-beam with the adjacent slabs.[62] Because this has a significant effect on the space required, they are often placed in service floors. In order to further improve outrigger systems, the pair of force can be distributed to all perimeter columns with a belt truss, see Figure 14 (a). But the stiffness of structure is mainly influenced by the position of the outrigger. With only one outrigger, the most effective location is between 40-60% of the height.[63] Additional level of outriggers increase the stiffness of the building, but each one by a smaller amount than the previous. However, in seismic zones, this should be provided in order to create additional stiffness reserves and thus a higher redundancy. The Taipei 101 (Taipei, 101 floors, 508m) shows that it is possible to build skyscrapers with over 100 floors with outrigger systems.

2.5.7 Mega structures

In mega structures, the number of vertical supporting elements is reduced to a few, particularly strong components. This is realised by mega columns or shear walls, with a large cross-section running through the entire height of the building.[64] To resist the bending moment

[60] Refer to Günel and Ilgin 2014, p. 87
[61] Refer to Günel and Ilgin 2014, p. 44
[62] Refer to Phocas 2005, p. 99
[63] Refer to Günel and Ilgin 2014, p. 45
[64] Refer to Günel and Ilgin 2014, p. 34

from the wind load, the columns should be arranged with the largest possible distance. The resulting tension can then be overcome by the concentrated vertical loads. As a secondary structural element, the bracing plays a major role. In the case of mega structures, the slabs are insufficient for bracing. Therefore, large belt-trusses, vierendeel frames or mega braces are used.[65] They couple the mega columns rigidly to each other and thus provide the stiffening and concentrated load transfer. The belt-trusses or the vierendeel frames must extend over several storeys to provide enough stiffness. Furthermore, they must also be arranged in several levels to ensure torsional rigidity. Therefore, mega structures can also be described as mega frames. Mega braces are multi-storey diagonals which can be installed over the entire height of the building. In this way, a three-dimensional framework is created, which can also be called space truss system.[66] Since the vertical coupling is not distributed over the entire height, the structure can be strongly dissected. Individual floors can be supported by interception or suspension systems. This allows great architectural freedom in terms of facade design and room layout. The idea of mega columns can also be used exclusively in the lower floors of a building.[67] In this case, the main structural system is transferred to the mega columns by means of heavy intercepting beams. This creates a column-free entrance area, compared to other structural systems. Different elements of mega structures are shown in Figure 14 (b). Mega structures can be used for high-rise buildings up to far more than 40 storeys.[68] Well-known examples are the Commerzbank Tower in Frankfurt (vierendeel-frame, 56 floors, 259 m) or the Bank of China Tower in Hong Kong (space-truss, 72 floors, 367 m).

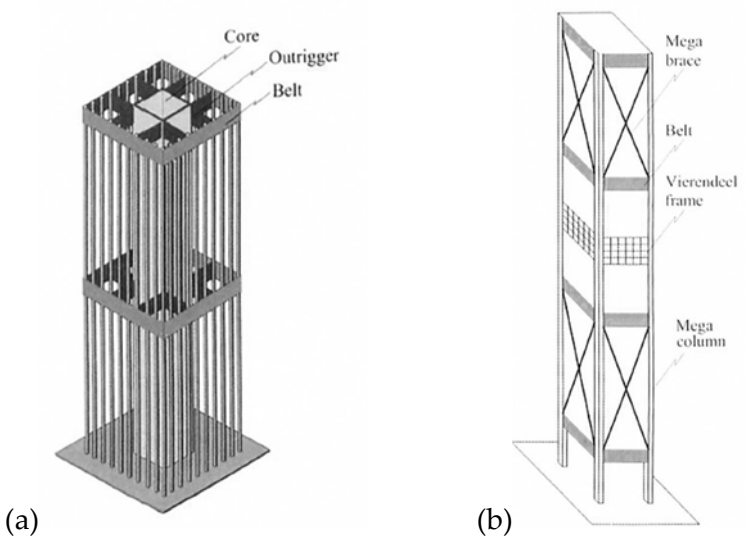


Figure 14 - Outrigger system (a) and mega structure (b)[69]

[65] Refer to Phocas 2005, p. 130
[66] Refer to Günel and Ilgin 2014, p. 37
[67] Refer to Günel and Ilgin 2014, p. 41
[68] Refer to Günel and Ilgin 2014, p. 37
[69] Günel and Ilgin 2014, pp. 37, 48

3 Basics of sustainable construction

Sustainability is a broad term which is nevertheless difficult to grasp. This chapter works out the definition of sustainability before studying what makes a sustainable construction. This is followed by a more detailed description of the three pillars of sustainability in terms of construction. Due to the large topic of sustainable construction, only the relevant aspects for the primary structure of the Tree of Life are dealt with in the context of this thesis.

The term sustainable development was first defined in 1987 by the World Commission on Environment and Development (Brundtland Commission) as follows:

> "Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs."[70]

The report itself, focuses on two key concepts. On the one hand, the concept of meeting the needs of the world's poor and on the other hand, the idea of limitation of environmental resources to meet present and future needs. The commission was inspired by the Oil Crisis (1973) and the Club of Rome Report (1972), which showed the limits of economic growth in the industrialized world. Then in 1992, representatives of all countries met for the first time at the UN conference in Rio to discuss the concept of sustainability worldwide. Further milestones in the development of sustainability were the ratification of the Kyoto Protocol, the UN Climate Conference in Paris and the adoption of the Sustainable Development Goals (SDGs) within the agenda 2030. Sustainable development, as we understand it today, is based on three pillars: ecology, economy and social affairs.[71]

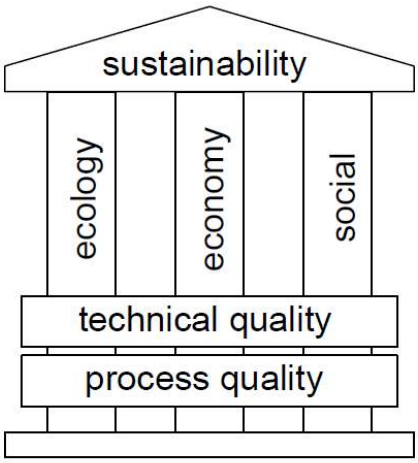


Figure 15 - Pillars of sustainable construction[72]

[70] World Commission on Environment and Development 1987, p.41
[71] Refer to Friedrichsen 2018, pp. 9-12
[72] Adapted from Friedrichsen 2018, p. 9

Sustainable development does not stop at the building industry. Every construction project interferes with nature and thus upsets the ecological balance. Due to the progressing industrialisation, any destruction can no longer be regarded as local, but rather has global effects as well. Thus construction activities show a great potential for the topic of sustainability. Sustainable construction means that the construction, operation and demolition of buildings should have the least possible negative impact on the environment, be economically efficient and guarantee the social and cultural well-being of people.[73] Beyond these three pillars of sustainability, sustainable construction should provide the technical quality as well as the process quality, see Figure 15. This should guarantee the quality of planning and execution as well as the reliable functionality of the building. Most of the standards for sustainable construction can be classified under the three pillars of sustainability, but however, there are construction-specific exceptions. This applies above all the classic building protection functions: moisture, wind, thermal, sound and fire protection. These properties have multiple functions as they are a basic condition for the health, comfort and protection of the residents on the one hand and guarantee the durability of the construction on the other. Durability itself cannot be clearly attributed either, as it guarantees both ecological and economic sustainability. Last but not least, the basic principle of every structure, the load transfer, must be given.[74]

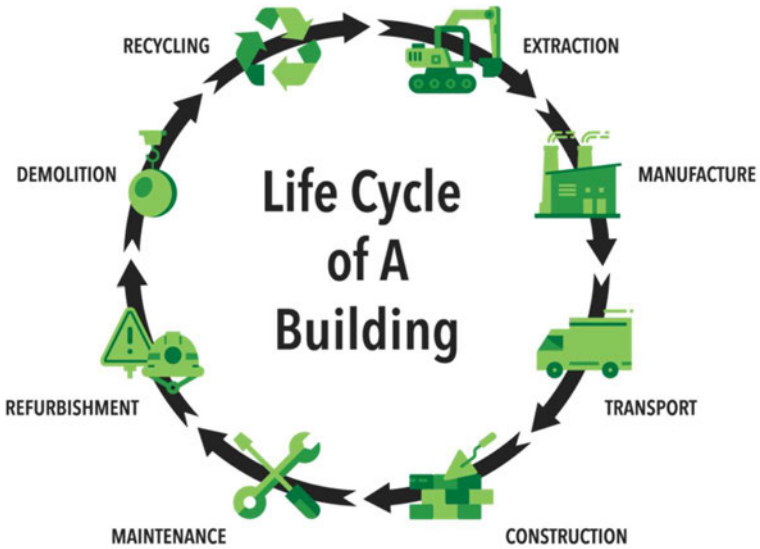


Figure 16 - Life cycle of a building[75]

Due to the long periods in which buildings are used, it is essential to consider energy consumption and environmental impacts over the entire life cycle. Sustainable optimisation

[73] Refer to Moro, 2020, p. 37
[74] Refer to Moro, 2020, p. 39
[75] IGBC Irish Green Building Council 2021

therefore includes not only the construction but also the utilization, the demolition and a lot more stages, see Figure 16. Within the utilization stage for example, maintenance and refurbishment can occur. The maximum costs and environmental impacts are therefore not occurring in the construction phase but in the utilisation phase. The demolition phase, in turn, offers the possibility of recycling energy and resources. It is important to consider the issue of sustainability right from the planning stage, as this is where the possibilities for influencing the building are the greatest. [76]

For the sustainability assessment of buildings, many procedures and certificates have been published in recent years. The best known at present is the US-American LEED (Leadership in Energy and Environmental Design). It was developed by the U.S. Green Building Council (USGBC), which is part of the World Green Building Council (WGBC). Since 2009, Germany has a quality label for sustainable construction as well. It was developed by the German Sustainable Building Council (Deutsches Gütesiegel für Nachhaltiges Bauen, DGNB). Originally, it was developed only for office and administrative buildings, but it is now also applicable to other types of use. Based on the DGNB, the German Federal Ministry of Construction has also developed a Sustainable Building Rating System for Federal Buildings (Bewertungssystem Nachhaltiges Bauen für Bundesgebäude, BNB). [77]

3.1 Ecological sustainability

Ecology is the first pillar of sustainability. It has already been defined in a general sense by the Brundtland Commission. Today, ecological construction describes on the one hand the protection of the environment from the negative effects of the building project and on the other hand it also means the protection of the humans in their environment.[78]

First and foremost, this means energy-efficient building. [79] The question of energy supply is one of the biggest problems in the world. As the energy sources are limited, the energy demand must be reduced. The global energy input of buildings is very high, especially through heating and cooling. This means that there is a great potential for savings in the use of buildings. However, energy-efficient construction means not only saving energy in the utilisation phase but also the reduction of the primary energy requirement in the construction phase as well as the recycling at the end of the life cycle. Recycling also enables the reduction of material consumption. This is important because the construction industry purchases about 90% of all mineral materials used. Buildings are therefore a huge material store, which can be

[76] Refer to Friedrichsen 2018, p.3
[77] Refer to Friedrichsen 2018, p. 23
[78] Refer to Friedrichsen 2018, p. 93
[79] Refer to Friedrichsen 2018, pp. 93-97

used efficiently to save raw materials and avoid construction waste.[80] Not only the raw materials used must be taken into account, but also the auxiliary materials and waste products that are produced. In addition, there are environmental impacts and pollutant emissions from building materials which not only burden nature but also have an impact on humankind.[81]

In the following, some criteria for ecological sustainability are examined in more detail. The general relevance is explained, as well as their particular importance for the project.

3.1.1 Energy efficient building design

An essential criterion for saving energy is an energy-efficient building design as the energy consumption of a building is largely determined by its architectural design[82]. One benchmark for this is the A/V-ratio, which reflects the relationship between the outer surface and volume of the building. Due to the compactness of the building, a low A/V-ratio, both heating or cooling energy as well as construction materials can be saved. Spheres have the lowest A/V-ratio, but cube-shaped sculptures are more useful because they are easier to construct. In addition, the ratio improves in larger buildings. Another important feature of the building design is the thermal zoning[83]. The orientation according to the solar radiation influences both winter and summer thermal insulation. In cold zones, using the sun's heat allows to save heating energy. While in summer, the orientation of the building is intended to provide passive cooling, mainly through shading.

Botswana is located in a subtropical climate zone. Maun itself has a steppe climate with little rainfall and many hours of sunshine. Since the average annual temperature is over 20°C, summer heat protection is crucial.[84] The building design can have a significant influence on this. The same applies to the A/V ratio. At present, however, neither an exact location nor an architect's design is existing. In this paper, a supporting structure for a simple fictional volume is designed. The sustainability of an energy efficient building design is therefore not included in the construction design.

3.1.2 Construction materials

Probably the greatest impact on ecology in the construction phase lies in the choice of material. Already the processing of the raw material into an industrially usable product causes extensive energy consumption and environmental impacts. The energy required to extract, process and install the materials is also known as embodied energy and can be recorded as primary energy

[80] Refer to Friedrichsen 2018, pp. 126-127
[81] Refer to Friedrichsen 2018, p. 130
[82] Refer to Friedrichsen 2018, p. 98
[83] Refer to Friedrichsen 2018, p. 100
[84] Refer to World Weather & Climate Information 2021

consumption. In addition, substances with greenhouse, acidification or ozone-layer-depletion potential are released over the life cycle and harm the environment.[85]

The choice of the material also influences the maintenance intensity, the service life and the recycling ability of the building products.[86] The aim is to make the building last as long as possible. For this purpose, components with different service lives should be separated by construction to enable easy replacement and repair. In addition, the clear separation of materials is important for recyclable construction. Composite materials are therefore generally critical.

Last but not least, possible harmful substances must be taken into account.[87] Building products contain numerous organic and inorganic substances which can enter the air, soil or groundwater. They not only endanger the environment but also human health. There are various eco-labels for evaluation of construction products. However, not only the products but also the way of installation shall be considered.

The environmental impact of building materials and thus their sustainability can be evaluated, for example, with the help of a life cycle assessment (LCA). The LCA is based on the compilation of input and output flows, the life cycle inventory analysis (LCI) and the assessment of their environmental impact, the life cycle impact assessment (LCIA).[88] Input flows such as raw material and energy consumption are not only recorded during extraction and further processing but also during installation, maintenance, disposal or recycling. Output flows, such as waste and emissions, are also tracked over the entire life cycle. With the help of public accessible databases, these flows can be quantified globally. Furthermore, the LCA provides a method for engineering analysis.

As already mentioned, the choice of building material probably has the greatest impact on the sustainability of the construction which is also the case in this project. Natural building materials such as wood or clay are generally more sustainable than man-made materials. In addition, the more industrial cycles the material has to pass through, the more environmentally damaging it is.

3.1.3 Building envelope

The design of the building envelope is largely responsible for energy consumption during the utilisation phase. The thermal insulation of a building allows energy savings while at the same time improving the well-being through a more pleasant indoor climate. At the same time, the

[85] Refer to Moro 2020, p. 43
[86] Refer to Friedrichsen 2018, p. 149
[87] Refer to Friedrichsen 2018, p. 130
[88] Refer to Moro 2020, p. 44

outer shell also closes off the room and thus provides protection against moisture, wind and sound. This prevents damage to the building and disturbance of the residents.[89]

The building envelope is an important feature of sustainable construction for any building. In the context of this thesis, however, only the primary structure is developed.

3.1.4 Building technology and renewable energies

Another way to minimize energy losses during the utilization phase is the efficient use of energy. This means in particular efficient building services engineering.[90] Heating systems, air conditioning, ventilation and hot water preparation are part of every building but require an ongoing energy supply. It is therefore important to ensure that the technology has the highest possible efficiency, in other words, a correspondingly low energy consumption for a given output. The installations should also be properly dimensioned and adapted to the building and its use in order to avoid wasting energy. Regulating and control units can be very helpful in this context. Considerations on whether to install a centralised or decentralised system must be evaluated by experts.

Another important issue in the context of building services engineering is the supply of energy. Sustainable buildings should be powered by renewable energies wherever possible.[91] One option for this is solar energy. It can be easily integrated into a building, is energy-saving and ensures a significant reduction in CO_2 emissions. Solar energy can either be used as a solar thermal system for hot water heating or as a photovoltaic system for electricity generation.

It is mandatory to take advantage of the sun energy opportunities in Maun. With an average of over approximately 250 hours of sunshine a month, Maun is perfectly suited for this.[92] The efficiency of the building technology depends on the technical planning of the building. Nevertheless, both parameters do not have a direct impact on the structural design, only the additional loads regarding solar panels may appear.

3.2 Economic sustainability

Another component of sustainable construction is economic sustainability, this refers to sustainable cost management.[93] The aim is to include all costs, not only the construction but also the costs of operation and demolition. In analogy to ecological sustainability, a realistic estimate of the total costs over the entire life cycle should be possible. The aim is to enable the building to be permanently marketable and thus contribute to sustainability. However, there

[89] Refer to Friedrichsen 2018, p. 103
[90] Refer to Friedrichsen 2018, p. 109
[91] Refer to Friedrichsen 2018, p. 120
[92] Refer to World Weather & Climate Information 2021
[93] Refer to Moro 2020, pp. 48-49

are methodological difficulties in measuring total life cycle costs. Already the determination of the actual construction costs is a projection into the future. Calculating in advance, the costs of constructing such a complex technical structure is naturally difficult. By looking at the costs over the entire life cycle, these difficulties increase considerably. Assumptions have to be made regarding the lifetime, renewal cycles, demolition, recycling and dumping of the component. Despite these assumptions, the difficulties in predicting the cost trend and future developments remain, caused by the assumed lifetime of 50-100 years. Although there are major difficulties and inaccuracies, it is a big step in the right direction to capture the whole life cycle costs.

Important aspects of economic sustainability include optimised planning and an adapted financing model. With comprehensive and coordinated planning, mistakes can be avoided at an early stage and target-oriented solutions can be found. A suitable form of financing allows the financing costs to be accurately estimated. The production costs should be determined as exactly as possible at the beginning. While the costs of utilization can be controlled with proper use and maintenance of the building. A profitability analysis is used to check the measures implemented.[94]

The approach of economic sustainability plays a very minor role in the context of this thesis. It has therefore only been mentioned for the sake of completeness. In relation to the Maun Science Park, however, it is an essential aspect for the realisation and feasibility of the project.

3.3 Social sustainability

The social pillar of sustainability is based on the principle of social equality and justice.[95] It is already a cornerstone of the Brundtland Report and in terms sustainable construction it covers the entire range of effects between individual users, buildings, cities and the entire community. This includes both functional and social aspects.[96]

Sustainability is achieved primarily through long-term building use. To ensure this, user satisfaction and acceptance are crucial. Functional aspects serve to improve the comfort and thus the residential quality of the users. This, together with a good feeling of security, ensures that the building will be used for a long time. User behaviour itself has a considerable influence on the service life of the building.[97] At building level, it is important to ensure the functionality. Therefore, it is important to clarify the requirements already at the planning stage. In addition, structural flexibility, both in terms of use and floor plan organisation, should be aimed for.

[94] Refer to Friedrichsen 2018, p. 181
[95] Refer to Hill and Bowen 1997, p. 226
[96] Refer to Friedrichsen 2018, p. 237
[97] Refer to Friedrichsen 2018, p. 253

The flexibility allows a simple conversion or adaptation of the building and therefore has the potential to extend the life cycle of the building.[98] Finally, the building must be integrated into the urban and social context. The integration into the surroundings is characterized by scale, architecture, green spaces, culture, mix of residents and much more.[99] All this has a great influence on the acceptance of the construction. In the social context, a building has the potential to have a positive impact on the community, not only in the short term but also in the long term. With a fair distribution of social costs and the provision of local materials and services, poverty can be fought effectively, equal opportunities can be guaranteed, and social relations can be strengthened.[100]

The social pillar of sustainable construction is probably the most difficult to consider and above all to evaluate. This is because the social impacts of each project are very individual. They must be considered and evaluated separately. Nevertheless, the task of social sustainability offers great opportunities. Below some of the social sustainability aspects for the Maun Science Park project are considered more detailed.

3.3.1 Comfort

Comfort is an essential factor for user satisfaction. It can be seen as a collective term for thermal comfort, visual comfort and sound protection.[101] Thermal comfort is composed of temperature and air sensation. Not only the room temperature, but also the surface temperature of the building components is decisive for the hot and cold feeling. For the air sensation, the humidity and the air velocity are of importance. Visual perception is mainly an architectural feature, but it also plays a key role in comfort. Daylight and sun are important for human psyche. Buildings should therefore be sufficiently flooded with light, but at the same time also allow some shading to avoid overheating. Sound insulation protects the residents from disturbing sound emissions, both among themselves and from outside. The building envelope, with its insulation, sealing and openings, is responsible for ensuring the necessary comfort in terms of construction. In combination with the building technology, the comfort should be optimised.

Comfort is of central importance for any building in which people live or work in. This also applies to the Tree of Life and the Maun Science Park. In the context of this elaboration, however, it will not be discussed further, since only the primary load-bearing capacity will be investigated. Nevertheless, attention should be paid to this in the further planning of the project.

[98] Refer to Friedrichsen 2018, p. 245
[99] Refer to Friedrichsen 2018, p. 239
[100] Refer to Hill and Bowen 1997, p. 227
[101] Refer to Friedrichsen 2018, p. 255

3.3.2 Flexibility of the construction

A flexible design is essential for a sustainable construction. A lack of flexibility in architecture means that buildings cannot adapt to the spirit of the times. They will become obsolete and are therefore no longer used. [102] Flexibility refers to the wholeness of the building. This means flexibility of use as well as flexibility of floor plan, flexibility of vertical arrangement, flexibility of the facade or flexibility of the building technology. Therefore, the flexibility of the supporting structure is of central importance. Due to the long lifecycle, the components have to meet constantly changing usage requirements and at the same time guarantee their main function, the load transfer. In order to achieve flexibility, attention should be paid to flexible, constant axis. Rooms of the same size enable simple configuration by connecting neighbouring rooms, while the floor height should be chosen in such a way that different uses are possible.[103] In general skeleton systems allow much greater freedom than shear wall systems, refer to chapter 2. The facade is the link between inside and outside. Thus, in addition to its original function, it must also be able to react flexible to the adaptations of the use. A small-scale facade structure is advantageous for this purpose. Furthermore, a facade that takes over the functions of the supporting structure loses considerable flexibility.

The flexibility of the construction plays a major role for the MSP. On the one hand, the use of the buildings is not planned down to the last detail and on the other hand, the project is intended to become a blueprint for the region and the whole of Africa. It must therefore be possible to realize the structures at different locations with only minor changes. In addition, the height should also be flexible in order to realize different numbers of floors, as mentioned in the project description. Finally, the first architectural design shows a shoebox design, compare to the Tree of Life (Figure 1), matching the housing tradition of the population, which is based on constant expansion of the living space according to the need. In addition to residential use, the building should also provide opportunities for urban farming, laboratories or training rooms. This results in a flexible mix of uses that should be considered.

3.3.3 Local architecture

Local architecture describes the aspect of the integration of the building into the urban and cultural context. Only if this is taken into account a construction project can be designed sustainably. Local architecture is crucial for the acceptance and use among the population as well as for the development of the surrounding region. The urban and cultural context are both strongly based on traditional building architecture. Which itself is closely linked to local, indigenous construction methods.[104] Indigenous construction methods, consisting of

[102] Refer to Keikut and Geier 2019, p. 32-34
[103] Refer to Friedrichsen 2018, p. 246
[104] Refer to Markovic n.d., pp. 1-2

traditional and local experience, were always applied in the past, but are now often replaced by modern, industrial methods. In some cases, however, local methods are still better suited to the existing conditions, such as temperature and weather. Moreover, the traditional building architecture represents a valuable cultural heritage. Awareness of this ensures the sustainable survival of knowledge, strengthens and connects the community. In this context, it is a mistake to try to impose modern building methods. Rather, it is important to take the knowledge of the past and adapt it to contemporary challenges. In addition to the traditional building architecture, the urban architecture must be considered as well for sustainable construction. A building should always be planned in the context of its surroundings.[105] It must fit in with the neighbourhood and the proportions must be coherent. Beside the visual effect, the social environment also plays a decisive role. The coexistence of living and working plays an increasingly important role. At the same time, green spaces provide the necessary balance and a diverse mix of residents ensures social peace. The attractiveness of the residential environment ensures a high level of identification with the building. In this way a long-term use can be ensured, and the desolation of new urban districts can be avoided. Another important aspect is the increasing urbanization of cities. As the world's population continues to grow, people are more and more moving to cities in search of work, housing and social interaction. As a result, the consolidation of cities is becoming increasingly important. On the contrary, urban sprawl leads to unnecessary impacts on the natural environment, which is not in the spirit of sustainability.

In summary, however, the most important thing about sustainable and local architecture is to involve the stakeholders. Only with a target group-oriented architecture a sustainably used and regional accepted building can be constructed.

With the MSP, a completely new urban district is to be created with the idea of sustainable living, working and research. In this overall approach, the sustainability of the social environment is already strongly included. Nevertheless, the visual aspects of the architecture have to be considered. The local architecture in Botswana, especially in the rural areas, is still characterized by traditional round huts, see Figure 17.[106] They consist of just one floor with a wooden skeleton construction. The supports are fixed in the ground and carry the wooden beams for the thatched roof, while the facade is a wall of sun-dried mud bricks with a ventilation opening under the roof. Of course, the Tree of Life cannot claim to completely copy the traditional construction method. The structural requirements alone are far too different for that. In addition, many people in the population also consider this construction method to be primitive and regressive. But in the meantime, attempts are already being made to adopt

[105] Refer to Friedrichsen 2018, pp. 239-241
[106] Refer to Markovic n.d., pp. 3-5

modern materials and techniques while at the same time preserving some traditional elements. In this way, a high level of acceptance among the population can be achieved while at the same time the cultural heritage can be preserved and the stability will be increased. Furthermore, the scale and proportions play an important role for the urban architecture in Maun. The city of Maun consists of sprawling, predominantly single-storey buildings on a flat plain. The construction of mega skyscrapers with more than a hundred meters would therefore be totally out of scale. The design of the Tree of Life, with 5-8 floors, is therefore a good solution to counteract the urban sprawl and at the same time not to blow up the scale. But tall buildings contradict the usual and accustomed way of living in Maun and are therefore not necessarily target group oriented. However, this break is necessary to deal with the increasing urbanization and thus the land consumption in Maun.

Figure 17 - Traditional round huts in Botswana, semi-finished and finished[107]

3.3.4 Local materials and services

The existing architecture and construction techniques used to be strongly related to the locally available building materials and services. This changed with industrialization. Since then, it is possible to import materials and outsource services. As a result, regions become economically desolate despite construction activities. But as already mentioned, the use of local materials and services can therefore have an enormous impact on the sustainable development of a region and thus on the sustainability of a project. Sourcing materials and employment locally supports the regional economy. Imports can be reduced, and exports may even be achieved. As a result, value creation and thus capital remains in the region, which can improve the quality of life for the entire population.[108] In addition to the macroeconomic aspects, local procurement is also reflected in the carbon footprint. Short transport distances have a direct impact on environmental sustainability.

[107] Markovic n.d., p. 2
[108] Refer to Markovic, pp. 1-2

The classic building materials in Botswana are, according to the traditional building methods, wood, mud and thatch.[109] Wood is mainly used in the form of naturally grown branches. While mud can be processed in the form of bricks or as rammed earth. Traditionally, water, earth and cow dung are mixed for this purpose, thatch is used for roofing. These building materials are hardly suitable in this way for the construction of a tall building structure. The most modern building materials and components are therefore usually imported from neighbouring countries, of South Africa, Namibia, Zimbabwe and Zambia. This is not particularly sustainable, but the construction industry in Botswana is not yet self-sufficient. [110] In the cement industry, only 1.1% of the materials used are local.[111] This is due to the fact that Botswana does not have adequate quality limestone for cement production. By contrast, sufficient raw materials are available for the glass and ceramics industry, so that nothing stands in the way of their production. Sufficient raw materials are also available for the production of masonry units. But the problem is that many of the bricks produced do not meet the requirements of the Botswana Bureau of Standards (BOBS). One reason for this is the use of Kalahari sand, which covers more than 75% of the country.[112] This sand is very fine with little silt and coarse materials. Thus, it has a low cohesion and poor strength. It is therefore unsuitable as construction sand. In addition to the masonry block, this also affects the concrete aggregate. The crushed rocks for the aggregates are mainly produced in the east of the country. This is another problem as the materials that are available are mainly found in the east and southeast of Botswana. This results in long transport routes regarding Maun. Also, the construction timber for Botswana has to be imported from neighbouring countries.[113] An alternative to wood from evergreen trees is the use of bamboo, where Africa provides good conditions for growth.[114] However, this building material is not yet well researched. Procuring local materials for the MSP and the Tree of Life is therefore a challenge. However, with a view to sustainability, individual solutions can be found. For local services, in addition to the existing ones, there is the possibility of training and start-ups. This should be made possible by the sustainable upswing that the project will bring.

[109] Refer to Markovic, p. 4
[110] Refer to Malumbela and Masuku, pp. 108-113
[111] Refer to Malumbela and Masuku, p. 108
[112] Refer to Malumbela and Masuku, p. 111
[113] Refer to Marais 2020
[114] Refer to Krötsch 2020

4 Structural designs

In this chapter now, different structural designs for the Tree of Life are worked out. The designs are developed from the previously determined basics in chapter 2 and 3. At the beginning, various assumptions and requirements have to be defined before four designs can be drawn. In the subsequent chapter 5, the designs are then evaluated on the basis of the requirements in order to find a preferred variant for the implementation of the Tree of Life.

4.1 Design assumptions and requirements

Since the first architectural design is just a sketch, various assumptions have to be made for the design. This concerns in particular the dimensions and cubature as well as the load assumptions which are important for a pre-dimensioning. In addition, requirements for the design are defined which are mainly derived from the basics discussed so far. They form the criteria for the evaluation of the designs. A special focus is on the life cycle assessment which is used to investigate the ecological sustainability.

4.1.1 Assumptions

For the general cubature, a simple cuboid with edge lengths of 18x18x30m is chosen. The construction of the various designs shall be within this box. In this way, all designs have the same general conditions and become comparable. The goal is to design a primary support structure to resist lateral and horizontal loads. The foundations are explicitly not included in the design. Their construction is highly dependent on the location and the existing subsoil conditions. Since both are unknown, the foundation design is not included. The design only examines the maximum number of storeys. A building height of 30m and 8 floors results in a floor height of 3.75m. This is generously dimensioned but must also include the ceiling structure and allow for various uses. The edge lengths of 18x18m result in a gross floor area of 324m². After taking into account an assumed construction area of 10%, a public traffic area of 12% and a technical area of 3%, a usable floor area of 243m² (75%) remains. This allows for example, two large families apartments per floor with 121.5m². Reference values for the space requirements of new buildings can be found, for example in the Building Cost Information Center of the German Chamber of Architects.[115]

In addition to the assumptions for the cubature, load assumptions are required for the pre-dimensioning of the components. The vertical surface load is assumed to be characteristic 5kN/m² in the form of the ceiling load. This includes the live load, a movable wall surcharge and the dead weight of a wooden ceiling including the floor build-up. Compared to a concrete

[115] Baukosteninformationszentrum Deutscher Architektenkammern 2020

ceiling, this is significantly lighter, which makes it more efficient and therefore more sustainable. A wooden ceiling is therefore specified as a ceiling construction. The dead weight of the remaining structure is considered in the pre-dimensioning. As horizontal design load, a wind load is applied. In general wind loads are determined using the gust wind speed and gust velocity pressure. Since no data is available for Botswana, gust speeds from South Africa are used. For the district of Ramotshere Moiloa (NW14), adjacent to Botswana's capital Gaborone, the gust wind speed is given with 36,0m/s.[116] This is used to determine the velocity pressure. In a simplified way, the gust velocity pressure is assumed to be constant over the height. With the help of pressure coefficients, the wind load can subsequently be defined. The resulting vertical load q_k=5kN/m² and the wind forces $w_{p,k}$=0,648kN/m² and $w_{t,k}$=0,405kN/m² represent characteristic values. For the pre-dimensioning, partial safety factors must be taken into account in order to obtain design values, however, combination factors are not used.

Vertical load:		**Wind load:**		
Life load:	$q_Q = 2{,}0\text{kN/m}^2$	Gust wind speed:	$v_p = 36{,}0\,\frac{m}{s}$	
Movable wall:	$q_{Q,2} = 1{,}2\text{kN/m}^2$	Gust velocity pressure:	$q_p = \frac{1}{2} * \rho * {v_p}^2 * 10^{-3}$	
Dead load:	$q_G = 1{,}8\text{kN/m}^2$	with $\rho = 1{,}25\text{kg/m}^3$	$q_p = 0{,}81\text{kN/m}^2$	
Total:	$\sum q_k = 5\text{kN/m}^2$	Wind load on surfaces:	$w_k = q_p(z) * c_{pe}$	
		Pressure windward:	$c_{pe} = 0{,}8$	$w_{p,k} = 0{,}648\,\frac{kN}{m^2}$
		Uplift leeward:	$c_{pe} = -0{,}5$	$w_{t,k} = 0{,}405\,\frac{kN}{m^2}$

4.1.2 Requirements

The requirements for the designs refer on the one hand to the structural requirements of a supporting structure and on the other hand to the required sustainability of the construction. As explained in chapter 3, there are meanwhile numerous sustainability labels in which different criteria have been established. The labels commonly used in Germany, DNGB[117] and BNB[118], define various sub-criteria for the five areas of sustainability, ecological quality, economic quality, social quality, technical quality and process quality, as well as for the quality of the location. But these requirements are not suitable for the planned designs, as they require a much higher level of detail and are not specific to this project. In addition, they evaluate a whole building and not just the primary structure. For this reason, individual requirements for sustainability are derived based on the knowledge worked out in chapter 3. The structural requirements result from the building support structure presented in chapter 2. Furthermore,

[116] Refer to Kruger et al. 2017, p. 23
[117] DGNB GmbH 2021
[118] Bewertungssystem Nachhaltiges Bauen (BNB) 2021

they also describe requirements for the construction and installation. The requirements of the designs can thus be divided into three categories. On the one hand, static and structural requirements and on the other hand, the requirements of sustainability in ecological and social terms.

The static requirements correspond to the typical structural engineering verifications, ultimate limit state (ULS) and serviceability limit state (SLS). The ULS verifies, among other things, the load-bearing capacity and stability of the structure. It must therefore be fulfilled 100% to ensure the safety of the structure. The SLS guarantees the limitation of deformations. On the one hand, these are decisive for the well-being of the users; on the other hand, they also have an impact on the load-bearing components. The deformations thus allow conclusions to be drawn about the stiffness of the structure. In the designs, the deformations serve as a quality control and a similar deformation of all structures is aimed for. The ULS and the SLS are important requirements for the design. They are fulfilled by means of pre-dimensioning and are therefore not criteria that will be evaluated. The construction requirements evaluate the complexity of the design in terms of its feasibility. They are divided into three criteria. The first criterion evaluates the degree of prefabrication and thus the effort required to install the building. This is decisive for a fast construction process. The need for equipment and manpower is considered here as well. Another criterion evaluates the complexity of the details. For example, rigid connections are far more complex than hinged ones. The last criterion evaluates the need for expertise. Some projects require a lot of expert knowledge in planning and construction, while for other projects unskilled labourer are sufficient. Many different trades also increase the need for expertise. The description of the construction requirements should enable the simplest possible construction. A high degree of complexity not only increases the effort required, but also the costs and the risk of faults. However, in contrast to the ULS and SLS, they represent soft criteria.

The requirements for environmental sustainability, as indicated in chapter 3.1, can best be assessed by means of a life cycle assessment (LCA). This can be used to evaluate both the efficiency of the building design, in terms of resource consumption, and the environmental compatibility of the materials. The LCA allows the comparison of the designs with data and facts and is therefore an important tool. The certification of the DNGB and the BNB also use a LCA for the evaluation of environmental sustainability. For this reason, the principles of a LCA are explained separately and in detail in chapter 4.1.3. The framework of the LCA for the designs is outlined and the requirements are defined in the form of indicators.

Table 1 - Design requirements

Category	Requirements	Description
Static and Constrution	Ultimate limit state (ULS)	The verification of the ULS is mandatory for the safety of the construction and must be fulfilled to 100%. For this purpose, the components are estimated by pre-dimensioning.
	Serviceability limit state (SLS)	The SLS mainly restricts the deformations. The deformations serve as reference parameter for stiffnesses of the designs.
	Prefabrication and installation effort	The degree of prefabrication has a high influence on the construction speed. In addition, the amount of construction equipment and manpower required for installation is assessed.
	Complexity of details	Complex details, such as rigid connections, result in a significantly higher construction effort. They should therefore be as simple as possible.
	Need for expertise	A high demand for expertise, as a result of a complex design, can lead to higher costs and greater risk of errors.
Ecological sustainability/ Life cycle assessment	Total renewable primary energy (PERT)	PERT describes the renewable energy demand for material use as well as for energy sources. It is determined via the heating energy [MJ].
	Total non-renewable primary energy (PENRT)	PENRT describes the non-renewable energy demand for material use as well as for energy sources. It is determined via the heating energy [MJ].
	Global warming potential (GWP)	GWP describes the global influence on climate change. It is given in the unit kg CO2 -Eq.
	Ozone depletion potential (ODP)	ODP describes the global destruction of the ozone layer in the stratosphere. It is given in the unit kg CFC-11 -Eq.
	Photochemical ozone creation potential (POCP)	POCP describes the formation of local smog near the earth's surface. It is given in the unit kg C2H4 -Eq.
	Acidification potential (AP)	AP describes regional acidification of soils and waters. It is given in the unit kg SO2 -Eq.
	Eutrophication potential (EP)	EP describes regional over fertilization of soils and waters. It is given in the unit kg PO4^3 -Eq.
Social sustainability	Flexibility	Flexibility means the possibility to change the utilization, the floor plan or the vertical arrangement over the life cycle of the building.
	Local architecture	Local architecture means to respecting traditional constuction elements, integrating the building into the site and landscape and considering the needs of the user group.
	Local materials and services	Where possible, locally mined and processed materials should be used. In addition, contractors should be chosen locally.

Some aspects of the social sustainability were described in detail in chapter 3.3. The flexibility, the architectural quality and the use of local resources are directly related to the primary support system studied. They are therefore adopted as requirements for the design as described. However, these requirements are difficult to compare with data. The evaluation of these soft factors must therefore be performed subjectively, on the basis of a detailed study of the project.

Table 1 shows a summary of the requirements or criteria with a brief description.

4.1.3 Life cycle assessment

As already mentioned in chapter 3.1.2, a LCA is an effective method for evaluating the potential environmental impact of buildings and building components. It is therefore also used in this thesis to compare the designs in terms of their environmental sustainability. The purpose of this chapter is to explain the LCA in detail and set up the project specific LCA.

A LCA enables the determination of material and energy flows of a product system, e.g. a building, over its entire life cycle, as well as the allocation of the resulting input and output flows to potential environmental impacts. It is important to note that a LCA is a relative, goal-oriented method with unavoidable uncertainties. In this sense, it is not suitable for predicting exact or absolute environmental impacts. Rather, it serves as a tool for planners in decision making. LCA's of buildings are well suited for the comparison of different variations of materials or building components. In principle, the entire life cycle of a building can be considered, but it is also possible to examine just certain phases for a specific LCA. At the material level, the focus is on the manufacturing and disposal phases. While at the building level, the interaction of building components and building technology with regard to energy requirements is important. The approach of the building LCA is explained in detail in DIN EN 15978 and is based on the LCA method according to the DIN EN ISO 14040 and the DIN EN ISO 14044. They divide the LCA into 4 stages, see Figure 18: Definition of the goal and scope of assessment, life cycle inventory (LCI), life cycle impact assessment (LCIA) and interpretation.[119]

Definition of the goal and the scope of the assessment

The definition of the goal helps to formulate the specific question to be answered with the help of the LCA.[120] These can be very diverse. For example, optimizing the choice of building materials in terms of durability or which phase of the building life cycle causes the greatest environmental impact. Based on the objective, the system boundaries of the assessment are

[119] Refer to El khouli et al. 2014, pp.23- 24
[120] Refer to El khouli et al. 2014, p. 24

defined. These define which components and processes are considered in the assessment.[121] In particular, the definition life cycle phases, which are taken into account is important here. The material flows are ideally mapped over the entire life cycle. This corresponds to the cradle-to-cradle concept, including reuse. However, a cradle-to-grave or cradle-to-gate concept is also possible.[122]

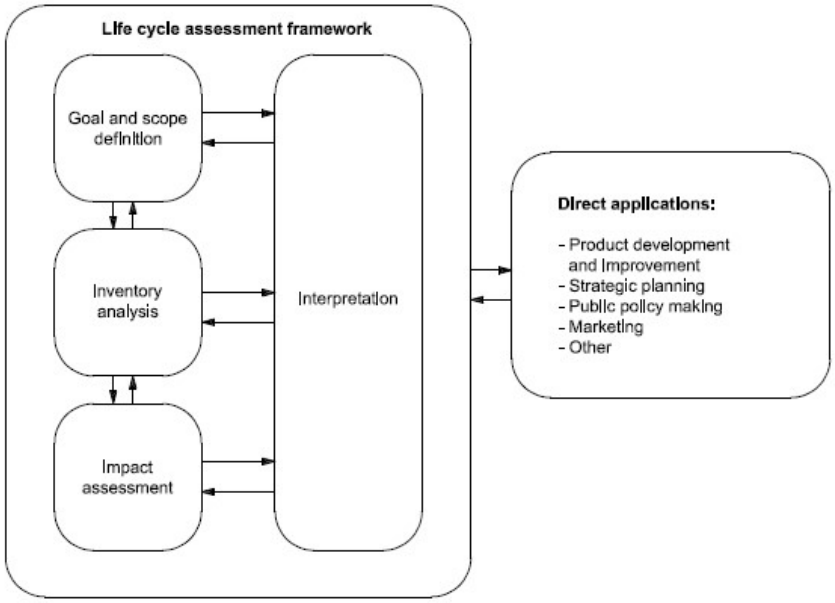


Figure 18 - Stages of a life cycle assessment[123]

In addition to the boundary conditions, the functional unit must also be defined. It defines the specific functions that a product system has to fulfil during its lifetime.[124] This standardizes the LCA both quantitatively and qualitatively towards a specific outcome. For example, it only makes sense to compare components that fulfil the same qualitative function. The functional unit is therefore the unit to which all input and output flows of the LCI and the results of the LCIA refer. Another important information for the scope of the study is the selection of the data source.[125] Data regarding the materials and energy processes used are needed for both the LCI and the LCIA. They can be obtained from professional databases or free accessible environmental product declarations. However, it is important that the data are of high quality, characterized by transparency and traceability. Once a data source has been found, further assumptions have to be made, especially with regard to material quantity and durability.[126] For a feasibility study, this can be done with the help of a pre-dimensioning. Last but not least, the impact categories and indicators for the further investigation must be determined in the

[121] Refer to El khouli et al. 2014, p. 25
[122] Refer to El khouli et al. 2014, p. 45
[123] DIN EN ISO 14040:2006-10, p. 17
[124] Refer to El khouli et al. 2014, pp. 27-28
[125] Refer to El khouli et al. 2014, p. 28
[126] Refer to El khouli at al. 2014, p. 29

first LCA phase. The impact categories describe the various environmental impacts, while the indicators represent quantifiable values for these impacts. The DIN EN 15978 lists several categories and indicators, the most relevant ones are explained below. Their harmful environmental impact is shown in Figure 19.

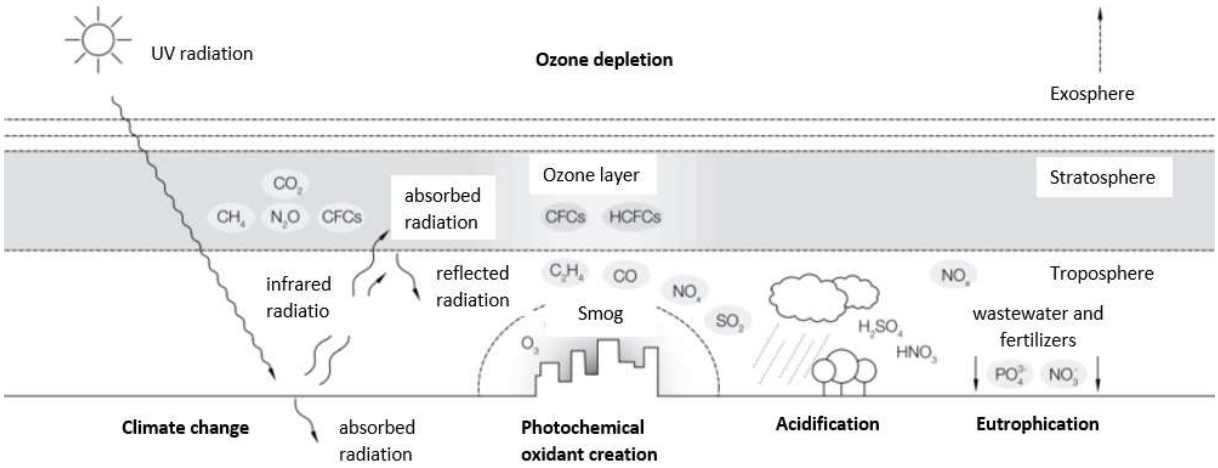


Figure 19 - Illustration of environmental impacts[127]

Impact categories and indicators[128]

The use of primary energy (PE) resources is an important indicator of resource consumption. It can be indicated with the help of the associated heating energy [MJ]. A distinction is made between renewable and non-renewable energy sources and between material use and energy sources. This results in four indicators: renewable primary energy as energy source (PERE), renewable primary energy for material use (PERM), non-renewable primary energy as energy source (PENRE) and non-renewable primary energy for material use (PENRM). Depending on the requirements, these can be cumulated to determine, for example, the total energy requirement. Further indicators of resource consumption relate to secondary materials and water consumption. The probably most important indicator of environmental impact is the global warming potential (GWP). It is directly related to climate change with the emission of greenhouse gases that affect the earth's atmosphere. Since there are a lot of greenhouse gases, CO_2 (carbon dioxide), the best-known greenhouse gas, is chosen as the reference unit. The effect of the other gases is converted to the effect of 1 kg CO_2 in the atmosphere. This is called CO_2-equivalent. Another category of environmental impact is stratospheric ozone depletion. The ozone layer filters out the sun's UV radiation. If this protection is lost, there is a risk of serious damage to human health. The indicator for this is the ozone depletion potential (ODP). It is measured in the unit kg CFC-11 (trichlorofluoromethane) equivalents. The photochemical ozone creation potential (POCP), on the other hand, is related to the effect of 1 kg of ethane

[127] Adapted from El khouli et al. 2014, p. 31
[128] Refer to El khouli et al. 2014, p. 30

(C_2H_4). Photooxidation refers to the formation of smog in the lower layers of the atmosphere. This consists of hazardous pollutants such as nitrogen oxides which, in high concentrations, endanger human health. The acidification potential (AP) of soil and water is determined using the kg SO_2 (sulphur dioxide) equivalent. It describes the damage to ecosystems as a result of acidification. Acidification results from the conversion of air pollutants to acid. The resulting acid rain leads to forest dieback and corrosive damage to building materials. Equally harmful to nature is the over fertilization of the soil. It is called eutrophication and significantly damages plants. The eutrophication potential (EP) is expressed as kg PO_2^3 (phosphate) equivalent. In addition to these impacts of input flows and emissions, output flows also contribute to the environment. These mainly include waste, but also reusable resource flows. In the context of this LCA, they are subordinate, so these indicators are not explained in detail.

Life Cycle Inventory (LCI)

In the LCI phase, all material flows of a product are recorded in detail over the entire life cycle.[129] Input flows represent resource and energy consumption, while waste and emissions are determined as output flows. Examples are crude oil demand, CO_2 or SO_2 emissions. In the LCI, the scope conditions defined in the first phase of the LCA must be applied. Accordingly, all material flows are related to the previously defined functional unit with the chosen database as data source. The stages of the life cycle which shall be considered also result from the scope conditions. The DIN EN 15978 divides the building life cycle into four stages. The product stage (A 1-3), the construction process stage (A 4-5), the use stage (B 1-7) and the end-of-life stage (C 1-4), see Figure 20. In additionally, a stage beyond the building life cycle is defined (D), which focuses mainly on the potential for recycling and recovery. All material flows can thus be assigned in detail to different phases and sub-phases over the life cycle. The result of the LCI is therefore a list of all input and output flows that identify and quantify the source of the environmental impact over the life cycle.

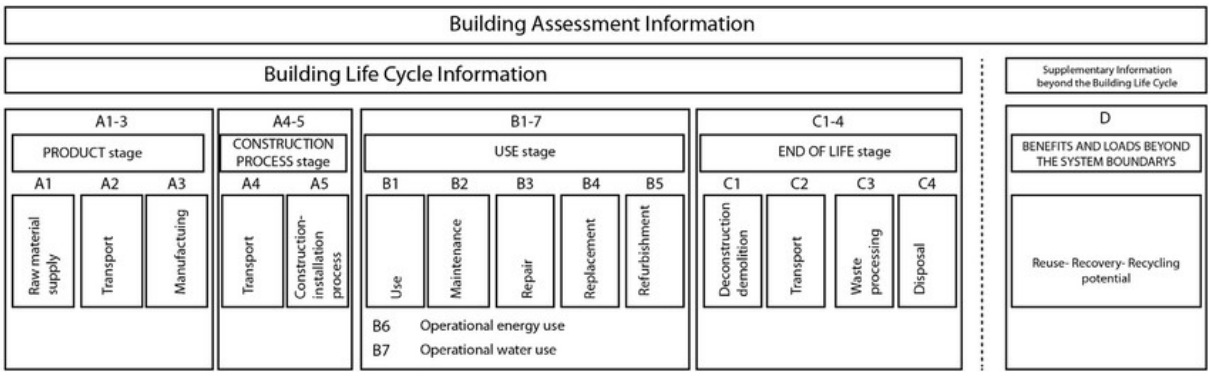


Figure 20 - Life cycle stages of the DIN EN 15978[130]

[129] Refer to El khouli et al. 2014, p. 32
[130] Adapted from DIN EN 15978:2012-10, p. 21

Life Cycle Impact Assessment (LCIA)

For the interpretation of the LCI phase, the LCIA is required. It assigns the input and output flows to specific environmental impacts.[131] For this purpose, the defined impact categories and indicators are needed. The assignment of the LCI outcome to the impact categories is referred to as classification. For example, the various greenhouse gases are assigned to the environmental impact climate change. Subsequently, the indicators of the impact category are calculated in the characterization. This is done by means of the explained reverence units. For example, methane is converted to kg CO_2 equivalent as global warming potential. With the help of the indicators, the result of the impact assessment can be used for the evaluation.

Interpretation

In the evaluation phase, the original question and objective are answered. For this purpose, the results of the life cycle inventory and the life cycle impact assessment are analysed, interpreted and compared.[132]

Project specific LCA

In the following, the project-specific framework for the LCA of the structural designs for the Tree of Life is presented. It corresponds to the explanation just given. The LCA for the individual designs themselves is presented in the associated chapters (4.2.3, 4.2.34.3.3, 4.4.3, 4.5.3). For this purpose, the building mass is determined based on the pre-dimensioning. In a next step, using the ÖKOBAUDAT[133] database and the associated reference flow, the LCI and the LCIA of the structural systems are set up. However, the impact indicators are only presented in total for the entire life cycle and are not assigned to the individual stages. The evaluation of the LCA is done in the context of the evaluation of the designs.

Life cycle assessment structural system Tree of Life:

Question:	Which structure performs best in terms of its environmental impact?
Goal:	To evaluate and compare the environmental sustainability of the analysed structures.
System boundaries:	The system includes all components of the primary structure that are used for load transfer. Explicitly excluded are the foundations. The entire life cycle of the building, according to DIN EN 15804, is considered, from product stag to the end-of-life stage. The additional

[131] Refer to El khouli et al. 2014, p. 34
[132] Refer to El khouli et al. 2014, p. 35
[133] ÖKOBAUDAT Informationsportal Nachhaltiges Bauen 2021

	reuse and recycling stage is also considered. Anyway, the utilization phase plays a subordinate role, since the energy demand during operation is not included in the LCA. Cause the primary structure is analysed, it is assumed that the lifetime of all involved components is equal to the lifetime of the whole building.
Data source:	The free online database ÖKOBAUDAT is used as data source. It has a high, verified data quality which is DIN EN 15804 compliant. Data sets for building materials, construction, transport, energy and disposal processes are available. These consist of generic, as well as company and association specific data. The ÖKOBAUDAT is the mandatory database for the German Sustainable Building Assessment System (BNB) and is also used internationally. It should be noted, however, that for Botswana no explicit data, e.g. for transport routes or energy mix, are available.
Assumptions:	The required material masses are determined by means of the component pre-dimensioning.
Impact categories:	Total renewable primary energy (PERT), Total non-renewable primary energy (PENRT), Global warming potential (GWP), Ozone depletion potential (ODP), Photochemical ozone creation potential (POCP), Acidification potential (AP), Eutrophication potential (EP)
LCI:	The material flows are recorded in detail in the ÖKOBAUDAT database. They refer to a reference flow of 1m³ or 1kg of material and are directly assigned to the life cycle phases of DIN EN 15804. Process information are provided for each data set and can be consulted.
LCIA:	The impact assessment is integrated in the database as well. For the reference flow, the environmental impacts are represented by means of the equivalents.
Interpretation:	The interpretation is provided in chapter 5.1.

4.2 Design 1: Concrete core structure

This first design is based on a core structure, refer to chapter 2.5.3. In order, to prevent complicated cantilevering of the slabs, pendulum columns support the vertical load transfer. Figure 21 shows the static system, while Figure 22 shows a sketch for rendering.

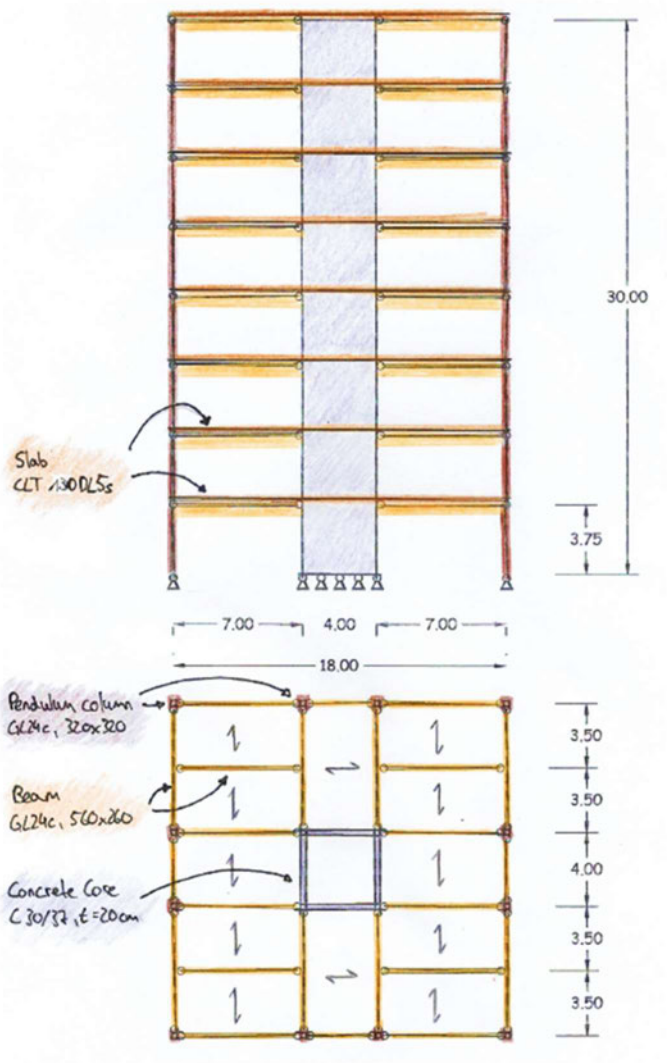


Figure 21 - Design 1: Structural system

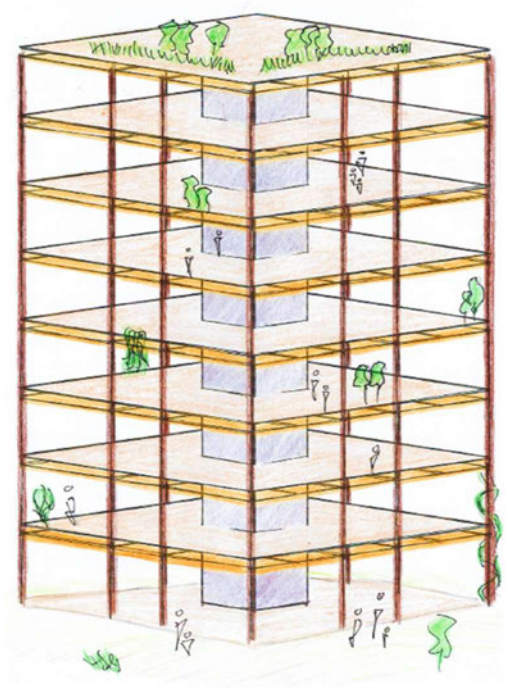


Figure 22 - Design 1: Rendering

4.2.1 Construction

The core structure is made of concrete and provides the stiffening of the building. Its dimensions are approximately 4x4m to include both stairwell and lifts. Typically, it is therefore made as a partially closed core to ensure accessibility. The central arrangement of the core allows a symmetrical architecture and avoids unwanted torsional stresses. To enable the 7m span of the slabs, additional pendulum columns are added. In contrast to a solution with continuous perimeter columns, they are only the height of the floors. Although this leads to slightly larger deformations, it allows a simpler construction and handling of the columns.

After the concrete core has been built, the floors with pendulum columns and beams can be constructed in segments. The beams support the slab construction by spanning from the core to the perimeter columns. In addition, spandrel beams connect the columns to each other. The columns and beams are made of glued laminated timber (glulam). This is more sustainable than the use of steel or concrete and at the same time allows the load bearing. For the same reason, wooden slab constructions are preferred to reinforced concrete slabs. In this design, a cross laminated timber (CLT) ceiling is chosen, which spans uniaxial. CLT is a flat, solid wooden product for load-bearing applications. It consists of at least three layers of wooden panels, glued together at right angles to each other. [134] Thanks to that planar gluing of the layers, the component has a useful shear stiffness and can provide the necessary diaphragm action of the slabs.

The total height of the building is determined by the height of the core. However, the structural system can also be modified to 4-10 floors without significant changes and thus offers a great flexibility. It would also be imaginable to build the core to the final height, but not to construct all the floors immediately. This would maintain flexibility in terms of height but requires additional measures in the building closure, especially in the sealing. The floor plan also offers a high degree of flexibility around the core. Apartments, greenery or other elements can be arranged in a modular way. This enables all kinds of combinations and conversions. Vertical openings, for additional freedom, can also be created if the stiffening of the ceiling is sufficient. Another advantage is the open facade. It allows interaction with the environment and thus picks up on the architectural features of a tree. Furthermore, the construction with a solid core and wooden supports is strongly similar to the traditional round hut architecture.

The hybrid construction of concrete and wood is a compromise between sustainability and load-bearing capacity. Concrete has a very poor carbon footprint but provides the necessary rigidity of the core. Due to the low structural depth of the core, an alternative more sustainable construction of rammed earth is hardly imaginable here. Wood as a building material is very sustainable, but sourcing glulam in Botswana is challenging, see chapter 3.3.4. For a project of this size, however, natural wood as a building material is not an option. In timber construction, in addition to the structural requirements, special attention must be paid to fire protection. For the secondary, modular elements, local building materials such as clay can also be used. This creates a deeper connection between site and construction.

4.2.2 Pre-dimensioning

For pre-dimensioning, the components are estimated using simple structural systems. The vertical load is transferred exclusively from the concrete core since it provides the complete

[134] Refer to Wallner-Novak et al. 2013, p. 8

stiffening. The horizontal loads, however, can be distributed to all vertical load-bearing elements, columns and core, with the help of load catchment areas. The beams only distribute the vertical loads, while the slab construction additionally transfers the horizontal load from the facade to the core. The loads correspond to the load assumptions specified in chapter 4.1.1 including safety factors. Due to some simplifications within the framework of the pre-dimensioning, a hundred percent utilization of the components is not aimed for.

Loads:		**Safety factors:**	
Wind load:		Wind load:	$\gamma_W = 1{,}5$
Pressure windward:	$w_{p,k} = 0{,}648 \frac{kN}{m^2}$	Life load:	$\gamma_Q = 1{,}5$
Uplift leeward:	$w_{t,k} = 0{,}405 \frac{kN}{m^2}$		
Life load:	$q_{,k} = 5{,}0 \frac{kN}{m^2}$		

Core – reinforced concrete

The core can be analysed as a simple cantilever. For the cross-section, the concrete walls are modelled using a squared hollow box cross-section. The height and width of the cross-section corresponds to the core length of 4m, while the wall and thus the web thickness is 20cm. The cantilever is loaded with the complete wind load, pressure and uplift, to determine the required reinforcement. The live load and the dead weight are not considered in this model. It is assumed that the resulting compressive stresses can be absorbed by the concrete without any problems.

Standard:	DIN EN 1992-1-1	**Load:**	
Cross-section:		Wind load:	$w_d = (w_{t,k} + w_{p,k}) * b * \gamma_W$
h/b/t [mm]:	4000/4000/200		$w_d = 28{,}431 \frac{kN}{m}$
Sectional area:	$A = 3{,}04m^2$	**Internal Force:**	
Section modulus:	$W = 3{,}368m^3$	Clamping Moment:	$M_{w,d} = w_d * \frac{h^2}{2}$
Material:			$M_{w,d} = 12794{,}0$ kNm
Concrete C30/37:	$f_{cd} = 17{,}0 \frac{MN}{m^2}$		
Steel B500A:	$f_{ywd} = 435 \frac{MN}{m^2}$		
System:			
Building width:	$b = 18m$		
Building hight:	$h = 30m$		

Dimensioning:

Stress: $\sigma = \frac{M_{w,d}}{W} = 3{,}8\,\frac{MN}{m^2}$ Membrane stress: $n_x = \sigma * t = 0{,}76\,\frac{MN}{m}$

Reinforcement: $a_{sw} = \frac{n_x}{f_{ywd}} * \frac{10000cm^2}{1m^2} = 17{,}47\,\frac{cm^2}{m}$ → Ø16 | 10cm, $a_{sw} = 20{,}11\,\frac{cm^2}{m}$

Reinforcement level: vertical $\rho_v = \frac{a_{sw}}{20cm*100cm} = 0{,}01$

horizontal $\rho_h = \rho_v * 0{,}2 = 0{,}002$

Compressive strength: $\sigma = 3{,}8\,\frac{MN}{m^2} \ll f_{cd} = 17{,}0\,\frac{MN}{m^2}$

Pendulum columns - glulam

For the compression-loaded pendulum columns, the buckling check is decisive. Accordingly, the 2nd Euler case can be chosen for the static system. The load is determined from the floor plan by means of the catchment areas. This results in a maximum area of 3.5x3.75m. The load is accumulated over all 8 storeys. Simplified, the pendulum supports are only loaded with the live load. The dead weight is much lower so that it can be ignored for pre-dimensioning. A square cross-section of glulam is analysed.

Standard: DIN EN 1995-1-1

Cross-section:

h/b [mm]: 320/320mm

Sectional area: $A = 1024{,}0cm^2$

Moment of inertia: $I = 87381{,}3cm^4$

Radius of gyration: $i = 9{,}24cm$

Material:

Glulam GL24h: $f_{c,0,d} = 14{,}76\,\frac{N}{mm^2}$

$k_{mod} = 0{,}8, \gamma_M = 1{,}3$

System:

Catchment area: $a_{max} = 19{,}25m^2$

Number of floors: $n = 8$

Load:

Life load: $q_{,d} = q_{,k} * \gamma_Q = 7{,}5\,\frac{kN}{m^2}$

Internal Force:

Normal force: $N_{q,d} = a_{max} * q_{,d} * n$

$N_{q,d} = 1155kN$

Dimensioning

Buckling length: $l_{ef} = 375cm$

Slenderness: $\lambda = l_{ef}/i = 40{,}6$

Coefficient: $k_c = 0{,}953$

Buckling: $\frac{N_{q,d}/A}{k_c * f_{c,0,d}} = 0{,}80 \leq 1$

Beams - glulam

The beams are designed as single span beams with a span of 7m each. They distribute the life loads to the pendulum columns and the core. The maximum catchment width results from the floor plan and can be used to determine the line load. The dead weight of the beam is neglected. Axial loads are not applied to the beams, which is why a pure bending check is crucial. Tilting is prevented by the attached slab, which serves to secure the position of beam. The material chosen is glulam with a simple rectangular cross-section.

Standard: DIN EN 1995-1-1	**Load:**
Cross-section:	Life load: $q_{,d} = q_{,k} * a * \gamma_Q = 28{,}125 \frac{kN}{m}$
h/b [mm]: 560/260	**Internal Force:**
Sectional area: $A = 1456{,}0cm^2$	Bending moment: $M_d = \frac{q_{,k} * l^2}{8} = 172, kNm$
Section modulus: $W = 13549{,}3cm^3$	**Dimensioning:**
Material:	Bending: $\frac{M_d/W}{f_{m,d}} = 0{,}86 \leq 1$
Glulam GL24h: $f_{m,y,d} = 14{,}76 \frac{N}{mm^2}$	
$k_{mod} = 0{,}8, \gamma_M = 1{,}3$	
System:	
Span: $l = 7m$	
Catchment width: $a_{max} = 3{,}75m$	

Slab construction – Cross laminated timber (CLT)

As already mentioned, a CLT slab consists of at least three layers of wooden panels glued to each other at right angles. Usually, softwood C24 is used as basic material. It can be made both as a uniaxial and as a biaxial spanned slab. But the main load bearing direction (0°), corresponding to the direction of the deck layers, has a much higher stiffness than the secondary load bearing direction (90°).Thus, the uniaxially spanning slab of the design can be dimensioned using a simple bending check. Therefore, only the layer that run in the load-bearing direction are taken into account. The crossing layers are only considered as distance keepers. This results in net cross-section values with the index "n". The general load-bearing behaviour of a CLT panel is indicated in Figure 23. [135]

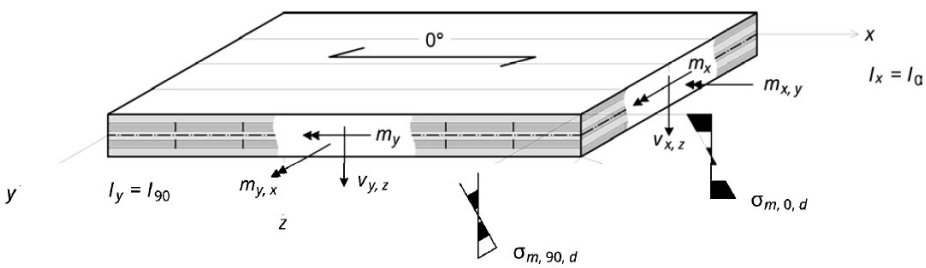


Figure 23 - Cross laminated timber slab, load transfer[136]

For the pre-dimensioning, the ceiling is examined as a single span beam with a reference width of 1m and a span of 4m. While the dead weight is included in the live load, the ceiling is additionally loaded axial as a result of wind pressure or uplift. Via the floor height, the wind loads on the slabs can be determined. Wind pressure is analysed, due to its higher amount. In addition to the bending analysis, shear, deformation, vibration and fire behaviour must also be investigated as part of the regular design. Therefore, only a utilization of approx. 50% is

[135] Refer to Wallner-Novak et al. 2013, pp. 8-12
[136] Wallner-Novak et al. 2013, p. 37

aimed for here. So far, CLT has not been included in the Eurocode. The pre-dimension is therefore realized to the guideline "Brettsperrholz Bemessung" published by proHolz Austria.[137]

Cross-Section:

CLT 130 DL5s [mm]: 30/20/30/20/30

Sectional area: $A = 1300{,}0\ \frac{cm^2}{m}$

$A_{0,n} = 900{,}0\ \frac{cm^2}{m}$

Moment of inertia: $I_{0,n} = 15675{,}0\ \frac{cm^4}{m}$

Section modulus: $W_{0,n} = 2411{,}54\ \frac{cm^3}{m}$

Material:

Panels C24: $f_{m,d} = 14{,}76\ \frac{N}{mm^2}$

$f_{c,0,d} = 12{,}92\ \frac{N}{mm^2}$

$k_{mod} = 0{,}8, \gamma_M = 1{,}3$

System:

Span: $l = 4m$

Floor hight: $h = 3{,}75m$

Load:

Life load: $q_{,d} = q_{,k} * \gamma_Q = 7{,}5kN/m^2$

Wind load: $W_d = w_{p,k} * h * \gamma_Q = 3{,}645\ \frac{kN}{m}$

Internal Force:

Bending moment: $M_d = \frac{q_{,d} * l^2}{8} = 18{,}98\ \frac{kNm}{m}$

Normal force: $N_{w,d} = W_d = 3{,}645\ \frac{kN}{m}$

Dimensioning:

Pressure and Bending: $\frac{N_d/A_n}{f_{c,0,d}} + \frac{M_d/W_n}{f_{m,d}} \leq 1$

$0{,}52 \leq 1$

4.2.3 Life cycle assessment

The LCA is based on the framework conditions as described in chapter 4.1.3. The mass is determined with the help of the pre-dimensioning and a simple model of the design. Table 2 shows the accumulated LCI and LCIA. The following datasets based on the ÖKOBAUDAT[138] database are used for this assessment: Beton der Druckfestigkeitsklasse C30/37, Bewehrungsstahl, Brettschichtholz - Standardformen (Durchschnitt DE) and Brettsperrholz (Durchschnitt DE). The results are compared with the LCA's of the other designs in chapter 5.1.2.

Table 2 - Design 1: LCI and LCIA

Component	Material	Section/ Thickness	Meter/ Surface	Volume	Mass	Comment
		[m²]/[m]	[m]/[m²]	[m³]	[kg]	
Core	C30/37	0,2000	480,00	96,00	230400,00	
Core-Steel	B500A			1,15	9043,20	Reinforcement level 1.2%
Colums	GL24c	0,1024	360,00	36,86	18690,05	
Beam	GL24c	0,1456	1248,00	181,71	92126,36	
Slab	CLT	0,1300	2592,00	336,96	164773,44	

[137] Refer to Wallner-Novak et al. 2013

[138] ÖKOBAUDAT Informationsportal Nachhaltiges Bauen 2021

Component	PERT	PENRT	GWP	ODP	POCP	AP	EP
	[MJ]	[MJ]	kg CO2	kg CFC-11	kg C2H4	kg SO2	kg PO4^3
Core	1,48E+04	1,13E+05	2,06E+04	5,72E-06	1,19E+00	3,42E+01	6,68E+00
Core-Steel	3,43E+04	8,00E+04	6,21E+03	1,89E-10	2,39E+00	1,17E+01	1,59E+00
Colums	3,42E+05	-1,04E+05	-5,91E+03	-3,51E-10	9,70E-02	1,13E+01	3,51E+00
Beam	1,68E+06	-5,11E+05	-2,91E+04	-1,73E-09	4,78E-01	5,58E+01	1,73E+01
Slab	2,85E+06	-9,57E+05	-5,70E+04	-2,92E-09	1,53E+01	5,80E+01	1,91E+01
TOTAL	**4,93E+06**	**-1,38E+06**	**-6,53E+04**	**5,72E-06**	**1,95E+01**	**1,71E+02**	**4,82E+01**

4.3 Design 2: Steel shear frame structure

The structure of this design can be described as a shear frame system. Four braced frames provide the transfer of the horizontal loads on each floor. The remaining columns can be designed as pendulum columns thanks to the stiff steel frames. Figure 24 shows the static system, while Figure 25 shows a rendering of the proposed design.

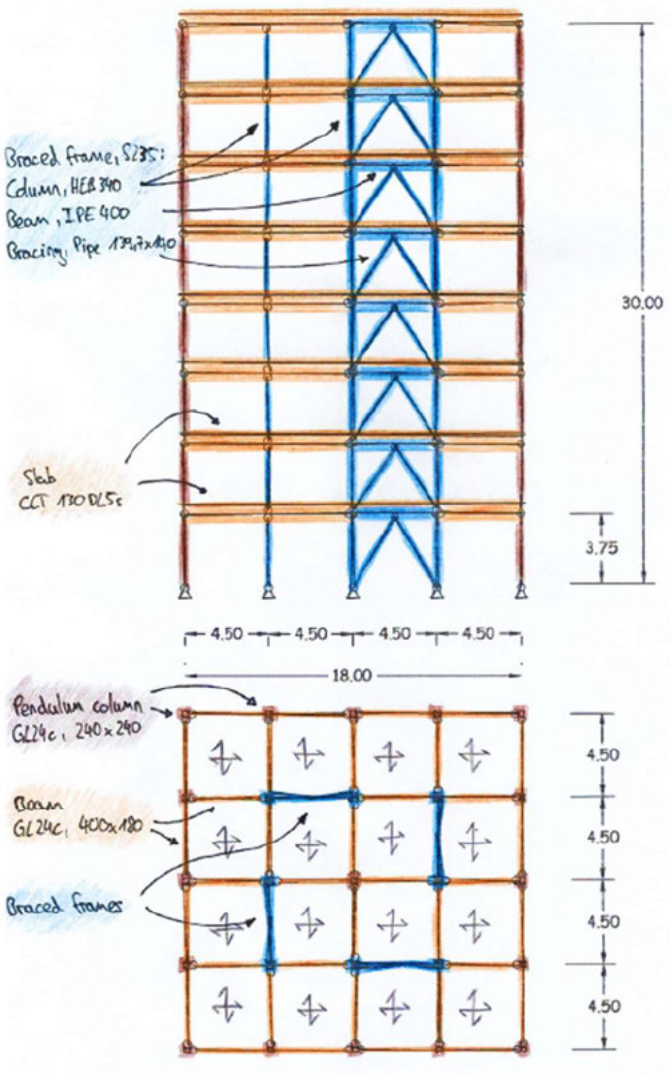


Figure 24 - Design 2: Structural system

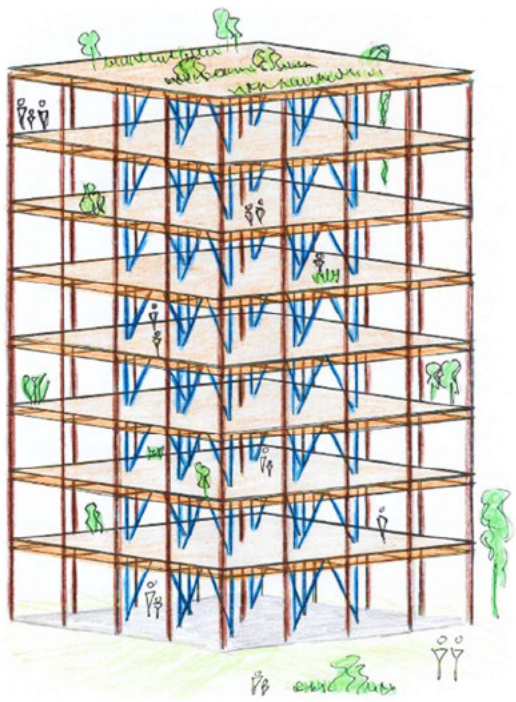


Figure 25 - Design 2: Rendering

4.3.1 Construction

Even if the name suggests it, the chosen system does not correspond exactly to the shear frame system presented in chapter 2.5.4. This design focuses on just one very stiff frame so there is no interaction between a rigid frame and a braced frame. Nevertheless, the existing stiffness is sufficient for the system. If greater stiffness is required, a secondary frame can be rigidly coupled to achieve interaction.

The columns and beams of the ridged frames are designed of steel profiles to withstand the additional loads caused by the bracing. At the same time, a steel structure simplifies the construction of rigid connections compared to a timber one. HEB profiles are chosen for the columns while the beams are made of IPE profiles due to the predominant moment load. For the bracing itself a K-bracing with tubular steel profiles is chosen. Compared to the X-bracing, this allows the integration of openings and if necessary, an eccentric arrangement for seismic loads. But the greatest advantage of K-bracing is the reduction of the buckling length. The bracings are arranged in the floor plan as a dissolved core. This allows a greater static depth than a classic core and at the same time does not obstruct the facade. The remaining pendulum columns and beams are made of glulam. The material glulam is chosen for the same reasons as in Design 1. It combines the properties of strength and sustainability and also permits the required cross-sections. The beams can be designed as single-span or double-span beams depending on their position. Continuous beams reduce the number of necessary beams and connections, the transport length is decisive for this. The beams create a uniform grid of 4.5x4.5m as a support structure for the slab system. The ceiling system consist of CLT elements, which biaxial span between the squared arranged beams. The concept of a cross laminated timber panel was explained in chapter 4.2.1. As an alternative to the shear stiff slab system, horizontal bracing can also be implemented at slab level. In this case, however, an additional plate construction must be planned as floor level and for vertical load distribution.

The design, in a form of a frame construction, offers the greatest possible freedom in terms of both floor plan and building height. The floors can be designed individually and in a modular way, while the architecture is only slightly restricted by the building grid. Within the open facade, balconies or terraces for greening and urban farming can be integrated. Although the rigid connections require complex details, they are necessary for the required stiffness of the construction. The total construction is nevertheless very simple. The bracing frames can be prefabricated in the factory and then only have to be installed on site. With the bracing in place, the other pendulum columns and beams can be erected. Thus, the building can be built floor by floor and the number of storeys is flexible.

As in Design 1, wood is used as a building material as much as possible, due to its sustainable properties. But the mentioned challenges for the procurement of glulam also occur here. The steel elements have a quite negative impact on the sustainability of the project. Anyway, they are necessary to ensure the stiffness. For both materials, factories for processing may be established in and around Maun as part of the MSP project. This way the population and the region can benefit from this project and a part of the social sustainability can be guaranteed.

4.3.2 Pre-dimensioning

For the design of the components, a simplified 2D model of a braced building axis is generated and analysed with the structural analysis software RFEM. Both wind loads and live loads are applied to the system. The dead weight of the elements is taken into account by the software. To determine the maximum internal forces, a simple load combination is analysed by means of Th. I. Ord.. The loads correspond to the load assumptions specified in chapter 4.1.1, with additional safety factors. They are applied to the corresponding axis using the axis distance and the building width. The static system is presented in Figure 26. As in Design 1, the preliminary design is based on simplified assumptions, so the components are not fully utilized.

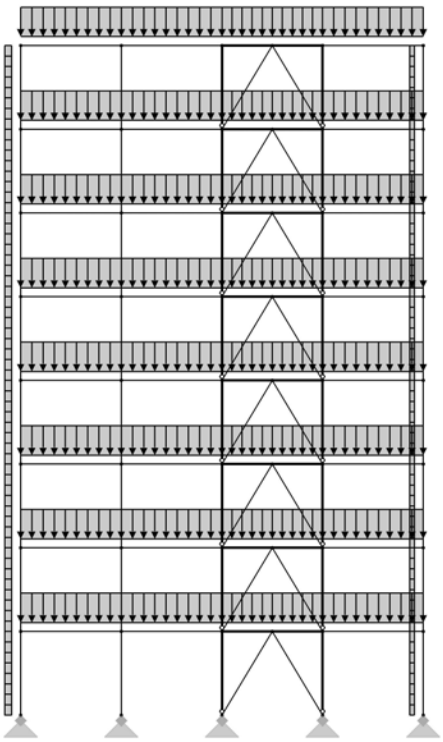


Figure 26 - Design 2: Structural system, RFEM-model

System:

Building width: $b = 18m$

Axis distance: $a = 4{,}5m$

Safety factor: $\gamma_G = 1{,}35$

$\gamma_Q = 1{,}5$

$\gamma_W = 1{,}5$

Load combination:

LK: $G_k * \gamma_G + Q_k * \gamma_Q + W_k * \gamma_W$

Load:

Wind loads:

Windward: $W_{p,k} = w_{p,k} * \frac{b}{2} = 5{,}832 \frac{kN}{m}$

with: $w_{p,k} = 0{,}648 \frac{kN}{m^2}$

Leeward: $W_{t,k} = w_{t,k} * \frac{b}{2} = 3{,}645 \frac{kN}{m}$

with: $w_{t,k} = 0{,}405 \frac{kN}{m^2}$

Life load: $q_{,k} = 5{,}0 \frac{kN}{m^2}$

$Q_k = q_{,k} * a = 22{,}5 \frac{kN}{m^2}$

Braced frame columns – steel

The columns of the braced frame are loaded with normal force and moments. However, the moment load is very small, so it is neglected in the pre-dimensioning. In Contrast, the normal force is very high because the columns have to carry the vertical load component of the bracing. They are therefore at high risk of buckling. In a simplified manner, the buckling length in both planes is assumed with the height of the floors. Since a HEB steel profile is chosen, buckling about the weak axis is decisive. The maximum normal force is determined with the analysis model.

Standard: DIN EN 1993-1-1

Cross section:

HEB 340

Sectional area: $A = 170{,}9cm^2$

Moment of inertia: $I_z = 9690{,}0cm^4$

Radius of gyration: $i_z = 7{,}53cm$

Material:

Steel S235: $f_{yd} = 21{,}36 \frac{kN}{cm^2}$

$\gamma_{M1} = 1{,}1$

Internal Force:

Normal force: $N_d = -2340{,}22kN$

Dimensioning:

Buckling length: $l_{ef} = 375cm$

Slenderness: $\lambda = \frac{l_{ef}}{i_z} = 49{,}8$

Slenderness level: $\bar{\lambda} = \frac{\lambda}{\lambda_1} = 0{,}53$

with $\lambda_1 = 93{,}3$

Buckling line: c

Coefficient: $\chi = 0{,}826$

Buckling: $N_{b,Rd} = \chi * A * f_{yd} = 3015{,}4kN$

$\frac{N_d}{N_{b,Rd}} = 0{,}77 \leq 1$

Braced frame beams – Steel

An IPE 400 is chosen for the beam of the braced frame. Due to the intermediate support, only a low moment load is expected. At the same time, the beam has to transfer the wind load from the slab toward the bracings, which means that it is subjected to normal force. Nevertheless, the expected utilization is very low. Anyway, the profil is chosen to achieve a constant height

with the glulam beams. The maximum loads are extracted from the static model. Lateral deflection of the profile, tilting, is prevented by the support of the slab construction.

Standard: DIN EN 1993-1-1	**Internal Force:**
Cross section:	Moment: $M_{y,d} = -44{,}97\text{kNm}$
IPE 400	Vertical force: $V_{z,d} = 61{,}01\text{kN}$
Sectional area: $A = 84{,}46\text{cm}^2$	Normal force: $N_{t,d} = 212{,}21\text{kN}$
Material:	$N_{p,d} = -149{,}86\text{kN}$
Steel S235: $f_{yd} = 23{,}5\ \frac{\text{kN}}{\text{cm}^2}$	**Dimensioning:**
$\gamma_{M0} = 1{,}0$	Criteria interaction: $\frac{V_{z,d}}{V_{pl,z,Rd}} = 0{,}1 < 0{,}5$
Plastic resistances:	$\frac{N_d}{N_{pl,Rd}} = 0{,}1 < 0{,}2$
Moment: $M_{pl,y,Rd} = 307{,}1\text{kNm}$	➔ No interaction
Vertical force: $V_{pl,z,Rd} = 579{,}8\text{kN}$	Bending: $\frac{M_{y,d}}{M_{pl,y,Rd}} = 0{,}15$
Normal force: $N_{pl,Rd} = 1986{,}0\text{kN}$	

Braced frame bracings – Steel

As already mentioned, a tubular cross-section is chosen for the bracing. The normal forces are determined using the static software. In comparison to the tensile forces, the compressive forces are decisive in the stiffening. Accordingly, a buckling analysis is done for pre-dimensioning. Due to the hinged connections, the buckling length corresponds to the bar length.

Standard: DIN EN 1993-1-1	**Internal Force:**
Cross section:	Normal force: $N_d = -455{,}24\text{kN}$
Pipe 139,7x10,0	**Dimensioning:**
Sectional area: $A = 40{,}70cm^2$	Buckling length: $l_{ef} = 437{,}3\text{cm}$
Moment of inertia: $I = 862{,}0cm^4$	Slenderness: $\lambda = \frac{l_{ef}}{i} = 95{,}06$
Radius of gyration: $i = 4{,}6\text{cm}$	Slenderness level: $\bar{\lambda} = \frac{\lambda}{\lambda_1} = 1{,}01$
Material:	with $\lambda_1 = 93{,}9$
Steel S235: $f_{yd} = 21{,}36\ \frac{kN}{cm^2}$	Buckling line: a
$\gamma_{M1} = 1{,}1$	Coefficient: $\chi = 0{,}66$
	Buckling: $N_{b,Rd} = \chi * A * f_{yd} = 572{,}77\text{N}$
	$\frac{N_d}{N_{b,Rd}} = 0{,}79 \leq 1$

Pendulum columns - Glulam

The wind loads are transferred directly into the slab via the facade, so the perimeter columns are only subjected to normal forces. Because of the compressive force, a buckling check is decisive. A square cross-section of glulam is analysed.

Standard:	DIN EN 1995-1-1
Cross section:	
h/b [mm]:	240/240mm
Sectional area:	$A = 576{,}0cm^2$
Moment of inertia:	$I = 27648{,}0cm^4$
Radius of gyration:	$i = 6{,}93cm$
Material:	
Glulam GL24h:	$f_{c,0,d} = 14{,}76 \frac{N}{mm^2}$
	$k_{mod} = 0{,}8, \gamma_M = 1{,}3$

Internal Force:	
Normal force:	$N_d = -627{,}91$
Dimensioning	
Buckling length:	$l_{ef} = 375cm$
Slenderness:	$\lambda = l_{ef}/i = 54{,}1$
Coefficient:	$k_c = 0{,}872$
Buckling:	$\frac{N_{q,d}/A}{k_c * f_{c,0,d}} = 0{,}85 \leq 1$

Beams - Glulam

The beams distribute the horizontal loads onto the columns. In contrast, the horizontal wind load is transferred to the stiffening elements via the floor slabs. Thus, the beams are only stressed in bending. The bending moment resulting from the simplified analysis model is reduced by 2/3 due to the realistic biaxial load transfer of the floor slabs. Tilting is prevented by the position securing of the slab. The material chosen is glulam with a rectangular cross-section.

Cross-Section:	
h/b [mm]:	400/180
Sectional area:	$A = 720{,}0cm^2$
Section modulus:	$W = 4800{,}0cm^3$
Material:	
Glulam GL24h:	$f_{m,y,d} = 14{,}76 \frac{N}{mm^2}$
	$k_{mod} = 0{,}8, \gamma_M = 1{,}3$

Internal Force:	
Bending moment:	$M_d{}^* = 86{,}66kNm$
	$M_d = M_d{}^* * \frac{2}{3} = 57{,}77kNm$
Dimensioning	
Bending:	$\frac{M_d/W}{f_{m,d}} = 0{,}81 \leq 1$

Slab construction – Cross laminated timber (CLT)

The ceiling system is not included in the 2D model. The load transfer should be biaxial as described in the design. The square beam grid serves as a support. Accordingly, the span of the slab is 4.5m in both directions. Since the determination of the bending stress for biaxial slabs depends on the directional stiffness, it is quite complex. The pre-dimensioning is therefore carried out as a uniaxially spanned slab with a reference width of 1m. The design principle of the CLT floor has already been presented under Design 1. The bending stress due the live load and the normal stress due the wind load are taken into account.

Cross-Section:

CLT 130 DL5s [mm]: 30/20/30/20/30

Sectional area: $A = 1300{,}0\ \frac{cm^2}{m}$

$A_{0,n} = 900{,}0\ \frac{cm^2}{m}$

Moment of inertia: $I_{0,n} = 15675{,}0\ \frac{cm^4}{m}$

Section modulus: $W_{0,n} = 2411{,}54\ \frac{cm^3}{m}$

Material:

Panels C24: $f_{m,d} = 14{,}76\ \frac{N}{mm^2}$

$f_{c,0,d} = 12{,}92\ \frac{N}{mm^2}$

$k_{mod} = 0{,}8, \gamma_M = 1{,}3$

System:

Span: $l = 4{,}5m$

Floor hight: $h = 3{,}75m$

Load:

Safety factors: $\gamma_Q = 1{,}5$

Life load: $q_{,k} = 5{,}0\ \frac{kN}{m^2}$

$q_{,d} = q_{,k} * \gamma_Q = 7{,}5kN/m^2$

Wind load: $w_{p,k} = 0{,}648\ \frac{kN}{m^2}$

$W_d = w_{p,k} * h * \gamma_Q = 3{,}645\ \frac{kN}{m}$

Internal Force:

Bending moment: $M_d = \frac{q_{,d} * l^2}{8} = 18{,}98\ \frac{kNm}{m}$

Normal force: $N_{w,d} = W_d = 3{,}645\ \frac{kN}{m}$

Dimensioning:

Pressure and Bending: $\frac{N_d/A_n}{f_{c,0,d}} + \frac{M_d/W_n}{f_{m,d}} \leq 1$

$0{,}59 \leq 1$

4.3.3 Life cycle assessment

The LCA is based on the framework conditions as described in chapter 4.1.3. The mass is determined with the help of the pre-dimensioning and a simple model of the design. Table 3 shows the accumulated LCI and LCIA. The following datasets based on the ÖKOBAUDAT[139] database are used for this assessment: Stahlprofil, Brettschichtholz - Standardformen (Durchschnitt DE) and Brettsperrholz (Durchschnitt DE). The results are compared with the LCA's of the other designs in chapter 5.1.2.

Table 3 - Design 2: LCI and LCIA

Component	Material	Section/ Thickness	Meter/ Surface	Volume	Mass	Comment
		[m²]/[m]	[m]/[m²]	[m³]	[kg]	
BF-Column	S235	0,0171	240,00	4,10	32197,56	Braced Frame
BF-Beam	S235	0,0084	144,00	1,22	9547,36	Braced Frame
BF-Bracing	S235	0,0041	279,87	1,14	8941,77	Braced Frame
Colums	GL24c	0,0576	510,00	29,38	14893,63	Pendulum Column
Beam	GL24c	0,0720	1296,00	93,31	47309,18	
Slab	CLT	0,1300	2592,00	336,96	164773,44	

[139] ÖKOBAUDAT Informationsportal Nachhaltiges Bauen 2021

Component	PERT	PENRT	GWP	ODP	POCP	AP	EP
	[MJ]	[MJ]	kg CO2	kg CFC-11	kg C2H4	kg SO2	kg PO4^3
BF-Column	4,98E+04	2,42E+05	2,30E+04	5,21E-05	7,22E+00	4,39E+01	4,93E+00
BF-Beam	1,48E+04	7,19E+04	6,81E+03	1,55E-05	2,14E+00	1,30E+01	1,46E+00
BF-Bracing	1,38E+04	6,73E+04	6,38E+03	1,45E-05	2,00E+00	1,22E+01	1,37E+00
Colums	2,72E+05	-8,26E+04	-4,71E+03	-2,80E-10	7,73E-02	9,02E+00	2,80E+00
Beam	8,65E+05	-2,62E+05	-1,50E+04	-8,88E-10	2,45E-01	2,87E+01	8,88E+00
Slab	2,85E+06	-9,57E+05	-5,70E+04	-2,92E-09	1,53E+01	5,80E+01	1,91E+01
TOTAL	**4,07E+06**	**-9,20E+05**	**-4,05E+04**	**8,21E-05**	**2,70E+01**	**1,65E+02**	**3,85E+01**

4.4 Design 3: Rammed earth shear wall structure

Design 3 is a shear wall system as explained in chapter 2.5.2. The shear walls are made of solid rammed earth and taper towards the top. In addition, pendulum columns are used at the facade level for vertical load transfer. Beams support the slab load and transfer it to the vertical load resisting elements. Figure 27 shows a simplified representation of the static system. A rendering for illustration is shown in Figure 28.

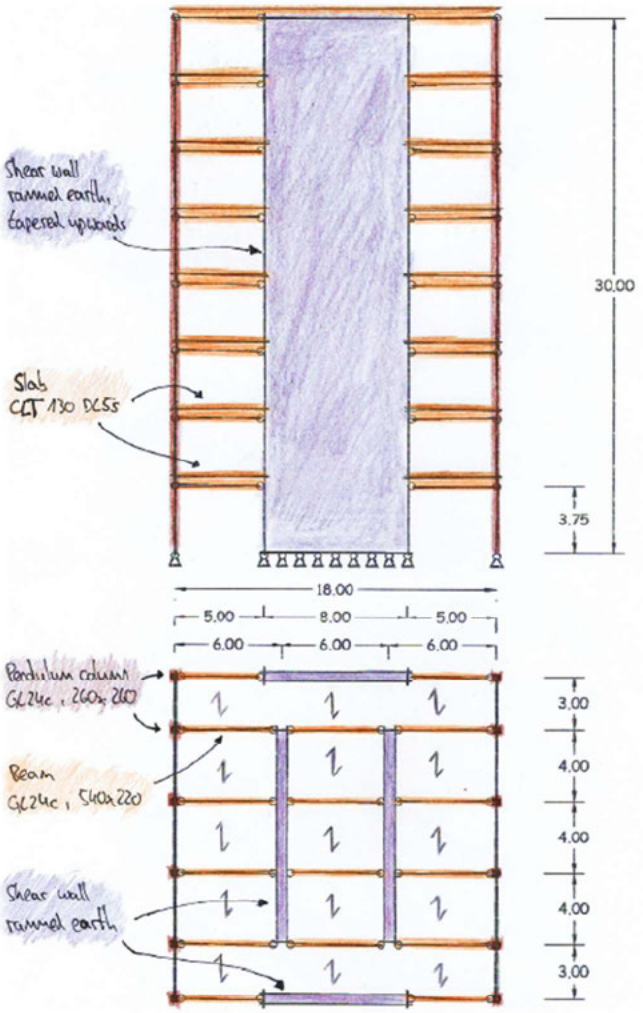


Figure 27 - Design 3: Structural system

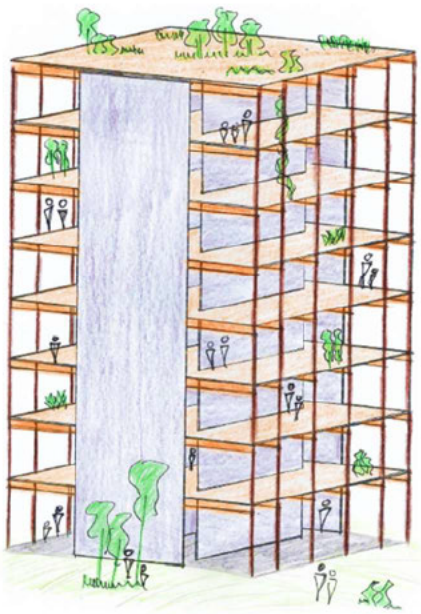


Figure 28 - Design 3: Rendering

4.4.1 Construction

The key elements of this design are the massive rammed earth walls. They strongly characterize the architecture as well as the load-bearing behaviour of the building. According to their shear wall function, the walls take over the horizontal load transfer. For each load direction, two walls are available, arranged in such a way that a stable ground plan against rotation is created. The parallel walls with a smaller inner lever arm are significantly longer, so that they can also transfer additional torsional loads. Of course, the walls are also used for vertical load transfer. They serve as supports for the beams and must of course bear their own weight. On the one hand, the normal force can create a frictional resistance to the shear load and on the other hand, it is possible to superimpose the resulting moment. In this way, tensile forces can be avoided or reduced. This is important because rammed earth is only suitable for pressure loads and additional measures in the form of reinforcement are necessary for significant tensile loads. The compressive strength of rammed earth is nevertheless considerably lower than that of comparable concrete. Wall thicknesses of up to 1m are therefore required. In order to reduce the dead weight at the same time, the wall thicknesses must be reduced towards the top. The additional pendulum columns at the facade level are made of glulam, as are the beams. The slab structure is planned to be made of CLT panels. The pendulum columns, beams and slabs are thus consistent with the construction from Design 1. Wooden constructions are preferred to solid or steel constructions because of their better environmental sustainability and their reduced weight.

As already mentioned, the walls have a strong influence on the architecture as well. For one thing, the solid walls require a lot of space due to their thickness. And in addition, closed walls without openings are designed to ensure the load-bearing capacity, which strongly limits the openness and flexibility of the floor plan. The displacement of two wall panels into the interior of the building divides the floor plan into two separate units. At the same time, this allows for an at least partially free, open facade and usable space. The closed interior can be used to accommodate the staircase and elevator, as with a core structure. The construction of the rammed earth is very time-consuming and labour intensive. The clay must be filled into the formwork and be compacted. In addition, the drying process must be taken into account. For the construction process, the rammed earth walls should be built first over one or two storeys before the columns, beams and slabs are connected. Through the shear stiff slabs, the bracing of the system is thus constantly ensured. The height of the structure is also determined by the continuous shear walls but is limited due to the load-bearing capacity of the rammed earth.

Rammed earth counters the lack of load-bearing capacity with the fact that it is a local building material, which is available almost everywhere. Clay has therefore an excellent ecological reputation, since it is not a mainly industrially manufactured product. In addition, clay has

very good building physics properties. The combination of earthen building materials and wood is still widespread in traditional architecture in Botswana. This allows the design to be closely linked to the culture and increases the acceptance of the population. For the wood, however, industrial glulam must be used in order to ensure strength and the required cross-sections.

4.4.2 Pre-dimensioning

As in Design 1, the pre-dimensioning is done using simple structural systems. The individual components correspond to Design 1 as well, except for the shear walls, which now replace the concrete core. The horizontal load is thus transferred by the shear walls and the vertical load transfer results proportionally from the catchment area. Beams and slabs provide for the distribution of the loads. The well-known load approaches and safety factors from chapter 4.1.1 are applied.

Loads:		**Safety factors:**	
Wind load:		Wind load:	$\gamma_W = 1{,}5$
Pressure windward:	$w_{p,k} = 0{,}648 \frac{kN}{m^2}$	Life load:	$\gamma_Q = 1{,}5$
Uplift leeward:	$w_{t,k} = 0{,}405 \frac{kN}{m^2}$		
Life load:	$q_{,k} = 5{,}0 \frac{kN}{m^2}$		

Shear walls – Rammed earth

As a static system for the shear walls, a vertical cantilever is used as described in chapter 2.5.2. The walls only carry horizontal loads parallel to their longitudinal axis since their stiffness in the transverse direction is too low. The vertical live loads result from the catchment width of the slabs. In addition, and in contrast to the concrete core, the dead weight must be taken into account here.

An issue for the design is that there is no generally applicable standard for earthen construction. Although some countries have published specific standards, such as the "Lehmbau Regeln" in Germany or the Engineering Design of Earth Buildings in New Zealand, but these are usually limited to a maximum of two-storey buildings. While the design methods can be adopted for taller buildings, the question of allowable strengths remains. Some reference values from different standards are given by Schroeder in Sustainable Building with Earth[140]. For example, New Zealand allows a compressive stress of 0.5 N/mm² and Australia of 0.7 N/mm² for unstabilized rammed earth. Therefore, the compressive strength of pure rammed earth is very low, but it is possible to stabilize it, especially by adding cement (India

[140] Refer to Schroeder 2016, p. 221

1.4 N/mm², Australia 5.2 N/mm²). In general, however, the strength of rammed earth is strongly dependent on the soil quality and is therefore determined in situ by means of individual samples. However, an empirical value for required wall thicknesses in rammed earth construction is the height-to-thickness ratio of 10/1.[141]

Neither the reference strengths nor the height-to-thickness ratio can be applied to the present design at first glimpse. For this reason, the pre-dimensioning is reversed. Practical, architecturally feasible cross-sections are specified, and the required material properties are determined. Due to the high specific mass of rammed earth (1.700-2.400kg/m³)[142], the wall thickness is decreased over the building height. The maximum wall thickness of 100cm at the base of the building is reduced every two floors, over 80cm and 60cm down to 40cm. This minimizes the dead weight and thus the load on the wall. The determination of the stresses and especially the coefficient of friction are derived from the New Zealand standard (Engineering Design of Earth Buildings).

Cross section:

$t_1/t_2/t_3/t_4$ [cm]:	100/80/60/40
Øt [cm]:	70 cm
Wall length:	$l_{min} = 8m$
Section modulus:	$W_1 = 10{,}67m^3$

Material:

Rammed Earth:	$g_k = 20{,}0\frac{kN}{m^3}$
Friction coefficient:	$k_v = 0{,}3$

System:

Building width:	$b = 18m$
Building hight:	$h = 30m$
Number of floors:	$n = 8$

Load:

Wind load: $w_d = (w_{t,k} + w_{p,k}) * b/2 * \gamma_W$

$w_d = 14{,}2\frac{kN}{m}$

Life load: $q_{,d} = q_{,k} * a * \gamma_Q = 45{,}0\frac{kN}{m}$

Dead load: $g_{,d} = g_{,k} * Øt * \gamma_G = 18{,}9\frac{kN}{m^2}$

Internal Force:

Clamping Moment: $M_d = w_d * \frac{h^2}{2} = 6{,}39\ MNm$

Normal Force: $N_{q,d} = q_{,d} * n = 0{,}36\frac{MN}{m}$

$N_{g,d} = g_{,d} * h = 0{,}567\frac{MN}{m}$

Vertical Force: $V_d = w_d * h = 0{,}426\ MN$

Dimensioning and material properties:

Normal stress: $\sigma_d = \frac{N_{q,d}+N_{q,d}}{t_1} + \frac{M_d}{W_1} = 1{,}52\frac{MN}{m^2}$

Required compressive strength: $f_k = \gamma_M * \sigma_d = 2{,}28\frac{MN}{m^2}$ $\gamma_M = 1{,}5$

Shear stress: $\tau = \frac{V_d - k_v * N_{g,d} * l_{min}}{t_1 * l_{min}} = -0{,}11\frac{MN}{m^2} < 0$ with friction

$\tau = \frac{V_d}{t_1 * l_{min}} = 0{,}053\frac{MN}{m^2}$ without friction

Required shear strength: $f_{v.k} = \gamma_M * \tau = 0{,}08\frac{MN}{m^2}$ $\gamma_M = 1{,}5$

[141] Refer to Marais 2020

[142] Refer to Schroeder 2016, p. 200

For the material properties of rammed earth, a compressive strength of f_k=2,28MN/m² and a shear strength of $f_{v.k}$=0,08MN/m² are thus required. Due to the long walls, the shear strength is not critical, especially if the frictional resistance resulting from the normal force is considered. The compressive strength, in contrast, is more crucial. Among the permissible compressive strengths listed by Schroeder, only cement-stabilized rammed earth from Australia achieves the required strength (5,2 N/mm²).[143] Alternatively, Martin Rauch, an expert in the field of earthen building, gives a compressive strength of 2,4MN/m² as the minimum characteristic value for a professionally produced earthen mixture.[144] However, this assumes a highly optimized soil mixture, which cannot always be guaranteed. It must therefore be expected that cement-stabilized rammed earth has to be used. Unfortunately, rammed earth loses some of its positive properties when mixed with cement. One is its ecological sustainability, since cement has to be produced in the same way as for concrete. Another is that cement provides a chemical bonding of the aggregates, which, unlike the physical bonding of clay minerals, is irreversible. Nevertheless, the implementation of a load-bearing rammed earth structure for high-rise buildings is within the realm of possibility.

It should be noted at this point that a material partial safety factor of 1.5 was chosen for the design, as it is used in the Eurocode for concrete. Whereas the German "Lehmbau Regeln" give a global safety factor, between the laboratory tests and the permissible compressive strength of earthen elements, of about seven.[145] Therefore, in case of realization, rammed earth samples should be carefully investigated in situ and a valid safety concept should be established.

Pendulum columns – Glulam

The pendulum columns can be pre-dimensioned as in Design 1 using the 2nd Euler case. The load bearing area results from the floor plan and corresponds to a maximum of 4x3m. While the dead load is neglected due to its small amount, the live load is summed up over all 8 floors. The buckling length results from the floor height. A simple square cross-section of glulam is studied.

[143] Refer to Schroeder 2016, p. 200
[144] Refer to Kapfinger and Sauer 2015, p. 125
[145] Refer to Schroeder 2016, p. 218

Standard:	DIN EN 1995-1-1
Cross section:	
h/b [mm]:	260/260mm
Sectional area:	$A = 676{,}0cm^2$
Moment of inertia:	$I = 38081{,}3cm^4$
Radius of gyration:	$i = 7{,}50cm$
Material:	
Glulam GL24h:	$f_{c,0,d} = 14{,}76 \frac{N}{mm^2}$
	$k_{mod} = 0{,}8, \gamma_M = 1{,}3$
System:	
Catchment area:	$a_{max} = 12{,}00m^2$
Number of floors:	$n = 8$

Load:	
Life load:	$q_{,d} = q_{,k} * \gamma_Q = 7{,}5 \frac{kN}{m^2}$
Internal Force:	
Normal force:	$N_{q,d} = a_{max} * q_{,d} * n$
	$N_{q,d} = 720kN$
Dimensioning	
Buckling length:	$l_{ef} = 375cm$
Slenderness:	$\lambda = l_{ef}/i = 50$
Coefficient:	$k_c = 0{,}907$
Buckling:	$\frac{N_{q,d}/A}{k_c * f_{c,0,d}} = 0{,}79 \leq 1$

Beams – Glulam

The beams have a span of 6m and are designed as single span beams. With the maximum catchment width of 4m, the live load can be calculated. Dead weight is neglected. It is intended that the slabs take over the transfer of the normal forces from the facade to the shear walls. Therefore, the beams are only subjected to bending. The supported slabs also prevent the beams from tilting. A rectangular glulam cross-section is dimensioned.

Standard:	DIN EN 1995-1-1
Cross-Section:	
h/b [mm]:	540/220
Sectional area:	$A = 1188{,}0cm^2$
Section modulus:	$W = 10682{,}0cm^3$
Material:	
Glulam GL24h:	$f_{m,y,d} = 14{,}76 \frac{N}{mm^2}$
	$k_{mod} = 0{,}8, \gamma_M = 1{,}3$
System:	
Span:	$l = 6m$
Catchment width:	$a_{max} = 4{,}0m$

Load:	
Life load:	$q_{,d} = q_{,k} * a * \gamma_Q = 30{,}0 \frac{kN}{m}$
Internal Force:	
Bending moment:	$M_d = \frac{q_{,k} * l^2}{8} = 135{,}0kNm$
Dimensioning:	
Bending:	$\frac{M_d/W}{f_{m,d}} = 0{,}85 \leq 1$

Slab construction – Cross laminated timber (CLT)

The slab construction corresponds exactly to the slab from Design 1. This is true for the type of floor, the CLT floor, as well as for the load transfer and the floor span (4m). The design situation is therefore exactly the same and can also be executed according to the guidelines of proHolz Austria, as described in chapter 4.2.2. The same requirements result in the same slab thickness of 130mm.

Cross-Section:

CLT 130 DL5s [mm]: 30/20/30/20/30

Sectional area: $A = 1300{,}0\ \frac{cm^2}{m}$

$A_{0,n} = 900{,}0\ \frac{cm^2}{m}$

Moment of inertia: $I_{0,n} = 15675{,}0 \frac{cm^4}{m}$

Section modulus: $W_{0,n} = 2411{,}54 \frac{cm^3}{m}$

Material:

Panels C24: $f_{m,d} = 14{,}76 \frac{N}{mm^2}$

$f_{c,0,d} = 12{,}92 \frac{N}{mm^2}$

$k_{mod} = 0{,}8, \gamma_M = 1{,}3$

System:

Span: $l = 4m$

Floor hight: $h = 3{,}75m$

Load:

Life load: $q_{,d} = q_{,k} * \gamma_Q = 7{,}5kN/m^2$

Wind load: $W_d = w_{p,k} * h * \gamma_Q = 3{,}645 \frac{kN}{m}$

Internal Force:

Bending moment: $M_d = \frac{q_{,d} * l^2}{8} = 18{,}98 \frac{kNm}{m}$

Normal force: $N_{w,d} = W_d = 3{,}645 \frac{kN}{m}$

Dimensioning:

Pressure and Bending: $\frac{N_d/A_n}{f_{c,0,d}} + \frac{M_d/W_n}{f_{m,d}} \leq 1$

$0{,}52 \leq 1$

4.4.3 Life cycle assessment

The LCA is based on the framework conditions as described in chapter 4.1.3. The mass is determined with the help of the pre-dimensioning and a simple model of the design. Table 4Table 1 shows the accumulated LCI and LCIA. The following datasets based on the ÖKOBAUDAT[146] database are used for this assessment: Stampflehmwand, Brettschichtholz - Standardformen (Durchschnitt DE) and Brettsperrholz (Durchschnitt DE). The results are compared with the LCA's of the other designs in chapter 5.1.2.

Table 4 - Design 3: LCI and LCIA

Component	Material	Section/ Thickness	Meter/ Surface	Volume	Mass	Comment
		[m²]/[m]	[m]/[m²]	[m³]	[kg]	
Shear Walls	Earth	0,7000	1200,00	840,00	1680000,0	
Colums	GL24c	0,0676	360,00	24,34	12338,35	
Beams	GL24c	0,1188	736,00	87,44	44330,46	
Slab	CLT	0,1300	2592,00	336,96	164773,44	

Component	PERT	PENRT	GWP	ODP	POCP	AP	EP
	[MJ]	[MJ]	kg CO2	kg CFC-11	kg C2H4	kg SO2	kg PO4^3
Shear Walls	1,47E+04	2,66E+05	1,74E+04	3,71E-11	4,12E+00	5,92E+01	1,40E+01
Colums	2,25E+05	-6,84E+04	-3,90E+03	-2,32E-10	6,40E-02	7,47E+00	2,32E+00
Beams	8,10E+05	-2,46E+05	-1,40E+04	-8,32E-10	2,30E-01	2,69E+01	8,32E+00
Slab	2,85E+06	-9,57E+05	-5,70E+04	-2,92E-09	1,53E+01	5,80E+01	1,91E+01
TOTAL	**3,90E+06**	**-1,01E+06**	**-5,75E+04**	**-3,95E-09**	**1,98E+01**	**1,52E+02**	**4,37E+01**

[146] ÖKOBAUDAT Informationsportal Nachhaltiges Bauen 2021

4.5 Design 4: Wooden diagrid structure

This last design is a typical diagrid structure as presented in chapter 2.5.5. The structural system consists of closed triangles with hinged struts. In order to support the vertical load transfer, four additional pendulum columns are placed in the middle of the building as supports for the beams. In Figure 29 the static system can be seen while Figure 30 shows a visualization.

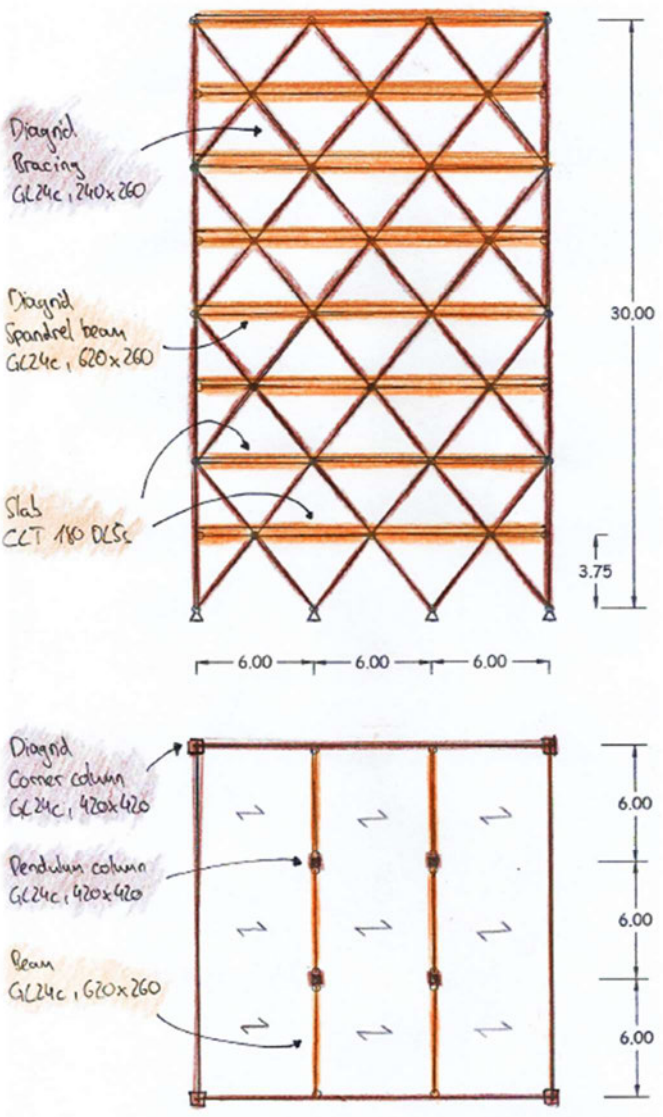


Figure 29 - Design 4: Structural system

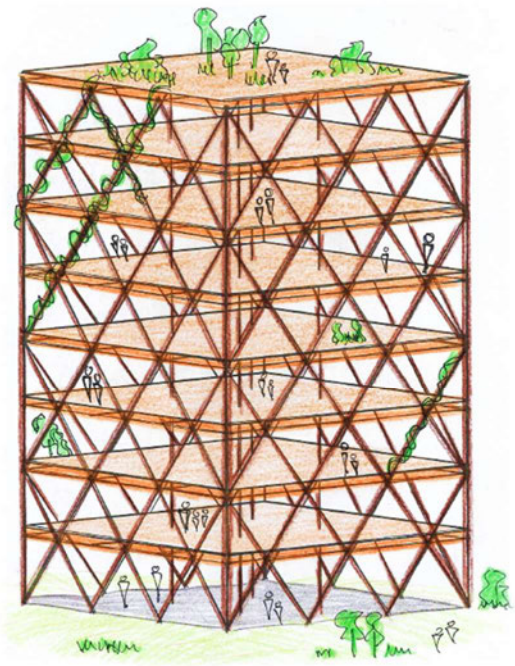


Figure 30 - Design 4: Rendering

4.5.1 Construction

The diagrid structure is the only tube structure that seems to make sense in the context of this project. A trussed tube is not flexible enough in the number of floors, while the ground plan area is too small for a practical implementation of a bundled or a tube-in-tube system. Furthermore, the framed tube, the basic system, is characterized by complex rigid connections

and usually only has sufficient stiffness in steel or concrete construction. But both construction methods are not particularly sustainable. These tube systems are therefore more suitable for high-rise buildings with more than 40 floors, as explained in chapter 2.5.5.

To enable the required sustainability, the entire supporting structure is made of wood. All beam elements are made of glulam. This allows the implementation of the required cross-sections and the necessary strength. The ceiling is manufactured as a two-dimensional equivalent of CLT. The diagonal bracings provide the stiffening and together with the spandrel beams, form a closed triangle of the truss structure. Thereby, the proportions are chosen in such a way that the struts run at a diagonal angle of approx. 45°. While the struts are only loaded axially, the spandrel beams, are also subjected to bending stress. This results from the vertical load since the spandrel beams function as supports for the slab construction or the beams. The corner columns form the lateral end of the truss facade and thus also link the neighbouring facades. They are correspondingly solid and run over two floors each. In the centre of the building, simple pendulum columns are arranged as supports for the beams. The beams, in turn, span in one direction and support the floor loads. A CLT floor was chosen as the slab system, spanning uniaxially as in Design 1. By connecting the slab to the facade, biaxial bending of the spandrel beams is avoided. At the same time, the slab contributes to a uniform distribution of the vertical loads.

The entire construction does not require any rigid connections, which is a great advantage. Still, complex details are created because a lot of elements meet in the nodes. The entire construction is nevertheless quite uncomplicated. Individual elements have manageable dimensions and can be prefabricated. The floors can be erected one after the other and function as a platform for each other. With two floors belonging together, due to the corner columns, the system is quite flexible in terms of height. The floor plan design and thus the possible utilizations are also very flexible. Only the four pendulum columns interfere with the free interior space. However, the floor plan area is fixed. The tubular structure allows only geometrically simple floor plans and not dissolved ones. Moreover, since the facade is part of the supporting structure, no cantilevering is possible. In this respect, the system is completely inflexible. But loggias and greenery may be arranged behind the facade with the construction as shading.

As already mentioned, the entire construction is made of wood. This has the advantage that only expertise in the use of one building material is required. Wood as a natural building material also fits perfectly into the nature of the Okavango Delta. However, it is debatable whether the architecture is suitable. The visible supporting structure creates a closed cubature that offers little architectural freedom and is difficult to integrate into the open space of the surrounding countryside. Furthermore, modern timber construction also has no relation to

traditional architecture. However, it is feasible to compensate this by means of secondary building elements, such as lightweight clay walls.

4.5.2 Pre-dimensioning

As described in chapter 2.5.5, tube systems are complex load-bearing structures that transfer the load three dimensionally like a hollow box section. For the preliminary design, however, only a 2D model of one diagrid facade is generated and loaded with half of the horizontal load, similar to the pre-dimensioning of Design 2. The vertical live loads are determined from the catchment width and areas of the floor plan. Due to the uniaxial slab support, the spandrel beams are loaded partly with a line load and partly with a point load caused by the beams. Both cases are investigated to determine the maximum internal forces of the struts. The analysis is performed using RFEM on the basis of Th. I. Ord.. Using the loads from the load assumptions and safety factors, a simple load combination is investigated. The static systems are presented in Figure 31. Due to some simplifications in the calculation of the pre-dimensioning, a 100% utilization for the components is not targeted here either. The beams and pendulum columns inside the building are independent of the grid structure and can be designed separately for vertical load transfer. The same applies to the slab structure which is not represented in the RFEM model as well.

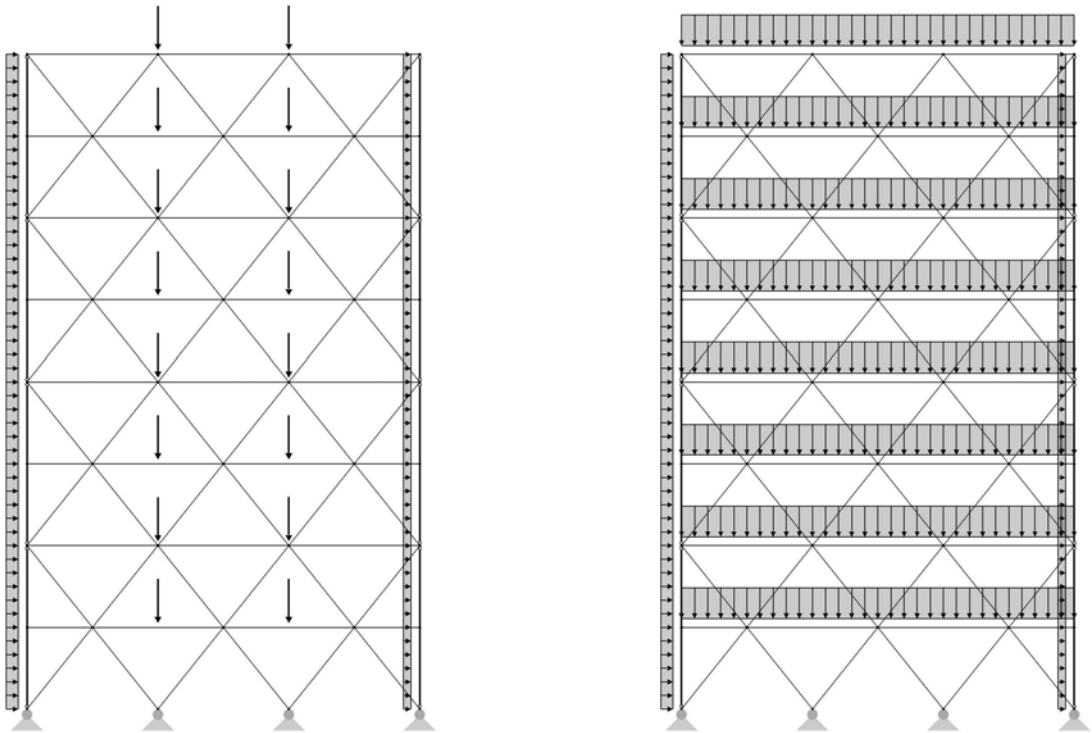


Figure 31 - Design 4: Structural systems, RFEM-model

System:	
Building width:	$b = 18m$
Catchment width:	$a = 3{,}0m$
Catchment area:	$A = 18{,}0m^2$
Safety factor:	$\gamma_G = 1{,}35$
	$\gamma_Q = 1{,}5$
	$\gamma_W = 1{,}5$
Load combination:	
LK: $G_k * \gamma_G + Q_k * \gamma_Q + W_k * \gamma_W$	

Load:

Wind loads:

Windward: $W_{p,k} = w_{p,k} * \frac{b}{2} = 5{,}832 \frac{kN}{m}$

with: $w_{p,k} = 0{,}648 \frac{kN}{m^2}$

Leeward: $W_{t,k} = w_{t,k} * \frac{b}{2} = 3{,}645 \frac{kN}{m}$

with: $w_{t,k} = 0{,}405 \frac{kN}{m^2}$

Life load: $q_{,k} = 5{,}0 \frac{kN}{m^2}$

Line load: $q_k = q_{,k} * a = 15{,}0 \frac{kN}{m}$

Point load: $Q_k = q_{,k} * A = 90{,}0kN$

Diagrid bracings – glulam

The diagonal bracings of the diagrid structure are hinged and run over one floor each. Depending on their position, they are subject to both compressive and tensile forces. However, it is the compressive forces within the lower members that is decisive. Due to the high compressive forces and their buckling length, they are vulnerable to stability. Their buckling length is equal to the bar length, as in the 2nd Euler case. A rectangular cross-section with the dimensions 240x260mm is investigated, with the help of the maximum normal force. The slightly higher width is chosen to allow the same width as the spandrel beams. Accordingly, buckling about the weak Y-axis is relevant.

Standard:	DIN EN 1995-1-1
Cross section:	
h/b [mm]:	240/260mm
Sectional area:	$A = 624{,}0cm^2$
Moment of inertia:	$I_y = 29952{,}0cm^4$
Radius of gyration:	$i_y = 6{,}93cm$
Material:	
Glulam GL24h: $f_{c,0,d} = 14{,}76 \frac{N}{mm^2}$	
$k_{mod} = 0{,}8, \gamma_M = 1{,}3$	

Internal Force:	
Normal force:	$N_d = -456{,}07kN$
Dimensioning	
Buckling length:	$l_{ef} = 480{,}2cm$
Slenderness:	$\lambda = l_{ef}/i = 69{,}29$
Coefficient:	$k_c = 0{,}702$
Buckling:	$\frac{N_{q,d}/A}{k_c * f_{c,0,d}} = 0{,}70 \leq 1$

Diagrid spandrel beam – glulam

The spandrel beams run in an articulated manner between the nodes of the diagrid and serve as supports for the slab construction or the beams. They are thus subjected to bending and shear forces via the live load. But they are also subjected to normal forces, due to the wind load and the connection to the bracings. Both compressive and tensile forces can occur. The critical verification is performed for tension and bending under a point load, caused by a supported

beam. Tilting is prevented by the attached slabs. Due to the high bending stress, a high rectangular cross-section is chosen.

Standard: DIN EN 1995-1-1	**Internal Force:**
Cross section:	Normal force: $N_{t,d} = 101{,}1kN$
h/b [mm]: 620/260mm	Bending moment: $M_{y,d} = 205{,}76kNm$
Sectional area: $A = 1612{,}0cm^2$	**Dimensioning**
Moment of inertia: $I_y = 516377{,}0cm^4$	Bending and Tension: $\frac{N_{t,d}}{A*f_{t,0,d}} + \frac{M_{y,d}}{W_y*f_{m,y,d}} \leq 1$
Section modulus: $W_y = 16657{,}3cm^3$	$0{,}90 \leq 1$
Material:	
Glulam GL24h: $f_{m,y,d} = 14{,}76 \frac{N}{mm^2}$	
$f_{t,0,d} = 10{,}455 \frac{N}{mm^2}$	
$k_{mod} = 0{,}8, \gamma_M = 1{,}3$	

Diagrid corner columns – glulam

The corner supports form the chords of the diagrid truss. They thus receive tensile and compressive forces from the wind load. While the tensile forces are overcome by the dead weight and the live load, these simultaneously increase the force in the compression zone. Due to the high compressive forces, buckling is decisive. The supports run continuously over two floors but are intermediately supported by the spandrel beams. The buckling length thus corresponds to the storey height in both directions. The bending moment due to the connected spandrel beams is neglected because of the small amount. In the design, it must be taken into account that the corner columns of the real building must carry the loads of two facades, so only a utilization of about 50% is aimed for. A rectangular cross-section of 420x420mm glulam is investigated.

Standard: DIN EN 1995-1-1	**Internal Force:**
Cross section:	Normal force: $N_d = -1309{,}79kN$
h/b [mm]: 420/420mm	**Dimensioning**
Sectional area: $A = 1764{,}0cm^2$	Buckling length: $l_{ef} = 375cm$
Moment of inertia: $I = 259308{,}0cm^4$	Slenderness: $\lambda = l_{ef}/i = 30{,}9$
Radius of gyration: $i = 12{,}12cm$	Coefficient: $k_c = 0{,}98$
Material:	Buckling: $\frac{N_{q,d}/A}{k_c*f_{c,0,d}} = 0{,}51 \leq 1$
Glulam GL24h: $f_{c,0,d} = 14{,}76 \frac{N}{mm^2}$	
$k_{mod} = 0{,}8, \gamma_M = 1{,}3$	

Pendulum columns – glulam

The pendulum columns inside the structure can be analysed independently of the diagrid structure. They only support the vertical load transfer of the live loads and the dead weight of

the slab. Accordingly, they are subject to pressure loads and a buckling check must be verified. The buckling length corresponds to the construction length and thus to the floor height. The load of the columns can be determined via the influence areas of the floor plan and then summed up over the floors. The same cross-section as for the corner columns of the diagrid is studied.

Standard: DIN EN 1995-1-1	**Load:**
Cross section:	Safety factor: $\gamma_Q = 1{,}5$
h/b [mm]: 420/420mm	Life load: $q_{,k} = 5{,}0 \frac{kN}{m^2}$
Sectional area: $A = 1764{,}0cm^2$	**Internal Force:**
Moment of inertia: $I = 259308{,}0cm^4$	Normal force: $N_{q,d} = a_{max} * q_{,k} * n * \gamma_Q$
Radius of gyration: $i = 12{,}12cm$	$N_{q,d} = 2160kN$
Material:	**Dimensioning**
Glulam GL24h: $f_{c,0,d} = 14{,}76 \frac{N}{mm^2}$	Buckling length: $l_{ef} = 375cm$
$k_{mod} = 0{,}8, \gamma_M = 1{,}3$	Slenderness: $\lambda = l_{ef}/i = 30{,}9$
System:	Coefficient: $k_c = 0{,}98$
Catchment area: $a_{max} = 36m^2$	Buckling: $\frac{N_{q,d}/A}{k_c * f_{c,0,d}} = 0{,}84 \leq 1$
Number of floors: $n = 8$	

Beams – glulam

The beams can be examined separately from the diagrid structure as well. Inside the building, they serve as supports for the floor structure. Since the floor takes over the distribution of the horizontal loads, the beams are only subjected to bending loads. The slab also prevents them from tilting. The resulting moment is determined from the static system of a single-span beam. Like the spandrel beams, the beams are also made of glulam. Since the bending stress is decisive, a correspondingly high rectangular cross-section of 620x260mm is analysed.

Standard: DIN EN 1995-1-1	**Load:**
Cross section:	Safety factors: $\gamma_Q = 1{,}5$; $\gamma_G = 1{,}35$
h/b [mm]: 620/260mm	Life load: $q_{,k} = 5{,}0 \frac{kN}{m^2}$
Sectional area: $A = 1612{,}0cm^2$	$q_{,d} = q_{,k} * a * \gamma_Q = 45{,}0 \frac{kN}{m}$
Moment of inertia: $I_y = 516377{,}0cm^4$	**Internal Force:**
Section modulus: $W_y = 16657{,}3cm^3$	Bending moment: $M_{q,d} = \frac{q_{,d} * l^2}{8} = 202kNm$
Material:	**Dimensioning**
Glulam GL24h: $f_{m,y,d} = 14{,}76 \frac{N}{mm^2}$	Bending: $\frac{M_d/W}{f_{m,d}} = 0{,}82 \leq 1$
$k_{mod} = 0{,}8, \gamma_M = 1{,}3$	
System:	
Span: $l = 7m$	
Catchment width: $a_{max} = 6{,}0m$	

Slab construction – Cross laminated timber (CLT)

The slab construction is not shown in the RFEM model either. But the design indicates that the slab is made of cross laminated timber and has a uniaxial load transfer. The design can therefore be carried out with net cross-section values as explained in the preliminary design of Draft 1. In addition to the bending load from the live load, the wind load is to be applied as an axial load. Cause, the shear stiff slab ensures a uniform distribution of the horizontal loads. A cross-section utilization of about 50% is aimed at in order to ensure additional reserves for the deformation and vibration checks.

Cross-Section:

CLT 180 DL5s [mm]: 40/30/40/30/40

Sectional area: $A = 1800{,}0\ \frac{cm^2}{m}$

$A_{0,n} = 1200{,}0\ \frac{cm^2}{m}$

Moment of inertia: $I_{0,n} = 40800{,}0\ \frac{cm^4}{m}$

Section modulus: $W_{0,n} = 4533{,}3\ \frac{cm^3}{m}$

Material:

Panels C24: $f_{m,d} = 14{,}76\ \frac{N}{mm^2}$

$f_{c,0,d} = 12{,}92\ \frac{N}{mm^2}$

$k_{mod} = 0{,}8, \gamma_M = 1{,}3$

System:

Span: $l = 6{,}0m$

Floor hight: $h = 3{,}75m$

Load:

Safety factors: $\gamma_Q = 1{,}5$

Life load: $q_{,k} = 5{,}0\ \frac{kN}{m^2}$

$q_{,d} = q_{,k} * \gamma_Q = 7{,}5kN/m^2$

Wind load: $w_{p,k} = 0{,}648\ \frac{kN}{m^2}$

$W_d = w_{p,k} * h * \gamma_Q = 3{,}645\ \frac{kN}{m}$

Internal Force:

Bending moment: $M_d = \frac{q_{,d} * l^2}{8} = 33{,}75\ \frac{kNm}{m}$

Normal force: $N_{w,d} = W_d = 3{,}645\ \frac{kN}{m}$

Dimensioning

Pressure and Bending: $\frac{N_d/A_n}{f_{c,0,d}} + \frac{M_d/W_n}{f_{m,d}} \leq 1$

$0{,}51 \leq 1$

4.5.3 Life cycle assessment

The LCA is based on the framework conditions as described in chapter 4.1.3. The mass is determined with the help of the pre-dimensioning and a simple model of the design. Table 5 shows the accumulated LCI and LCIA. The following datasets based on the ÖKOBAUDAT[147] database are used for this assessment: Brettschichtholz - Standardformen (Durchschnitt DE) and Brettsperrholz (Durchschnitt DE). The results are compared with the LCA's of the other designs in chapter 5.1.2.

[147] ÖKOBAUDAT Informationsportal Nachhaltiges Bauen 2021

Table 5 - Design 4: LCI and LCIA

Component	Material	Section/ Thickness	Meter/ Surface	Volume	Mass	Comment
		[m²]/[m]	[m]/[m²]	[m³]	[kg]	
D-Bracings	GL24c	0,0624	921,98	57,53	29168,50	Diagrid Bracings
D-Beams	GL24c	0,1612	576,00	92,85	47075,56	Diagrid Beams
D-Columns	GL24c	0,1764	120,00	21,17	10732,18	Diagrid Corner Columns
P-Columns	GL24c	0,1764	120,00	21,17	10732,18	Pendulum Columns
Beams	GL24c	0,1612	288,00	46,43	23537,78	
Slab	CLT	0,1800	2592,00	466,56	228147,84	

Component	PERT	PENRT	GWP	ODP	POCP	AP	EP
	[MJ]	[MJ]	kg CO2	kg CFC-11	kg C2H4	kg SO2	kg PO4^3
D-Bracings	5,33E+05	-1,62E+05	-9,22E+03	-5,48E-10	1,51E-01	1,77E+01	5,48E+00
D-Beams	8,60E+05	-2,61E+05	-1,49E+04	-8,84E-10	2,44E-01	2,85E+01	8,84E+00
D-Columns	1,96E+05	-5,95E+04	-3,39E+03	-2,02E-10	5,57E-02	6,50E+00	2,02E+00
P-Columns	1,96E+05	-5,95E+04	-3,39E+03	-2,02E-10	5,57E-02	6,50E+00	2,02E+00
Beams	4,30E+05	-1,31E+05	-7,44E+03	-4,42E-10	1,22E-01	1,43E+01	4,42E+00
Slab	3,95E+06	-1,33E+06	-7,90E+04	-4,05E-09	2,13E+01	8,02E+01	2,64E+01
TOTAL	**6,17E+06**	**-2,00E+06**	**-1,17E+05**	**-6,32E-09**	**2,19E+01**	**1,54E+02**	**4,92E+01**

5 Evaluation and Conclusion

In the previous chapter, four designs of a sustainable structural concept for the MSP were developed. In order to determine a preferred design, these are now compared and evaluated. However, a favourite design does not necessarily represent a perfect sustainable structure. Furthermore, the question arises whether there is a perfect structure at all. For this reason, additional general conclusions for the development of sustainable load-bearing systems will subsequently be drawn.

5.1 Evaluation

The evaluation is based on the requirements described in chapter 4.1.2. Accordingly, the designs are first compared with each other in terms of their structural and constructional aspects. This is followed by the interpretation of the LCA's, which represents the ecological sustainability. As the last category of requirements, the criteria of social sustainability are compared with each other. With the help of these comparisons, the designs are then evaluated and assessed in a utility value analysis.

5.1.1 Static and constructive criteria

The designs are all based on a different static system as presented in chapter 2.5. The other structural systems for which no design has been prepared are not suitable for the current project in the eyes of the author. This has several reasons, but above all, complex structures such as most tube systems, outrigger systems or mega structures require a much higher effort, which is only worthwhile if the building is more than 40 storeys high. Whereas the classic rigid frame system, requires rigid connections which can usually only be implemented with sufficient stiffness in steel construction or reinforced concrete. However, both construction materials have a negative effect on environmental sustainability, see chapter 5.1.2. For the same reason, only wooden pendulum columns and beams are used in all designs. Thereby, glulam must be used, since most of the cross-sections are clearly too large for structural solid timber. In the load assumptions, reference was made to a wooden slab because of the weight savings. To ensure comparability, the same construction method, a CLT slab, was chosen in all designs. As long as the ceiling has the required shear stiffness, it has no further influence on the structure. It can therefore be optimized separately with other wooden ceiling systems at a later stage. The main difference in the construction is therefore related to the stiffening systems. Thus, they have the greatest impact on the evaluation of all criteria.

Prefabrication and installation effort

In terms of prefabrication and installation effort, the bar structures have a clear advantage over the solid structures. All glulam elements in Design 4 as well as the steel frames in Design 2 can

be prefabricated without any problems. Thus, an incredibly high degree of prefabrication can be achieved, which also significantly simplifies the installation. In addition, the structure can be erected one floor at a time, using the slabs of the previous floor as a working platform. Due to the smaller components, the shear frame system has a slight advantage over the diagrid. The solid construction elements, concrete core and rammed earth wall, in contrast, have to be manufactured on site. For both in-situ constructions a formwork, a scaffolding and a crane are needed. The construction is therefore much more complex and takes significantly longer. Especially within the rammed earth construction, the individual layers must be compacted separately, by hand or by machine, and a long drying process must be waited for. In comparison, the concrete core can be erected rather quickly.

Complexity of details

The articulated connections between glulam pendulum columns and beams are required in any design. However, they are usually not a problem as long as the specifics of timber construction are taken into account. In contrast, more attention must be paid to the shear connections of the slab elements as well as to the shear force transfer to the bracing elements. But this applies in a similar way to all designs. Beyond that, the beam structures have a slightly simpler detailing. In Design 2, the details are limited to predominantly simple, articulated and right-angled bar connections. Only the construction of the rigid frame connection is critical, but this can be solved during prefabrication. Whereas Design 4 consists entirely of articulated connections, but a certain complexity still arises due to the large number of bars in a node. An additional challenge is the integration of the structure into the facade. In the case of the solid concrete core, Design 1, the challenge lies particularly in the reinforcement for the partially closed core. This required proper planning and execution. In contrast, the load application into the concrete core is beneficial, as the material strength usually does not pose any problems. The opposite is the detailing in rammed earth construction. So far, there are only few empirical values for corresponding connections. Moreover, the limited strength requires extra measures wherever disturbances, such as load application, occur.

Need for expertise

The expertise required depends on the experience in the construction methods and the number of trades required. Reinforced concrete and steel construction are probably the best-known and most widespread construction methods, especially in high-rise building construction. Accordingly, the required expertise in construction and planning is low. In contrast Design 4 requires a higher level of expertise due to the modern timber construction and a more complex load-bearing system. Because of the lack of international standards, Design 3, rammed earth construction, is also far behind. Although many people around the world live in mud huts, there are hardly any tall buildings in mud construction and most rammed earth buildings are

limited to two storeys. The commercial construction of eight-storey rammed earth buildings is therefore uncharted territory which requires a great deal of expertise and research.

5.1.2 Ecological sustainability criteria

The life cycle assessment is intended to assess the environmental sustainability of the structures. For each design, the LCA was calculated based on a preliminary design. For the interpretation of the results, the main materials are considered first on their own to understand their influences and impacts. This is of central importance for the subsequent interpretation of the results. The primary building materials are wood or glulam, structural steel, concrete and rammed earth.

LCA of the building materials

The main impacts in the LCA of timber products occur in the manufacturing phase (Module A). The main environmental impact parameters are the GWP, the AP and the POCP. In all impact categories, both wood supply and transport play a central role. But the GWP and the AP are mainly affected by the electricity consumption and the heat demand in the factory. Whereas the binder of wooden products, e.g. glulam, has a large influence on the emission of the POCP equivalents. A special role within the GWP is taken by the carbon storage from the photosynthesis of trees. Its influence is so great that the other indicators seem negligible next to it. However, the carbon is bound to the biomass and leaves the system at the end of the life cycle, either in the form of waste wood or through reduction to ashes. The same effect can be seen with the energy demand. The majority of the energy required comes from the growth of the tree and is therefore contained in the material or biomass itself (PERM). A small amount of energy is still used for the manufacturing of the product, while the share of transport is negligible. At the end of the life cycle, the wood component can therefore be used either materially, as reused waste wood, or for energy recovery, through combustion (Module D). In both cases, the bonded CO_2 and the PERM leave the system. However, in the case of material reuse, the CO_2 remains captured in the wood and the lost PERM can be credited as PERE. In the case of energy use, the CO_2 is released through combustion, but energy is gained at the same time. The material energy PERM is converted accordingly into non-renewable energy PENRE. Due to the compensation of the energy production, by means of renewable energies instead of fossil energies, credits result on all indicators. In total, the compensation results in a credit instead of a consumption for the indicators PENRT, GWP and ODP. It must be warned against the mistake of thinking the more wood the better. The storage effect of the CO_2 as well as the energy remain above all also in a living tree. Therefore, wood products should always be sourced from sustainable forestry. The LCI of the ÖKOBAUDAT assumes an energetic use

at the end of the life cycle. In principle, however, the aim should be to reuse wood products, as this extends the storage effect of CO_2.[148]

Unlike wood, structural steel is a completely industrially manufactured product. The impact of production (Module A) on the environmental effects and energy requirements is correspondingly high. The main component of steel is iron, which is sintered in the blast furnace and then converted to liquid steel in the converter. In a secondary production route, recycled steel waste can be melted directly in the electric arc furnace. The liquid steel can then be further processed into semi-finished products. The majority of the GWP, AP, EP as well as the POCP are therefore caused by emissions in the energy-intensive steel production. Credits in Module D arise from recycled secondary steel replacing primary steel in production. In this context, structural steel has a very good recycling potential overall. Up to 99% of the steel mass can be recovered, of which about 11% is reused as a component and 88% is returned to production as steel scrap. The resulting credits amount to approximately 30-40% of the primary emissions caused. The majority of the resource requirements, PERT and PENRT, are needed for the extraction and processing of the raw materials and the provision of electrical energy for the manufacturing process. Recycling can save the energy required for the complex sintering process.[149]

Concrete is produced by mixing cement, aggregate, water and, if necessary, admixtures. As with the other materials, the production phase (Module A) with extraction of the raw materials, transport and installation has by far the greatest impact on the environment. Cement production dominates all impact categories by far, even though it accounts for only about 13% of the masses. During the sintering and deacidification of the lime marl to clinker, numerous environmentally harmful emissions are produced. Another driver, especially for the consumption of resources, is the extraction and transport of the raw materials. The share of processes in the concrete factory, in contrast, is rather small. Demolition in the disposal phase (Module C) can be neglected in comparison almost as much as positive carbonation in the use phase. The recycling potential is also rather low, since only the aggregates can be reused as crushed concrete material.[150]

Rammed earth, unlike steel and concrete, is not an industrially manufactured product. It does not have to be produced by sintering or other energy- and emission-intensive processes. Accordingly, no module stands out in the life cycle. Due to the simple manufacturing process, a cubic meter of rammed earth thus has a much lower environmental impact than concrete,

[148] Refer to Rüter and Diederichs 2012, pp. 153-159
[149] Refer to bauforumstahl e.V., pp. 3-4 and 7-8
[150] Refer to Informationszentrum Beton GmbH, pp. 3 and 6-8

for example. Yet, this is combined with lower strengths. The material rammed earth itself, without cement, can be recycled very well, since the consolidation process is reversible. However, the manufacturing processes are not recyclable, which means that the recycling potential of rammed earth in terms of LCA indicators is rather low. Due to the significantly higher mass, which is usually required for rammed earth, transport must also be taken into account, especially in comparison with other building materials.

LCA of the designs - interpretation

For the comparison of the different LCA's, not the individual building materials, but the entire structures with all masses are taken into account. The environmental impacts and resource consumption are summed up over all life cycle modules, according to the cradle-to-cradle principle. Table 6 shows the comparison of the impact categories for the different designs. In Figure 32, the individual impact categories are nominated to the maximum value and shown comparatively as a bar chart.

Table 6 - Comparison of the LCA for the designs

	PERT	PENRT	GWP	ODP	POCP	AP	EP
	[MJ]	[MJ]	kg CO2	kg CFC-11	kg C2H4	kg SO2	kg PO4^3
	10^6	10^6	10^4	10^-7	10^1	10^1	10^1
Design 1	4,927	-1,378	-6,529	57,158	1,950	17,094	4,815
Design 2	4,068	-0,920	-4,054	820,559	2,703	16,470	3,851
Design 3	3,903	-1,006	-5,751	-0,039	1,976	15,152	4,369
Design 4	6,166	-1,997	-11,730	-0,063	2,188	15,369	4,918

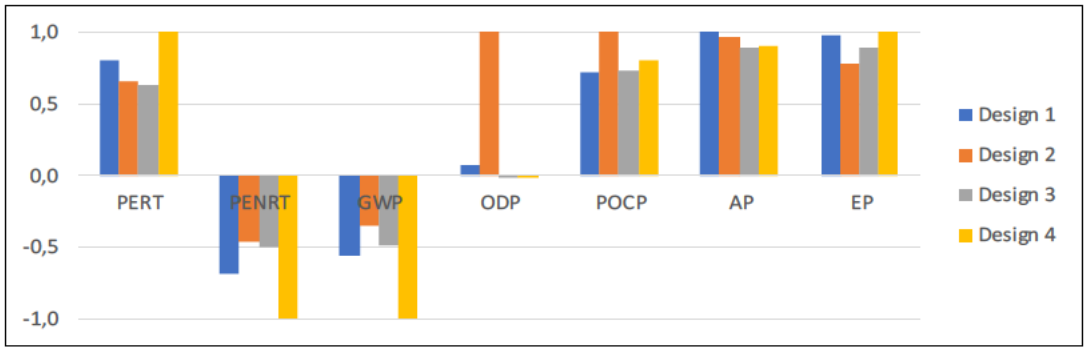


Figure 32 - Diagram: Comparison of the LCA for the designs

For a better classification of the results, the evaluation for exclusively the horizontal load-bearing elements is also shown. These are, according to the designs, a concrete core, braced steel frames, rammed earth shear walls and a peripheral wooden diagrid structure. The results are shown in Table 7 and Figure 33. The interpretation is made for each impact category separately.

Table 7 - Comparison of the LCA for the horizontal load resisting elements

	PERT	PENRT	GWP	ODP	POCP	AP	EP
	[MJ]	[MJ]	kg CO2	kg CFC-11	kg C2H4	kg SO2	kg PO4^3
	10^6	10^6	10^4	10^-7	10^1	10^1	10^1
Design 1	0,049	0,193	2,678	57,208	0,358	4,586	0,827
Design 2	0,078	0,382	3,616	820,600	1,136	6,907	0,776
Design 3	0,015	0,266	1,744	0,000	0,412	5,924	1,397
Design 4	1,589	-0,482	-2,750	-0,016	0,045	5,268	1,633

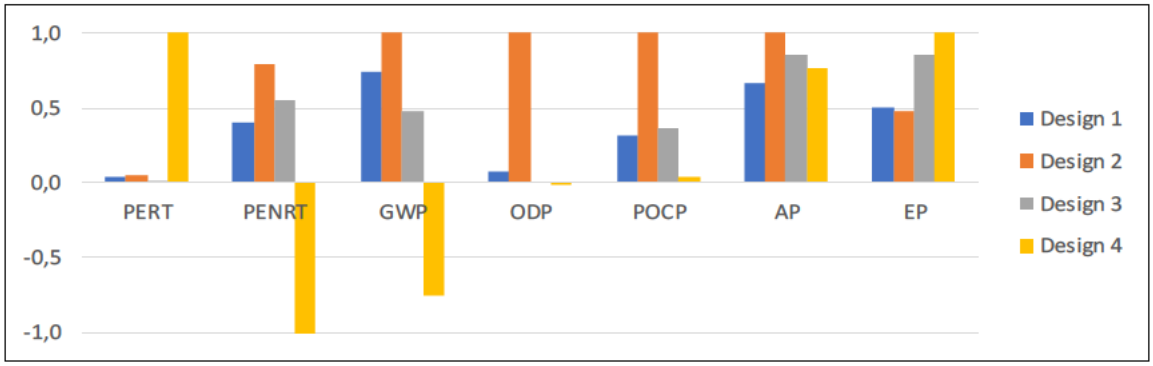


Figure 33 - Diagram: Comparison of the LCA for the horizontal load resisting elements

The primary energy demand of renewable energies (PERT) is clearly highest for the complete wooden structure from Design 1. This is due to the fact that it correlates directly with the installed wooden mass, from bars and ceilings. This can be seen, among other things, in the fact that the PERT for the stiffening made of concrete, steel and rammed earth is very low compared to the total demand. The compensation of wood as a renewable energy source compared to fossil energy sources is the reason that wood has such a large impact. The wooden mass of the designs thus reflects the demand, compare Design 1 with 275t; Design 2 with 226t; Design 3 with 221t and Design 4 with 349t of wood.

The opposite effect can be seen in the primary energy demand for non-renewable energy (PENRT). The compensation of wood results in a credit to PENRT across all designs. However, the primary energy demand of the remaining building materials is no longer negligible. Summed up, the steel frames have the highest primary energy demand for production. But what is most surprising, is that rammed earth requires more PENRT than concrete. One cubic meter of clay requires significantly less energy than concrete, but there is a compensation in the volume required, compare to concrete 97m² and rammed earth 840m². The overall credit is the greatest for Design 1 due to the use of exclusively wooden components.

In the case of the global warming potential (GWP), the same effect can be seen as a result of wood compensation as in the case of PENRT. The crediting of wood as well as its large-scale installation results in a negative value in total. However, the industrial production of concrete and steel degrades the balance in the corresponding designs; both are very GWP intensive.

Rammed earth does not require industrial production, but a high mass is needed. In addition, the least amount of wood is used. Thus, the pure timber construction, the diagrid, scores best here once again.

Wood and clay materials have almost no influence on the ozone depletion potential. However, the emission for the steel frames is noteworthy here, it forms a very clear peak compared to the other designs. A closer look reveals that this impact result from the life cycle module D of the steel product.

The binders of the cross-laminated timber slab are predominantly responsible for the formation of smog, the photochemical ozone creation potential (POCP). The ceiling thickness of the structures therefore plays a significant role in here. In addition, the energy-intensive production of steel causes noticeable POCP emissions. The influence of concrete and rammed earth is greater than that of the wooden elements, but still subordinate compared to the slabs.

All building components have a negative influence on the acidification potential (AP). No building material stands out negatively or positively. Rather, wood as well as concrete, steel and clay cause emissions of a similar magnitude. A balance between the materials takes place via the required masse.

For the eutrophication potential (EP), the same applies as for the AP; all designs perform in a similar magnitude. The amount of wood construction material has an influence here again, whereas the influences of steel and concrete are rather small. Furthermore, the noticeable influence of rammed earth, which is certainly related to the mass used, is surprising.

Uncertainty of the LCA

At this point, once again, it should be pointed out that the LCA is not entirely accurate. The data for the life cycle inventory (LCI) and impact assessment (LCIA) are taken from ÖKOBAUDAT, which usually refers to average values from Germany. For Botswana, there are certainly deviations, especially for transport routes and the mix of electricity. But there is no comparable database for Botswana. Furthermore, the assumptions and the preliminary measurement are subject to additional uncertainties. The preliminary design is only done for the load transfer, additional measures for fire protection, weather protection, insulation or similar are not considered. In the opinion of the author, however, a comparison of the designs is possible and useful despite this uncertainty.

5.1.3 Social sustainability criteria

The requirements of social sustainability are strongly dependent on the project. In chapters 3.3 and 4.1.2, the project-specific requirements and criteria for this thesis were elaborated. In this chapter, the comparison of the designs with regard to flexibility, architectural quality and the possible use

of local resources is elaborated. Since these are soft factors, the evaluation is subject to a certain degree of subjectivity.

Flexibility

The flexibility of a building is closely linked to the primary support structure. For example, the shear walls from Design 3, strongly dissect the building and additionally require a lot of space due to their thickness. They thus limit the flexibility of the floor plan enormously. Furthermore, two of the rammed earth walls are located in the facade plane, which also restricts flexibility, compare chapter 3.3.2. In Design 4, even the entire load-bearing structure is part of the facade. Thus, resolved floor plans can hardly be created. Otherwise there is a large free interior area with wide spans and few columns. The flexibility inside the building is thus maximized. Design 1 also consists of shear walls, but here they are combined into a core. The restriction is thus reduced to the centre of the building, where the utilities are located. This, in combination with a free facade and large spans, ensures a high degree of flexibility. The floor frame construction from Design 2 is not restricted by any closed walls and thus offers maximum flexibility in terms of floor plan, height and facade.

Local architecture

The architectural quality of a design is often very subjective. From the author's point of view, however, Design 1 and Design 3 fit best into the local architecture. Design 1 most closely resembles the structure of a tree and thus the first design of the Tree of Life. It also resembles traditional architecture, with a mixture of solid and skeletal structures. The open design also allows for modularity and interaction between the building and the environment. Design 3, as explained under flexibility, is more limited in modularity, but with the use of traditional building materials, it allows an even closer link to traditional architecture. Design 2 offers a good, open modularity, but as a modern bar structure it has no relation to the traditional building culture or the land. In Design 4, this is even more extreme, as the closed construction does hardly allow any interaction with the surroundings and may look like a foreign body in the open space around Maun.

Local materials and services

The possibility of using local materials and services is very difficult to evaluate from abroad. However, the supply of building materials for Maun is rather difficult, as shown in chapter 3.3.4. Most of the building materials, such as concrete or steel, have to be brought in from the industrial southwest of the country. Timber usually even has to be imported from neighbouring countries. Designs 1, 2 and 4 are thus all rated rather poorly in the use of local materials. Design 3 is an exception, since clay is a locally available building material almost everywhere. However, it must be checked whether the soil in Maun and the surrounding area is sufficient for the structural requirements. The rammed earth design is also the best for local

services. It is generally very labour-intensive and workers can be trained very well. For the prefabrication of the other designs, it is advisable to build the necessary infrastructure on site as part of the project and thus train and involve the local population.

5.1.4 Utility analysis

With the help of the comparisons made, the designs are evaluated in this section with a utility analysis. For this purpose, the individual criteria are weighted and then rated with points. For the weighting, 100 points, corresponding to 100%, are distributed among the categories or the individual criteria. The evaluation then takes place for each criterion with points between 1-10. The value 1 thereby stands for "not at all" or "most badly fulfilled", the value 10 however for "completely" or "best fulfilled". The evaluation of the soft factors from static and construction as well as of the social sustainability is performed subjectively based on the comparisons in the previous sections. The evaluation of the LCA, in contrast, is based on the data determined. Thereby. the best value in each impact category receives a rating of 10, the worst a rating of 1. Intermediate values are interpolated.

In the following, the weighting is explained before Table 8 shows the utility analysis with the evaluation. Finally, the results are briefly summarized.

Weighting

In the weighting of common sustainability certifications, such as DGNB[151] and BNB[152], approximately 2/3 of the points are distributed equally among the three pillars of sustainability. The remaining third is allocated to technical quality, process quality and site characteristics. The weighting of the designs cannot follow this model, since only primary structures were examined rather than entire buildings. Nevertheless, the evaluation sheets provided by the DGNB and the BNB are suitable as an orientation for the weighting of the criteria, so that proportionality is maintained.

The first category of requirements, static and construction, is weighted with a total of 15 points. The complexity of the construction is an important factor for the feasibility of the project, but it should not be a knockout criterion. As Botswana is not an industrialized country, there is an inherent risk in lack of construction equipment, industrial manufacturing and expertise. All of these risks can affect the realization of the project, so the aim should be to simplify the construction as much as possible. All three specific criteria of prefabrication and construction effort, complexity of details and need for expertise are given the same importance of 5% of the total score. Similar criteria of the DGNB or the BNB system can be summed up to about 7-10%, especially in the technical and process-related quality.

[151] DGNB GmbH 2021
[152] Bewertungssystem Nachhaltiges Bauen (BNB) 2021

The ecological sustainability of the designs is assigned 60% of the weighting. On the one hand, this can be assessed by means of an LCA, on the other hand, the primary supporting structure and the associated building materials represent a large part of the mass of the entire building. Their share of grey energy and environmental impact is correspondingly high. The indicators are weighted differently depending on their importance. The GWP is given the highest weighting with 20%. It currently has the greatest significance, as it is directly linked to global climate change. The other environmental impacts ODP, POCP, AP and EP are weighted equally at 5% each. For resource consumption, both PERT and PENRT are weighted at 10% each. The consumption of PENRT is more critical because it is limited. However, the high weighting of PERT is intended to punish excessive wood consumption. Otherwise, due to the high credits from compensation, the impression the more wood the better is created. The BNB system also evaluates the ecological sustainability with a LCA, it corresponds to 22.5% of the overall rating. The weighting of the individual indicators is based on relevance factors and is comparable to the selected weighting.

The category of social sustainability is weighted at 25% overall. While it is one of the three pillars of sustainability, it is also difficult to relate it to the primary structure alone. This is best achieved with the criteria of flexibility, which has a significant impact on service life and is therefore weighted at 15%. In addition, local architecture and the use of local materials and services are important for the "made in Botswana" project. However, both criteria are difficult to evaluate from abroad and are therefore only weighted with 5%. Similar criteria are weighted with about 10% via the BNB system.

Table 8 - Utility analysis of the structural designs

			Design 1		Design 2		Design 3		Design 4	
Category	Critaria	Weighting	Rating	W. Rating	Rating	W. Rating	Rating	W. Rating	Rating	W. Rating
Static and Constrution	Prefabrication and installation effort	5	4	20	9	45	2	10	8	40
	Complexity of details	5	6	30	8	40	3	15	7	35
	Need for expertise	5	8	40	8	40	2	10	6	30
Ecological sustainability / Life cycle assessment	Total renewable primary energy (PERT)	10	6	60	9	90	10	100	1	10
	Total non-renewable primary energy (PENRT)	10	5	50	1	10	2	20	10	100
	Global warming potential (GWP)	20	4	80	1	20	3	60	10	200
	Ozone depletion potential (ODP)	5	9	45	1	5	10	50	10	50
	Photochemical ozone creation potential (POCP)	5	10	50	1	5	10	50	7	35
	Acidification potential (AP)	5	1	5	4	20	10	50	9	45
	Eutrophication potential (EP)	5	2	10	10	50	6	30	1	5
Social sustainability	Flexibility	15	8	120	10	150	3	45	7	105
	Local architecture	5	9	45	4	20	9	45	2	10
	Local materials and services	5	3	15	3	15	7	35	3	15
	Total	**100**	**5,70**	**570**	**5,10**	**510**	**5,20**	**520**	**6,80**	**680**

Results

Design 4, the wooden diagrid structure, was identified as the preferred variant by the utility value analysis. It is followed at a distance by Design 1, the concrete core structure. Design 2 and Design 3 are roughly equal, with the steel frame structure at the end of the list. Table 9 shows the performance of the designs in the individual requirement categories.

Table 9 - Results of the requirement categories

		Design 1		Design 2		Design 3		Design 4	
Static and Constrution	15	**6,00**	90	**8,33**	125	**2,33**	35	**7,00**	105
Ecological sustainability /Life cycle assessment	60	**5,00**	300	**3,33**	200	**6,00**	360	**7,42**	445
Social sustainability	25	**7,20**	180	**7,40**	185	**5,00**	125	**5,20**	130

The winning design is very well rated in the category of structural design and construction and especially in environmental sustainability. This is due to the extensive use of wood. The design of the concrete core is rated moderate in all categories and thus reaches second place. In contrast, the remaining two designs have clearer advantages and disadvantages. The steel frame as a skeleton structure is a simple, fast and flexible construction, but scores very poorly in environmental terms. The opposite is true for the rammed earth design, which performs well ecologically but is the most challenging and least flexible in construction.

5.2 Conclusion

The wooden diagrid structure, Design 4, has proven to be the preferred design from the evaluation. However, as mentioned above, this is by no means a perfect structure. Design 4 also has identifiable disadvantages and the weighting as well as the evaluation is based on the author's assessment. Nevertheless, the following conclusions can be drawn for the development of a sustainable structural system.

From an ecological point of view, wooden structures are clearly the best choice in building material. This is mainly due to the positive balance of the PENRT and the GWP indicators. The compensation, by using renewable wood instead of fossil fuels for energy production, at the end of the life cycle provides a credit within the analysed system. Overall, however, reuse of wood components should be preferred to thermal recycling. Although the extension of the life cycle delays the compensation, it allows a long-term saving effect of CO_2 and the PERM. This is because both are bound to the biomass. For the same reason, it is essential to source the wood exclusively from sustainable forestry. A living tree is still more valuable for the

environment than a dead tree, used for timber construction. However, if this is considered, the wood used must not be limited from an ecological point of view. In fact, the more wood is used, the greater the storage effect. This aspect confirms the assumption of the wooden pendulum columns, beams and ceilings. Apart from wood, the performance "room" is mainly independent of the building materials used for the structure, in terms of resource conservation.[153] The reason therefore is a trade-off between the LCA indicators, the strength and the required mass of a building material. For example, industrially manufactured products require a lot of primary energy and emissions but are at the same time more efficient and require less mass. This is clearly visible when comparing the concrete core and rammed earth shear walls. In the case of a multi-storey timber structure, however, attention must be paid to feasibility. In principle, industrial timber products must be used since solid wood is hardly available with required cross-sections. The low stiffness of the connections can also be critical, which can lead to high deformations. For serviceability vibrations, fire protection and sound insulation must also be given particular attention.

As a structural system, a member structure is preferred to a solid structure. The main reason is the basically greater flexibility. This is particularly important for the MSP since no detailed utilization has yet been defined and the entire project should be transferable. In addition, a member structure can be prefabricated to a large extent, which leads to simple and fast installation. Besides that, the load-bearing system for an 8-storey building structure can be freely chosen, as it has no significant impact on the ecology or the mass. This is due to the fact that the premium for height effect only becomes relevant above a height of about 10 storeys. Below this height, the vertical load transfer dominates the required mass. And this is more or less equal for all systems. However, the structure itself is only one part of the sustainability of a building. It is therefore important that a structure fits into the overall sustainability concept of a building and project. This explicitly means working together with a sustainable architectural design, which requires an interdisciplinary cooperation of all parties involved.

Sustainable, local material procurement is also an important point for a sustainable building. For the MSP, however, this is a major challenge. The city of Maun lies in a very secluded area, which means that most of the building materials have to be transported over long distances. Especially the sustainable timber procurement is challenging, often it is imported from neighbouring countries, compare chapter 3.3.4. One solution can be the development and application of bamboo as construction timber. There is already research into the use of glue-laminated bamboo (glubam) for structural applications.[154] But it certainly needs more study and development for widespread use.

[153] Refer to El khouli et al. 2014, p.48
[154] Refer to Xiao et al. 2014

The only true local building material is earth or clay. A structure made of earth is quite sustainable, but less than expected in terms of the LCA. In addition, the implementation of a high-rise earthen structure is very challenging due to the lack of strength. This requires the development of a detailed design concept. In the author's opinion, such an earthen structure is thus feasible, but not suitable for widespread construction. A realization of an 8-storey earth structure would therefore be merely a prestige project.

In summary, for a sustainable structure, a wooden skeleton structure is needed. Which is sourced from sustainable forestry. Furthermore, the structure must fit into the sustainability concept of the entire project and architectural design.

6 Static analysis of the favourite design

After various designs have been developed and evaluated in the last chapters, this one now examines the favourite design, the wooden diagrid structure, in more detail. The investigation is intended to confirm the structural and constructional feasibility of the design. For this purpose, the structural analysis software Dlubal RFEM is used to set up a system analysis in order to verify the assumed cross-sections and dimensions of the pre-dimensioning. This is followed by a detailing of the main connections to prove their feasibility as well.

The calculation files of the structural analysis in RFEM are digitally attached to the appendix of this thesis, refer to Appendix B2 and B3.

6.1 System statics

The system statics is set up as a 3D model type, thus the three-dimensional load-bearing behaviour of the tube system can be represented. Furthermore, the use of the finite element software RFEM allows the implementation not only of the members but also of the slabs. Figure 34 shows the final visualization of the analysis model.

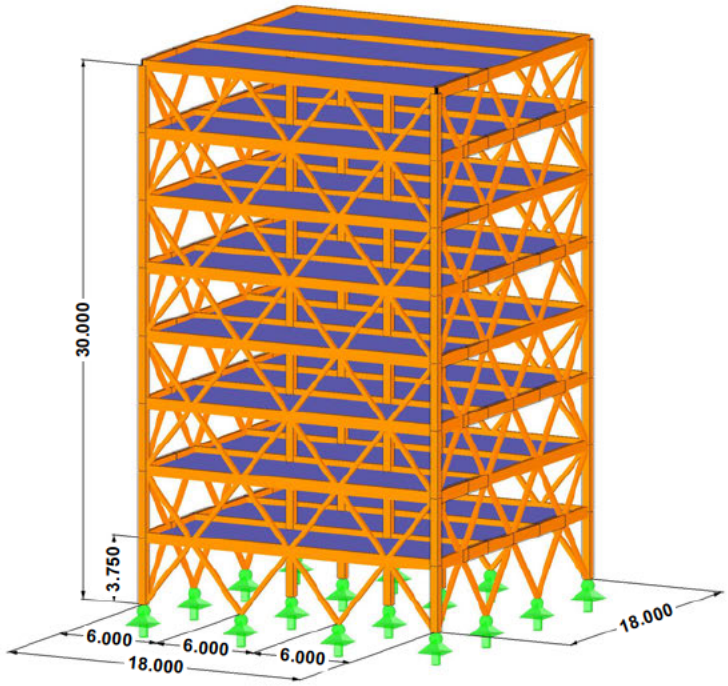


Figure 34 - Visualization of the analysis model

The current version of the Eurocode is chosen as standard, with the DIN EN 1990 as the basis for structural design. Therefore, the load assumptions are guided by DIN EN 1991. For timber construction, in particular for the design, DIN EN 1995 applies. The German National Annex is taken into account.

In the ultimate limit state (ULS), the load combinations are calculated on the basis of Theory II. Ord., taking into account pre-deformations and pre-curvature. For the serviceability limit state (SLS), the deflection is determined separately for each load case under Theory I. Ord., the results are then superposed for the evaluation.

6.1.1 Model structure

The model structure with nodes, sizes, members and planes is based on the design in chapter 4.5, see Figure 34. For the member structure, glulam GL24h is chosen as material. The cross-sections of the members themselves are defined as in the pre-dimensioning (Table 10).

Table 10 - Cross-sections of the analysis model

Cross-sections		
Nr.	Name	b x h
1	Diagrid Bracing	260x240
2	Diagrid Spanrel Beam	260x620
3	Diagrid Corner Column	420x420
4	Beam	260x620
5	Pendulum Column	420x420

The moment joints of the beams, spandrel beams and corner columns are given a fictive stiffness of 10kNm/rad to stabilize the system. In addition, the stiffness of the axial connection of the bracings is modelled. Thus, the influence of the flexibility of the connections is taken into account in the determination of the internal forces. The estimation of the spring stiffness is based on the internal forces of the pre-dimensioning.

In addition, the stiffnesses for the ULS under Theory II. Ord. must be reduced. For the SLS, in contrast, the mean values of the stiffness characteristics may be used.

For the cross laminated timber (CLT) slabs, softwood C24 is defined as material. However, it is only used to determine the dead weight of the slab, since the stiffness of the orthotopic slab is defined directly by the stiffness matrix. The weight of the material is manipulated according to the weight for CLT panels (γ=5,5 kN/m³). The stiffness matrix of the CLT panel is set up using the data in proHolz Austria[155]. When setting up the stiffness matrix, it should be noted that the torsional stiffness $D_{3,3}$ of CLT panels tends to be overestimated, which is why it is reduced by a factor $k_{drill} \approx 0{,}65$. For the shear stiffness due to stress $v_{x,y}$ and $v_{y,x}$ ($D_{4,4}$, $D_{5,5}$), a shear correction coefficient shall be considered. For rectangular cross-sections it is known to be $\frac{1}{K_z} = \frac{5}{6}$. For five-layer CLT panel a shear correction coefficient between $0{,}18 \leq \frac{1}{K_z} \leq 0{,}2$ can be chosen, based on standard values for CLT. Since CLT is an inhomogeneous material, the shear stiffness under shear stress ($D_{8,8}$) must also be reduced compared to homogeneous materials,

[155] Refer to Wallner-Novak et al. 2013, pp. 37 and 42

an empirical factor of 0.75 is chosen. All the slabs are modelled articulated for each field, as intended in the design. However, the eccentricity due to the support on the beams is not shown. Simplified, the slabs are connected to the gravity lines of the beams. In the same way, openings for staircases and elevators are not shown. They are not defined yet and can be implemented later.

As in the design, the entire structure is supported by articulated node bearings, see Figure 34.

Estimation of the stiffness for the bracing connection

Steel plate-timber connection with pin dowels

Verification method according DIN EN 1995 German national annex

Load:

Max. normal force: $N_d = 456{,}07\text{kN}$

Force/fiber angle: $\alpha = 0°$

Components:

Pin dowel: $\emptyset 16, S235$

Steel plate: $2x\ t = 15\text{mm}, S235$ → 4 shear planes per pin dowel

Diagrid bracing: $GL24h, \rho_k = 380\text{kg}$

Load-bearing capacity per shear joint:

$F_{v,Rd} = 11{,}44\text{kN}$

with $F_{v,Rk,C24} = 15{,}0\text{kN};\ k_{GL25h} = 1{,}049;\ k_{mod} = 0{,}8;\ \gamma_M = 1{,}1$

Number of pin dowls:

$n_{req} = \frac{N_d}{4*F_{v,Rd}} = 9{,}96$ and $n_{ef} = n^{0,9} * \sqrt[4]{\frac{a_1}{13*\emptyset}} * 4$ with 4 rows and $a_1 = 5\emptyset$

with $n = 4 \rightarrow n_{ef} = 10{,}969 \geq n_{req}$ → 4 rows with 4 rod anchors each, $\sum n = 16$

Stiffness of the Connection:

Rod anchor and steel plate: $K_{ser} = 2 * \frac{\rho_{mean}{}^{1,5} * \emptyset}{23} = 12710\ \text{N/mm}$

Resulting Stiffness: $K_{res} = \sum n * K_{ser} = 203360\ \text{N/mm}$

System stiffnesses of the member structure:

According DIN EN 1995 German national annex

Stiffness ULS: $\gamma_M = 1{,}3$

$E_d = \frac{E_{mean}}{\gamma_M} = 892{,}3\text{kN/cm}^2$

$G_d = \frac{G_{mean}}{\gamma_M} = 55{,}38\text{kN/cm}^2$

$K_d = \frac{2}{3} * \frac{K_{res}}{\gamma_M} = 104287\ \text{N/mm}$

Stiffness SLS:

$E_{mean} = 1160\text{kN/cm}^2$

$G_{mean} = 67{,}0\text{kN/cm}^2$

$K_{res} = 203360\ \text{N/mm}$

Stiffness matrix of the CTL slab

According to proHolz Austria

Cross Section:

CLT 180 DL5s [mm]: 40/30/40/30/40

Sectional area: $A_{n,0} = 1200{,}0\ \frac{cm^2}{m}$ $A_{n,90} = 600{,}0\ \frac{cm^2}{m}$

Moment of inertia: $I_{n,0} = 40800{,}0 \frac{cm^4}{m}$ $I_{n,90} = 7800 \frac{cm^4}{m}$

Material: Panels C24

Stiffness: $E_{0,mean} = 11000 N/mm^2$ $E_{90,mean} = 370 N/mm^2$

$G_{mean} = 690 N/mm^2$ $\vartheta_{x,y} = \vartheta_{y,x} = 0$

Stiffness matrix:

Bending and torsion:

$D_{1,1} = \frac{E_{0,mean} * I_{n,0}}{1 - \vartheta_{x,y} * \vartheta_{y,x}} = 4488\ \frac{kNm^2}{m}$ $D_{2,2} = \frac{E_{0,mean} * I_{n,90}}{1 - \vartheta_{x,y} * \vartheta_{y,x}} = 858 \frac{kNm^2}{m}$

$D_{1,2} = \sqrt{\vartheta_{x,y} * \vartheta_{y,x} * D_{1,1} * D_{2,2}} = 0 \frac{kNm^2}{m}$ $D_{3,3} = k_{drill} * G_{mean} \frac{b * d^3}{12} = 858 \frac{kNm^2}{m}$

Shear:

$D_{4,4} = \frac{1}{K_{0,z}} * G_{mean} * A_{n,0} = 16550 \frac{kN}{m}$ $D_{5,5} = \frac{1}{K_{90,z}} * G_{mean} * A_{n,90} = 8280 \frac{kN}{m}$

Membrane:

$D_{6,6} = E_{0,mean} * A_{n,0} = 1320000\ \frac{kN}{m}$ $D_{7,7} = E_{0,mean} * A_{n,90} = 1320000 \frac{kN}{m}$

$D_{6,7} = \vartheta_{y,x} * D_{6,6} = 0\ \frac{kN}{m}$ $D_{8,8} = 0{,}75 * G_{mean} * b * d = 93150 \frac{kN}{m}$

6.1.2 Loads and combinations

For the load assumptions, the assumptions from chapter 4.1.1 are adopted and adapted. For the ULS analysis, load combinations are generated using the three load cases dead load, live load and wind load, including partial safety factors and combination coefficients. For each load combination, imperfections are also taken into account. They are needed to consider the stability issue.

Load case 1: Dead weight (G)

The dead weight of the construction is taken into account by the program itself via the cross-sections and the choice of materials.

Load case 2: Live load (Q)

The sum of the live loads was assumed to be 5kN/m², including the dead weight of the ceiling construction. This can now be removed, since it is included in the dead weight. Accordingly, 4kN/m² remain as live load.

Life load:

Total vertical load: $\sum q_k = 5kN/m^2$

Dead weight CLT slab: $g_k = 1{,}0kN/m^2$ with $\gamma = 5{,}5kN/m^3$, $t = 18cm$

Life Load: $q_k = \sum q_k - g_k = 4{,}0kN/m^2$

Load case 3: Wind load (W) in +X

For the wind loads, the assumption that the gust velocity pressure is constant over the entire building height, is adopted from the pre-dimensioning is. In addition to the wind load in leeward and windward, the wind loads on the transverse directions as well as on the roof are included. The horizontal wind loads are transferred from the facade directly into the floor slabs. Accordingly, the loads are summed up over the influence width and plotted as a line load. The vertical wind loads on the roof, on the other hand, are retained as distributed loads. The geometries for the loaded wall and roof zones are taken into account, according to the DIN EN 1991. Since the building is double symmetrical, it is sufficient to investigate only one wind direction.

Wind load:

Gust velocity pressure: $q_p(z) = 0{,}81\frac{kN}{m^2} = \text{const.}$

Influence width slab: $a = 3{,}75m$

Wind load on surfaces: $w_k = q_p(z) * c_{pe}$

Walls:

Zone A: $c_{pe,10,A} = -1{,}2\, w_{A,k} = -0{,}972\frac{kN}{m^2}$ → $w_{A,k} = -3{,}645\frac{kN}{m}$

Zone B: $c_{pe,10,B} = -0{,}8\, w_{B,k} = -0{,}648\frac{kN}{m^2}$ → $w_{B,k} = -2{,}43\frac{kN}{m}$

Zone C: Does not occur, due to the geometry

Zone D: $c_{pe,10,D} = 0{,}8 \quad w_{D,k} = 0{,}648\frac{kN}{m^2}$ → $w_{D,k} = 2{,}43\frac{kN}{m}$

Zone E: $c_{pe,10,E} = -0{,}5\, w_{E,k} = -0{,}405\frac{kN}{m^2}$ → $w_{E,k} = -1{,}519\frac{kN}{m}$

Roof:

Zone F: $c_{pe,10,F} = -1{,}8\, w_{F,k} = -1{,}458\frac{kN}{m^2}$

Zone G: $c_{pe,10,G} = -1{,}2\, w_{G,k} = -0{,}972\frac{kN}{m^2}$

Zone H: $c_{pe,10,H} = -0{,}7\, w_{H,k} = -0{,}567\frac{kN}{m^2}$

Zone I: $c_{pe,10,I} = -0{,}6\; w_{I,k} = -0{,}486\frac{kN}{m^2}$

Load case 4: Imperfections (I)

Stability is verified by means of a stress analysis under Theory II. Ord. For this purpose, imperfections in the form of initial inclination and pre-curvature must be considered. The imperfections are specified in DIN EN 1995-1-1. The inclination is considered over the total

building height and is imposed in wind direction on all vertical columns (corner columns and pendulum columns). The pre-curvature is taken into account separately for each member.

Imperfections:

Pre-curvature: $e = 0{,}0025 * l$ $1/l = 400$

Inclination: $\Phi = 0{,}005 * \sqrt{\frac{5}{h}}$ $1/\Phi = 489{,}9$ with $h = 30\text{m}$

Load combinations:

The load combinations for the ULS are summarized below.

Load combinations:

LK1: $1{,}35 * G_k + I$

LK2: $1{,}35 * G_k + 1{,}5 * Q_k + I$

LK3: $1{,}35 * G_k + 1{,}5 * W_k + I$

LK4: $1{,}35 * G_k + 1{,}5 * Q_k + 1{,}5 * 0{,}6 * W_k + I$

LK5: $1{,}35 * G_k + 1{,}5 * W_k + 1{,}5 * 0{,}7 * Q_k + I$

6.1.3 Results

In a simplified way, only the maximum internal forces for the different cross-sections and the slabs are plotted here as results. They are not linked to a specific position or load combination. For more detailed investigations, please refer to the calculation itself. The numbering of the cross-sections is based on Table 10.

Table 11 - Maximum internal forces of the cross-sections

	N [kN]		Vy [kN]		Vz [kN]		My [kNm]		Mz [kNm]		Mt [kNm]	
	max	min	max	min	max	min	max	min	max	min	max	min
1	154,6	-351,7	0,0	0,0	5,7	-5,7	7,6	0,0	0,0	0,0	0,1	-0,1
2	165,1	-102,8	9,6	-9,6	54,7	-54,7	139,8	-1,1	2,3	-1,0	0,2	-0,2
3	0,0	-1888,9	0,6	-0,6	11,7	-14,6	6,1	-8,1	1,8	-1,8	0,1	-0,1
4	13,8	-11,3	2,9	-2,9	92,6	-92,6	164,9	-2,4	0,7	-0,9	0,0	0,0
5	0,0	-2221,9	0,0	0,0	26,1	-26,1	26,2	0,0	0,0	0,0	0,0	0,0

Table 12 - Maximum internal forces of the slabs

plate stress	mx [KNm/m]		my [kNm/m]		mxy [kNm/m]		vx [kN/m]		vz [kN/m]	
	max	min	max	min	max	min	max	min	max	min
	40,2	-22,4	7,4	-14,5	4,5	-4,5	24,7	-25,2	25,0	-25,0
diaphragm stress	nx [kN/m]		ny [kN/m]		nxy [kN/m]					
	max	min	max	min	max	min				
	64,2	-64,3	64,3	-45,3	41,4	-41,4				

For the internal forces of the slabs, it should be noted that singularities occur for the transverse and the diaphragm forces at the support points, especially above the columns. This was already been considered when extracting the maximum internal forces.

6.1.4 Dimensioning of the members ULS

The design of the members is done using the RFEM module RF-TIMBER Pro. The module performs the design for all members and load combinations. It also considers the service classes and the load action durations that are responsible for the reduction factor k_{mod}. Service class SECL 2 is chosen for all components, since it is assumed that loggias and openings are integrated into the building, which corresponds to a roofed open structure. The load duration class LDC depends on the different loads and is determined by the program itself.

As already mentioned several times, the stability check is performed with the help of a stress verification according to Theory II. Order and imperfections. Independently of this, however, the check against torsional buckling of bending members must also be performed. The RF-TIMBER Pro module takes this into account by defining effective member lengths. All beams and spandrel beams are continuously supported in the compression chord, so their coefficient k_{crit} can be assumed to be 1.0.

Table 13 shows the maximum utilization rates for the different cross-sections. Compared to the pre-dimensioning, noticeably smaller utilization values are achieved. Nevertheless, the cross-sections are not optimized here, since the deformations are already critical, as shown in the SLS (6.1.6). A reduction of the cross-sections would lead to a softer system and thus to further deformations.

Table 13 - Maximum utilization of the cross-sections

Cross-sections			Max. Utilisation
Nr.	Name	b x h	
1	Diagrid Bracing	260x240	0,51
2	Diagrid Spanrel Beam	260x620	0,59
3	Diagrid Corner Column	420x420	0,68
4	Beam	260x620	0,67
5	Pendulum Column	420x420	0,87

6.1.5 Dimensioning of the slabs ULS

The design of the CLT ceiling is realised using the internal forces of the slabs. Since the structural design of CLT slabs has not yet been included in the DIN EN 1995, the design is based on the guidelines published by proHolz Austria[156]. In general, this guideline is also based on the Eurocode and the associated partial safety factor concept.

[156] Wallner-Novak et al. 2013

Material strength characteristics

Characteristic values for CLT are not standardized and result from the technical approval. However, they lie within a certain range of variation, and the guideline specifies reliable values which are now also used.[157] As it is usual in timber construction, the permissible strengths are reduced by a safety value and modification coefficient. According to the national annex, $\gamma_M = 1{,}3$ is applied as the partial safety factor. The modification coefficient $k_{mod} = 0{,}8$ results from the service class SECL2 and the load duration class for the live load (LDC medium-term). It is possible to increase the component resistance with a system coefficient k_{sys}, since the load is transferred simultaneously via several components or boards. However, this is not done here.

Dimensioning of the slabs - Material strength characteristics

According to proHolz Austria

Tensile strength:	$f_{t,0,k} = 14{,}0 \frac{N}{mm^2}$	→	$f_{t,0,d} = 8{,}62 \frac{N}{mm^2}$
Compressive strength:	$f_{c,0,k} = 21{,}0 \frac{N}{mm^2}$	→	$f_{c,0,d} = 12{,}92 \frac{N}{mm^2}$
Bending strength:	$f_{m,k} = 24{,}0 \frac{N}{mm^2}$	→	$f_{m,d} = 14{,}76 \frac{N}{mm^2}$
Shear strength:	$f_{V,k} = 2{,}5 \frac{N}{mm^2}$	→	$f_{V,d} = 1{,}54 \frac{N}{mm^2}$
Rolling shear strength:	$f_{V,R,k} = 1{,}10 \frac{N}{mm^2}$	→	$f_{V,R,d} = 0{,}68 \frac{N}{mm^2}$
Torsional strength:	$f_{T,0,k} = 2{,}5 \frac{N}{mm^2}$	→	$f_{T,0,d} = 1{,}54 \frac{N}{mm^2}$
Diaphragm shear strength:	$f_{V,S,k} = 5{,}0 \frac{N}{mm^2}$	→	$f_{V,S,d} = 3{,}08 \frac{N}{mm^2}$

Cross-section values

In addition to the material properties, the cross-section values resulting from the slab structure are required for the design. As already explained, net cross-section values occur due to the orthotropic component. A cross-laminated timber panel, CLT 180 DL5s [mm]: 40/30/40/30/40, was chosen for the floor structure. This results in the following cross-section values:

Dimensioning of the slabs - Cross-section values:

Sectional area:	$A_{n,0} = 1200{,}0 \frac{cm^2}{m}$	$A_{n,90} = 600{,}0 \frac{cm^2}{m}$
Moment of inertia:	$I_{n,0} = 40800{,}0 \frac{cm^4}{m}$	$I_{n,90} = 7800{,}0 \frac{cm^4}{m}$
Section modulus:	$W_{n,0} = 4533{,}3 \frac{cm^3}{m}$	$W_{n,90} = 1560{,}0 \frac{cm^3}{m}$
Static moment:	$S_{n,0,R} = 2800{,}0 \frac{cm^3}{m}$	$S_{n,90,R} = 1050{,}0 \frac{cm^3}{m}$
Torsional resistance:	$W_T = 9611{,}80 \frac{cm^3}{m}$	, simplified for a homogeneous rectangle

[157] Refer to Wallner-Novak et al. 2013, pp. 22-24

Verification

The ULS verifications are performed at the internal forces level, with the maximum forces according Table 12. In general, both load directions 0° and 90° can be analysed separately, since it is assumed that only the boards in the longitudinal direction participate in the load transfer. The axis X of the panels in the static model corresponds therefore with the layer 0°, parallel to the deck layer. Accordingly, axis Y corresponds to the 90° layer.

Dimensioning of the slabs - Verification:

According to proHolz Austria

Tension:

Layer 0°: $n_{x,d} = 64{,}2\,\frac{kN}{m}$ $\sigma_{t,0,d} = \frac{n_{x,d}}{A_{n,0}} = 0{,}054\,\frac{kN}{cm^2}$ $\frac{\sigma_{t,0,d}}{f_{t,0,d}} = 0{,}06$

Layer 90°: $n_{y,d} = 64{,}3\,\frac{kN}{m}$ $\sigma_{t,90,d} = \frac{n_{y,d}}{A_{n,90}} = 0{,}11\,\frac{kN}{cm^2}$ $\frac{\sigma_{t,90,d}}{f_{t,0,d}} = 0{,}124$

Compression:

Layer 0°: $n_{x,d} = 64{,}3\,\frac{kN}{m}$ $\sigma_{c,0,d} = \frac{n_{x,d}}{A_{n,0}} = 0{,}054\,\frac{kN}{cm^2}$ $\frac{\sigma_{c,0,d}}{f_{c,0,d}} = 0{,}04$

Layer 90°: $n_{y,d} = 45{,}3\,\frac{kN}{m}$ $\sigma_{c,90,d} = \frac{n_{y,d}}{A_{n,90}} = 0{,}076\,\frac{kN}{cm^2}$ $\frac{\sigma_{c,90,d}}{f_{c,0,d}} = 0{,}06$

Bending:

Layer 0°: $\max m_{x,d} = 40{,}2\,\frac{kNm}{m}$ $\sigma_{m,0,d} = \frac{m_{x,d}}{W_{n,0}} = 0{,}88\,\frac{kN}{cm^2}$ $\frac{\sigma_{m,0,d}}{f_{m,d}} = 0{,}60$

Layer 90°: $\max m_{y,d} = 14{,}5\,\frac{kNm}{m}$ $\sigma_{m,90,d} = \frac{m_{y,d}}{W_{n,90}} = 0{,}93\,\frac{kN}{cm^2}$ $\frac{\sigma_{m,90,d}}{f_{m,d}} = 0{,}63$

Shear due to plate stress:

Layer 0°: $\max v_{x,d} = 25{,}2\,\frac{kN}{m}$ $\tau_{v,0,R,d} = \frac{v_{x,d} * S_{n,0,R}}{I_{n,0} * b} = 0{,}02\,\frac{kN}{cm^2}$ $\frac{\tau_{v,0,R,d}}{f_{V,R,d}} = 0{,}25$

Layer 90°: $\max v_{y,d} = 25{,}0\,\frac{kNm}{m}$ $\tau_{v,90,R,d} = \frac{v_{y,d} * S_{n,90,R}}{I_{n,90} * b} = 0{,}03\,\frac{kN}{cm^2}$ $\frac{\tau_{v,90,R,d}}{f_{V,R,d}} = 0{,}5$

Torsion:

$\max m_{xy,d} = 4{,}5\,\frac{kN}{m}$ $\tau_{T,d} = \frac{m_{xy,d}}{W_T} = 0{,}047\,\frac{kN}{cm^2}$ $\frac{\tau_{T,d}}{f_{T,0,d}} = 0{,}3$

Shear due to diaphragm stress:

Shear failure of the boards along a joint:

$\max n_{xy,d} = 41{,}4\,\frac{kN}{m}$ $\tau_{V,S,d} = \frac{n_{xy,d}}{\min(A_n)} = 0{,}07\,\frac{kN}{cm^2}$ $\frac{\tau_{V,S,d}}{f_{V,S,d}} = 0{,}224$

Shear failure of the entire plane:

$\max n_{xy,d} = 41{,}4\,\frac{kN}{m}$ $\tau_{V,d} = \frac{n_{xy,d}}{A_{brutto}} = 0{,}023\,\frac{kN}{cm^2}$ $\frac{\tau_{V,d}}{f_{V,d}} = 0{,}15$

The normal forces and the bending moments can be analysed separately, since the maxima do not occur at the same locations. The maximum normal forces occur near the supports, while the maximum moment occurs in the centre of the plate.

The shear force failure due to bending of cross laminated timber is usually determined by rolling shear failure. This refers to a failure of the transverse layers which, as pure distance holders, are only subjected to shear under flexural load. The consequence of this is a fracture

tangential to an annual growth ring of the transverse layers as soon as the rolling shear strength is exceeded.

The torsional stress results from the bending torsional moments. Simplified, the torsional resistance is determined analogously to a homogeneous rectangular cross section.

In addition to the shear failure due to plate stress, the shear failure due to diaphragm stress must also be investigated. This can result in shearing of the boards along a joint or in shear failure of the entire sheet. The failure of the glued surfaces is not investigated in this context.

Summary

The maximum utilization of the slab occurs as a result of the bending stress. It is 0.6 for the 0° layer and 0.63 for 90° layer. The utilization thus correlates approximately with the pre-dimensioning. Further optimization of the slab is not recommended here because, as with the bars, the deformation is already in the critical range, compare to the SLS.

6.1.6 Serviceability limit state SLS

For the analysis of the SLS, a separate structural model is set up with adjusted stiffness and without imperfections. The deformations are determined by calculation according to Theory I. Ord. Both the total horizontal deformation and the deflection of the slabs and beams are checked. An estimation of the vibration behaviour of the floor slabs is realized with the help of a simple hand calculation.

Total horizontal deformation

The maximum horizontal deformation results from a characteristic load combination with wind as main load. The maximum horizontal deformation is $u_x = 12{,}0\text{mm}$, which is very low considering the height of the building.

Vertical deflection

For the determination of the deflection, only the vertical loads are considered and superposed. Furthermore, only the deformations of the first-floor slab including the associated beams are analysed. This is since the net deflection is wanted. In the upper floors, the compression of the vertical load-bearing elements adds up, so that they have a significant impact on the deformation. For example, the compression of the pendulum columns, under dead load and live load, adds up to $u_z = 12{,}5\text{mm}$.

For the verification of the deflection, the elastic initial deflection w_{inst} and the final deflection w_{fin} with creep deformations should be investigated, according to DIN EN 1995. No superelevation of the components will be considered. The initial deflection is determined under the characteristic load combination. For the final deflection, the creep deformation under quasi-continuous load must also be taken into account.

Deflections

According to DIN EN 1995-1-1

Deflections:		**Beam:**	**Slab:**
Dead load:	w_G	$w_G = 3{,}4\text{mm}$	$w_G = 6{,}6\text{mm}$
Life load:	w_Q	$w_Q = 11{,}4\text{mm}$	$w_Q = 25{,}0\text{mm}$
Initial deflection:	$w_{inst} = w_G + w_Q$	$w_{inst} = 14{,}8\text{mm}$	$w_{inst} = 31{,}6\text{mm}$
Quasi-continuous:	$w_{qs} = w_G + \psi_2 * w_Q$	$w_{qs} = 6{,}82\text{m}$	$w_{qs} = 14{,}1\text{mm}$
Creep deflection:	$w_{creep} = k_{def} * w_{qs}$	$w_{creep} = 4{,}09\text{mm}$	$w_{creep} = 8{,}46\text{mm}$
Final deflection:	$w_{fin} = w_{inst} + w_{creep}$	$w_{fin} = 18{,}89\text{mm}$	$w_{fin} = 40{,}06\text{mm}$

Limits of deflection:

Span: $l = 6{,}0\text{m}$ for beams and slabs

Initial deflection: $w_{inst} \leq \frac{l}{300} = 20\text{mm}$

Final deflection: $w_{fin} \leq \frac{l}{200} = 30\text{mm}$

The standard gives recommended deflection limits depending on the span. The deflection of the beams, on one hand, is within the limits for both the initial state and the final state. The deformation of the slabs, on the other hand, exceeds the values. However, it must be noted that the deformations are strongly influenced by the interaction of the components and the system. If the deformation of the slabs is nevertheless classified as too large, a stronger and thus stiffer slab structure must be chosen. Overall, the vertical deformations are already in the critical range. The stiffness of the vertical load-bearing elements should therefore not be reduced further.

Vibration analysis

For structural systems and components, the DIN EN 1995 requires an investigation of whether vibrations occur, as a result of frequent loads, that impair the function or the use of the element. This applies in particular to residential slabs. The vibration behaviour can be determined by measurements or estimated by calculations. The most important parameters are the first natural frequency, the stiffness and the damping behaviour of the slab. To avoid resonance, a sufficient distance between the excitation frequency and the first natural frequency should be aimed for. Since an excitation frequency of about 4.00 Hz is generated as a result of simple walking, the DIN EN 1995 requires 8.00 Hz as the minimum value of the first natural frequency.[158] As an alternative to the regulation in the Eurocode, the vibration verification for the slabs can be carried out according to Hamm and Richter. The slabs are therefore divided into three classes with respect to their vibration behaviour as shown in Figure 35. The first

[158] Refer to Wallner-Novak et al. 2013, p. 74

natural frequency and the stiffness of the slab, in the form of the static deflection, are decisive for the classification.

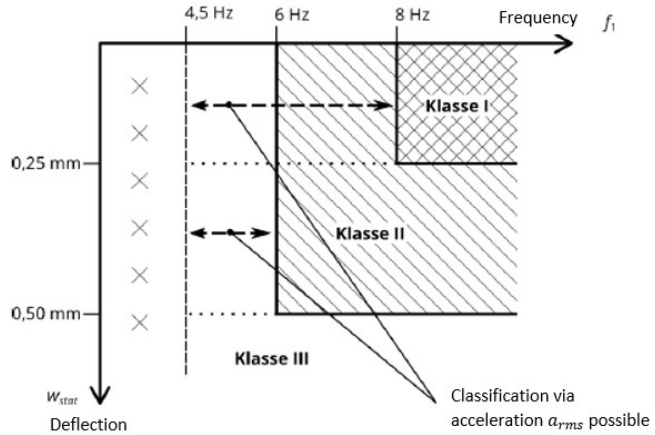


Figure 35 - Classification of vibration behaviour[159]

For the determination of the natural frequency and the deflection, the slab system can be reduced to a single-span beam with uniformly distributed mass for manual calculation. The influence of the transverse load-bearing effect can be taken into account via factors, as long as the requirement $\frac{EI_{90}}{EI_0} < 0{,}05$ is fulfilled. The following verification is carried out accordingly without further derivation, additional information can be taken from proHolz Austria[160].

Proof of vibration – slab system

According to Hamm and Richter; proHolz Austria

Slab system

Equivalent system: single-span beam with uniformly distributed mass and influence of the transverse load-bearing effect

Span/Width/Thickness: $l = 6m$ $b = 18m$ $t = 0{,}18m$

Stiffness: $EI_0 = 4488000\frac{Nm^2}{m}$ $EI_{90} = 858000\frac{Nm^2}{m}$

Vibration mass: $m = \rho_{mean} * t = 81kg/m^2$ with $\rho_{mean} = 450kg/m^3$

Natural frequency:

$$f_1 = \frac{\pi}{2*l^2} * \sqrt{\frac{EI_0}{m}} * k_{trans} = 10{,}373Hz$$

Influence of the transverse effect: $k_{trans} = \sqrt{1 + \left(\frac{l}{b}^2 + \frac{l}{b}^4\right) * \frac{EI_{90}}{EI_0}} = 1{,}01$

Static deflection:

$$w_{stat} = \frac{1000N*l^3}{48*EI_0} * \frac{1}{b_f} = 0{,}278mm$$

Influence of the transverse effect: $b_f = \frac{l}{1{,}1} * \sqrt[4]{\frac{EI_{90}}{EI_0}} = 3{,}6$

[159] Refer to Wallner-Novak et al. 2013, p. 79

[160] Refer to Wallner-Novak et al. 2013, p. 74-85

Using the first natural frequency and the static deflection, the slab of the design can be classified in vibration class II, based on Figure 35. Typical applications for vibration class II are slabs within a single unit or in a single-family home. More desirable, however, would be a classification in class I, which is used for ceilings between different usage units, such as apartment separation ceilings or office ceilings.[161] However, the deflection of the slab is insufficient for this. To improve this, a thicker and stiffer slab structure may be chosen, as already mentioned in the deflection analysis. The criterion of the natural frequency is not critical and clearly exceeds the required minimum value of DIN EN 1995. However, it should be noted that just the ceiling dead weight was taken into account for the vibration mass and not the floor build-up. An increase in the mass results in a reduction of the natural frequency. This must be considered when further optimizing the load-bearing system.

6.2 Detail statics

After examining the system statics of the diagrid structure in the previous section, the associated details are studied in this chapter. This concerns in particular the details of the member structure. The feasibility of the connections for the bracings, spandrel beams and beams are verified with pin dowels and nails. The load transfer in the joints of the vertical columns can be realised by means of pressure contact. Last but not least, options for the detailing of the cross laminated timber slab are shown.

For the feasibility, only the load-bearing capacity of the fasteners is examined. Additional steel components, such as slotted plates and head plates, are not verified in detail within the context of this design. Due to their high strength, it is assumed that their realisation is not critical. The same applies to the connection of the steel components with welds. As in the system statics, service class SECL 2 is assumed for the connections. The load duration class LDC is set to medium-term, due to the live load. The two specifications result in a modification coefficient of $k_{mod} = 0{,}8$.

The details are shown individually in drawings. For merging the connections, 3D modelling is recommended to check the implementation.

6.2.1 Detail diagrid bracings

The detail of the bracings can be made as a steel plate-timber connection with pin dowels, as shown in Figure 36. The corresponding verification is shown below. The maximum load results from the normal force, refer to Table 11. The small vertical forces can be neglected. The minimum distances of the fasteners according to the DIN EN 1995 must be taken into account in the design; they are observed here accordingly. It should be noted that both tension and

[161] Refer to Wallner-Novak et al. 2013, p.80

compression forces occur in the bracings. The verification for block shear failure is not exclusively listed here, yet it is not critical.

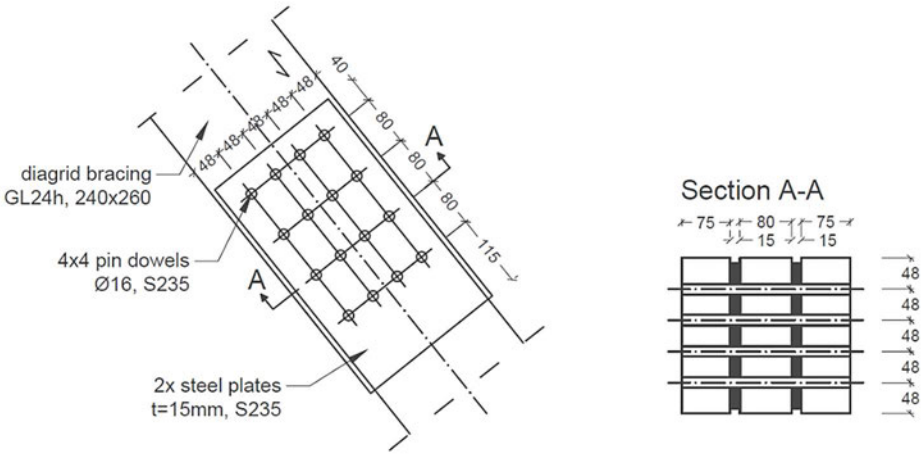


Figure 36 - Detail: Connection diagrid bracing

Connection diagrid bracing

Steel plate-timber connection with pin dowels

Verification method according to DIN EN 1995 German national annex

Load:

Internal forces: $N_d = 154{,}6\text{kN (tension)}/-351{,}7\text{kN (pressure)}$

Decisive load: $F_{Ed} = 351{,}7\text{kN}$

Force/fiber angle: $\alpha = 0°$

Components and environmental conditions:

Pin dowel: $\emptyset 16, S235$

Diagrid bracing: $GL24h, \rho_k = 380\text{kg}$

Steel plate: $2x\ t = 15\text{mm}, S235$

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}1$

Load-bearing capacity per shear joint:

$$F_{v,Rk} = \sqrt{2} * \sqrt{2 * M_{y,Rk} * f_{h,k,\alpha} * \emptyset} = 15633\ \text{N}$$

Plastic moment pin dowel: $M_{y,Rk} = 145927\ \text{Nmm}$

Hole bearing strength timber: $f_{h,k,\alpha} = 26{,}17\text{N/mm}^2$

Minimum timber thickness:

$$t_{req} = 1{,}15 * 4 * \sqrt{\frac{M_{y,Rk}}{f_{h,k,\alpha} * \emptyset}} = 85{,}87\text{mm} \leq t_{min} = 75\text{mm} \qquad \rightarrow \text{Reduction: } \frac{t_{min}}{t_{req}}$$

Effective number of pins in a row/ number of shear joints / number of rows:

$$\text{with } n = 4: \quad n_{ef} = n^{0{,}9} * \sqrt[4]{\frac{a_1}{13\emptyset}} = 2{,}74 \qquad s = 4 \qquad r = 4$$

Loading capacity of the joint:

$$F_{Rd} = \frac{t_{min}}{t_{req}} * r * s * n_{ef} * F_{v,Rk} * \frac{k_{mod}}{\gamma_M} = 435{,}34\text{kN} \leq F_{Ed} = 351{,}7\text{kN}$$

In combination with the connection of the spandrel beams, the detail forms the truss node for the diagrid structure. In the node itself, the steel plates of the different connections can be joined by means of welds. The same applies to the connection detail to the corner column.

6.2.2 Details diagrid spandrel beams

The spandrel beams are also connected with a steel plate-timber connection and pin dowels, see Figure 37. The connection is realized with two inner steel plates to enable the truss node in combination with the connection of the bracings. The corresponding verification is given below. For the design load, the maximum normal and shear forces of the spandrel beams, given in Table 11, are superposed in a simplified way. For the minimum distances, the values for $\alpha = 0°$ and $\alpha = 90°$ should be applied to be on the safe side, due to the superposition. These have already been taken into account in the design according to Figure 37. The verification for block shear failure is not listed here as it is not decisive. In addition, however, a forked bearing for a torsional moment of $M_{tor,d} = M_{Ed}/80$ is to be designed according to DIN EN 1995 German national annex for flexural members. The detail with the two steel plates inside is excellent for this. Without further proof, it is assumed that the load-bearing reserves of the pin dowels can transfer the additional moment.

The slotted plates of the spandrel beams are brought together in the truss nodes with the connections of the bracings. However, at every second floor, a direct load application into the corner columns occur. In this case, the load is transferred via the slotted plates into an end plate and via nails into the column. The calculation of the steel plate-timber connection with nails is shown below. The loads for the verification are significantly lower than the maximum internal forces of the beams. Therefore, they are taken directly from the calculation model at the appropriate locations. As for the pin dowel connections, the minimum distances are taken into account in the design of the connection, see Figure 38. The minimum timber thickness is given as well.

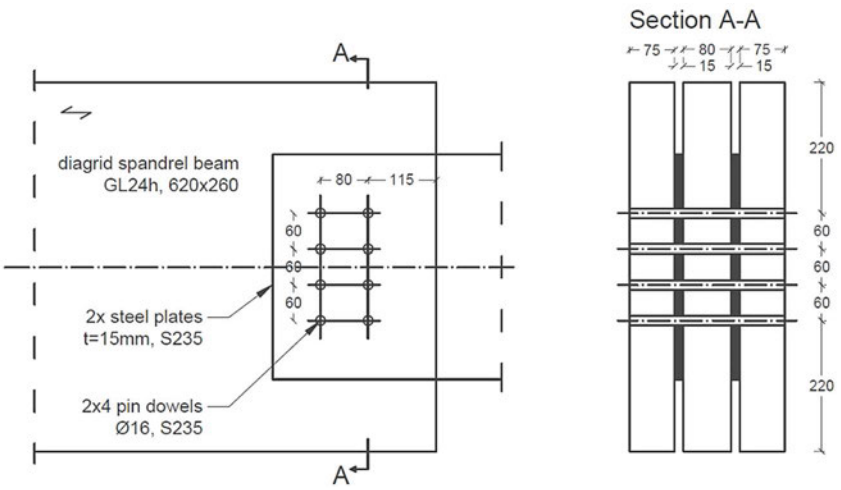


Figure 37 - Detail: Connection diagrid spandrel beam

Connection diagrid spandrel beam

Steel plate-timber connection with pin dowels

Verification method according to DIN EN 1995 German national annex

Load:

Internal forces: $N_d = 165{,}1kN\ (tension)/-102{,}8kN\ (pressure)$

$V_d = 54{,}7kN$

Decisive load: $F_{Ed} = 175{,}0kN$

Force/fiber angle: $\alpha = 18{,}2°$

Components and environmental conditions:

Pin dowel: $\emptyset 16, S235$

Diagrid spandrel beam: $GL24h, \rho_k = 380kg$

Steel plate: $2x\ t = 15mm, S235$

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}1$

Load-bearing capacity per shear joint:

$$F_{v,Rk} = \sqrt{2} * \sqrt{2 * M_{y,Rk} * f_{h,k,\alpha} * \emptyset} = 15203\ N$$

Plastic moment pin dowel: $M_{y,Rk} = 145927\ Nmm$

Hole bearing strength timber: $f_{h,k,\alpha} = 24{,}75N/mm^2$

Minimum timber thickness:

$$t_{req} = 1{,}15 * 4 * \sqrt{\frac{M_{y,Rk}}{f_{h,k,\alpha} * \emptyset}} = 88{,}3mm \leq t_{min} = 75mm \qquad \rightarrow \text{Reduction: } \frac{t_{min}}{t_{req}}$$

Effective number of pins in a row/ number of shear joints / number of rows:

with $n = 2$: $n_{ef} = n^{0,9} * \sqrt[4]{\frac{a_1}{13\emptyset}} = 1{,}47$ $\quad s = 4 \quad r = 4$

Loading capacity of the joint:

$$F_{Rd} = \frac{t_{min}}{t_{req}} * r * s * n_{ef} * F_{v,Rk} * \frac{k_{mod}}{\gamma_M} = 220{,}88kN \leq F_{Ed} = 175{,}0kN$$

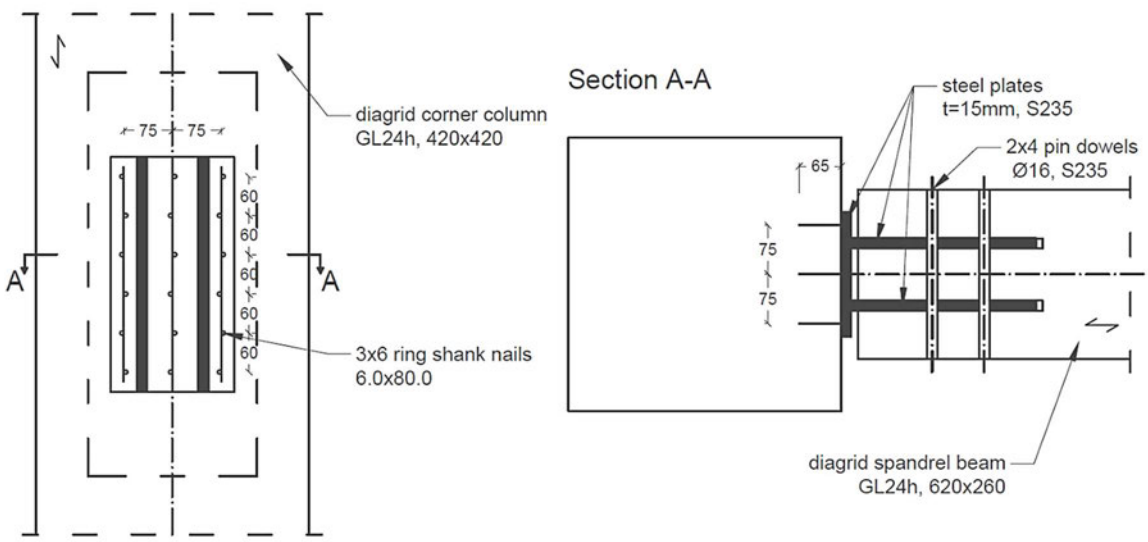


Figure 38 - Detail: Load application diagrid spandrel beam to corner column

Load application diagrid spandrel beam to corner column

Steel plate-timber connection with nails

Verification method according to DIN EN 1995 German national annex

Load:

Internal forces: $N_d = 22{,}1kN$ and $V_d = 17{,}2kN$

Decisive load: $F_{ax,Ed} = 22{,}1kN$ and $F_{V,Ed} = 17{,}2kN$

Components and environmental conditions:

Ring-shank nail: $6{,}0x80{,}0mm$; $\emptyset_h = 12mm$

load class 3C ; $f_u > 600N/mm^2$; not predrilled

Diagrid spandrel beam: GL24h, $\rho_k = 380kg$

Steel plate: $t = 15mm$, S235

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}1$

Shear off:

Load-bearing capacity per shear joint: $F_{v,Rk} = A * \sqrt{2 * M_{y,Rk} * f_{h,k} * \emptyset} = 2851N$

Plastic moment nail: $M_{y,Rk} = 18987\ Nmm$

Hole bearing strength timber: $f_{h,k} = 18{,}2N/mm^2$

Factor A: $A = 1{,}4$

Increase of the load capacity: $\Delta F_{v,Rk} = \min \begin{cases} 0{,}5 * F_{v,Rk} = 1425N \\ 0{,}25 * F_{ax,Rk} = 703N \end{cases} = 703N$

Effective number of nails: $n_{ef} = 18$

Loading capacity of the joint: $F_{V,Rd} = n_{ef} * (F_{v,Rk} + \Delta F_{v,Rk}) * \frac{k_{mod}}{\gamma_M} = 46{,}52kN$

$F_{V,Rd} = 46{,}52kN \leq F_{V,Ed} = 17{,}2kN$

Pull out:

Load-bearing capacity per nail: $F_{ax,Rk} = f_{ax,k} * \emptyset * t_{pen} = 2815N$

Pull-out resistance: $f_{ax,k} = 50 * 10^{-6} * \rho_k^2 = 7{,}22N/mm^2$

Penetration depth: $t_{pen} = 65mm$

Effective number of nails: $n_{ef} = 18$

Loading capacity of the joint: $F_{ax,Rd} = n_{ef} * F_{ax,Rk} * \frac{k_{mod}}{\gamma_M} = 36{,}85kN$

$F_{ax,Rd} = 36{,}85kN \leq 22{,}1kN$

Combined load:

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{V,Ed}}{F_{V,,Rd}}\right)^2 = 0{,}50 \leq 1$$

6.2.3 Details beams

The beams are primarily loaded with bending moments which results in large vertical forces at the support connections. The connection of the beams is therefore realized with an internal steel plate and pin dowels. Figure 39 shows the detail. The maximum load results from the internal forces given in Table 11. The verification for the bar dowels is carried out analogue to the previous detail. Once again, the minimum distances for $\alpha = 0°$ and $\alpha = 90°$ are considered. Due to the low tensile force, block shear failure is not decisive here either. But for the fork bearing, additional measures are needed, because the single slotted plate does not have any lever arm. However, these will not be discussed further in this paper.

Depending on the location of the beam, the steel plate can be connected directly to the truss node or must be attached to the side of the spandrel beam. Within the diagrid structure, the beam must also be attached to the pendulum column. The load application into the pendulum column or the spandrel beam is realized with an end plate and ring shank nails. The detail, for example of the connection to the pendulum column is shown in Figure 40 and is dimensioned accordingly. The loads are the same as for the pin dowel connection. The minimum distances for the nails are taken into account in the design, as well as the minimum thickness of the timber.

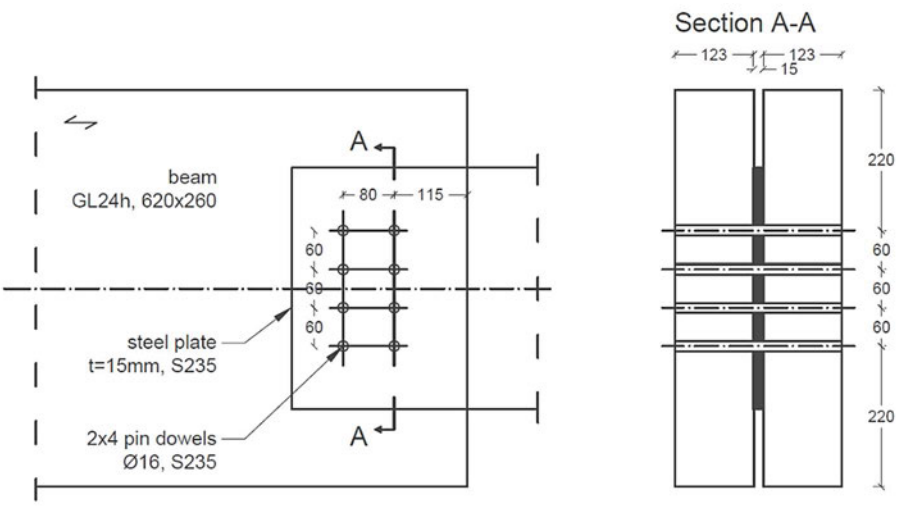


Figure 39 - Detail: Connection beam

Connection beam

Steel plate-timber connection with pin dowels

Verification method according to DIN EN 1995 German national annex

Load:

Internal forces: $N_d = 13{,}8kN\ (tension)/-11{,}3kN\ (pressure)$

$V_d = 92{,}6kN$

Decisive load: $F_{Ed} = 93{,}7kN$

Force/fiber angle: $\alpha = 81{,}22°$

Components and environmental conditions:

Pin dowel: $\emptyset 16, S235$

Diagrid bracing: $GL24h, \rho_k = 380kg$

Steel plate: $t = 15mm\ , S235$

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}1$

Load-bearing capacity per shear joint:

$$F_{v,Rk} = \sqrt{2} * \sqrt{2 * M_{y,Rk} * f_{h,k,\alpha} * \emptyset} = 12451\ N$$

Plastic moment pin dowel: $M_{y,Rk} = 145927\ Nmm$

Hole bearing strength timber: $f_{h,k,\alpha} = 16{,}6N/mm^2$

Minimum timber thickness:

$$t_{req} = 1{,}15 * 4 * \sqrt{\frac{M_{y,Rk}}{f_{h,k,\alpha} * \emptyset}} = 107{,}8mm \leq t_{min} = 122{,}5mm$$ → no reduction

Effective number of pins in a row/ number of shear joints / number of rows:

with $n = 2$: $n_{ef} = n^{0,9} * \sqrt[4]{\frac{a_1}{13\emptyset}} = 1{,}47$ $s = 2$ $r = 4$

Loading capacity of the joint:

$$F_{Rd} = r * s * n_{ef} * F_{v,Rk} * \frac{k_{mod}}{\gamma_M} = 106{,}49kN \leq F_{Ed} = 93{,}7kN$$

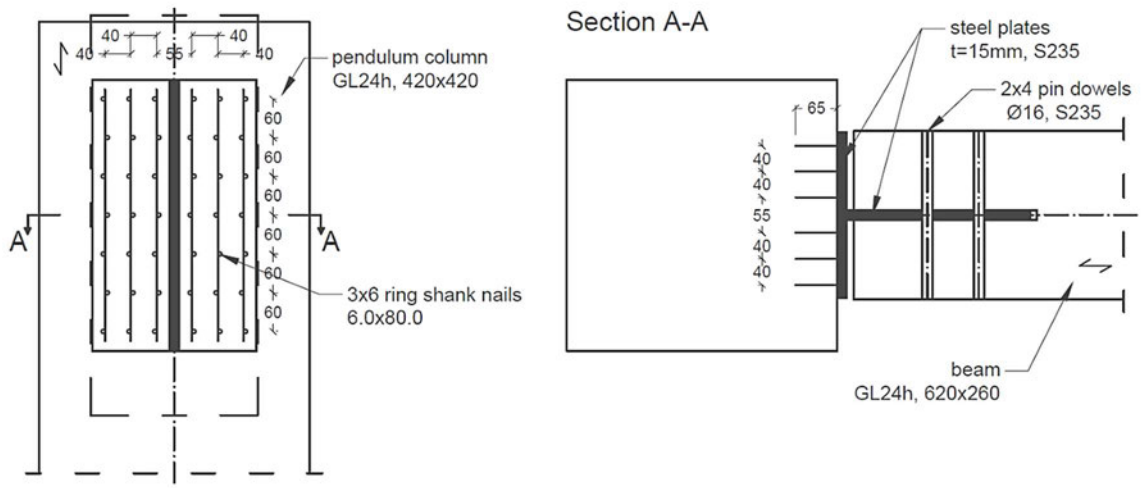


Figure 40 - Detail: Load application beam to pendulum column

Load application beam to pendulum column

Steel plate-timber connection with nails

Verification method according to DIN EN 1995 German national annex

Load:

Internal forces: $N_d = 13{,}8kN$ and $V_d = 92{,}6kN$

Decisive load: $F_{ax,Ed} = 13{,}8kN$ and $F_{V,Ed} = 92{,}6kN$

Components and environmental conditions:

Ring-shank nail: 6,0x80,0mm ; $\emptyset_h = 12mm$

load class 3C ; $f_u > 600N/mm^2$; not predrilled

Diagrid spandrel beam: GL24h, $\rho_k = 380kg$

Steel plate: $t = 15mm$, S235

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}1$

Shear off:

Load-bearing capacity per shear joint: $F_{v,Rk} = A * \sqrt{2 * M_{y,Rk} * f_{h,k} * \emptyset} = 2851N$

Plastic moment nail: $M_{y,Rk} = 18987\ Nmm$

Hole bearing strength timber: $f_{h,k} = 18{,}2N/mm^2$

Factor A: $A = 1{,}4$

Increase of the load capacity: $\Delta F_{v,Rk} = \min \begin{cases} 0{,}5 * F_{v,Rk} = 1425N \\ 0{,}25 * F_{ax,Rk} = 703N \end{cases} = 703N$

Effective number of nails: $n_{ef} = 42$

Loading capacity of the joint: $F_{V,Rd} = n_{ef} * (F_{v,Rk} + \Delta F_{v,Rk}) * \frac{k_{mod}}{\gamma_M} = 108{,}5kN$

$F_{V,Rd} = 108{,}5kN \leq F_{V,Ed} = 92{,}6kN$

Pull out:

Load-bearing capacity per nail: $F_{ax,Rk} = f_{ax,k} * \emptyset * t_{pen} = 2815N$

Pull-out resistance: $f_{ax,k} = 50 * 10^{-6} * \rho_k^2 = 7{,}22N/mm^2$

Penetration depth: $t_{pen} = 65mm$

Effective number of nails: $n_{ef} = 42$

Loading capacity of the joint: $F_{ax,Rd} = n_{ef} * F_{ax,Rk} * \frac{k_{mod}}{\gamma_M} = 87{,}08kN$

$F_{ax,Rd} = 87{,}08kN \leq 13{,}8kN$

Combined load:

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{V,Ed}}{F_{V,,Rd}}\right)^2 = 0{,}75 \leq 1$$

6.2.4 Detail pendulum columns

The pendulum supports are connected as a compressive joint with force transmission exclusively via contact surfaces. This is possible because the columns are axially loaded only by compression. Attention is paid to the fact that the load is transmitted directly between the columns, in order to avoid critical bearing pressure on the CLT slab. For this purpose, a square hollow cross-section made of steel is chosen as distance holder, see Figure 41. Steel plates are welded to the profile in the form of an open box section to support the column. This fitting also serves to absorb the small transverse forces. These can be transferred by means of transverse pressure on the pendulum columns. The additional nails, which are hammered in all around, are only used to secure the position. The entire connection can thus be realized by means of pressure contact. In the following, the transverse pressure on the pendulum column and the steel profile are verified. The design loads are the maximum internal forces of the pendulum columns according to Table 11. It should be noted that the connection of the beams is decoupled from the connection of the pendulum supports, this is done as described above with the help of a steel plate - nail connection.

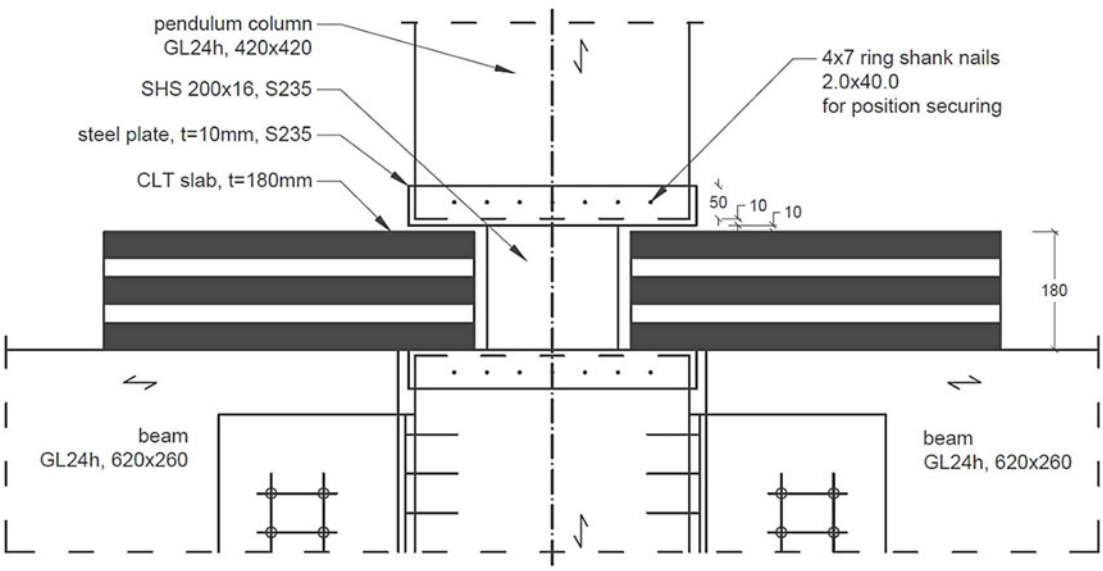


Figure 41 - Detail: Connection pendulum column

Connection pendulum column

Steel-timber connection with pressure contact

Verification method according to DIN EN 1995 and DIN EN 1993

Load:

Internal forces: $N_d = -2221{,}9\text{kN (pressure)}$ and $V_d = 26{,}1\text{kN}$

Components and environmental conditions:

Pendulum column: $\text{GL24h}, \rho_k = 380\text{kg}$

Steel plates: $t = 10\text{mm}, \text{S235}$

Steel section: $\text{SHS 200x16}, \text{S235}$

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}3$

Transverse pressure timber:

Load-bearing capacity: $F_{v,Rd} = A_{ef} * k_{c,90} * f_{c,90,k} * \frac{k_{mod}}{\gamma_M}$

Contact area: $A_{ef} = 420 * 50 = 2100\text{mm}^2$

Coefficient: $k_{c,90} = 1{,}5$

Strength: $f_{c,90,k} = 2{,}7\text{N/mm}^2$

→ $F_{v,Rd} = 52{,}3\text{kN} \leq V_d = 26{,}1\text{kN}$

Steel profile, SHS 200x16:

Strength: $f_{y,k} = 23{,}5\text{kN/cm}^2$

Partial safety factor: $\gamma_{M0} = 1{,}0$

Compressive stress: $N_{c,Rd} = A * \frac{f_{y,k}}{\gamma_{M0}}$

Area: $A = 111{,}0\text{cm}^2$

→ $N_{c,Rd} = 2608{,}5\text{kN} \leq N_d = 2221{,}9\text{kN}$

Shear stress: $V_{Rd} = A_V * \frac{f_{y,k}}{\sqrt{3} * \gamma_{M0}}$

Shear area: $A_V = 55{,}5\text{cm}^2$

→ $V_{Rd} = 753{,}0\text{kN} \leq V_d = 26{,}1\text{kN}$

6.2.5 Detail diagrid corner columns

The connection of the corner columns is probably one of the most complex details, eight members run together in one node. In addition, two facade directions have to be brought together. Figure 42 shows how this can be realized. The double slotted plates for the connections of the bracings and spandrel beams are merged with an end plate and inserted into the column with the help of a single slotted plate. The slotted plate inside the columns is fixed with pin dowels. The pin dowels for the different directions must be staggered in height, so that they do not cross each other. The slotted plates and the end plates are also welded to a horizontal steel plate which is positioned between the end-grains of the columns. Via this plate, the normal forces are transferred to the column by means of pressure contact. The vertical forces of the corner columns are transmitted by the steel plate-pin dowel connection,

in the upper and lower columns respectively. The transmission of forces between the bracings and the spandrel beam takes place within the steel plates themselves.

In the following, only the verification of the pin dowels for the shear force is given. The load results from the maximum shear force loading of the supports, see Table 11. The application of the normal force via the end-grains does not require any further verification, it is fulfilled with the stress verification for pressure in the direction of the fibre. As mentioned at the beginning, the stress distribution and the welds of the steel plates are not further investigated within here.

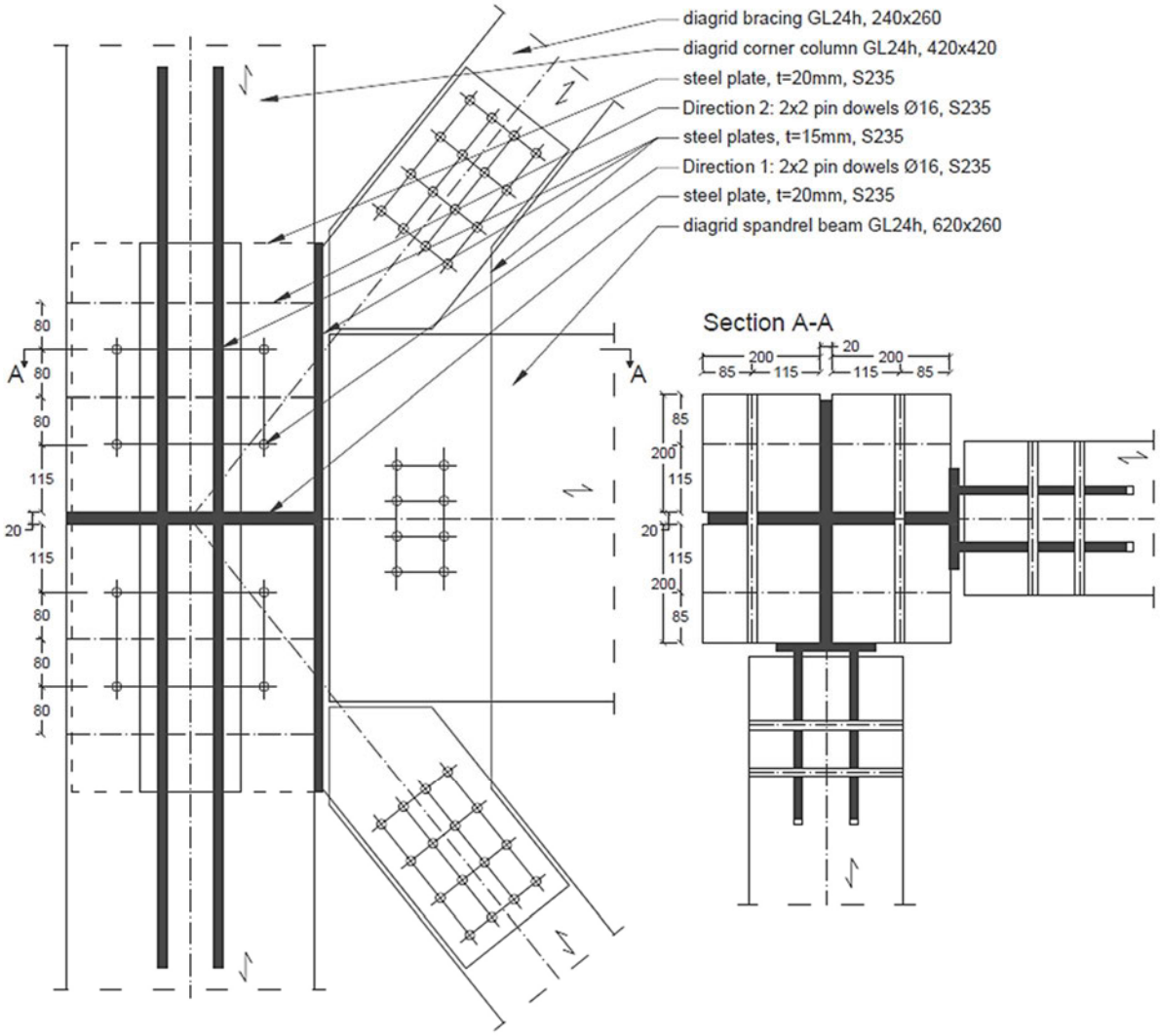


Figure 42 - Detail: Connection diagrid corner columns

Connection diagrid corner column

Steel plate-timber connection with pin dowels

Verification method according to DIN EN 1995 German national annex

Load:

Internal forces:	$V_d = 14{,}6kN$
Decisive load:	$F_{Ed} = 14{,}6kN$
Force/fiber angle:	$\alpha = 90°$

Components and environmental conditions:

Pin dowel:	$\emptyset 16, S235$
Diagrid bracing:	$GL24h, \rho_k = 380kg$
Steel plate:	$t = 20mm\ , S235$
Modification coefficient:	$k_{mod} = 0{,}8$
Partial safety factor:	$\gamma_M = 1{,}1$

Load-bearing capacity per shear joint:

$$F_{v,Rk} = \sqrt{2} * \sqrt{2 * M_{y,Rk} * f_{h,k,\alpha} * \emptyset} = 15633\ N$$

Plastic moment pin dowel:	$M_{y,Rk} = 145927\ Nmm$
Hole bearing strength timber:	$f_{h,k,\alpha} = 26{,}17N/mm^2$

Minimum timber thickness:

$$t_{req} = 1{,}15 * 4 * \sqrt{\frac{M_{y,Rk}}{f_{h,k,\alpha} * \emptyset}} = 85{,}87mm \leq t_{min} = 200mm \qquad \rightarrow \text{No reduction}$$

Effective number of pins in a row/ number of shear joints / number of rows:

$$\text{with } n = 2: \quad n_{ef} = n^{0,9} * \sqrt[4]{\frac{a_1}{13\emptyset}} = 1{,}47 \qquad s = 2 \qquad r = 2$$

Loading capacity of the joint:

$$F_{Rd} = \frac{t_{min}}{t_{req}} * r * s * n_{ef} * F_{v,Rk} * \frac{k_{mod}}{\gamma_M} = 66{,}85kN \leq F_{Ed} = 14{,}6kN$$

6.2.6 Details cross laminated timber slab

In addition to the details for the member connections, solutions must be found for the connections and joints of the cross-laminated timber (CLT) slab. Besides contact joints, mainly pin-type fasteners are used as connection technology.[162] Wood screws, either fully or partially threaded, are typically used. The load-bearing capacity is generally regulated by product approvals, although reference for the calculation is usually made to the DIN EN 1995. But due to the layered structure, the real load-bearing and deformation behaviour of fasteners in CTL differs from structural solid timber or Glulam. This was investigated in an extensive research project at the University of Karlsruhe, which resulted in a detailed design proposal.[163] For the practical construction of the designed connections, the engineer usually specifies a structural minimum screwing, with screw type, number, spacing and penetration depth. For the present

[162] Refer to Wallner-Novak et al. 2013, p. 105

[163] Blaß and Uibel 2007

design, details are required for the joints in the secondary bearing direction and for the support on the beams.

Joints in CLT slabs can be either articulated or rigid.[164] Articulated joints are usually made by means of a stepped overlap or a single-sided top layer. For the transmission of bending moments, in contrast, a top layer on both sides is required. The layers are fixed with screws and glue and transmit the corresponding moment thanks to the lever arm. For the associated shear forces as well as for normal forces, a screw cross made of fully threaded screws can also be mounted. In this case, the screws are axially stressed to pull out, which helps their load-bearing capacity. In the RFEM model, the CLT slab is modelled as a continuous panel; accordingly, flexural stiff joints are required in the secondary load-bearing direction. They can be designed as shown in Figure 43.

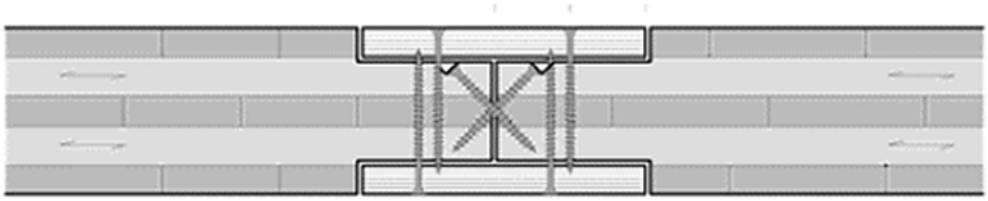


Figure 43 - Detail: Cross laminated timber slab joint in secondary bearing direction[165]

The support of the slabs is primarily achieved via the pressure contact. The vertical forces can thus be transferred to the beams. In addition, however, a transfer of the shear forces is required. This is due the fact that the slabs participate in the stiffening of the building in form of a diaphragm slab. The shear forces can be transmitted by means of a wood-material-wood connection with wood screws. Shear forces occur both axially and tangentially to the support axis.

In the following, the support detail is verified as an example. Thereby, the maximum bearing pressure as well as the shear of the screws are verified according to the DIN EN 1995. For the maximum loads, the forces of the line joints form the RFEM model are consulted. For the shear forces, it should be noted that a superposition of the maximum forces is analysed, which occurs primarily in the corners of the facade. Towards the middle, the loads decrease considerably, so that a gradation of the fasteners seems to be reasonable for this design. Figure 44 shows a section of the support with the arrangement of the fasteners. The minimum distances for the screws are observed.

[164] Refer to Wallner-Novak et al. 2013, pp. 99-100
[165] Wallner-Novak et al. 2013, p. 101

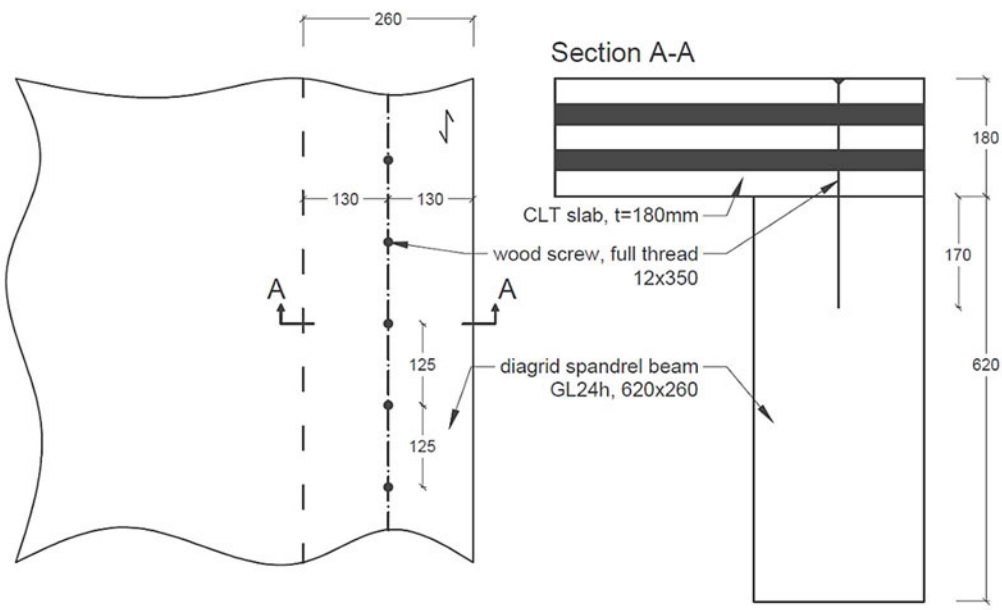


Figure 44 - Detail: Support cross laminated timber slab

Support CLT slab

Derived timber product – timber connection with pressure contact

Verification method according to DIN EN 1995

Load:

Decisive load: $F_{v,Ed} = 89{,}8\text{kN/m}$ $F_{c,90,Ed} = v_z = 81{,}4\text{kN/m}$

Components and environmental conditions:

Beam: GL24h; $f_{c,90,k} = 2{,}7\ \text{N/mm}^2$

Slab: CLT, $f_{c,90,k} = 2{,}5\ \text{N/mm}^2$

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}3$

Transverse pressure timber:

Load-bearing capacity: $F_{c,90,Rd} = A_{ef} * k_{c,90} * \min f_{c,90,k} * \frac{k_{mod}}{\gamma_M}$

Contact area: $A_{ef} = 260 * 1000 = 260000\text{mm}^2/\text{m}$

Coefficient: $k_{c,90} = 1{,}5$

→ $F_{c,90,Rd} = 600\text{kN/m} \geq F_{c,90,Ed} = 81{,}4\text{kN/m}$

Support slab on beams

Derived timber product – timber connection with wood screws

Verification method according to DIN EN 1995 German national annex

Load:

Line joints: $n_d = 62{,}59\text{kN/m}$ $v_y = 64{,}38\text{kN/m}$

Decisive load: $F_{v,Ed} = 89{,}8\text{kN/m}$

Force/fiber angle: $\alpha = 45{,}8°$

Components and environmental conditions:

Wood screw: 12,0x350; $f_u > 600\text{N/mm}^2$; full thread

Beam: GL24h, $\rho_k = 380\text{kg}$

Slab: CLT, $\rho_{B,k} = 350\text{kg}$ raw material C24

Modification coefficient: $k_{mod} = 0{,}8$

Partial safety factor: $\gamma_M = 1{,}1$

Load-bearing capacity per shear joint:

$$F_{v,Rk} = \sqrt{\frac{2*\beta}{1+\beta}} * \sqrt{2 * M_{y,Rk} * f_{h,1,k,\alpha} * \emptyset} = 7701\text{ N}$$

Plastic moment screw: $M_{y,Rk} = 145927\text{ Nmm}$

Hole bearing strength slab: $f_{h,1,k} = 0{,}019 * {\rho_{B,k}}^{1,24} * \emptyset^{-0,3} = 12{,}87\text{N/mm}^2$

Hole bearing strength beam: $f_{h,2,k,\alpha} = 21{,}55\text{N/mm}^2$

Coefficient: $\beta = \frac{f_{h,2,k,\alpha}}{f_{h,1,k}} = 1{,}67$

Minimum timber thickness:

Slab: $t_{1,req} = 130\text{mm} \leq t = 180\text{mm}$

Beam: $t_{2,req} = 90\text{mm} \leq t = 620\text{mm}$ → no reduction

Increase of the load capacity:

$$\Delta F_{v,Rk} = \min\begin{cases} F_{v,Rk} = 7701\text{N} \\ 0{,}25 * F_{ax,Rk} = 14820\text{N} \end{cases} = 7701\text{N}$$

Pull-out resistance: $F_{ax,Rk} = 59282\text{N}$

Loading capacity per screw:

$$F_{V,Rd} = (F_{v,Rk} + \Delta F_{v,Rk}) * \frac{k_{mod}}{\gamma_M} = 11{,}2\text{kN}$$

Number of screws/meter:

$n_{req} = F_{v,Ed}/F_{V,Rd} = 8{,}0\frac{1}{m}$ chosen 1x screw Ø12 /12,5cm

7 Summary and Outlook

This paper was intended to develop a sustainable structural concept for the Maun Science Park in Botswana. The focus was on an up to 8-storey building, the Tree of Life, which is intended for various uses such as living, research or working.

At the beginning, the general principles for tall building structures were elaborated. In particular, the horizontal load-bearing behaviour and the premium for height effect were explained. The resulting commonly used structural systems for tall buildings and their specific properties were subsequently presented. As a second cornerstone, the topic of sustainable construction was outlined. In particular, the three pillars of sustainability were introduced: ecological sustainability, economic sustainability and social sustainability. For each pillar individually, the reference to sustainable building and the MSP has been established. It was possible to demonstrated how diverse the topic of sustainable construction is. Ecological sustainability can best be assessed with a life cycle assessment, whereby the explicit procedure was explained in detail in chapter 4.1.3 While the economic sustainability is subordinate in this thesis, the flexibility of the structure and the local reference are of particular importance for the social sustainability.

Based on the fundamentals, four structural designs were developed in the main part: a concrete core structure, a steel shear frame structure, a rammed earth shear wall structure and a wooden diagrid structure. For each design, the construction was presented and a preliminary design of the load-bearing components was carried out. In order to assess the environmental sustainability, a life cycle assessment was set up for each design using the ÖKOBAUDAT database.

Subsequently, in chapter 5.1, the designs were compared and evaluated with regard to predefined requirements. With the help of a utility value analysis, Design 4, the wooden diagrid structure, was determined as the preferred variant. The evaluation of the designs, with individual advantages and disadvantages, was used to draw general conclusions for the development of sustainable structural systems for the MSP. It was established that wood as an ecological building material is definitively the most sustainable choice for construction. However, the precondition is that it is sourced from sustainable forestry. Furthermore, it could be demonstrated that there is a mass balance between ecological and industrially manufactured products in terms of strength and environmental impact. The local situation in terms of availability of building materials, society and architecture is difficult to assess from abroad. However, it can be assumed that the procurement of materials for Maun poses challenges due to its remote location. In order to maximise flexibility and simplicity of the construction, a member structure is generally recommended. The “premium for height” effect,

however, was noted to play a minor role in the 8-storey design. Overall, the author believes that one of the most important factors for a sustainable structural design is the integration of the structure into an overarching sustainability concept for the entire project.

As a completion of this paper, the last chapter proves the feasibility of the favourite design, the wooden diagrid structure. This was realised with the help of a system statics in Dlubal RFEM as well as the verification of relevant details.

The knowledge gained about the construction of sustainable tall building structures is not only relevant for the Maun Science Park, but partly also for general use. It can therefore be transferred, with regional adaptations, to buildings all over the world. The life cycle assessment in particular is an important tool in this context, as it enables a traceable evaluation of ecological sustainability. Thanks to free access to databases, structural engineers can thus contribute significantly to the development of sustainable buildings. The trend towards more and more wooden tall buildings is therefore absolute consequent. Stunning examples such as the HoHo in Vienna, Austria, or the Mjøstårnet in Brumunddal, Norway, show the possibilities of realisation. However, in some regions of the world, especially in Africa and thus also in Botswana, timber is a rare resource. The use of bamboo as a building material can provide a change. Clay, another local and ecological building material, is less suitable for load-bearing structures in tall buildings. The positive ecological properties are countered by poor structural properties such as high dead weight and low strength. It is therefore more suitable for subordinate load-bearing components, for example in the form of lightweight clay walls. However, both ecological building materials, bamboo and clay, will certainly play a decisive role in the further development of the Maun Science Park and sustainable buildings all over the world.

Due to the diversity of the topic of sustainable construction, the author recommends that the next step should be to develop an overall sustainability concept for the buildings of the Maun Science Park. Sustainable construction can be implemented in many different ways, so in what specific way should the sustainability of MSP buildings be achieved? Based on this concept, the various specialist planners can work towards the goal in an interdisciplinary manner. In this way, for example, the structural system can be further optimised. With a common concept, each specialist group can contribute to goal of sustainable construction.

It will be exciting to further pursue this extraordinary project and to see where it leads to.

List of References

bauforumstahl e.V. (2018): *Baustähle: Offene Walzprofile und Grobbleche; Umwelt-Produktdeklaration*. Institut Bauen und Umwelt e.V. (IBU) online. Available at https://bauforumstahl.de/wissen/nachhaltigkeit/umwelt-produktdeklarationen. Accessed 05.03.2021.

Baukosteninformationszentrum Deutscher Architektenkammern (2020): *Baukosten Gebäude Neubau 2020; Statistische Kostenkennwerte Teil 1*. BKI, Stuttgart.

Bewertungssystem Nachhaltiges Bauen (2021): *Bewertungskriterien für Bürogebäude*. Bundesministerium des Innern, für Bau und Heimat online. Available at https://www.bnb-nachhaltigesbauen.de/bewertungssystem/buerogebaeude/. Accessed 08.02.2021.

Blaß, Hans Joachim; Uibel, Thomas (2007): *Tragfähigkeit von stiftförmigen Verbindungsmitteln in Brettsperrholz; Band 8 der Reihe Karlsruher Bericht zum Ingenieurholzbau*. Universität Karlsruhe (TH), Lehrstuhl für Ingenieurholzbau und Baukonstruktion.

Bühler, Michael (2020): *Weltraumtechnologie für nachhaltiges Leben auf der Erde*. Hochschule Konstanz, Technik, Wirtschaft und Gestaltung online. Available at https://www.linkedin.com/pulse/space-technology-sustainable-life-earth-michael-max-buehler/. Accessed 05.12.2020.

Bühler, Michael; Michalski, Alexander; Hollenbach, Pia; Krötsch et al. (2020): *Internationale Kooperationen & Interdisziplinäre Projekte Botswana*. Hochschule Konstanz, Fakultät Bauingenieurwesen online. Available at https://moodle.htwg-konstanz.de/moodle/course/view.php?id=4262. Accessed 05.12.2020

Council on Tall Buildings and Urban Habitat (2020): *CTBUH Height Criteria; for Measuring & Defining Tall Buildings*. Council on Tall Buildings and Urban Habitat online. Available at https://www.ctbuh.org/resource/height#tab-tall-supertall-and-megatall-buildings. Accessed 17.11.2020.

DGNB GmbH (2021): *Übersicht aller Kriterien für Gebäude Neubau*. DGNB System online. Available at https://www.dgnb-system.de/de/gebaeude/neubau/kriterien/. Accessed 08.02.2021.

Eisele, Johann (2014): *Grundlagen der Baukonstruktion; Tragsysteme und deren Wirkungsweise*. 2nd ed. DOM Publ, Berlin.

El khouli, Sebastian; John, Viola; Zeumer, Martin (2014): *Nachhaltig konstruieren; Vom Tragwerksentwurf bis zur Materialwahl: Gebäude ökologisch bilanzieren und optimieren*. Institut für internationale Architektur-Dokumentation, München.

Friedrichsen, Stefanie (2018): *Nachhaltiges Planen, Bauen und Wohnen; Kriterien für Neubau und Bauen im Bestand*. 2nd ed. Springer, Berlin/Heidelberg.

Green, Michael; Taggart, Jim (2017): *Hoch Bauen mit Holz; Technologie, Material, Anwendung*. Birkhäuser, Basel.

Günel, Mehmet Halis; Ilgin, Hüseyin Emre (2007): *A proposal for the classification of structural systems of tall buildings*. In: Building and Environment, 42 (7), pp. 2667–2675.

Günel, Mehmet Halis; Ilgin, Hüseyin Emre (2014): *Tall buildings; Structural systems and aerodynamic form*. Routledge, Abingdon/New York.

Gupta, Abhishek; Gupta, S. M. (2017): *Structural Development of Skyscrapers*. In: International Journal of Advances in Mechanical and Civil Engineering, 4 (3), pp. 6–10.

haascookzemmrich STUDIO 2050, Freie Architekten PartG mbB (2020).

Hill, Richard C.; Bowen, Paul A. (1997): *Sustainable construction: principles and a framework for attainment*. In: Construction Management and Economics, 15 (3), pp. 223–239.

IGBC Irish Green Building Council (2021): *What is embodied carbon?*. IGBC Irish Green Building Council online. Available at https://www.igbc.ie/what-is-embodied-carbon/. Accessed 06.02.2021.

Informationszentrum Beton GmbH (2018): *Beton der Druckfestigkeitsklasse C 30/37; Umwelt-Produktdeklaration*. Institut Bauen und Umwelt e.V. (IBU) online. Available at https://www.beton.org/wissen/nachhaltigkeit/umweltproduktdeklarationen/. Accessed 05.03.2021.

Kapfinger, Otto; Sauer, Marko (2015): *Martin Rauch: Gebaute Erde; Gestalten & Konstruieren mit Stampflehm*. 2nd ed. DETAIL, München.

Keikut, Frank; Geier, Sonja (2019): *Modul17; Hochhaustypologie in Holzhybridbauweise*. vdf, Zürich.

Krötsch, Stefan (2020): *Architektur der Notwendigkeit; Vortrag.* Hochschule Konstanz, Fakultät Bauingenieurwesen online. Available at https://moodle.htwg-konstanz.de/moodle/mod/page/view.php?id=178505. Accessed 13.12.2020.

Kruger, Andries; Retief, Johan; Goliger, Adam (2017): *Development of an updated fundamental basic wind speed map for SANS 10160-3*. In: Journal of the South African Institution of Civil Engineering, 59 (4), pp. 12–25.

Malumbela, Goitseone; Masuku E. U. (2017): *Resources and Strategies towards the Development of a Sustainable Construction Materials Industry in Botswana*. In: International Journal of Civil and Environmental Engineering, 11 (2), pp. 108–114.

Marais, Paul (2020): *Q&A with Paul Marais on Rammed Earth; Interview*. Available at https://moodle.htwg-konstanz.de/moodle/mod/page/view.php?id=187393. Hochschule Konstanz, Fakultät Bauingenieurwesen online. Accessed 13.12.2020.

Markovic, Zoran (n.d.): *Traditional Cultural Elements in Built Environment Design in Botswana; Interior Design; Exterior Design; Built Techniques and Technologies Decorations.* University of Botswana.

Maun Science Park (2021): *Maun Science Park*. Available at https://www.maunsciencepark.com/. Accessed 13.03.2021.

Moro, José Luis (2020): *Baukonstruktion - vom Prinzip zum Detail*. Springer, Berlin/Heidelberg.

ÖKOBAUDAT Informationsportal Nachhaltiges Bauen (2021): *ÖKOBAUDAT; Datenbank*. Available at https://www.oekobaudat.de/. Accessed 23.02.2021.

Phocas, Marios C. (2005): *Hochhäuser, Tragwerk und Konstruktion*. Teubner B.G., Stuttgart/Leipzig/Wiesbaden.

Rüter, Sebastian; Diederichs, Stefan (2012): *Ökobilanz-Basisdaten für Bauprodukte aus Holz; Abschlussbericht*. Johann Heinrich von Thünen-Institut, Institut für Holztechnologie und Holzbiologie. Available at https://epub.sub.uni-hamburg.de/epub/volltexte/2013/20099/pdf/dn050490.pdf. Accessed 05.03.2021.

Schroeder, Horst (2016): *Sustainable Building with Earth*. Springer, Cham/Heidelberg/New York.

Wallner-Novak, Markus; Koppelhuber, Josef; Pock, Kurt (2013): *Brettsperrholz Bemessung; Grundlagen für Statik und Konstruktion nach Eurocode*. proHolz Austria online. Available at https://www.proholz.at/shop/proholz-information. Accessed 20.12.2020.

Widjaja, Eddy (2012): *Aussteifung von Bauwerken*. In: *Entwurfshilfen für Architekten und Bauingenieure; Faustformeln für die Vorbemessung, Vorbemessungstafeln, Bauwerksaussteifung*. 2nd ed.. Eds. Schneider, Klaus-Jürgen and Widjaja, Eddy. S. 149–162. Beuth (Bauwerke), Berlin/Wien/Zürich.

World Commission on Environment and Development (1987): *Report of the World Commission on Environment and Development: Our Common Future*. Sustainable Development Knowledge Platform online. Available at https://sustainabledevelopment.un.org/milestones/wced. Accessed 02.12.2020.

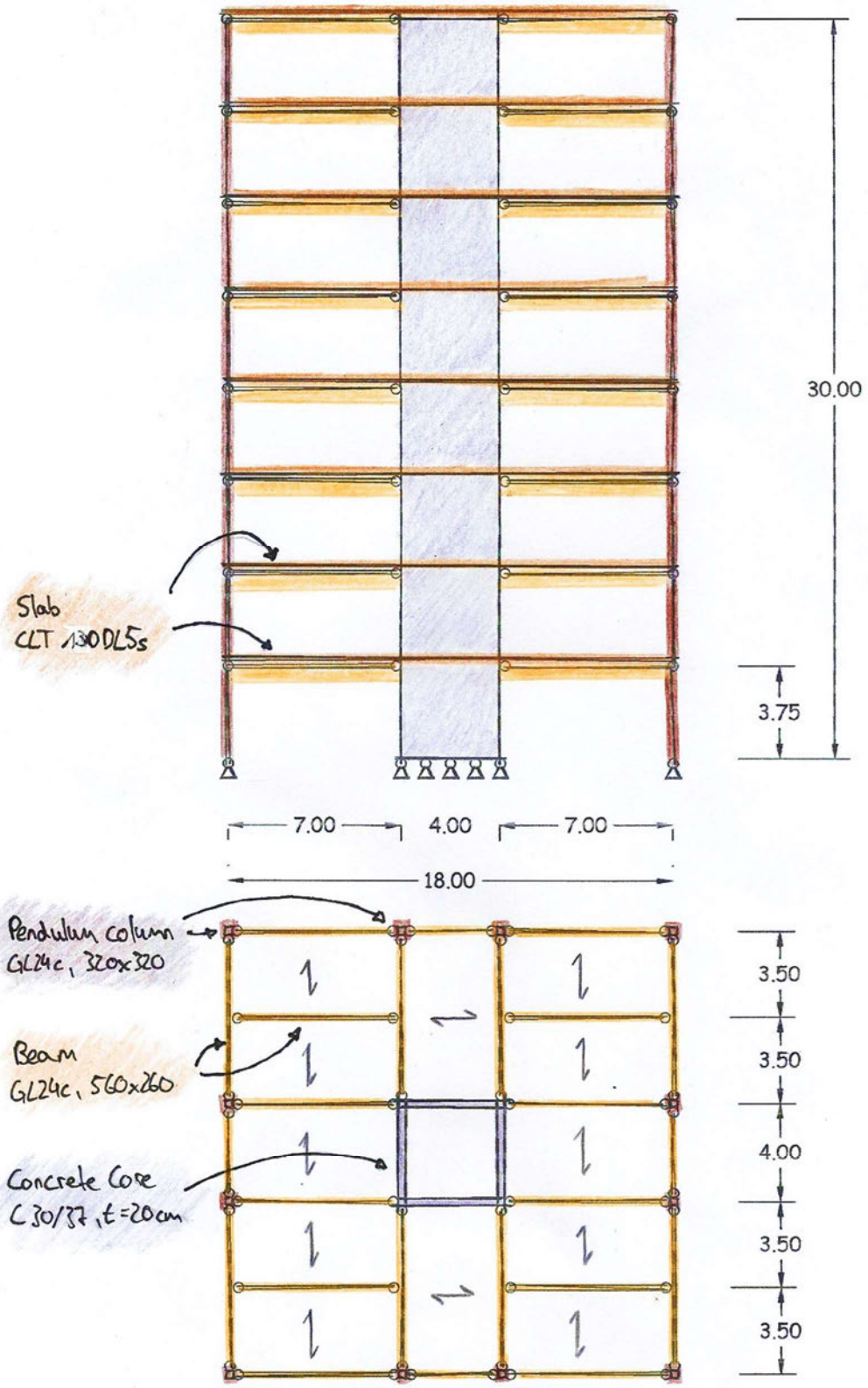
World Weather & Climate Information (2021): *Climate in Maun, Botswana*. World Weather & Climate Information online. Available at https://weather-and-climate.com/average-monthly-Rainfall-Temperature-Sunshine-fahrenheit,Maun,Botswana. Accessed 22.03.2021.

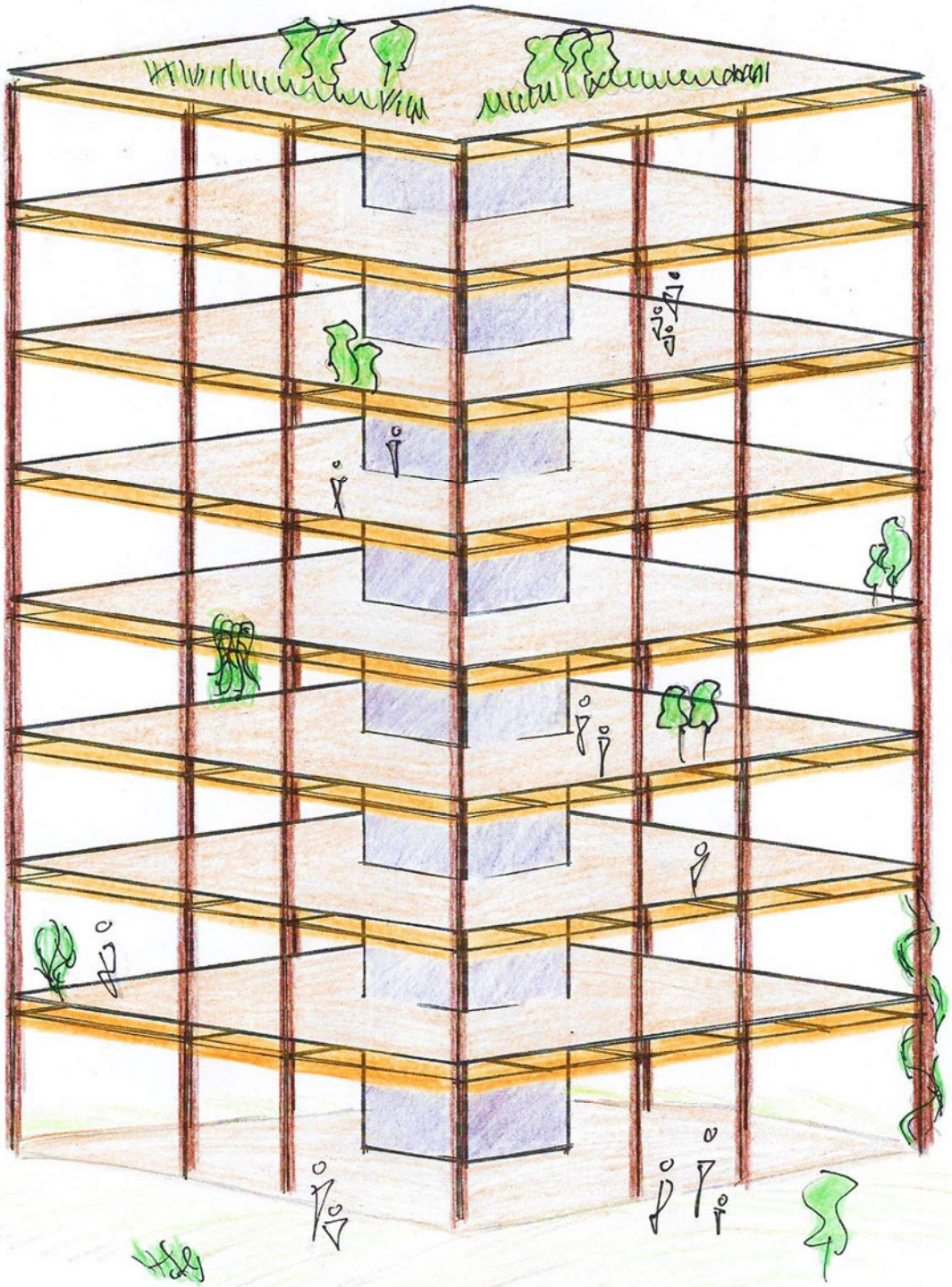
Xiao, Y.; Shan B.; Yang, R. Z.; Li, Z.; Chen, J. (2014): *Glue Laminated Bamboo (GluBam) for Structural Applications*. In: *Materials and Joints in Timber Structures; Recent Developments of Technology*. Eds. Aicher, Simon; Reinhardt, H.-W. and Garrecht, Harald. pp. 589–603. Springer, Dordrecht/Heidelberg/New York.

Appendix

A1:	Design 1: Structural system and Rendering			
A2:	Design 2: Structural system and Rendering			
A3:	Design 3: Structural system and Rendering			
A4:	Design 4: Structural system and Rendering			
B1:	Calculation file LCA:	Life_Cycle_Assessment	Excel	(digital)
B2:	Calculation file ULS:	Diagrid_Structure_ULS	RFEM5	(digital)
B3:	Calculation file SLS:	Diagrid_Structure_SLS	RFEM5	(digital)

A1: Design 1: Structural system and Rendering





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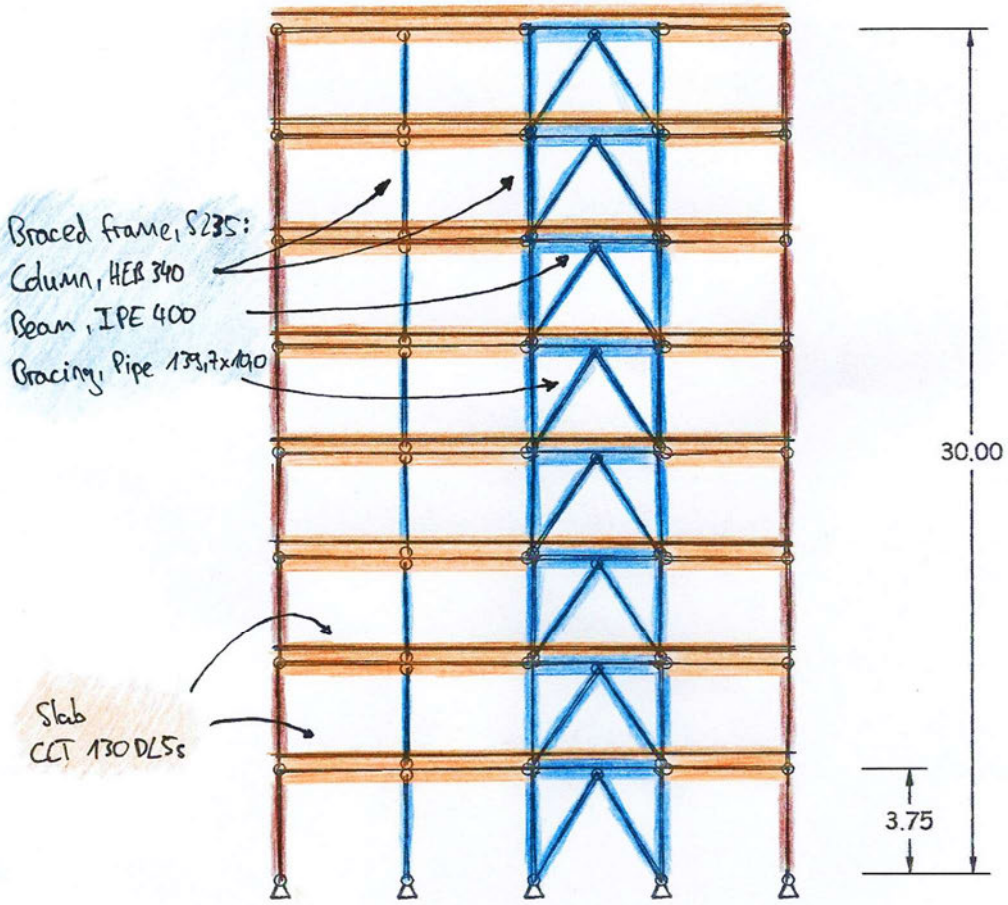

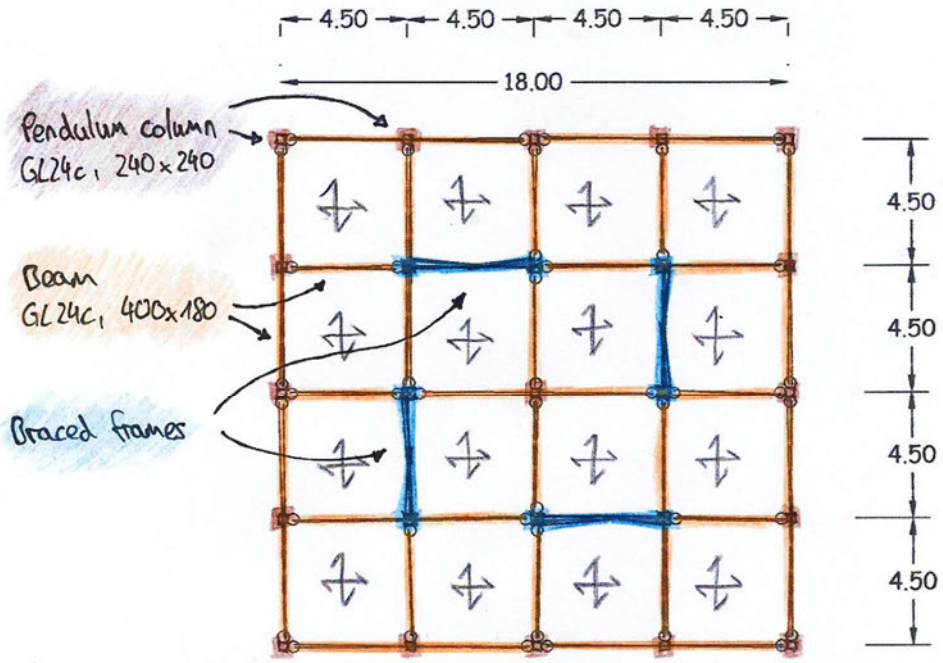

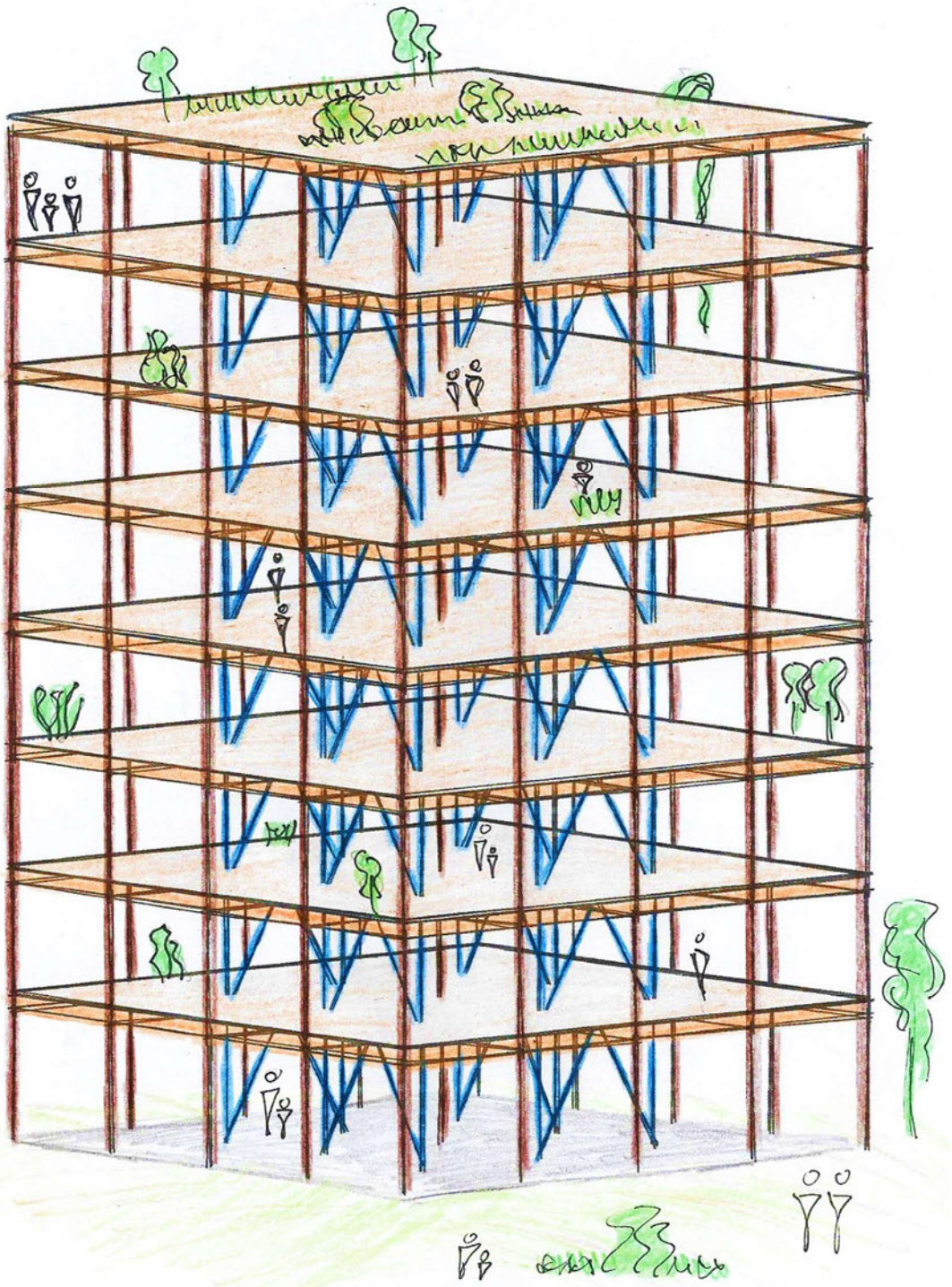

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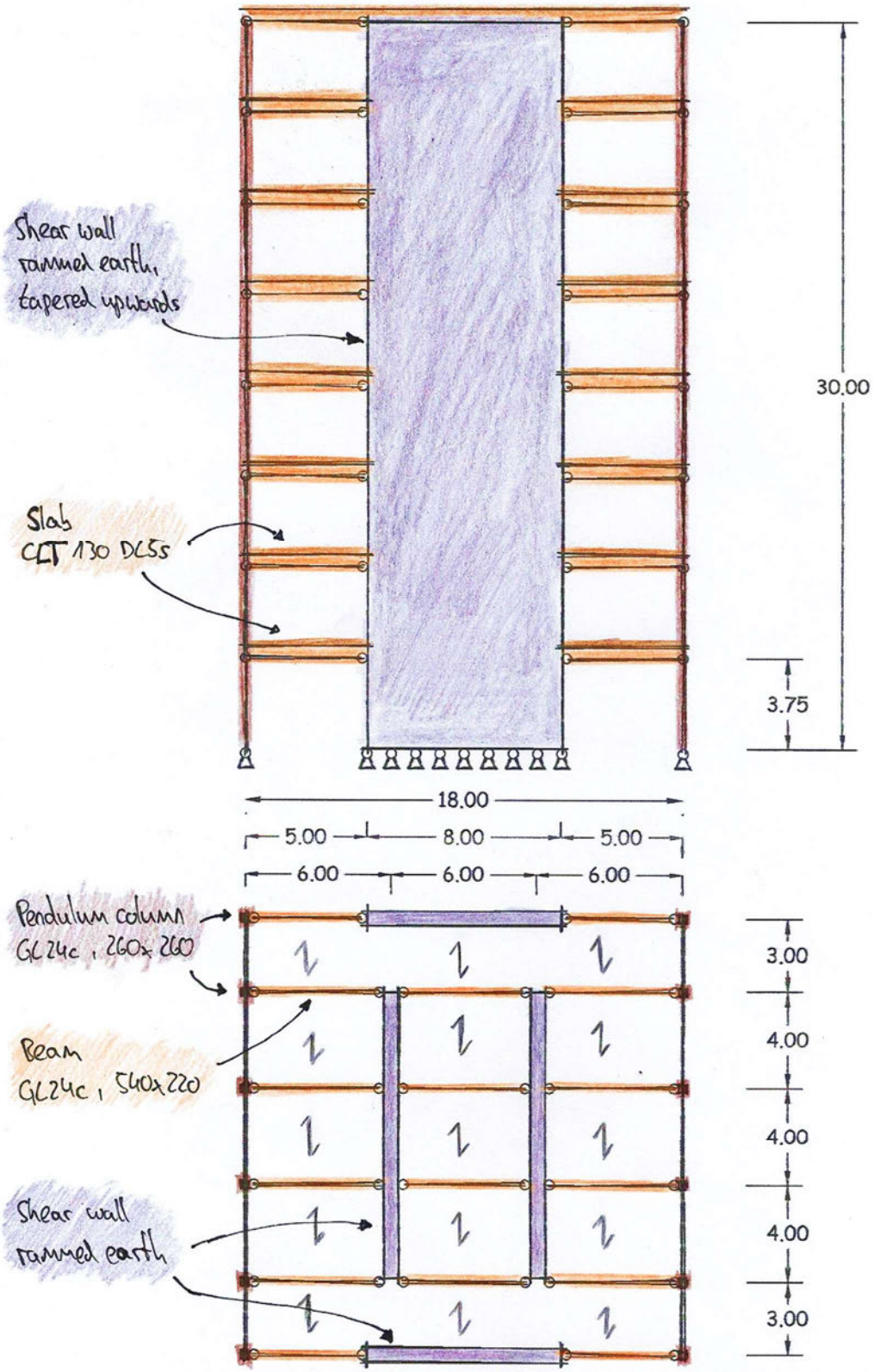

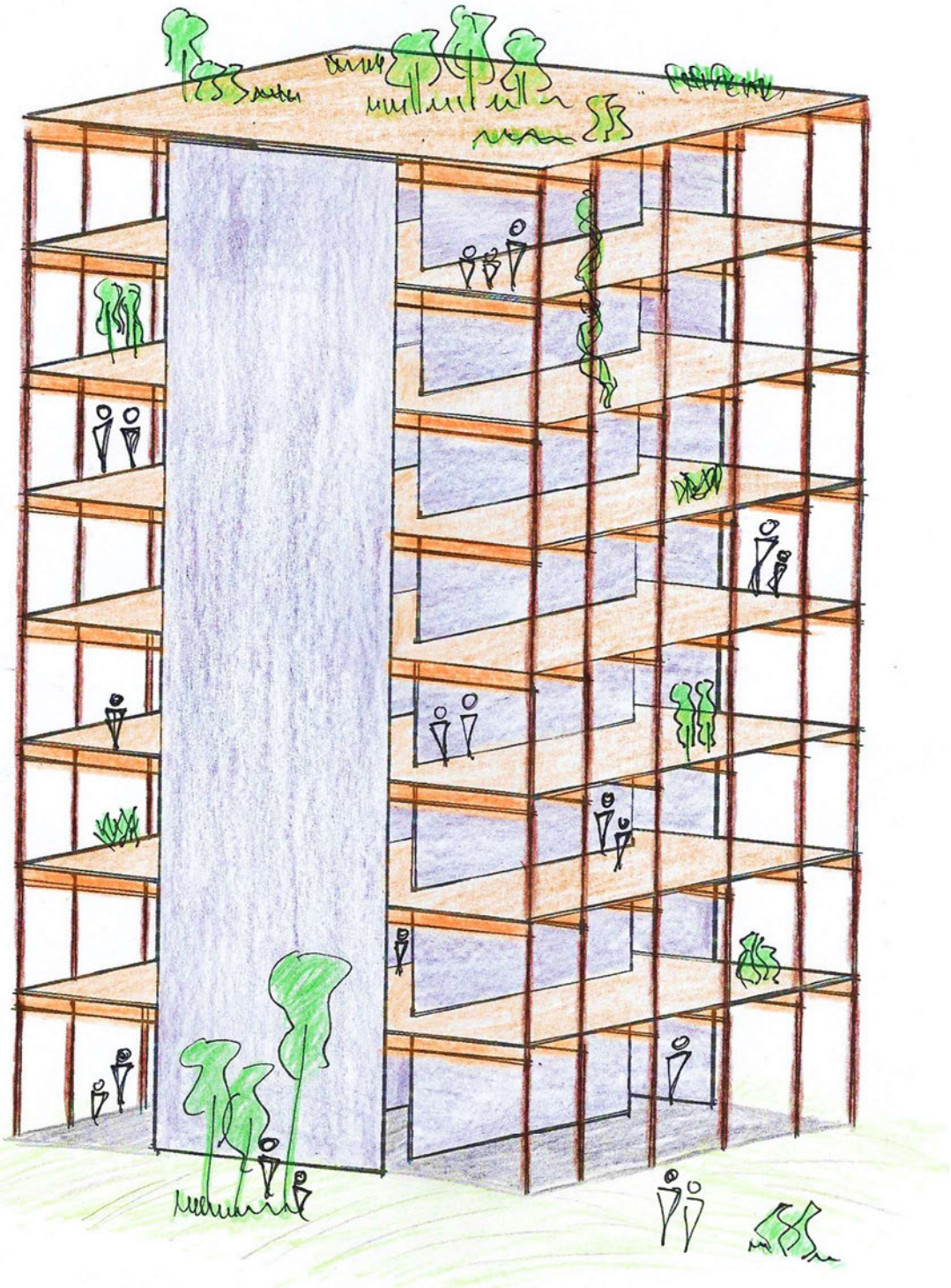

A4: Design 4: Structural system and Rendering