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Publication date
2021

Document Version
Accepted author manuscript

Citation (APA)
Ravenshorst, G., Gijzen, R., Willebrands, O., Slooten, E., & van de Kuilen, J. W. (2021). *Students in high-rise: Aspects influencing the design of tall timber buildings*. Paper presented at World Conference on Timber Engineering 2021, WCTE 2021, Santiago, Chile.

STUDENTS IN HIGH-RISE: ASPECTS INFLUENCING THE DESIGN OF TALL TIMBER BUILDINGS

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ABSTRACT: Tall timber buildings are of growing interest worldwide and also in the Netherlands. With the introduction of CLT, higher timber buildings are possible compared to traditional timber frame housing. Tall timber buildings can be made as modular buildings or as buildings with a CLT or concrete core and a timber wall/column-floor system that can contribute to the overall stability. In this paper, master student thesis projects dealing with various aspects of timber high-rise buildings are presented. These include the possibilities of using 3D-modules in high-rise buildings, design aspects of differential vertical shortening, parametric design, as well as wind-induced behaviour. For tall timber buildings specific aspects as the influence of the connections on the stability and comfort (accelerations), and the necessity of performing a phased analysis to account for the different creep behaviour of timber and concrete. When this is done properly, tall timber buildings can be designed with various structural appearances, fulfilling the requirements and the wishes of architects and users. The raised technical questions have been incorporated in the teaching program of Delft University of Technology and formed the basis for the master thesis projects presented in this paper.

KEYWORDS: Master thesis, Highrise buildings, Modular buildings, Differential shortening, Wind induced behaviour.

1 INTRODUCTION

Tall timber buildings are of growing interest worldwide an also in the Netherlands. With the introduction of CLT higher timber buildings are possible compared to timber frame housing. In the 1990s in Delft in the Netherlands a 6-storey timber frame housing was considered as a tall timber building. However, 6 storeys is kind of a natural limit for timber frame housing. With the introduction of CLT higher timber buildings are possible. This trend started worldwide with the 9-storey Stadhaus building in London and after that, higher buildings arose around the world. In the Netherlands, recent developments are the 8-storey hotel Jakarta in Amsterdam that was finished in 2018 and the 73 meter high apartment building HAUT, also in Amsterdam, currently under construction.

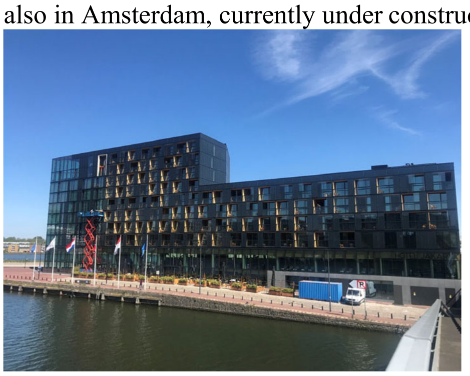

Figure 1*: 8-storey hotel Jakarta in Amsterdam made of prefabricated 3D-CLT modules, completed in 2018.*

Architects also discovered the material CLT to design (highrise)buildings. An extra challenge for timber engineers is that architects want to show the construction material timber, also in high-rise, raising additional questions about fire safety.

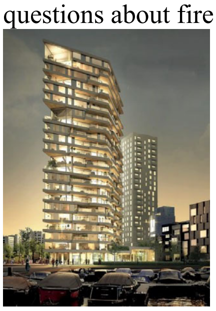

Figure 2*: 73 m high residential building HAUT in Amsterdam made up of a concrete core and CLT walls and floors. Under construction, completion planned in 2021. (photo taken from hautamsterdam.nl)*

In this paper, 3 Master thesis projects performed at Delft University of Technology are presented. All studies investigate the limits in tall timber buildings for different aspects. These topics are the design, the use and mechanical performance of 3D timber modules, the influence of differential vertical shortening (DVS) between timber and concrete and the application of outriggers in ultra high-rise. The studies show that when these aspects are taken into account, efficient tall timber buildings can be efficient and designed realistically.

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2 MODULAR CLT BUILDINGS

In Amsterdam, a hotel building of 8-storeys was built and finished 2018. The hotel rooms consisted of modules of 9 m x 3.5 m x 2.9 m that were prefabricated in a factory outside of Amsterdam and after completion transferred to the building site, where they were stacked together. The hotel was triangular shaped and had some concrete stabilizing walls, but due to the lay-out, the modular system had to maintain its own stability in 2 directions, the long direction where the modules are coupled, and the short direction where only the length of a module can be used. The modules were placed on a ground storey consisting of a concrete "table" construction, allowing for architectural freedom at ground level.

Figure 3*: Front view of the modular system. The modules have to maintain its own stability for wind load in both directions.*

The modules had only one stabilizing wall in the short direction of the module (this is the direction where the modules can be coupled). The floor was made of concrete and the CLT walls and ceiling were in sight. The basic fire resistance required for the CLT walls according to the Dutch building code is 120 minutes. Because a sprinkler system was used, after consultation with the local authorities, the required fire resistance could be reduced by 30 minutes.

Gijzen [1] took the Hotel Jakarta building as a starting point for a study to the governing aspects for a building consisting of stacked modules that had to provide its own stability in 2 directions. In the short direction of the modules only one stability wall of 1.8 length is present. Figure 4 shows the final design of a module. In this research a concrete floor of 140 mm thickness was decided after a trade-off against a CLT floor. Two long CLT walls of 142 mm thickness and one stability wall of 142 mm thickness perpendicular to the length of the module. The bathroom is located directly adjacent to this internal wall. The stability walls are connected to the concrete floors by glued in rods with a diameter of 20 mm.

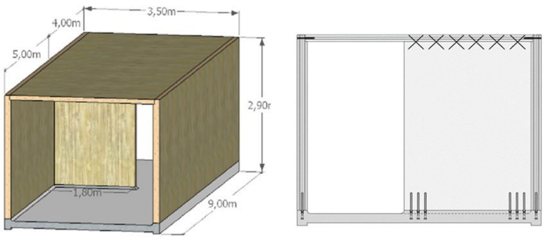

Figure 4: *Structural design of a module representing one hotel room.*

The ceiling CLT element of 99 mm thickness was connected to the perpendicular stability wall with the use of inclined screws with a diameter of 8 mm.

The inter-modular connections designed consist of a steel T-shaped angle plate with pins. See figure 5. The steel pins fit accurately into the steel cones that should be casted into the pre-fabricated floor slab. The reasoning behind the connection consists of the following steps. After two horizontal modules are placed next to each other, the T-shaped angle plate can be screwed on top of two neighbouring side walls. During screwing, the three T-shaped angle plates in a row should be held accurately into position by using a mould over the 6 pins that corresponds with the mould used for precasting the steel cones in the concrete floor. The next step is to place the third module over the angle plate (as seen in the right bottom figure) and to secure the angle plate onto the concrete slab by using (drilled or glued) anchors. After that, the next module can be placed with the accurately pre-casted cones over the steel pins.

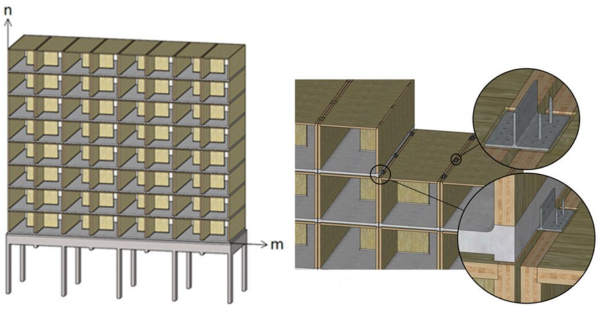

Figure 5: *Structural design the modular structure (left) and the connection of the modulus during construction (right).*

To examine the structural behaviour of the modular assembly in terms of deformations and force distribution a 3D finite element model was made. Then, resistance verifications have been done to find the critical limits for this design case. For timber structures, deformations can be critical, so it is important to make the calculations with realistic input values. To be able to do this the strength and stiffness values of all elements have to be known for model input.

A way to improve the stiffness of screwed connections is the use of inclined screws, as drawn in figure 4 in the upper connection of the stability wall with the ceiling.

Stiffness values for inclined screws are not given in Eurocode 5 yet. Therefore, results from Flatscher, Bratulic and Schickhofer [4] were used. Their research showed that for lateral forces the stiffness of a pair of inclined screws can be a factor of 20 higher than a pair of screws place perpendicular to the load direction. Because of the system, the axial deformation of the screws and glued in rods are also important parameters in the design.

In figure 6 the displacement of a system of 8 by 8 modules under wind loading in the long direction of the building is shown. In figure 7 two adjacent modules under wind loading in the long direction are shown. It indicates the relevance of the bending stiffness of the floor and ceiling, apart from the in-plane rigidity of the wall element and its connections.

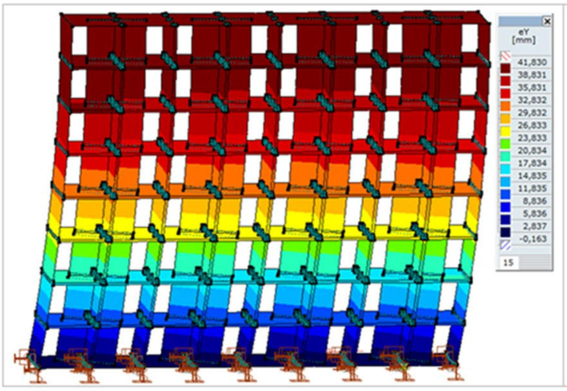

Figure 6: *Displacement of the 8x8 module structure under wind loading in the long direction.*

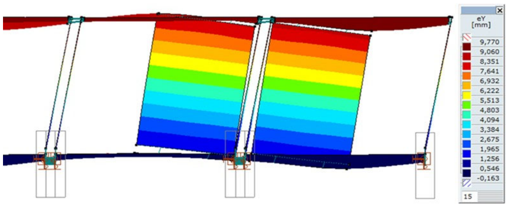

Figure 7: *Displacement of the stability walls due to wind loading in the long direction.*

The aspects that caused the horizontal displacement of the stability walls and their contribution as a percentage to the total displacement were:

- CLT stability wall (16% contribution to the horizontal deformation): shear and bending of the CLT wall
- Slip in the connections (9% contribution to the horizontal deformation):lateral slip in the screws at the ceiling and in the glued in rods at the concrete floor)
- Rotation (75% contribution to the horizontal deformation): rotation due to axial deformation in screws in the ceiling and glued in rods in the concrete floor, and due to bending of the ceiling and bending of the floor.

The maximum horizontal displacement at the top for this case study was approximately 42 mm, which was below the deformation limit that was defined as H/500 = 46.4 mm. Displacement due to wind in the other direction was much lower (below 10 mm) and not governing.

However, the load distribution of the vertical forces of the long walls to the support needed special attention because of the table structure. The walls of the modules were placed on a concrete beam. However, the stacked CLT walls are much stiffer than the concrete beam. This causes a transfer of the vertical forces by arch working, giving high axial forces at the sides if the long walls. The CLT walls were verified by putting the axial forces to a one meter length and verify this part with a hand calculation. It turned out that in the case of CLT walls of 142 mm thickness, the walls exactly met the requirements for buckling in the case of fire, and for the ultimate limit state under normal loads, the number of storeys could be increased to 13.

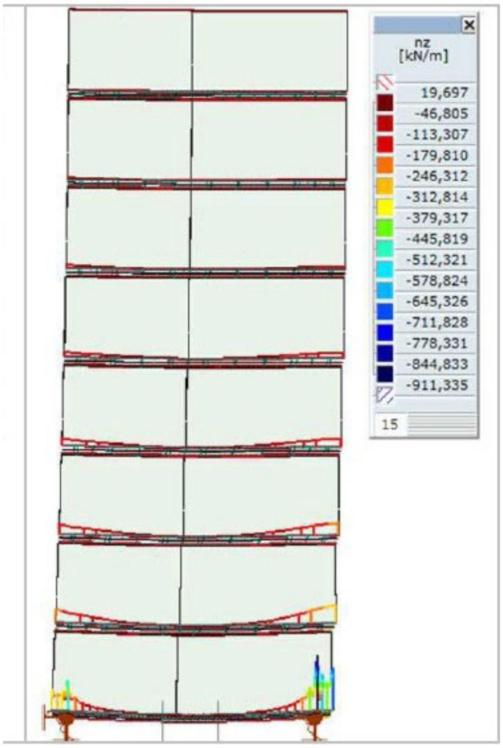

Figure 8: *Force distribution to due to vertical loads in the long walls of the modules.*

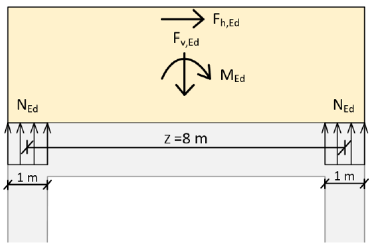

Figure 9: *Assumed mechanical scheme for the verification under axial load of the long walls of the modules.*

The study showed that with the investigated stacked modules a structure of 8 x 8 modules can be designed. The study also showed that the factors to determine the maximum number of stacked modules in length and height are governed by deformations under wind load in the long direction and the buckling capacity of the CLT walls under fire load.
To increase the number of modules, important factors are the stiffness, number of the connections and the dimensions of the CLT panels. Only an analysis combining these aspects can achieve an optimal design.

3 DIFFERENTIAL VERTICAL SHORTENING IN TIMBER-CONCRETE HIGH-RISE

A common way to design high-rise buildings is to use a concrete stability core with a connected glued laminated column and CLT floor system. Figure 10 shows this system for the Brock Commons residential building in Vancouver.

Figure 10: *Structural system of the Brock Commons residential building in Vancouver: A concrete core, glued laminated columns and CLT floors. (photo taken from [5].*

However, the different visco-elastic behaviour of timber and concrete causes differential vertical shortening (DVS) in the different stages of the construction process and over the service life of the structure. So both elastic and creep deformations over time need to be addressed. Willebrands [2] investigated how these differential shortenings occur and how they can be controlled. Figure 11 shows the principle of DVS compared with vertical displacements due to lateral wind loads. These two effects have to be combined to determine the total vertical deformation.

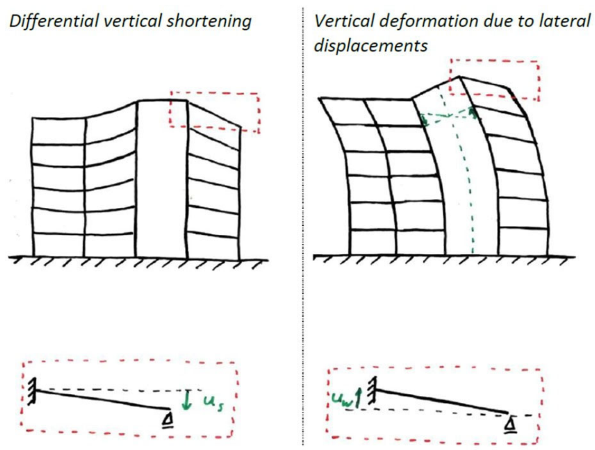

Figure 11: *The effect of differential vertical shortening (DVS) of a concrete core and timber columns (left) and vertical deformations due to wind loads (right).*

The effects of DVS mainly reveal themselves in the serviceability limit state. Codes give limits for deformations to maintain a certain comfort level for people that use the building. These can be verified against mechanical loadings (permanent loads and live loads by people and wind). But vertical deformations also have to be prevented to stay within the limits for the tolerances needed to prevent damage to façades, internal partition walls, ventilation and water supply systems.

For the case studies performed in this study, the following limits for the difference of vertical displacement between concrete core and timber columns as shown in figure 11 were defined:

- Related to damage on secondary building elements: caused by vertical displacements due to the post-DVS and lateral loads: 0.006 *l
- Related to the appearance of the structure: caused by vertical displacements due the total DVS and lateral loads: 0.008* l

The post-DVS deformations are deformations that occur after placement of secondary building systems. l is the span of the floor between concrete core and timber column.

The factors for shrinkage and creep for concrete were based on Eurocode 2. For the timber columns the following assumptions were made:

- The dimensions of the columns are chosen in such a way that in the serviceability limit state the occurring stresses are about 45% of the characteristic compression strength. In that range a linear elastic response is assumed to short term loads.
- Based on a metastudy, the red curve in figure 12 was used for the development of timber creep over time, giving a creep factor of 1.0 over a period of 50 years.
- For shrinkage parallel to the grain of the columns the moisture content at construction was assumed to be 15% and the drying over time to 8% according to the graph of figure 13.

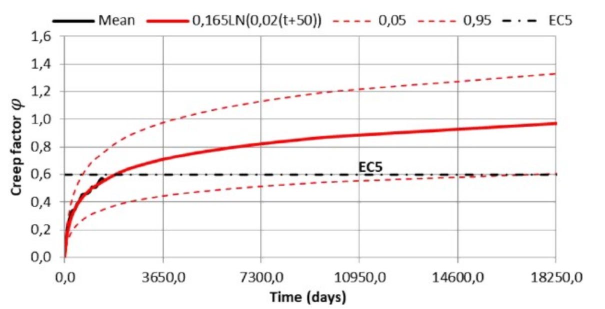

Figure 12: *Assumed creep model for timber loaded parallel to the grain.*

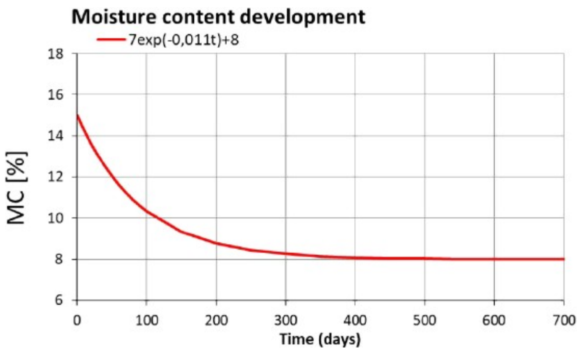

Figure 13: *Assumed moisture content development for the structure after construction.*

The last thing that has to be determined is the building sequence in time. The concrete core will be built ahead of the timber structure. Then the timber structure will be placed, after that concrete toppings on the floors and at last the finishes. That means that creep will start at different moments for different loads. See figure 14.

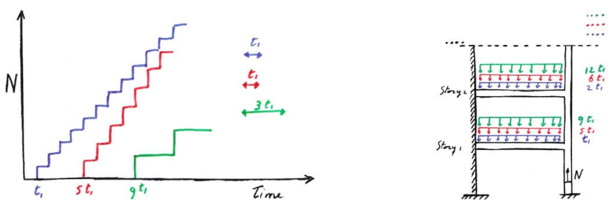

Figure 14: *Staged construction of the timber structure (blue line), concrete toppings (red line) and finishes (green line)*

For instance, in the Brock Commons building, timber columns and a CLT floor were added every 4 days, and the concrete toppings were made in the same sequence, but 5 floors behind. In this building, the total expected shortening of the columns was 45 mm [6]. Compensation of 8 mm was applied by the use of shim plates at the 7th, 11th and 15th level, so 24 mm in total.

The geometry of the case study building in this study is presented in figure 15. Half of the building was modelled in ETABS.

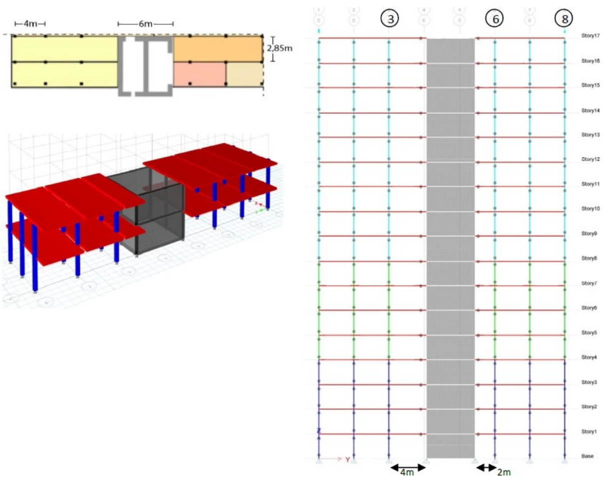

Figure 15*: Geometry of the case study building.*

In figure 16 the model with the dimensions of the columns are shown. The concrete core wall is 450 mm thick and made of concrete strength class C40/50. The columns are made of glued laminated timber GL24h and parallel strand lumber PSL 2.0E.
It is assumed that the concrete core is built 2 months before the timber structure. The erection sequence time of a timber construction storey is 4 days, the 40 mm concrete topping is 5 floors behind the timber construction, and the finishes are 6 storeys behind the concrete toppings.

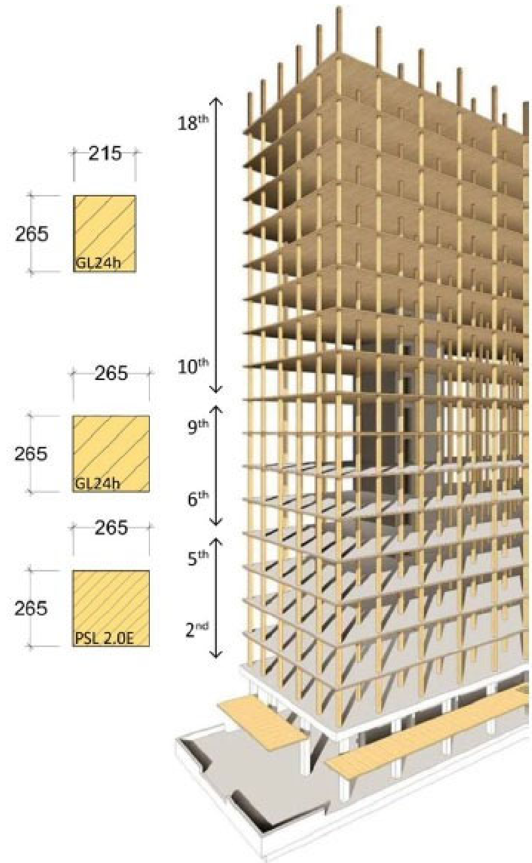

Figure 16: *Dimensions and configurations of the timber columns in the model.*

The ETABS model was validated by hand calculations at different stages. This provided enough confidence to use the model to investigate different aspects. The results will be shown for an example building with 25 storeys.

Figure 17 shows the vertical displacements of the columns on axis 3 over the height of the building due to differential shortening when no compensation is applied.
The green horizontal bar shows the total differential shortening and the orange part of the bar shows the post-DVS, the differential shortening due to the finishing. The post-DVS cannot be compensated. The yellow band shows the defined limit values for the post-DVS and the light green band the defined limit for the total DVS. For this building the post-DVS fulfils the limit requirements. The total DVS largely exceeds the limit requirements, so a compensation strategy is required.

In figure 18 the compensation strategy used in the Brock Commons building is presented, by the use of shim plates compensating for 8 mm starting at storey 6 and then every 4th floor level above. However, figure 18 shows that still the requirements are exceeded at certain levels.

Figure 19 shows another compensation strategy. When the calculated DVS exceeds the requirement (in this case 24 mm), a compensation of 24 mm is applied. At the floor where without compensation 48 mm displacement would occur, again a compensation of 24 mm is applied. At the floor where without compensation 72 mm displacement would occur another compensation is applied. For that position only a compensation of 12 mm is necessary.

Besides this compensation strategy it was also investigated what the influence was on the DVS for aspects such as the time sequence, the column dimensions and floor spans.

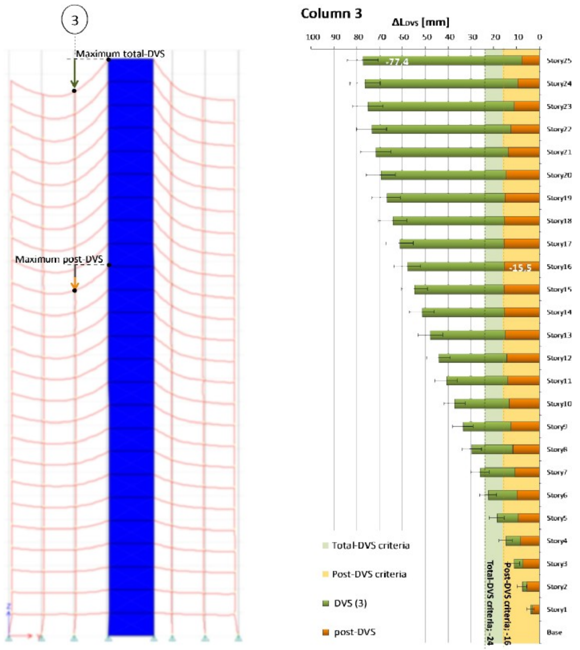

Figure 17: *Vertical deformations due to DVS and limits for the model without compensation.*

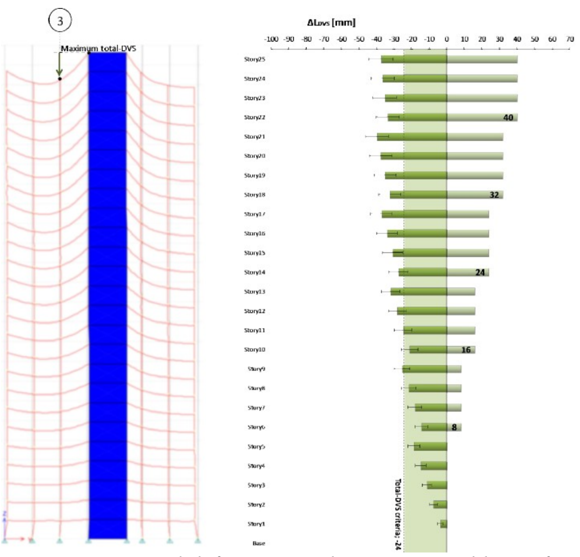

Figure 18: *Vertical deformations due to DVS and limits for the model with 8 mm compensation at storey 6 and every 4th floor up.*

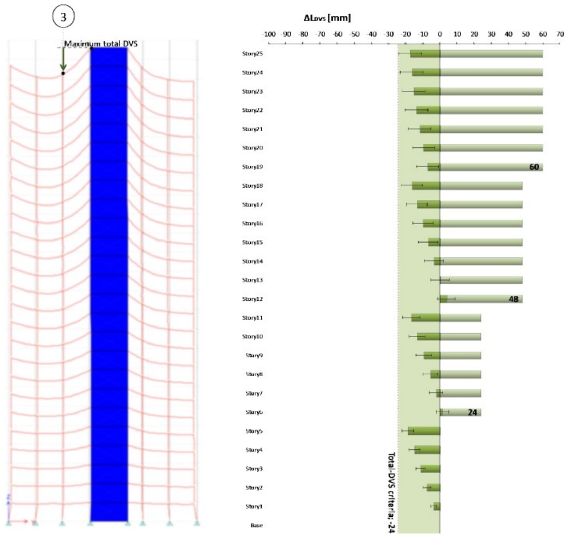

Figure 19: *Vertical deformations due to DVS and limits for the model with 24 mm compensation at storey 6 and 12 and 12 mm compensation at storey 19.*

It can be concluded that vertical differential shortening cannot be neglected in the design of hybrid tall timber buildings using a concrete core and timber columns.

With the use of a good model the effect of compensation strategies and building sequence in time can be investigated to take this into account in the design and construction of these buildings.

4 HYBRID WOOD-CONCRETE SKYSCRAPER IN ROTTERDAM

Slooten [3] investigated the feasibility of hybrid wood-concrete skyscraper in Rotterdam. A 300 m tall building was designed having 76 storeys, a floorplan of 32 m x 32m and a concrete core of 14 m x 14 m. Columns of GLT on a grid of 4.5 m x 4.5 m had a varying cross size from 1.2 m x 1.2 m at the base and 0.75m x 0.75 m at the top. At three levels, storey high outriggers from LVL trusses were placed, through which the cooperation of concrete core and timber columns was achieved for the stability of the system.

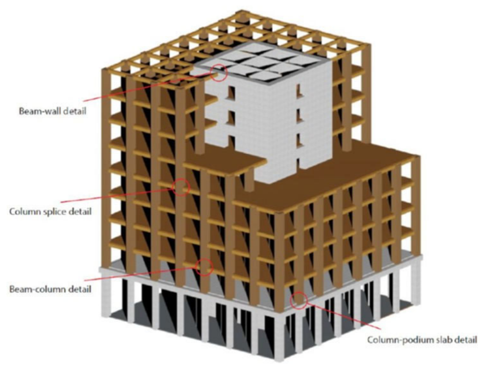

Figure 20: *Main geometry of the building with concrete core and timber columns and floors.*

The main focus in this study was the dynamic behaviour under wind load and with which measures this could be improved. There are two properties that can be used to evaluate the behaviour under wind load: the maximum horizontal deflection at the top and the peak acceleration response of the building. The fire safety engineering of the building was also addressed in this study, by looking at the required the timber dimensions and the effect of encapsulation with gypsum panels.
The structure was modelled in ETABS software. The outriggers introduce extra normal (tension) forces in the columns. Therefore, the stiffness of the column to column connections had to be taken into account. This connection was made by welding rods to base plates. The rods are then glued into the columns ends. The base plates can be bolted together on site. This solution offers the possibility of adding extra steel plates in between columns and floors, to compensate for differential vertical shortening.
The influence of the stiffness of these connections was taken into account by applying lateral springs between the columns. For the loadings, the values according to the Dutch National Annex to the Eurocode 0 were applied with wind loads representative for the coastal area of the Netherlands.
Figure 21 shows the influence of the number of outriggers on the horizontal displacement due to wind loading. This clearly shows the effectiveness of the outriggers, and a number of 3 outriggers was chosen [7]. Then the requirements for the horizontal displacements at the top were met. Figure 22 shows the configuration of the outriggers. They were made of LVL and the connections were designed to transfer the forces.

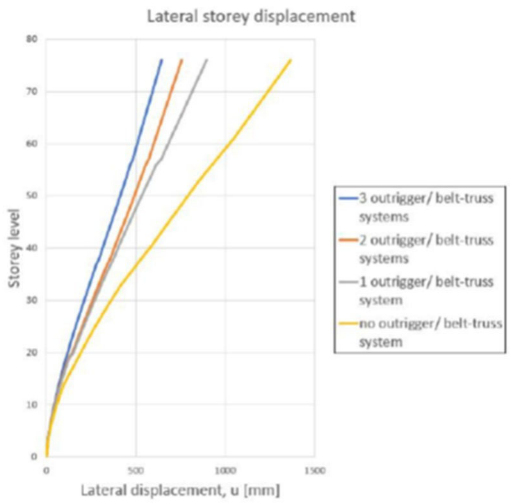

Figure 21: *Influence of the number of outriggers on the horizontal displacement due to wind loading.*

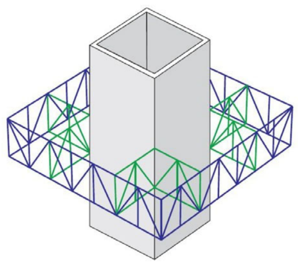

Figure 22: *Configurations of the outriggers*

The configuration with 3 outriggers met the requirements for the horizontal deflection, but not the requirements for the peak acceleration response. For this configuration a peak acceleration response of 0.421 m/s^2 in the along wind direction and 0.674 m/s^2 in the across wind direction, where the limit depends on the natural frequency of the building, but should at least be lower than 0.4 m/s^2.

Another aspect that was studied was if it would be useful to make the column to column and beam to column connections moment resisting instead of hinged.
Figure 23 shows that the difference in horizontal displacement at the top could be around 15%. However, since moment-resisting connections are very expensive, it was decided to continue with hinged connections and find improvements by other measures.

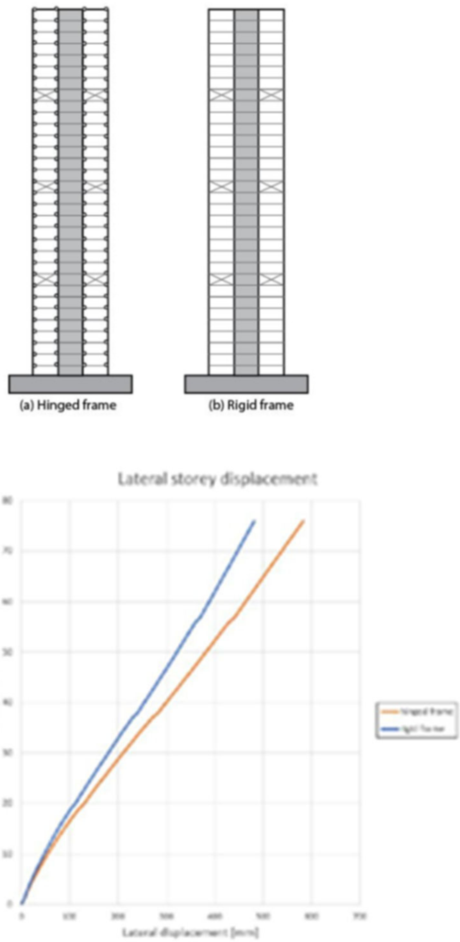

Figure 23: *Influence of the use of moment resisting or hinged connections for column to column and column to beams.*

Two other measures were investigated. The first was a geometrical adaptation of the shape. By applying chamfered corners the wind flow is more favourable leading to lower accelerations.
The second measure was to apply a tuned mass damper on the top of the building. With these two measures the peak acceleration responses in the two directions could be reduced to acceptable levels. Figure 24 shows the adapted building. Figure 25 shows the requirements for the peak acceleration response related to the natural frequency of

the building according to Eurocode 1 and the found peak response accelerations in the along wind direction ($a_{W,max}$) and the across wind direction ($a_{D,max}$).

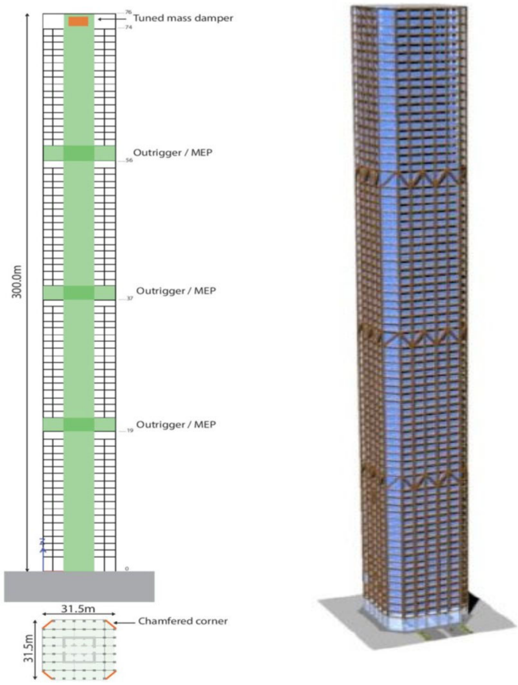

Figure 24: *Adaptions to meet the requirements for peak response accelerations: chamfered corners and a mass tuned damper.*

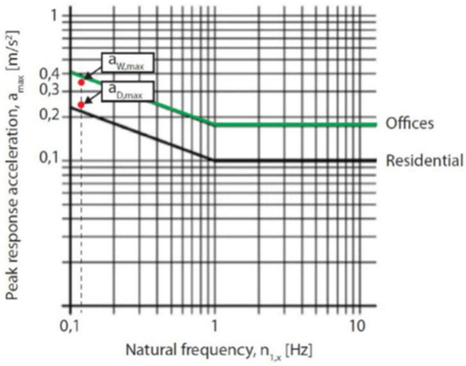

Figure 25: *Peak response accelerations for along and across wind directions of the building (red dots), with the limit values according to Eurocode 1.*

The main conclusions from the research were that a hybrid timber high-rise is feasible and can be optimised for horizontal deflection and accelerations by applying outriggers made of LVL, chamfered corners and the application of a tuned mass damper on the top of the building. In this way all the connections between columns and between beams and columns can constructed as hinges.

CONCLUSIONS

Tall timber buildings can be made of timber modular buildings or hybrid buildings with a concrete core and a timber column floor system that can contribute to the overall stability.
For tall timber buildings specific aspects as the influence of the connections on the stability and comfort, and the different behaviour of timber and concrete have to be taken into account. When this is done in a proper way, tall timber buildings can designed with different appearances to fulfil the requirements and the wishes of architects and users.

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