MSc Program

Wood Based Building Design for Sustainable Urban development

Bracing for slender multi-storey Timber Buildings

A Master Thesis submitted for the degree of

"Master of Science"

Supervised by

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Preface

The present work was developed in the context of the research project “8+”, the possibilities of multi-storey timber buildings in urban context.

I would like to express my deep gratitude to O. Univ.-Prof. Dipl.-Ing. Dipl.-Ing. Wolfgang Winter who made my attendance to this Master course possible and who advised me as his research assistant at the Institute of Architectural Sciences, Structural Design and Timber Engineering at Vienna University of technology.

I also would like to thank Dipl.-Ing. Dr. Kamyar Tavoussi who advised me and with whom I had a lot of productive discussions during the research project.

I also would like to thank Dipl.-Ing. Dr. Yoshiaki Amino who kindly read this draft and advised me.

Cordial thanks to my wife Olivia who supported me during the whole time. We got married during a three weeks brake of the master course and she joined me to Torino for honeymoon.

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List of abbreviations

CLT	cross laminated timber
DIN	“Deutsches Institut für Normung”, German institute for standardization
EC	Eurocode
GL	glue laminated
RFEM	3D Finite element analyses software from Ing. Software DLUBAL
RSTAB	3D framework program from Ing. Software DLUBAL
U_{total}	total displacement

Abstract

The objective of the research work is to investigate a possible twenty storey high office timber building and to discuss about the problem of load transfer and bracing.

Considering boundary conditions like geographical, geometrical and structural criteria the design is done according to the significant lateral loads. Therefore the dynamic behaviour due to the load cases wind and earthquake is analysed.
In a research approach several possibilities of load bearing, bracing and floor systems are compared. For load bearing systems two dimensional linear or surface elements or a combination of both are practicable. For bracing systems the stiffening with diagonals and the stiffening with rigid frame connections are investigated. The floor structure has various functions like sound insulation, load bearing and stiffening.
On the basis of two examples the different structures are investigated. The first one is a square type partial crosswise stiffened by diagonal elements and the second one is a rectangular type with rigid connections of cross laminated timber elements in vertical and horizontal direction.
The dynamic calculations are carried out by framework software and finite element program.

Due to the softness of slender multi-storey timber structures the buildings show good behaviour in case of earthquake. Dynamic wind loads have more influence considering Viennese circumstances.
For optimal stiffening the bracing system is located in the façade connected by a shear stiff floor structure.
Cross laminated timber panels seem to be efficient solution combining high shear stiffness for bracing, high load bearing capacity and good characteristic values as façade elements. Rigid or partial rigid connections of theses cross laminated timber elements might be carried out by simple steel connectors. The behaviour of such connections must be investigated.

1. Introduction and objective

Beginning 2007 the architectural office “schluderarchitektur” started a research project about the possibilities of multi-storey timber buildings in urban context. The target was the investigation of an eight or more storey timber office building. It was funded by “Haus der Zukunft”, “BM VIT”.
The motivation is linked to the obvious signs of changing of climate. To maintain the standard of living it will be necessary to use and to consume raw materials in a careful way. Therefore the material wood which grows again seems to be predestined. The strategy is to apply timber structures not only in the periphery but as well in city centres. According to that the new or different requirements represent a real challenge. In the field of timber products there has already been an intensive development in the last years. Also the new guidelines of the “Wiener Bauordnung” allow since 2001 four storeys in wood. Prof. DDI Wolfgang Winter of the Vienna University of Technology made significant contributions to these proceedings.
The specific aims of this research project are related to economical, safety, static and architectural questions. Therefore several partners were asked to work together. The Institute of Architectural Sciences, Structural Design and Timber Engineering leaded by Prof. Winter was in charge of the investigation of possible structures in terms of load bearing and bracing systems. The approach and results of these investigations are carried out in this master’s thesis.

For the study following questions will be analysed:

- Do multi-storey timber buildings exist?
- What structures are possible?
- Where are the limits?

Dealing with possible timber structures for multi-story buildings it is crucial to know about existing multi-storey timber buildings and their structures.

A first example of a multi-storey timber building is the Todaj-ji temple in Japan, which is a building of the first millennium.

figure 1.1: Todaj-ji Tempel (Nara, Japan)

(Source: Institute of Architectural Sciences, Structural Design and Timber Engineering, Vienna University of Technology)

The Todaj-ji temple was constructed in 752 in Nara, Japan. It is the highest timber structure worldwide with about 50 meters. The columns are made out of cedar logs with diameter of 2 meters.

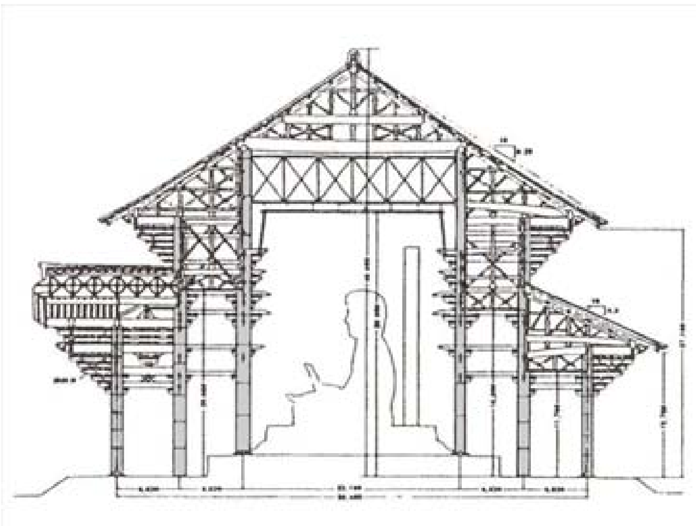


figure 1.2: Section of Todaj-ji Tempel (Nara, Japan)

(Source: Preservation and seismic Retrofit of the traditional Wooden Buildings in Japan; Journal of temporal Design in Architecture and the Environment, Vol. 1, No. 1; K. Katagihara; 2001)

Other examples of modern multi-storey timber buildings with different usage and different structures are given below.

A four storey timber building designed by architects Kaufmann is part of a completed project for low-rise social housing with European passive building standards at “Mühlweg” in 1210 Vienna. The structure consists in massif wooden panels and industrialized pre-manufactured timber frames.

figure 1.3: Timber Passive House at Mühlweg, 1210 Vienna, Austria 2006

(Source: Bruno Klomfar)

A five storey timber building designed by architects Dietrich and Untertrifaller is part of the same project located at “Mühlweg” as mentioned before. The structure consists in massif wooden panels highly prefabricated.

figure 1.4: Timber Passive House at Mühlweg, 1210 Vienna, Austria 2006

(Source: Bruno Klomfar)

The first six storey timber building in Switzerland by architects Scheitlin, Syfrig and Partner is a completed project in summer 2006 in Steinhausen. It is a residential and office building. The structure consists in a massif concrete core and stiffening wooden walls. The fast and inexpensive construction method is due to the high level of prefabrication.

figure 1.5: Steinhausen timber building, Switzerland 2006

(Source: http://www.holzhausen.ch/, 04.09.2008)

In 2007 a seven storey residential building with timber skeleton construction designed by architects Kaden and Klingbeil was completed in Esmarchstrasse 3, Berlin, Germany.

figure 1.6: e3 timber building, Germany 2007

(Source: http://www.e3berlin.de/haus/index.php, 04.09.2008)

In the same year the block Limnologen in Växjö, Sweden consisting in four eight storey houses started to grow. Two of the four houses are already finished, and the other two are being mounted in the autumn of 2008. The structure is made by cross laminated timber (CLT) panels and the system two by four.

figure 1.7: Limnologen - eight storey timber houses, Sweden 2007

(Source: „Holzbau in nachhaltiger und moderner Stadtentwicklung – ein Praxisbeispiel aus Schweden, umsetzbar in Deutschland?“, Tobias Schauerte)

The architects Waugh Thistleton designed a nine storey timber residential tower in London built up in less than eight weeks. The structure is made out of CLT panels.

figure 1.8: Timber residential tower Murray Grove, Great Britain 2008

(Source: dezeen, design magazine)

As these examples shows timber constructions up to fifty meters for special usage buildings and up to nine storeys for residential buildings were realized.

The objective of the research work is to investigate a possible twenty storey high office timber building and to discuss about the problem of load transfer and bracing.

2. Boundary conditions

2.1 Criteria

To limit the extent of the research work and the results following criteria were specified:

- The location for this building is Vienna, Austria.
- The type is an office building.
- The structure consists in three reinforced concrete storeys and seventeen timber storeys.
- The floor to floor height is 3,25 meters.
- The overall height amount to 65 meters.
- The plan area should reach about 750 square meters.

2.2 Parameters

The location has an important influence on the design and the construction of a building.
First there exist specific building regulations for every country and every state. The design must be carried out according to these regulations. For being flexible in the choice of the location the design should be adaptable. The design should as well convince for possible changes of requirements.
Second the wind zone and earthquake zone are significant for the dynamic loads applied on the building. The design depends definitively on these parameters.

2.2.1 Building regulations

"In 2001, the building regulations of Vienna were modified to allow the construction of multi-storied timber houses with up to five storeys, provided that the supporting elements for the ground floor are made of mineral materials.
The legal basis for this analysis is the draft of the new building regulations of the City of Vienna which draw from directives of the Austrian Institute of Construction Engineering OIB (directive 2 on fire protection). The structural components of buildings with up to seven storeys (building class 5) are required to have a fire resistance of 90 minutes and, in addition, need to be composed of building materials with at least class A2 fire performance according to EN 13502-1. However, OIB directive 2 also notes that these requirements may not necessarily apply if there is solid proof of measures that guarantee equivalent protection."

(Source: Feasibility Analysis of Seven-Storied Timber Houses,
DI Dr. Martin Teibinger, Thomas Busch)

2.2.2 Wind zone

The wind velocities shown in the following maps represent average wind speeds. Values used for calculations of wind loads according to codes will differ. They have to respect the range of the wind velocity. The exact procedures are given in chapter 3.2 codes.

The map below is a world, year-round average wind speed map. We see that the greatest wind velocities are above the oceans and on the coasts with about 6 m/s and faster marked in yellow, orange, green and red in the figure. The midcontinent regions reach wind speeds of 1 to 5 m/s which are marked in purple and different blue colours in the figure.

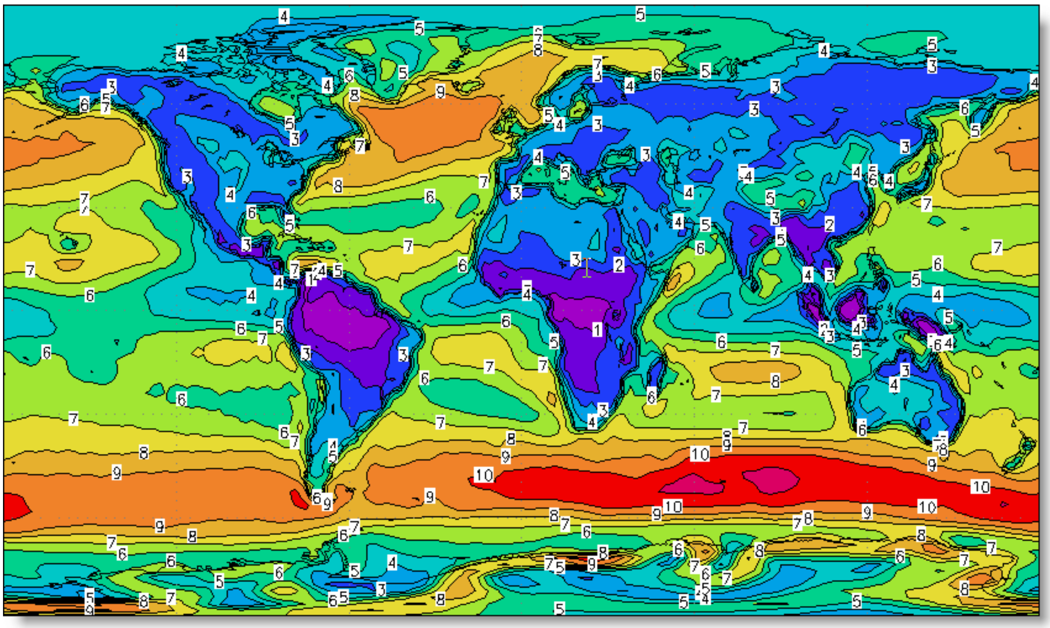


figure 2.1: World wind map

The wind map of central Eastern Europe shows the range of wind velocity in Austria more precise. Lower wind speeds from 2 to 3 m/s are measured in Vienna. In western parts of Austria the wind speed reaches up to 10 m/s. As we are speaking about average values the wind speeds used for calculating the wind loads are higher but in between a lower range.

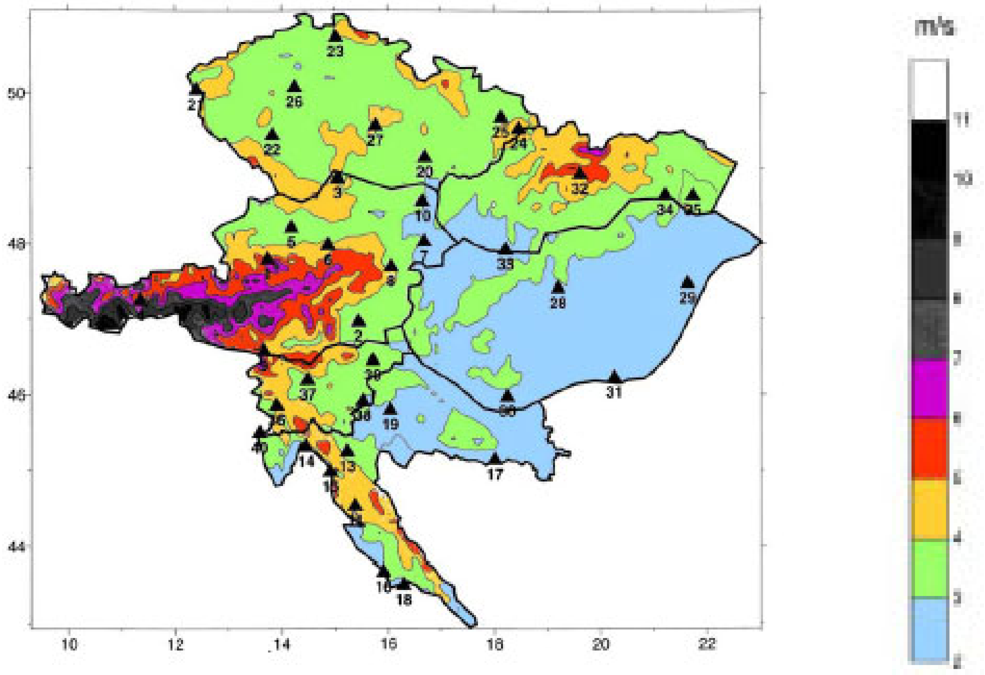


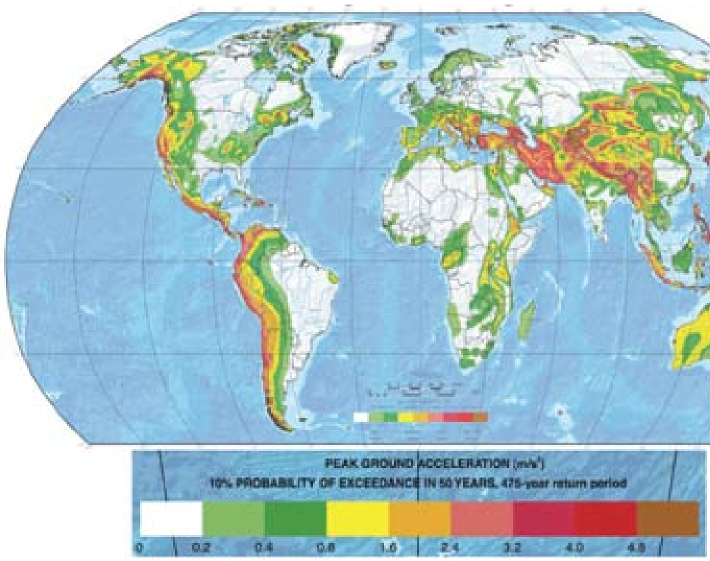
figure 2.2: Wind map of central eastern Europe

(Source: Jungbauer, 1998)

2.2.3 Earthquake zone

The following maps show the global and local peak ground accelerations and the seismicity in Austria.
The maps show the peak ground acceleration (PGA) that a site can expect during the next 50 years with 10 percent probability.

The global seismic hazard map below shows the strongest earthquake zones in red located on the pacific coasts and the mountain chain von south east Europe across the middle east until the Himalaya. Central Europe is located in a low to medium earthquake hazard zone marked in green in the map.



[SED 1999)]

figure 2.3: Global seismic hazard map

(Source: „Schweizerischer Erdbebendienst SED“: Global Seismic Hazard Assessment Program (GSHAP), Global Seismic Hazard Maps, seismo.ethz.ch/GSHAP/, ETH Zürich.)

The European seismic hazard map shows the north half of Europe in a peak ground acceleration zone up to 0,8 m/s². The southern half of Europe is the higher hazard zone with peak ground accelerations up to 4,8 m/s².

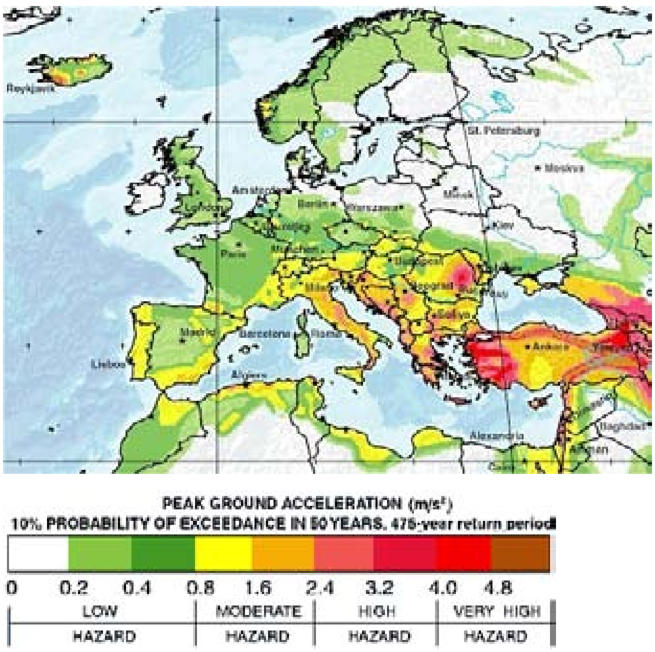


figure 2.4: European seismic hazard map

(Source: http://geology.about.com/library/bl/maps/bleurope.htm, 05.09.2008)

The peak ground acceleration in Austria can reach a value of about 1,6 m/s². The strongest earthquake zones are the so called "Mur-Mürz-Furche" and the region around Innsbruck marked in red. Vienna is situated in a medium zone. In Vienna the ground acceleration applied for calculations is 0,87 m/s².

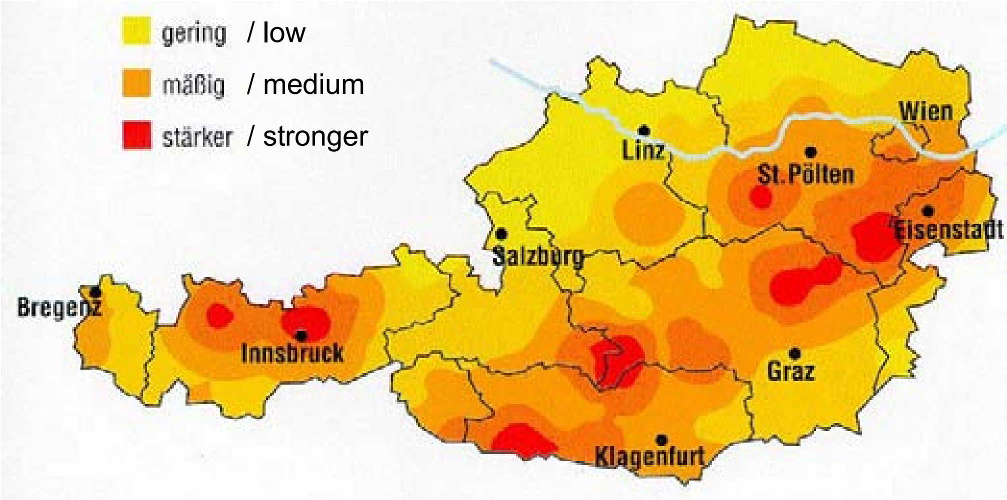


figure 2.5: Austrian seismic hazard map

(Source: ÖNORM B 4015, ÖSTERREICHISCHE GESELLSCHAFT FÜR ERDBEBENINGENIEURWESEN UND BAUDYNAMIK; http://www.oge.or.at/oge_norm.htm, 05.09.2008)

The seismicity maps below of Austria indicate not only the geographic position and the frequency of earthquakes but as well the intensity and the depth.

The intensity is given by the European macroseismic scale EMS-98 which is based on the Mercalli- Sieberg. The scale is increasing from 1-impalpable to 12-complete destruction.
The relevant range for constructions in Austria is from 6 to 8 which present ground accelerations of 0,25 to 2 m/s². The consequences are low to high damages on structures.

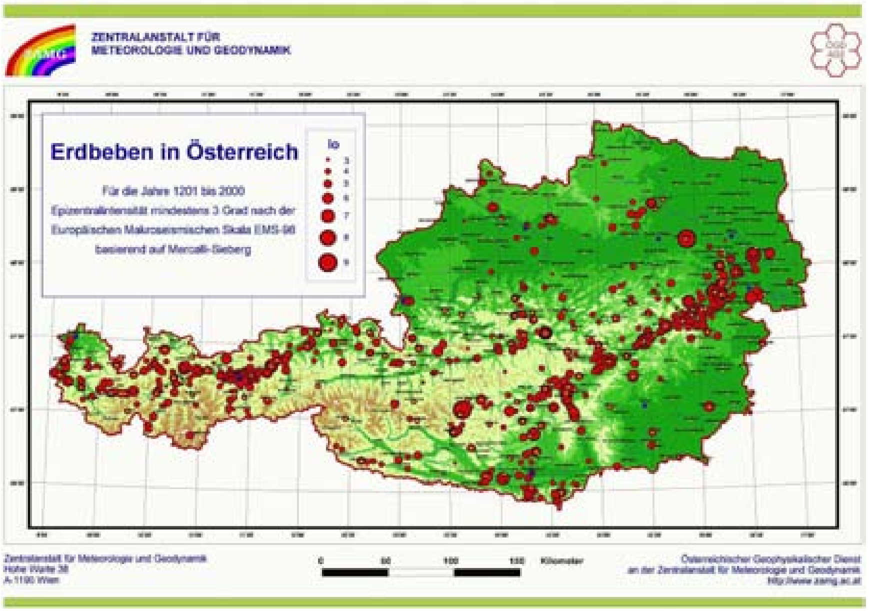


figure 2.6: Seismicity of Austria, 1201-2000

(Source: ZAMG, „Zentralanstalt für Meteorologie und Geodynamik“)

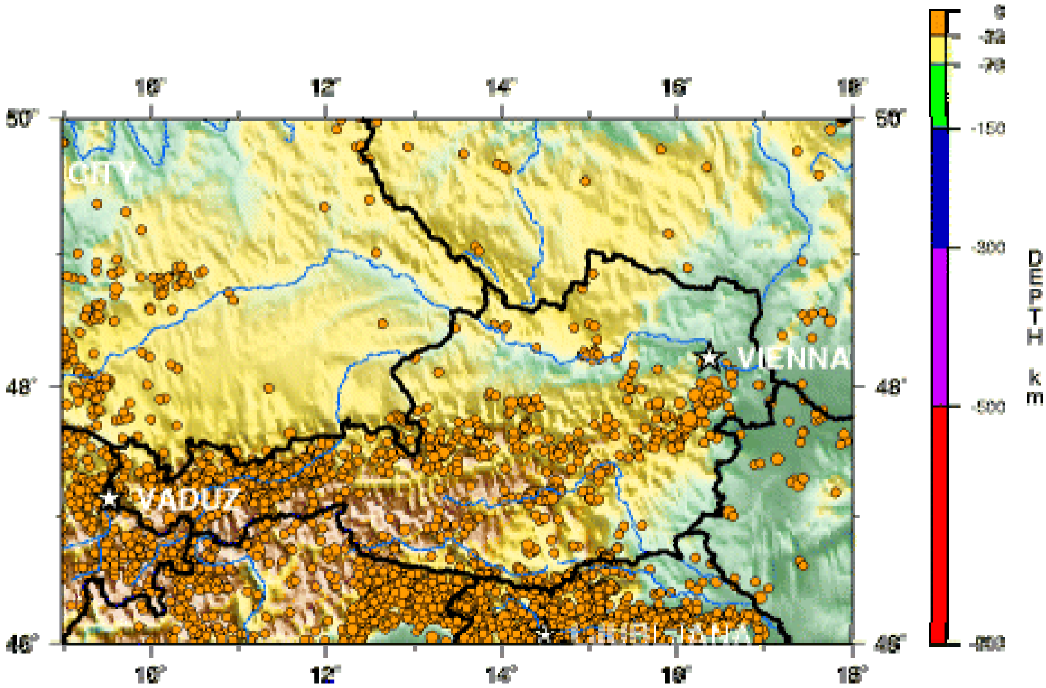


figure 2.7: Seismicity of Austria, 1990-2006

(Source: USGS, http://earthquake.usgs.gov/regional/world/austria/seismicity.php, 05.09.2008)

3. Design

3.1 Load assumptions

The load assumptions are made according to EC 1.
The overall floor load calculated in the table below doesn't take into account the factors for load case combinations.

table 3.1: Load assumptions

Loads		
Live load	3 kN/m²	(300 kg/m²)
Live load staircase	5 kN/m²	(500 kg/m²)
Dead load floor (incl. CLT - panel)	2,5 kN/m²	(250 kg/m²)
Load increase for partitions	0,5 kN/m²	(50 kg/m²)
Overall load floor	6 kN/m²	(600 kg/m²)

3.2 Codes

3.2.1 Load case wind

For the load case wind the design was made according to ÖNORM B 4014.
It is different to the design according to EC concerning the factors respecting the dynamic influences. Nevertheless the calculated values for wind loads should not differ too much.
As a first step it is relevant to analyse if the building is susceptible to oscillations. Buildings with heights up to 40 meters are not susceptible to oscillations as well as buildings with heights between 40 to 100 meters if the logarithmic damping ratio δ satisfy following inequality (see formula 3.1).

$$\delta = \delta_1 + \delta_2 + \delta_3 \geq \frac{3{,}3\left(\frac{h}{h_0}\right)^{2/3}}{\left(1+\frac{3\,f_e h}{\bar{v}_h}\right)\cdot\left(1+6\,\frac{f_e h}{\bar{v}_h}\cdot\frac{l_e}{h}\right)\cdot\left(\frac{f_e h}{\bar{v}_h}\right)^{2/3}\left\{s^2-1+1{,}7\left(\frac{h}{h_0}\right)^{0{,}5}\left[1+0{,}5\left(\frac{l_e}{h}\right)^{0{,}5}\right]\right\}} \qquad \textbf{3.1}$$

where

δ is the logarithmic damping ratio

δ_1 is the component of the damping ratio according to the material

δ_2 is the component of the damping ratio according to the structure

δ_3 is the component of the damping ratio according to the foundation

h is the height in m

h_0 is 1200 m

fe is the 1. natural frequency in Hz

le is the determinant length (bracing members) in m

$\overline{v_h}$ is the hourly average wind velocity depending on the height of the building in m/s

s is the size factor

The wind velocity $\overline{v_h}$ depends on the basic wind value v_{10}, the height over the ground and the landform. v_{10} for Vienna is 135 km/h. There are three categories of landforms, from 1 with no obstacles for wind to 3 with numerous tall obstacles like houses. To take into account the maximum possible wind load category 1 is chosen. The wind velocity $\overline{v_h}$ increase from 22,6 m/s in 6 meters height to 32,9 m/s in 75 meters height.

The static wind load is calculated with this formula:

$$w = c_p \cdot q_p(z) \text{ [kN/m}^2\text{]} \qquad \mathbf{3.2}$$

For the dynamic wind load the static wind load is multiplied by the gust response factor φ:

$$w = \varphi \cdot c_p \cdot q_p(z) \text{ [kN/m}^2\text{]} \qquad \mathbf{3.3}$$

$$\varphi = 1 + g_s \cdot \sigma_w \qquad \mathbf{3.4}$$

$$\sigma_w = \frac{4}{\sqrt{3}} \cdot I_h \cdot \sqrt{\frac{s_1 \cdot F_1}{\delta + \delta_a} + B_1} \qquad \mathbf{3.5}$$

$$\delta_a = \frac{\rho \cdot I_e}{m_G} \cdot \frac{\overline{v_h}}{f_e} \cdot \frac{c}{2} \qquad \mathbf{3.6}$$

where

w is the wind pressure in kN/m²

c_p is the pressure coefficient

$q_p(z)$ is the peak velocity pressure in kN/m²

(z) is the reference height in m

φ is the gust response factor

g_s=3,6 is the peak factor

σ_w is the standard deviation

I_h is the turbulence intensity

s_1 is the dynamic size factor

F_1 is the gust energy factor

δ is the logarithmic damping ratio

δ_a is the aerodynamic decrement

B_1 is the gust basic component

ρ is the air density (1,25kg/m³)

m_G is the generalized mass in kg/m

3.2.2 Load case earthquake

The load case earthquake is designed according to EC 8.

The calculations are done with equivalent lateral forces. The conditions are fulfilled (see table 3.4, figure 3.2).

They are generated according to standard EUROCODE 8:2004-11.

The type of spectrum used is a design spectrum for linear calculation.

For Austria the spectrum type 1 is relevant.

The lower bound factor β = 0,2 is taken as limit value for the horizontal design spectrum.

As mentioned before the ground acceleration for Vienna is a_g = 0,87 m/s².

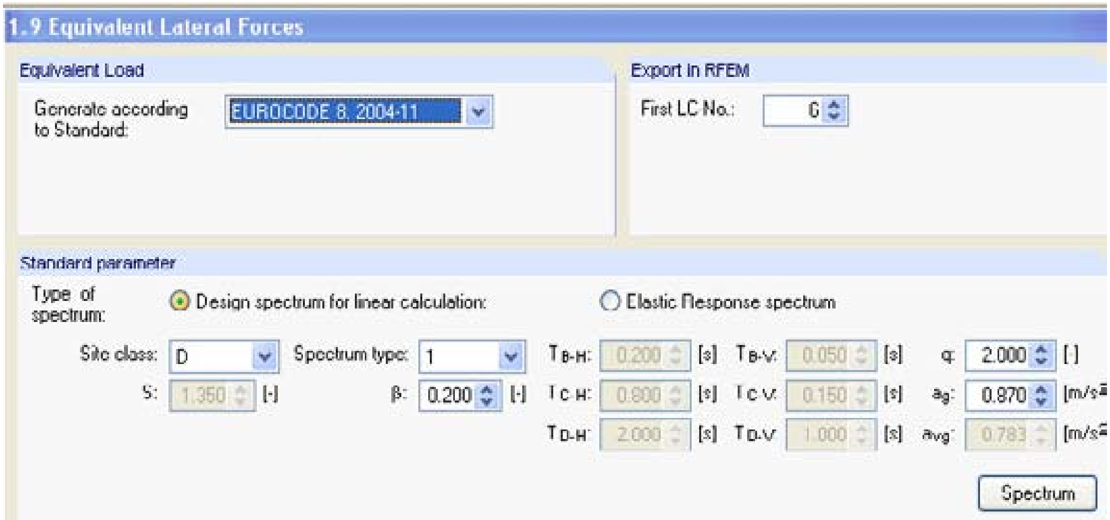


figure 3.1: Input values load case earthquake, screen shot DYNAM

As a low quality ground type the site class D is chosen (see table 3.2).

table 3.2: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 – 50	70 – 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 – 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

(Source: EN 1998-1:2004 (E) - 4)

The importance class 3 (see table 3.3) for the building is respected for the displacement limits.

table 3.3: Importance classes for buildings

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

NOTE Importance classes I, II and III or IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, Annex B.

(Source: EN 1998-1:2004 (E) - 4)

table 3.4: Consequences of structural regularity on seismic analysis and design

Regularity		Allowed Simplification		Behaviour factor
Plan	Elevation	Model	Linear-elastic Analysis	(for linear analysis)
Yes	Yes	Planar	Lateral force[a]	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial[b]	Lateral force[a]	Reference value
No	No	Spatial	Modal	Decreased value

(Source: EN 1998-1:2004 (E) - 4)

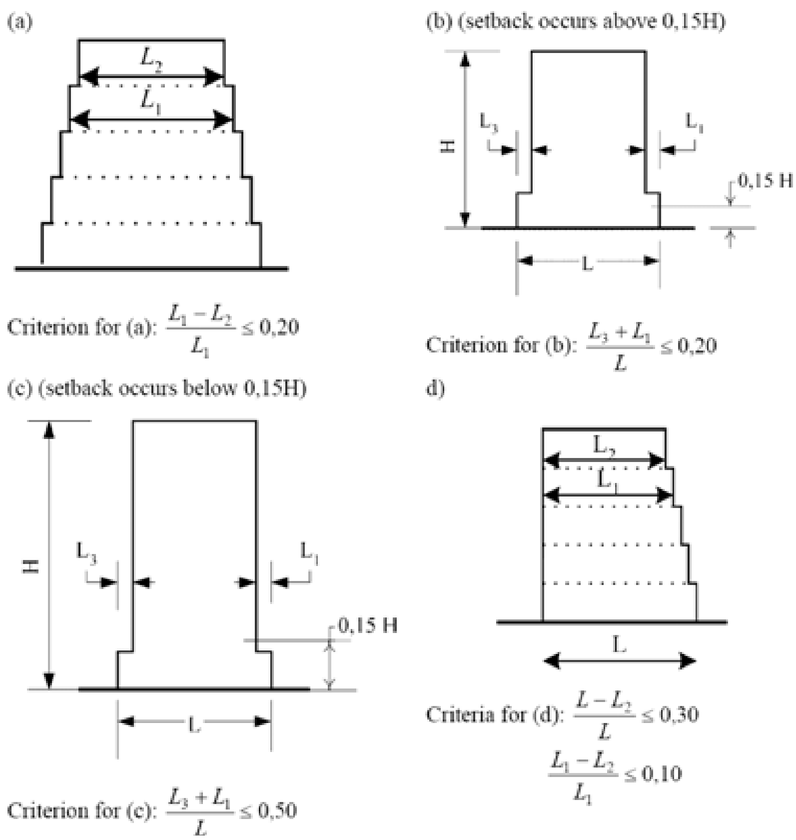


figure 3.2: Criteria for regularity of buildings with setbacks

(Source: EN 1998-1:2004 (E) - 4)

The influence of the ductility of the construction is taken to account with the behaviour factor q = 2. For wood constructions the minimum behaviour factor to expect is q=1,5 and the maximum is q=4. That means the behaviour factor chosen for the calculations is considered on the safe side.

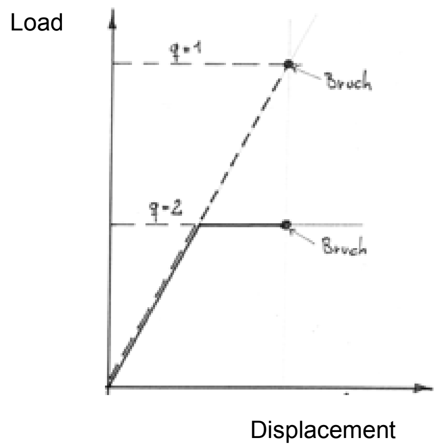


figure 3.3: Load-displacement diagram for behaviour factor q

(Source: Institute of Architectural Sciences, Structural Design and Timber Engineering, Vienna University of Technology)

For the horizontal components of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions:

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2{,}5}{q} - \frac{2}{3} \right) \right] \qquad \textbf{3.7}$$

$$T_B \le T \le T_C : S_d(T) = a_g \cdot S \cdot \frac{2{,}5}{q} \qquad \textbf{3.8}$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_C}{T} \right] \\ \ge \beta \cdot a_g \end{cases} \qquad \textbf{3.9}$$

$$T_D \le T : S_d(T) \begin{cases} = a_g \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_C T_D}{T^2} \right] \\ \ge \beta \cdot a_g \end{cases} \qquad \textbf{3.10}$$

where

a_g is the design ground acceleration on type A ground

S is the soil factor

T_C is the upper limit of the period of the constant spectral acceleration branch

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

Sd (T) is the design spectrum;

q is the behaviour factor;

β is the lower bound factor for the horizontal design spectrum.

NOTE: The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for β is 0,2.

The parameters used for the horizontal elastic response spectrum for ground type D are marked in red in table 3.4.

table 3.5: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

(Source: EN 1998-1:2004 (E) – 4)

According to these parameters the horizontal design spectrum is shown in figure 3.4.

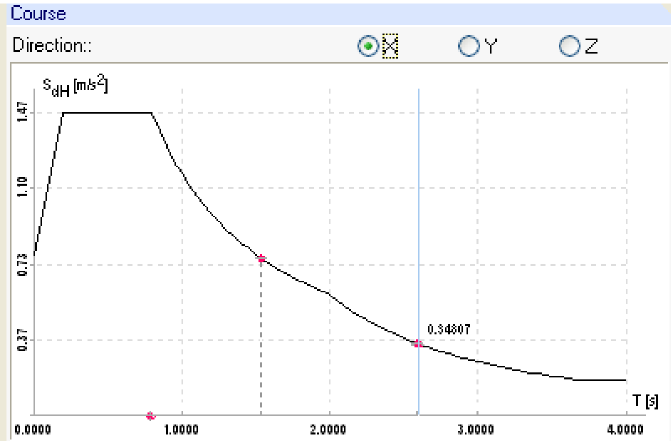


figure 3.4: Horizontal design spectrum, screen shot DYNAM

For the vertical components of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions:

$$0 \le T \le T_B : S_{vd}(T) = a_{vg} \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{3}{q} - \frac{2}{3} \right) \right] \qquad \textbf{3.11}$$

$$T_B \le T \le T_C : S_{vd}(T) = a_{vg} \cdot \frac{3}{q} \qquad \textbf{3.12}$$

$$T_C \le T \le T_D : S_{vd}(T) = a_{vg} \cdot \frac{3}{q} \cdot \left[\frac{T_C}{T} \right] \qquad \textbf{3.13}$$

$$T_D \le T : S_{vd}(T) = a_{vg} \cdot \frac{3}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \qquad \textbf{3.14}$$

The parameters used for the vertical elastic response spectrum for spectrum type 1 are marked in red in table 3.6.

table 3.6: Recommended values of parameters describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

(Source: EN 1998-1:2004 (E) – 4)

According to these parameters the vertical design spectrum is shown in figure 3.5.

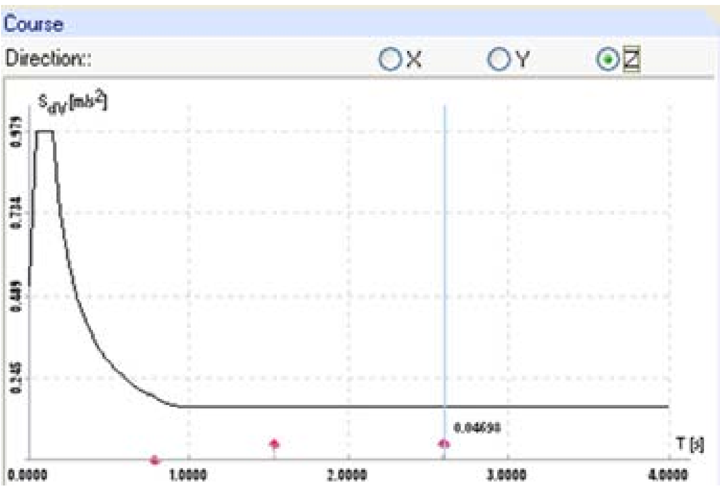


figure 3.5: Vertical design spectrum, screen shot DYNAM

The combinations for components of the design spectrum are described in following expressions:

E_{Edx} "+" 0,30 E_{Edy} "+" 0,30 E_{Edz} **3.15**

0,30 E_{Edx} "+" E_{Edy} "+" 0,30 E_{Edz} **3.16**

0,30 E_{Edx} "+" 0,30 E_{Edy} "+" E_{Edz} **3.17**

where

"+" implies "to be combined with";

E_{Edx} represents the action effects due to the application of the seismic action along the chosen horizontal axis x of the structure;

E_{Edy} represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure.

EEdz represents the action effects due to the application of the vertical component of the design seismic action as defined.

3.2.3 Displacement limits

The displacement limits are calculated for the load cases wind and earthquake according to following formulas:

- Load case wind

u_{total} < h/500 **3.18**

u_{total} < 65m/500

u_{total} < 13 cm

The displacement limit chosen for the load case wind is very strict. The limit might vary with the different national codes.

- Load case earthquake

The displacement limit for the load case earthquake is given by the limitation of interstorey drift:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$d_r v \leq 0{,}005h$ **3.19**

b) for buildings having ductile non-structural elements:

$d_r v \leq 0{,}0075h$ **3.20**

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$d_r v \leq 0{,}010\ h$ **3.21**

where

d_r is the design interstorey drift;

h is the storey height in meter;

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.

The value of the reduction factor v may also depend on the importance class of the building. Implicit in its use is the assumption that the elastic response spectrum of the seismic action under which the "damage limitation requirement" should be met has the same shape as the elastic response spectrum of the design seismic action corresponding to the "ultimate limit state requirement".
NOTE: The values to be ascribed to v for use in a country may be found in its National Annex. Different values of v may be defined for the various seismic zones of a country, depending on the seismic hazard conditions and on the protection of property objective. The recommended values of v are 0,4 for importance classes III and IV and v = 0,5 for importance classes I and II.

(Source: EN 1998-1:2004 (E) - 4)

Formula 3.19 is used to limit the displacement for the load case earthquake:
dr*v ≤ 0,005*h
dr ≤ 0,005*h/v
dr ≤ 0,005*3,25m/0,4
dr ≤ 4 cm (per storey)
u_{total} < 68 cm (for 17 timber storeys)

3.2.4 Load case combination

The load case combinations are defined according to standard Eurocode EN 1990:2002.

As the wind loads are taking into account the dynamic behaviour, they are influenced by the first natural frequency of the building. This value depends on the load applied on the building. So for every load case combination the applied wind load will differ.

For the ultimate limit states (ULS) following four load case combinations are investigated:

LK1: $1{,}35g+1{,}5p+1{,}5*0{,}6w_1$ **3.22**

LK2: $1{,}35g+1{,}5w_2+1{,}5*0{,}7p$ **3.23**

LK3: $1g+1{,}5w_3$ **3.24**

LK4: $1g+1e+0{,}3p$ **3.25**

For the serviceability limit states (SLS) following two load case combinations are investigated:

LK5: $1g+1w_5+0{,}5p$ **3.26**

LK6: $1g+1p+0{,}5w_6$ **3.27**

where

LK is the load case combination

g is the dead load

p is the live load

w is the wind load

- w_1 is the result of $1{,}35g+1{,}5p$
- w_2 is the result of $1{,}35g+1{,}05p$
- w_3 is the result of $1g$
- w_5 is the result of $1g+0{,}5p$
- w_6 is the result of $1g+1p$

e is the load due to earthquake

4. Research approach

This chapter should point out the reflections made entering the topic of multi-storey timber buildings considering the geometry, the load bearing system, the bracing system and the floor structure.
Different options according to construction history and completed projects are investigated.

4.1 Geometry

For the geometric conception two alternatives are investigated. One is a square type with a core for staircases, elevators and restrooms. The other one is a rectangular type. The relevant aspect for the office building is a flexible space with column free areas. The space should easily be separated for private owners who need about 250 m². Also the guidelines for evacuation need to be taken into account. Constructions must be realised that in case of fire the users can fast and safe escape the building. For example the Austrian regulations prescribe escape routes of maximum forty meters from any point of the building to the staircases or a safe place. The precise details can be found in the Austrian regulations.

The square type has a plan area of 27*27 = 729 m².

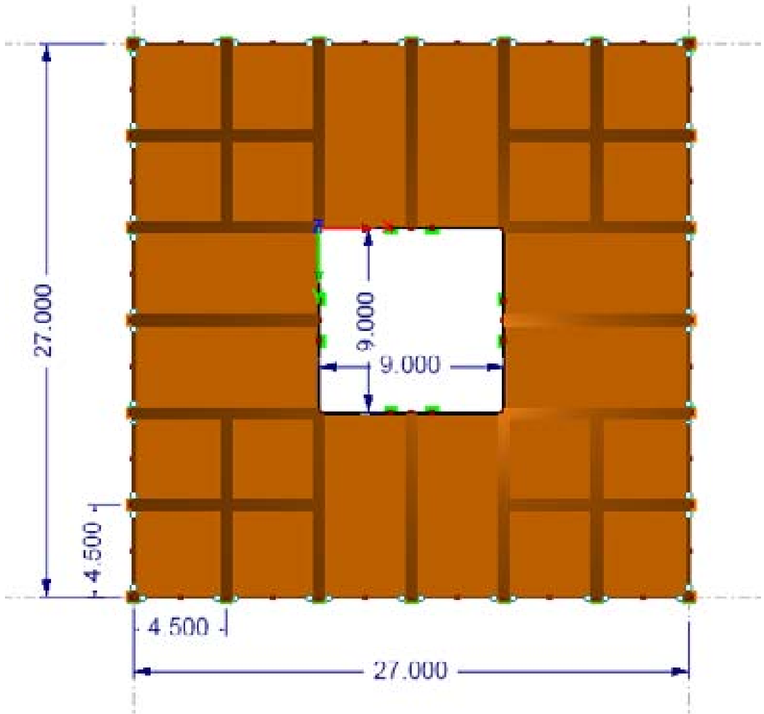


figure 4.1: Plan view of square type geometry, screen shot RFEM

The rectangular type has a plan area of 16,5*45 = 742,5 m².

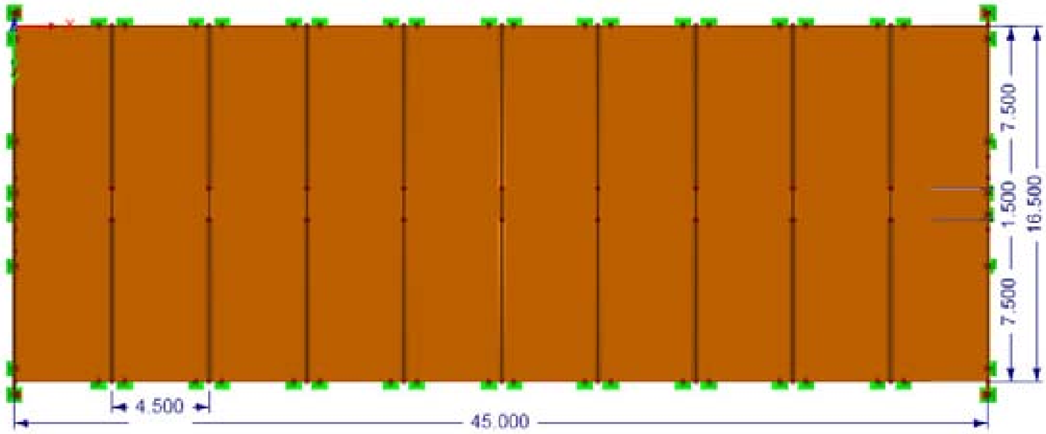


figure 4.2: Plan view of rectangular type geometry, screen shot RFEM

A detailed description of the two types will be given in chapter 6.

4.2 Load bearing system

The load bearing system must be well chosen for high rise timber buildings. Different timber structures are possible as construction method.
The structures can be divided in two parts. On one hand there are two dimensional surface elements as shown in the left part of figure 4.3. On the other hand constructions can be done with two dimensional linear elements as the right part of figure 4.3 shows.
The level of prefabrication is very important in terms of construction time and economical aspects. It increases from linear elements to surface elements.
In ascending order considering the level of prefabrication for linear elements there are following construction methods:

- logs
- timber framing (Historic post and beam with diagonals)
- two by four
- skeleton structure (Post and beam engineered)

Logs, timber framing and two by four are single pieces assembled on the construction site. Skeleton structure is a combination of single pieces with elements.

In ascending order considering the level of prefabrication for surface elements there are following construction methods:

- Massive panel out of e.g. cross laminated timber
- Panel construction prefabricated

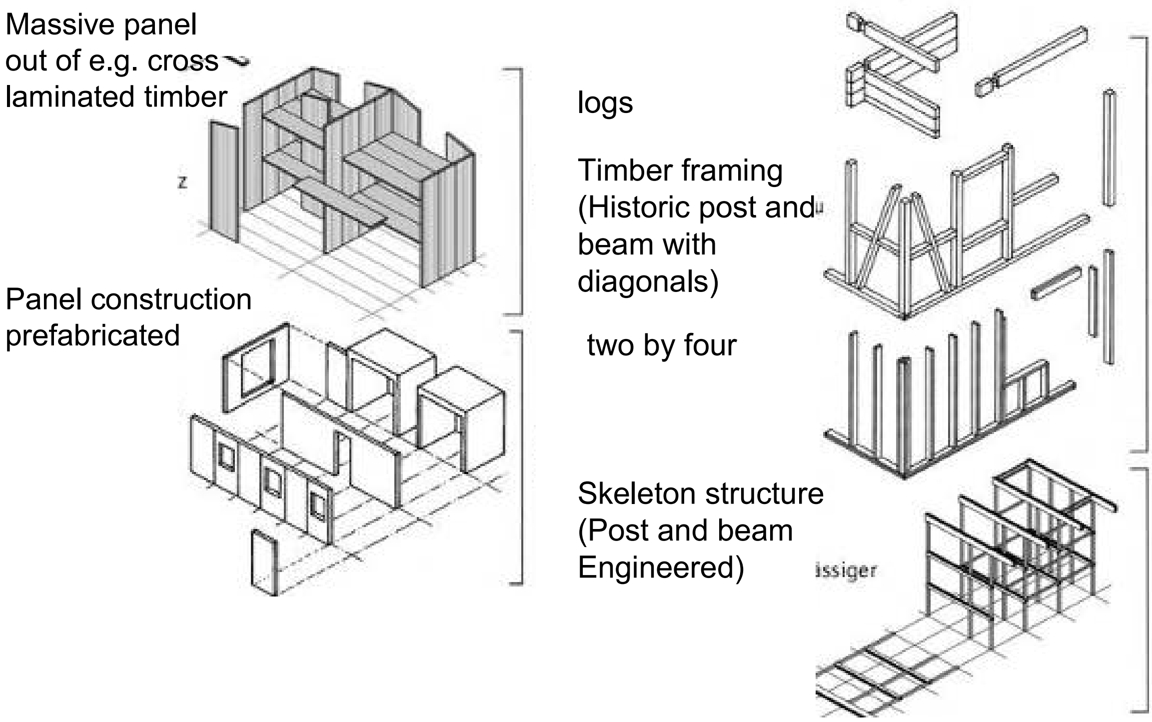


figure 4.3: Timber construction methods

(Source: Institute of Architectural Sciences, Structural Design and Timber Engineering, Vienna University of Technology)

4.3 Bracing system

For timber high rise building the bracing system must be studied very precisely. It has not only an influence on the structural considerations. As well it is influential on architectural perspectives and it contributes to requirements of building physics considering the façade of a building. These considerations must be taken into account while designing the bracing system.
In structural points of view the bracing system has to stiffen the building against lateral loads resulting from wind and earthquake. Due to the dimension of high rise buildings the resulting lateral forces are very significant. The stiffness of the building affects the value of the wind loads and the response of the building on earthquake. For such buildings the target is a symmetric bracing system.
To face these requirements several bracing systems are investigated.

One possible bracing system is the bracing with stiffening cores out of CLT panels. The core can be located in the centre (see figure 4.4, left image) or at the edge (see figure 4.4, right image). Of course the core would be located at both edges for symmetric behavior.

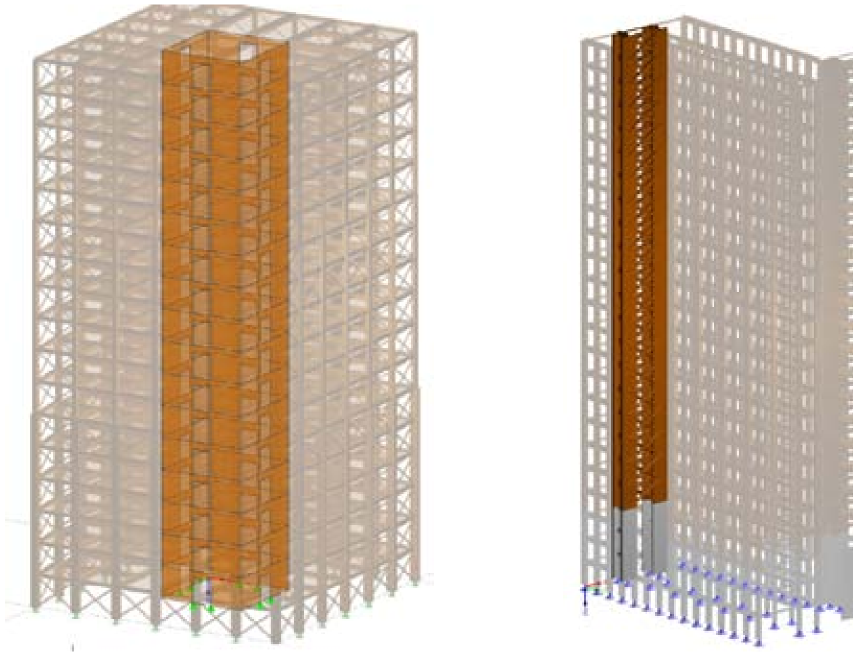


figure 4.4: Stiffening cores, screen shot RFEM and RSTAB

Another possibility is to stiffen the building through rigid frames (see figure 4.5).

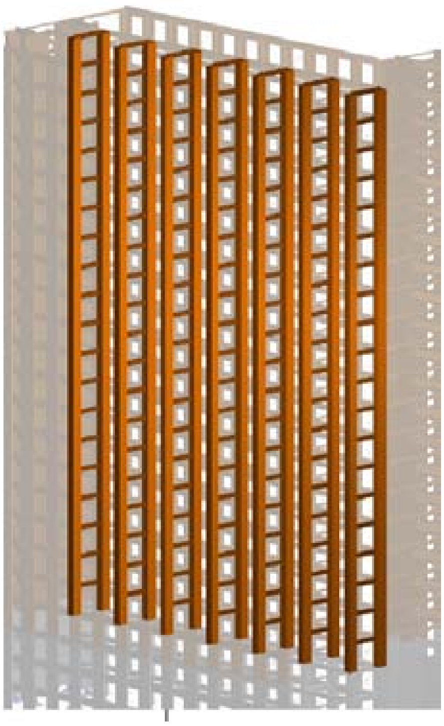


figure 4.5: Stiffening frames, screen shot RSTAB

A further method is to stiffen the building with panels e.g. out of cross laminated timber (see figure 4.6).

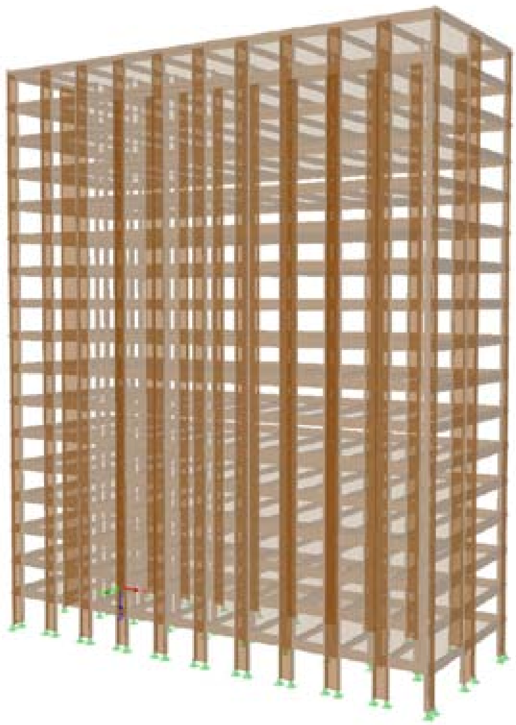


figure 4.6: Stiffening panels, screen shot RFEM

An additional method is the bracing with diagonal linear elements. The building can be partial crosswise stiffened (see figure 4.7, left image) or overall crosswise stiffened (see figure 4.7, right image).

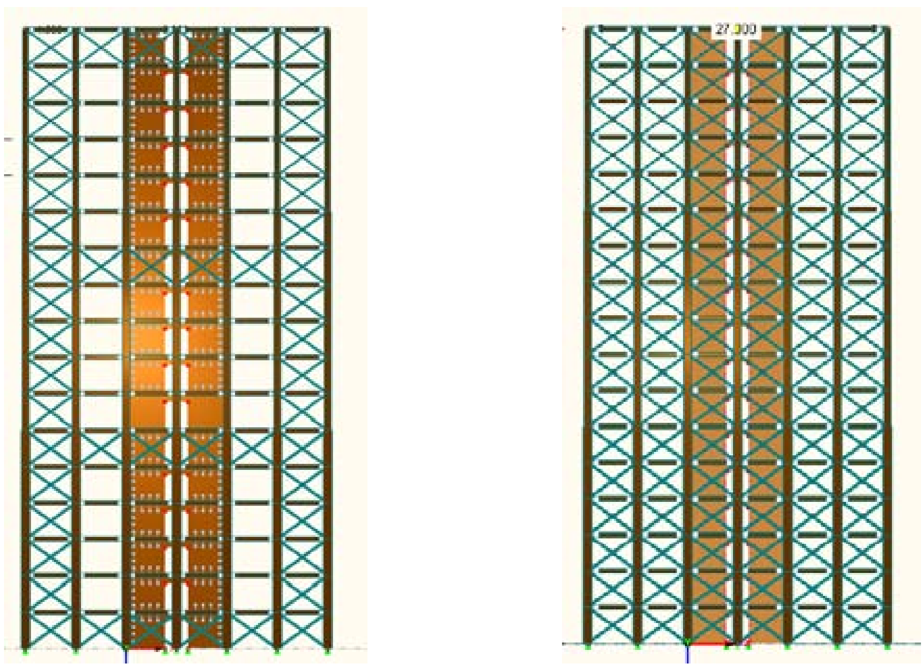


figure 4.7: Bracing with diagonals, screen shot RFEM

4.4 Floor structure

The floor structure needs to fulfil several requirements. For static reason the structure should be shear stiff. The sound insulation needs to obey the standard requirements of building physics. Therefore a certain mass and dependent on that a certain height is necessary. Nevertheless the storey to storey height needs to be limited for economical reasons. A target is to reduce the geometrical height.

The floor structure consists of beams and slabs. The height is variable with the configuration. The figures below show three different possibilities. One is a non composite structure with beams with rectangular cross sections (see figure 4.8) and the slab put on it. Another is a composite structure in T-section shape (see figure 4.9). A further one is a composite structure in box-girder shape (see figure 4.10).

figure 4.8: Rectangular cross section

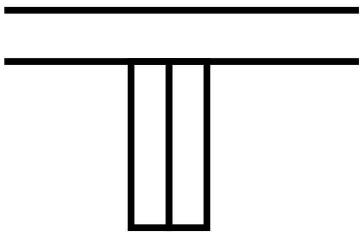


figure 4.9: T-section

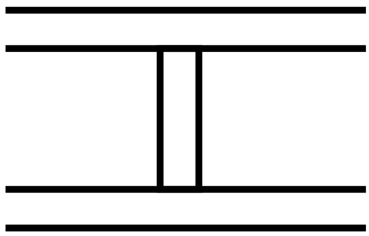


figure 4.10: Box-girder

In a first approach it was helpful to find out the necessary static parameters of the rectangular cross section of the beams, like the required dimension for the necessary moment of resistance and moment of inertia according to geometric conditions.
The graphs show curves for the width of 50, 100 and 200 cm for rectangular beams and straight curves for the required moment of resistance or moment of inertia depending on the effective loaded area determined by the span and the column spacing (e.g. 20*10 means 20 meters span and 10 meters column spacing). The span varies from 15 to 20 meters and the column spacing from 1,25 to 10 meters. The crossing point between the curve and the straight curve determine the necessary depth of the beam related to the specified parameters.
For the ultimate limit states the moment of resistance of the beam is significant (see figure 4.11). For the serviceability limit states the moment of inertia is significant (see figure 4.12 and figure 4.13).

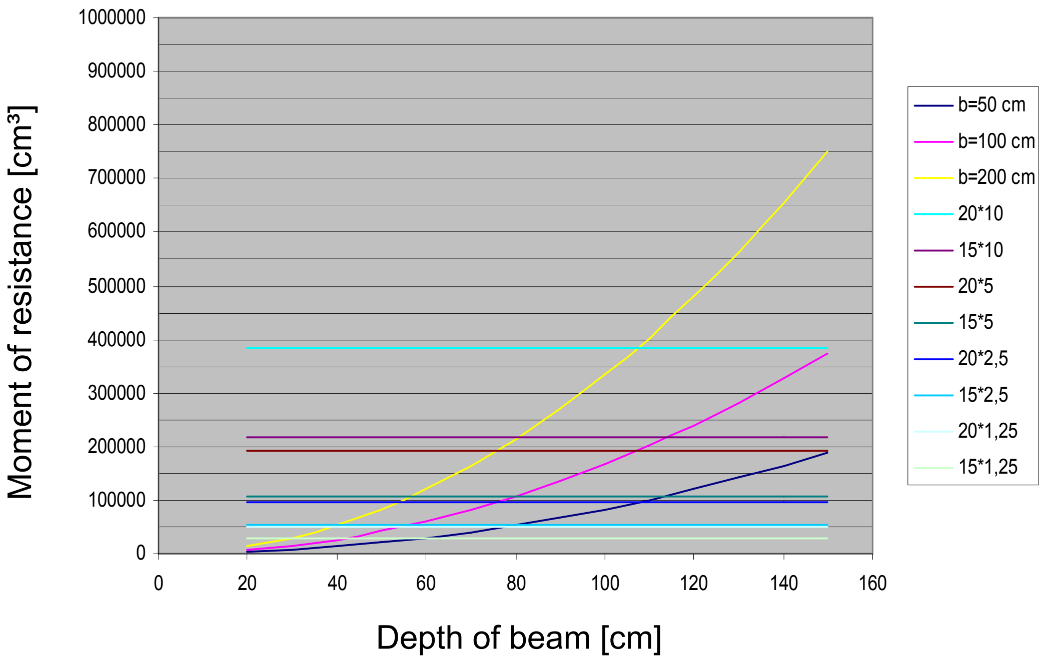


figure 4.11: Required beam dimension for ultimate limit states

For the serviceability limit states two limitations are investigated.
In figure 4.12 the limit for the deflection of the beam is u=l/300.

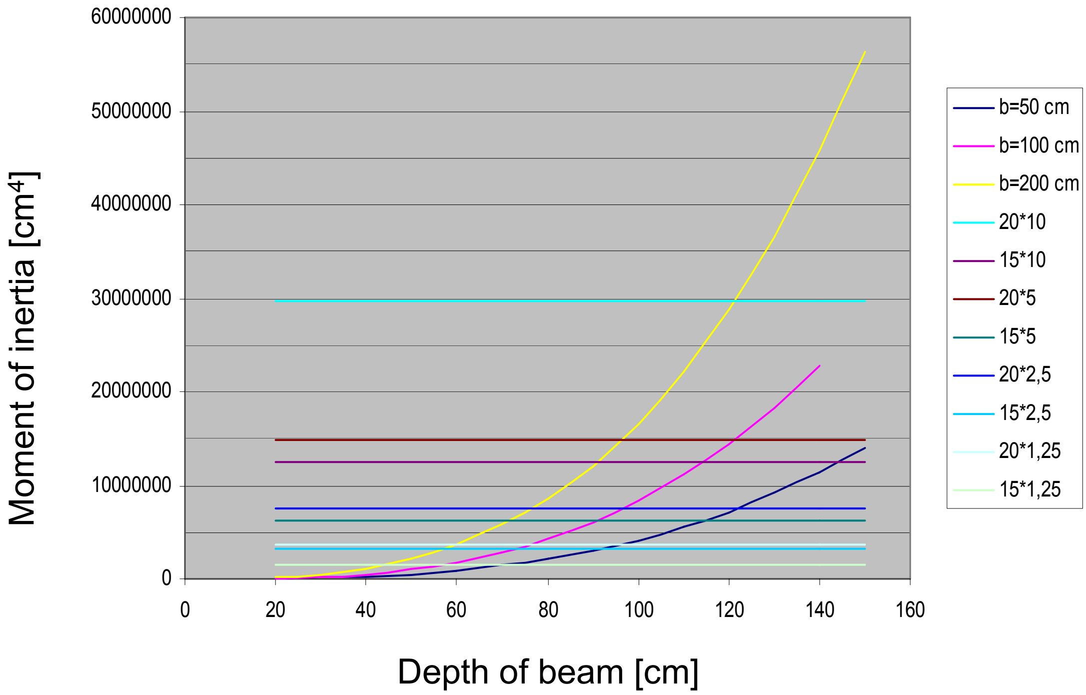


figure 4.12: Required beam dimension for serviceability limit states (u=l/300)

In figure 4.13 the limit for the deflection of the beam is u=l/500.

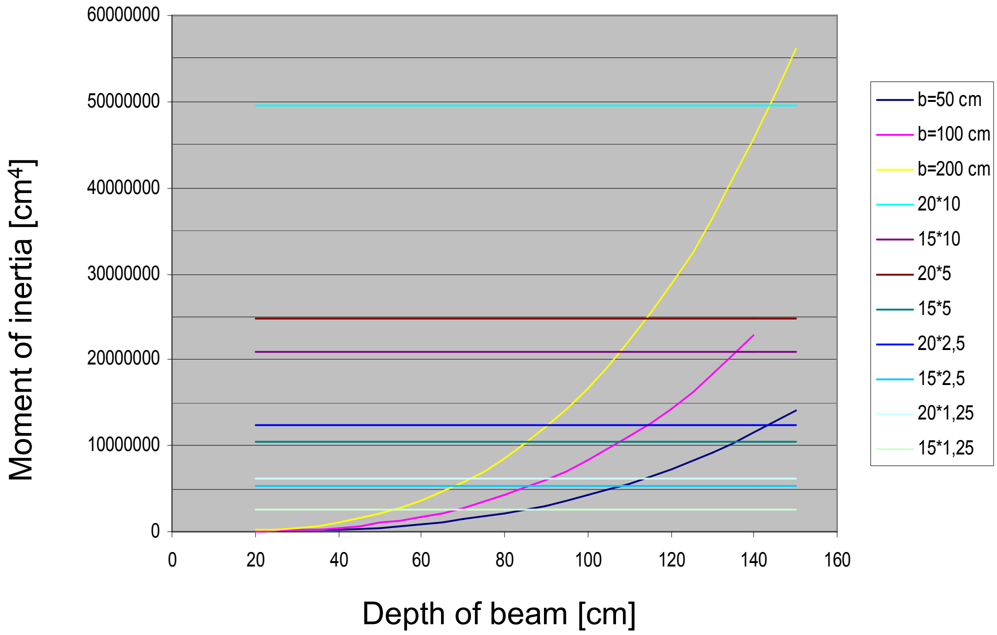


figure 4.13: Required beam dimension for serviceability limit states (u=l/500)

4.5 Column

As well as for the beams the necessary dimensions of columns for ultimate limit states according to specified parameters are evaluated.

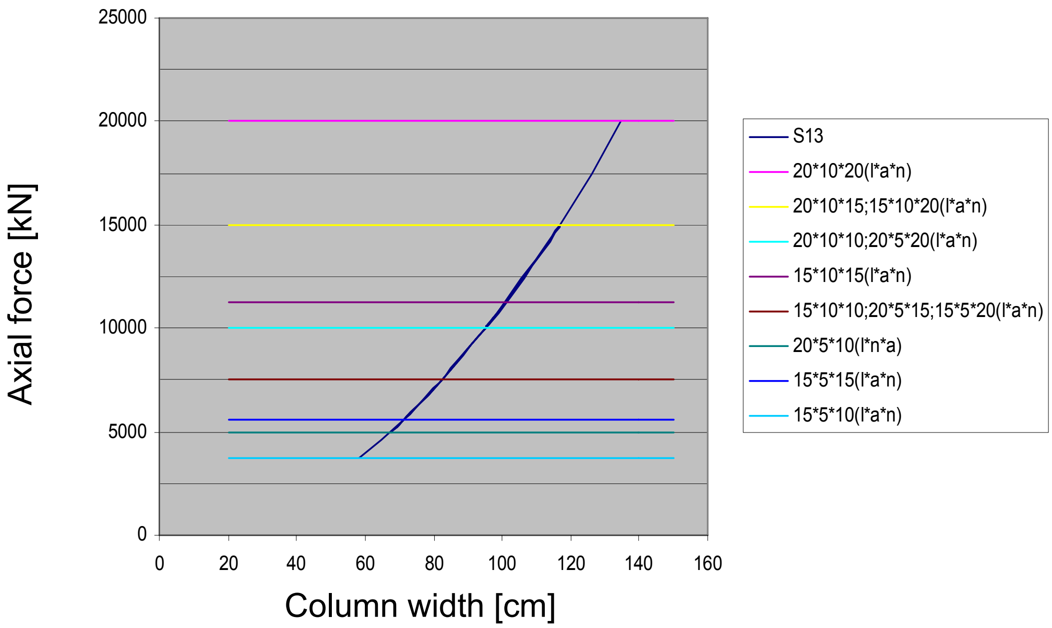


figure 4.14: Required column dimension for ultimate limit states

The graph (see figure 4.14) shows a curve for the material S13 according to ÖNORM DIN 4074-1 and straight curves for the required axial force depending on the span, the column spacing and the number of storeys (e.g. 20*10*20 means 20 meters span, 10 meters column spacing and 20 storeys). The crossing points of the curve with the straight curves define the column width needed for the required parameters.

5. Results of research approach

In this chapter the results of the research approach with a multiplicity of ideas and possibilities are described. The target is to find out the relevant systems to be investigated and optimised.

5.1 Bracing

In a first step the bracing with stiffening cores is analysed.
Therefore a calibration on the timber core for the framework software RSTAB with the finite element program RFEM is executed. The timber core with 7,5 meters length by 2,5 meters width consists of 16 cm thick CLT panels. Twenty storeys with a storey height of 3,25 meters are simulated.
In RSTAB the core is simulated by big dimensional vertical linear elements connected and stiffened by linear diagonal elements. The core is calibrated by the three natural frequencies. Therefore the stiffness of the diagonals is varied until reaching a good correspondence for all three natural frequencies.

The figure 5.1 shows the calibration of the first natural frequency for the timber core.

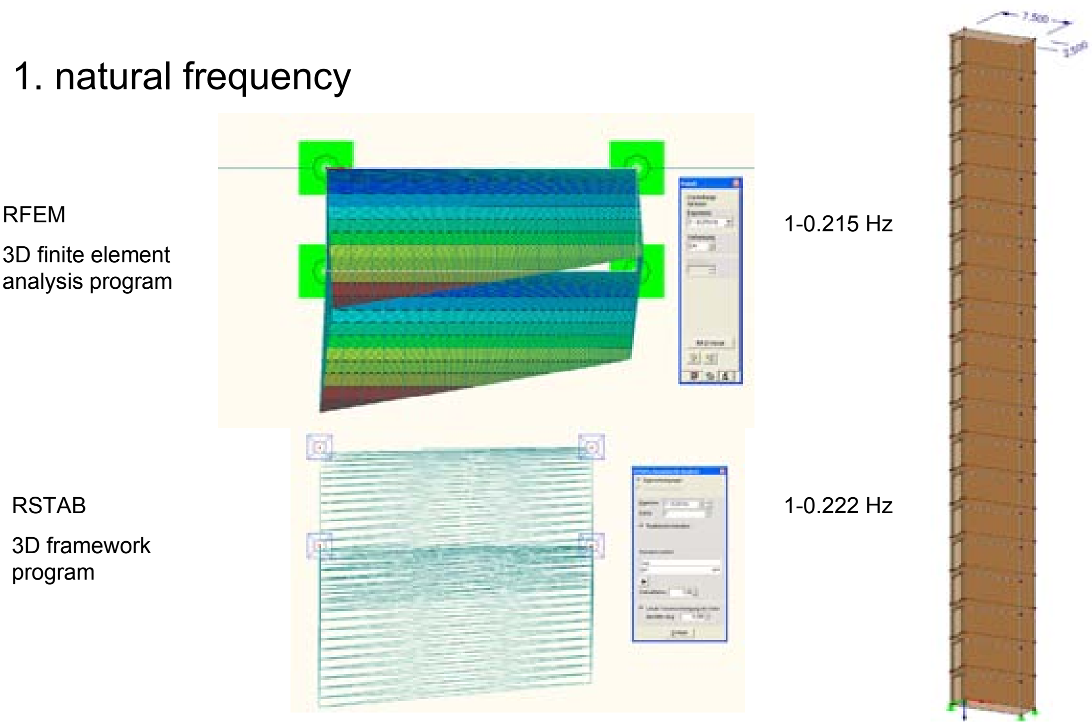


figure 5.1: Calibration of the 1. natural frequency for timber core in RSTAB and RFEM

The figure 5.2 shows the calibration of the second natural frequency for the timber core.

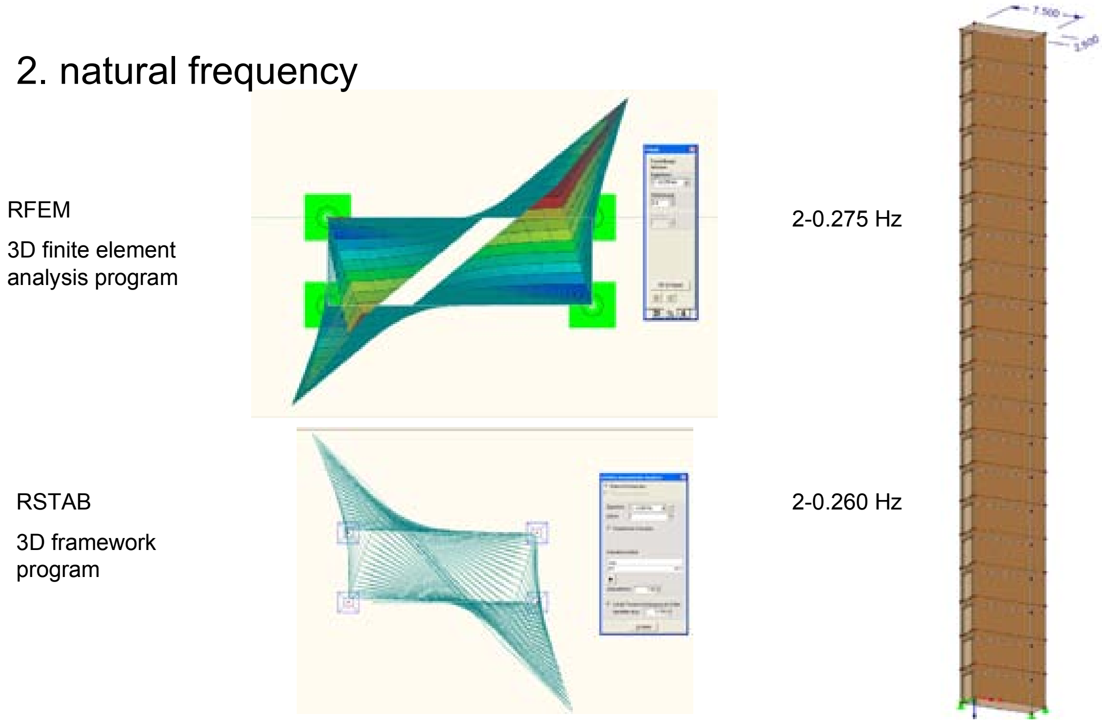


figure 5.2: Calibration of the 2. natural frequency for timber core in RSTAB and RFEM

The figure 5.3 shows the calibration of the third natural frequency for the timber core.

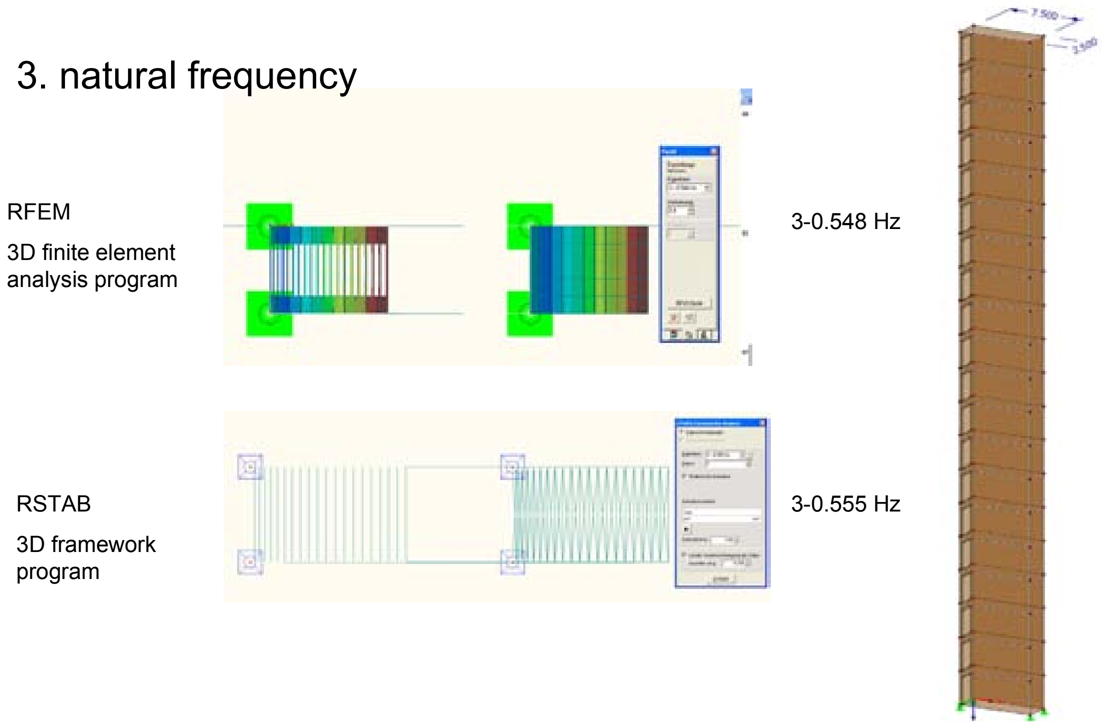


figure 5.3: Calibration of the 3. natural frequency for timber core in RSTAB and RFEM

In a second step the stiffness of the calibrated timber core of 7,5 meters by 2,5 meters is compared to a wood steel composite frame of 3 meters span (see figure 5.4). The core was already described. The frame consists in glulam columns with 80 by 60 cm rectangular cross section and a rigid connected steel truss of 50 cm height with U 200 cross sections. The relevant natural frequency for the significant stiffness in longitudinal direction is compared.
The results show a similar performance. The stiffness of the composite frame is about 10% higher than the stiffness of the timber core.

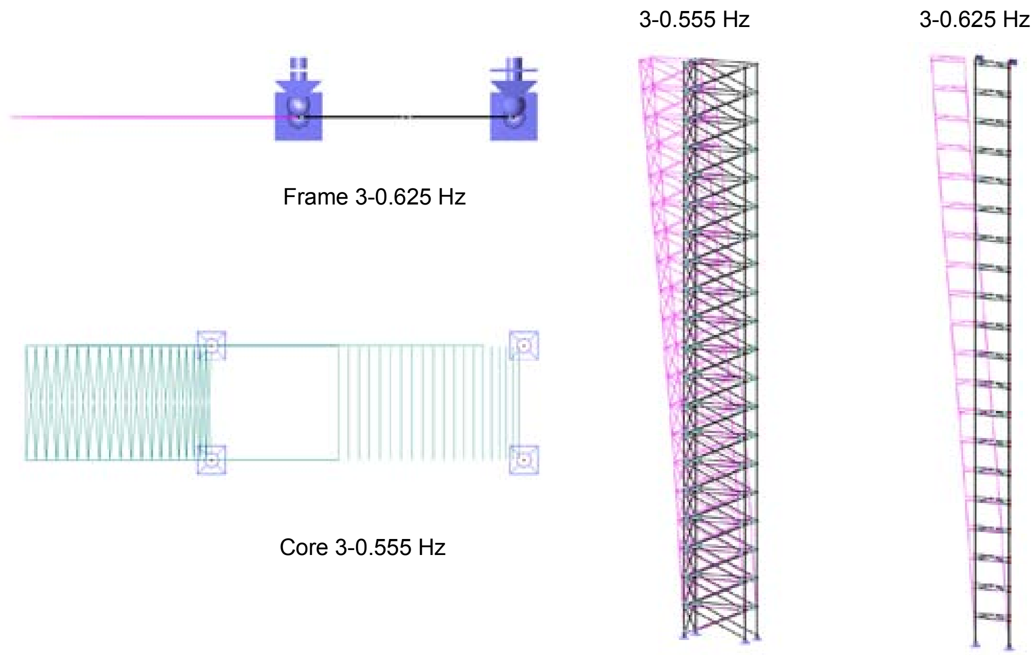


figure 5.4: Stiffness comparison between timber core with 7,5 by 2,5 m and composite frame with 3 m span, screen shot RSTAB

In a next step the stiffness of a system with rigid connections is compared to the stiffness of a system with diagonal bracing. The systems consist in three bays.
For the comparison the same amount of cubage is used. Just the cross sections are adapted to the system (see figure 5.5).
In the rigid connections system the posts have rectangular cross section of 10cm/90cm means 900 cm² and the beams have 2cm/50cm means 100 cm².
In the diagonal bracing system the posts have quadratic cross section of 30cm/30cm means 900 cm² and the beams have 10cm/10cm means 100 cm².
These elements are made out of glued laminated timber with an E-modulus of 1100 kN/cm².
For the diagonal bracing system the diagonals are simulated with the ½ E-Modulus for taking into account the diagonal connection. The cross section is 10cm/10cm.

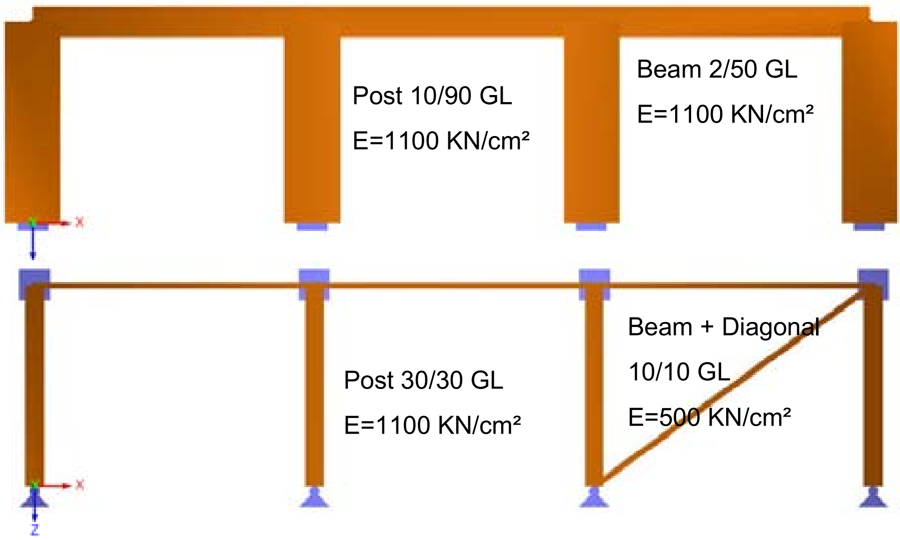


figure 5.5: Rigid connections system and diagonal bracing system, screen shot RSTAB

The results of the comparison between the rigid connections system and the diagonal bracing system are shown in the following figures. To all systems a unit load of 100 kN is applied.

For a one storey diagonal bracing system the horizontal displacement is $U_{horiz.}$ ~ 44 mm (see figure 5.6).

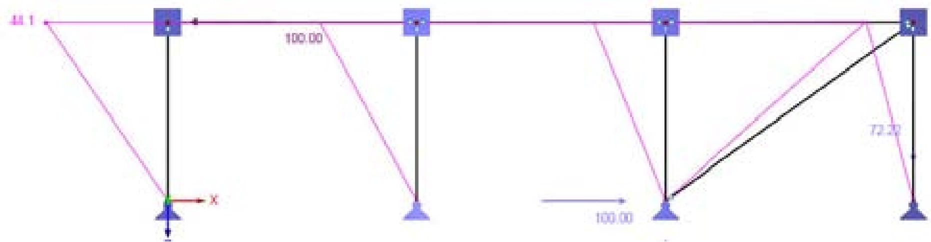


figure 5.6: Displacement diagonal bracing system for one storey, screen shot RSTAB

For a one storey rigid connections system with fixed support the horizontal displacement is $U_{horiz.}$ ~ 6 mm (see figure 5.7).

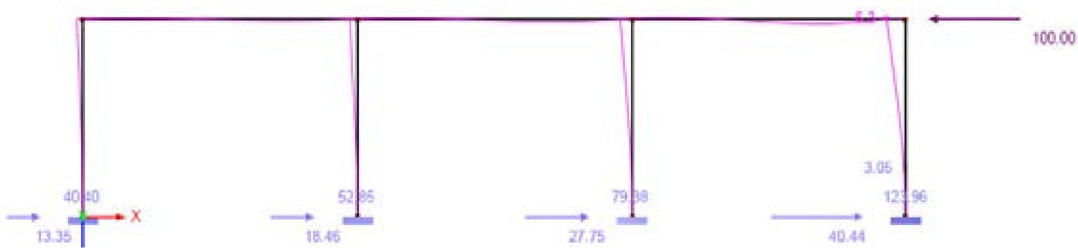


figure 5.7: Displacement rigid connections system fixed support for one storey, screen shot RSTAB

For a one storey rigid connections system with hinged support the horizontal displacement is $U_{horiz.}$ ~ 66 mm (see figure 5.8).

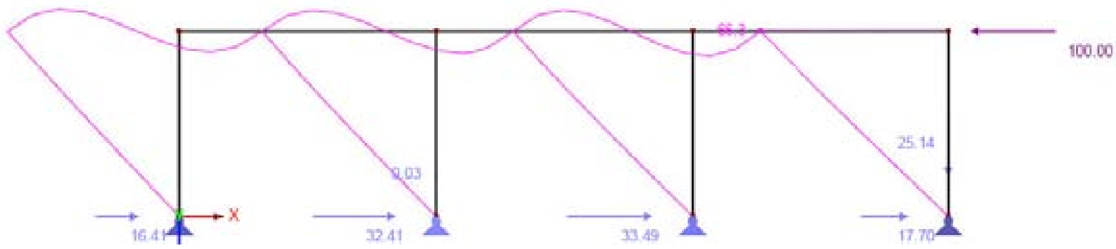


figure 5.8: Displacement rigid connections system hinged support for one storey, screen shot RSTAB

The results show that fixing the support for rigid connections system would decrease the displacement to one-tenth.
The comparison between the diagonal bracing system and the rigid connections system with hinged support shows a one and a half time better performance in terms of stiffness for the diagonal system.

The same comparisons with same loading are performed on two storeys for analysing the deformation picture.

For a two storey diagonal bracing system with column splice hinged the horizontal displacement is $U_{horiz.}$ ~ 44 mm (see figure 5.9).

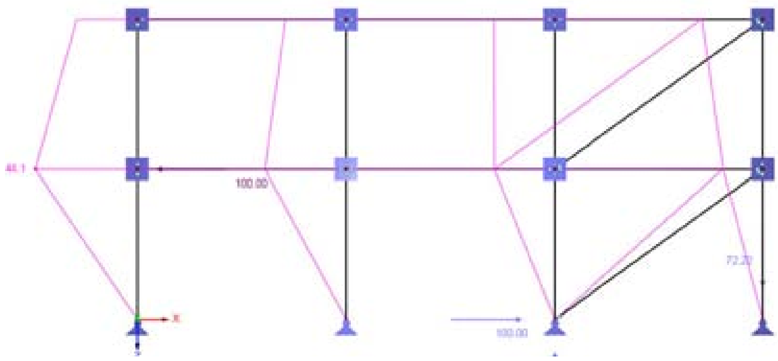


figure 5.9: Displacement diagonal bracing system for two storeys, screen shot RSTAB

For a two storey rigid connections system with fixed support the horizontal displacement is $U_{horiz.}$ ~ 8 mm (see figure 5.10).

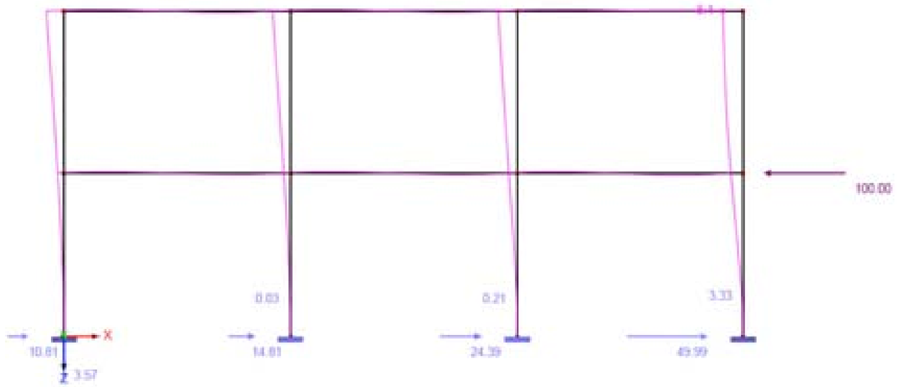


figure 5.10: Displacement rigid connections system fixed support for two storeys, screen shot RSTAB

For a two storey rigid connections system with hinged support the horizontal displacement is $U_{horiz.}$ ~ 66 mm (see figure 5.11).

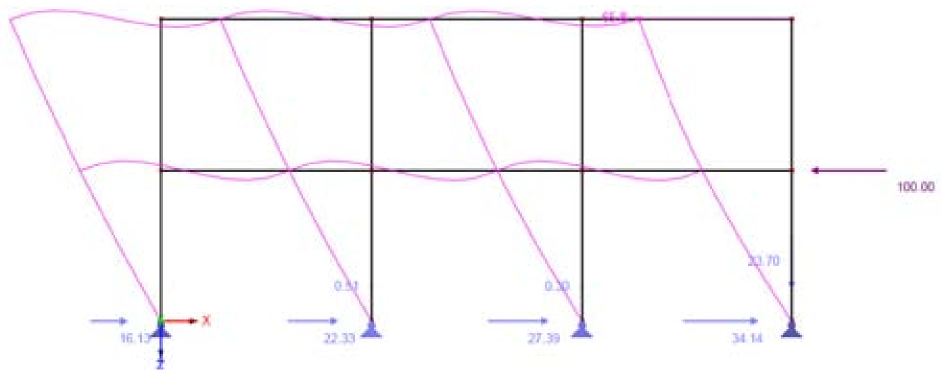


figure 5.11: Displacement rigid connections system hinged support for two storeys, screen shot RSTAB

The results of the two storey systems comparisons are similar to the one storey systems comparison. The diagonal bracing system is still one and a half time stiffer than the rigid connections system with hinged support. The maximum displacement for the diagonal system is in the first storey, while the maximum displacement for the rigid connections system is in the second storey.

In a further step the diagonal bracing system is investigated more precisely. The stiffness of an overall and a partial crosswise stiffened building is compared (see figure 5.12 and figure 5.13). For the partial crosswise stiffened building just every fifth storey is entire stiffened.
The lateral force applied is the wind load. The wind load is depending on the first natural frequency as the gust response factor is taken into account. Therefore the applied wind load for the overall crosswise stiffened building which is stiffer is about

1,4 kN/m² without live load to 1,5 kN/m² with live load and for the partial crosswise stiffened building about 1,5 kN/m² without live load to 1,9 kN/m² with live load.

Applied wind load
1,4 -1,5 kN/m²

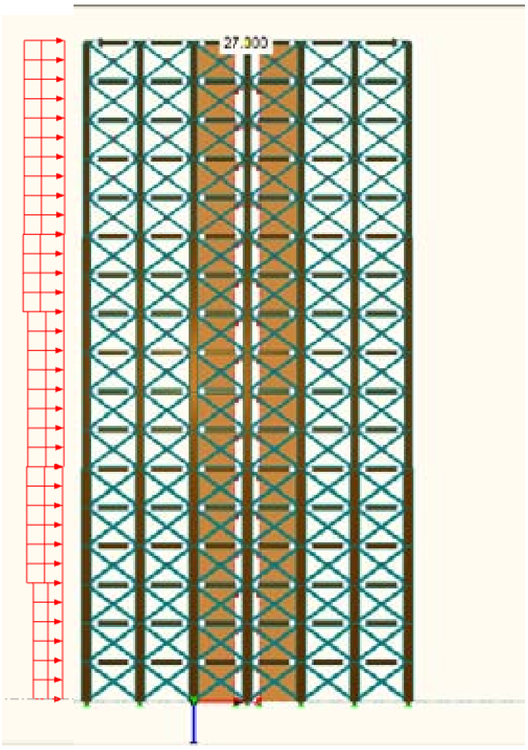


figure 5.12: Overall crosswise stiffening, screen shot RFEM

Applied wind load
1,5 -1,9 kN/m²

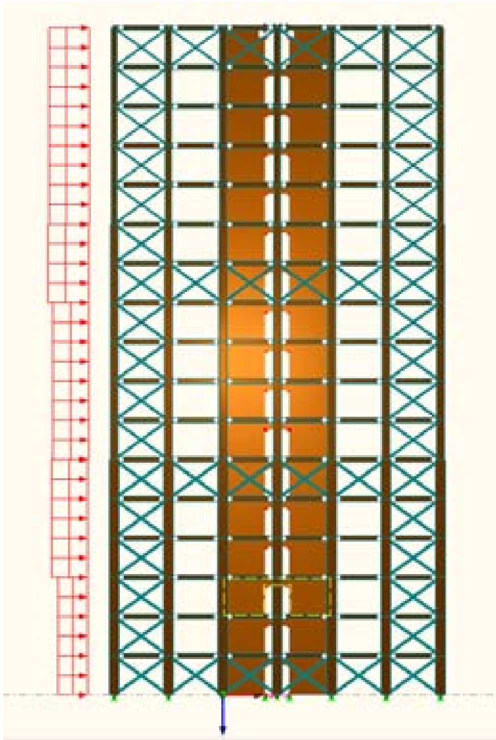


figure 5.13: Partial crosswise stiffening, screen shot RFEM

table 5.1: Results of comparison between a overall and a partial crosswise stiffened building

	Variant 1 – overall		Variant 2 - partial	
	With live load	no live load	With live load	no live load
f [hz]	0,521	0,84	0,302	0,488
T [s]	1,92	1,19	3,304	2,047
u_x max [cm]	3,33	2,68	11	8,77
Support compression max [kN]	2500	1265	3380	1800
Support tension max [kN]	-	-	-	420

The results of table 5.1 indicate the influence of vertical load applied to the building. Without the live load the systems are about 25 % stiffer. The overall crosswise stiffened system is about three times stiffer than the partial crosswise stiffened system. Nevertheless the performance of the partial crosswise stiffened system seems to be sufficient regarding the displacement.

5.2 Floor structure

First results show in the following three figures the dimensions of beams needed for different cross sections according to the span and the column spacing. Additionally the cubage in m³/m² needed for every cross section is evaluated.

The figure 5.14 points out the dimension of beams with rectangular cross section depending on span and column spacing. Several alternatives with varied width are compared in matter of cubage needed. As the moments of inertia ($I=b*h^3/12$) and resistance ($W=I/(h/2)$) are significant for the dimension of the beams the height is taken into account with the cube respectively the square. As a result the cubage decreases with increasing height of the beam.

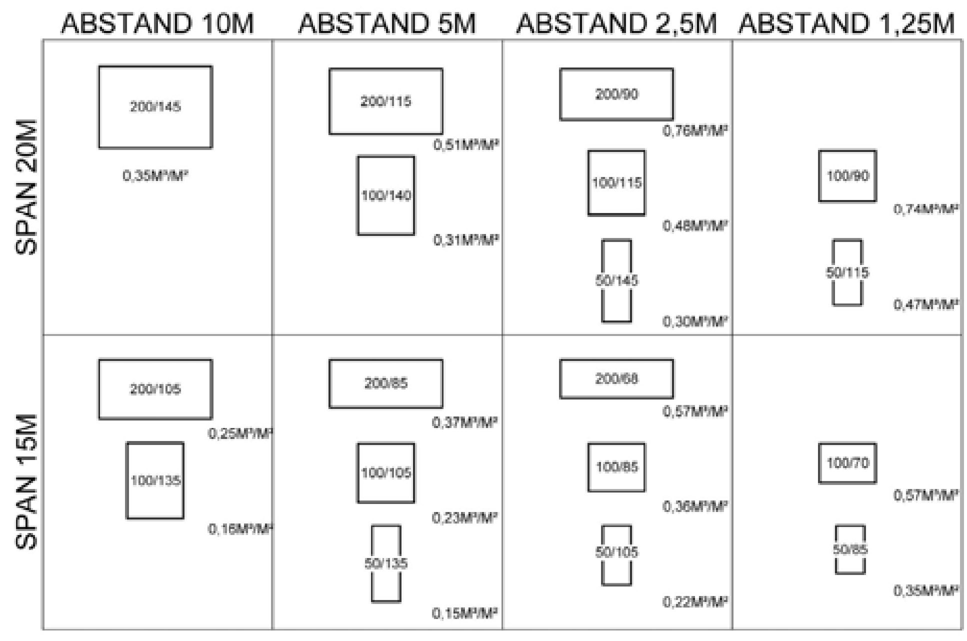


figure 5.14: Dimension of beams with rectangular cross section depending on span and column spacing

The figure 5.15 points out the dimension of beams with T - section depending on span and column spacing.

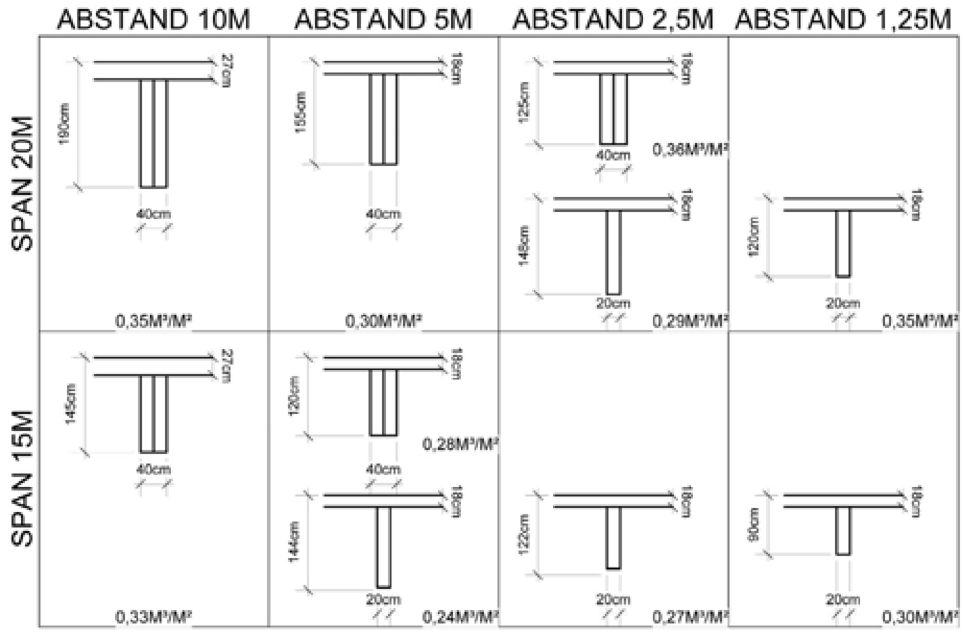


figure 5.15: Dimension of beams with T - section depending on span and column spacing

The figure 5.16 points out the dimension of beams with box - girder depending on span and column spacing.

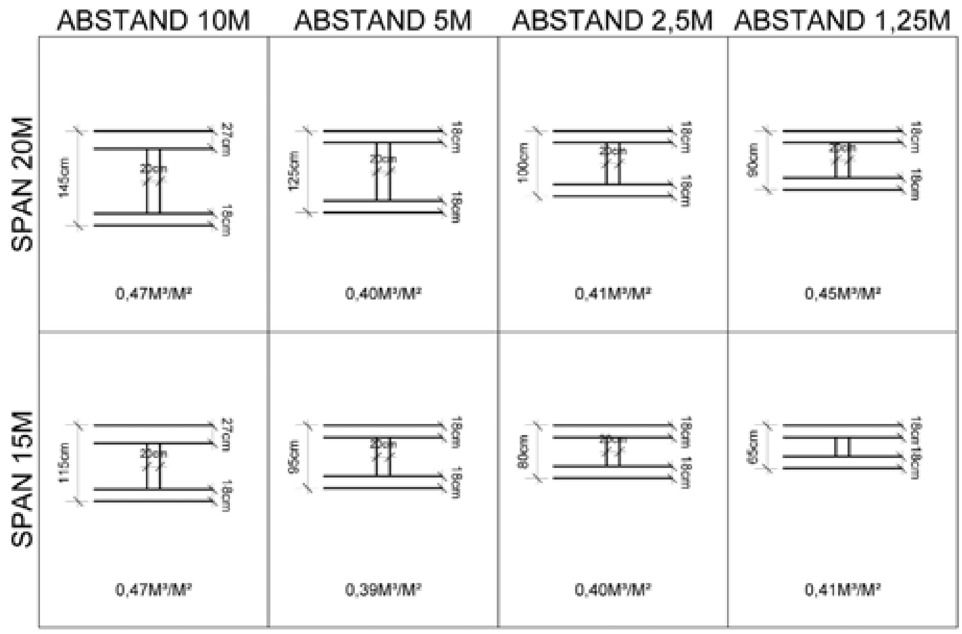


figure 5.16: Dimension of beams with box - girder depending on span and column spacing

The results of the three kinds of cross sections show the most effective performance for a span of 15 meters and a column spacing of 5 meters. Still the required construction heights of these systems need to be very big.

In the following part several setups according to static considerations for floor constructions are investigated.
For the column spacing of 4,5 m following variants are worked out.

The first one is a classic floor structure with glulam beams with a span of 7,5 m and a CLT slab with a span of 4,5 m (see figure 5.17).

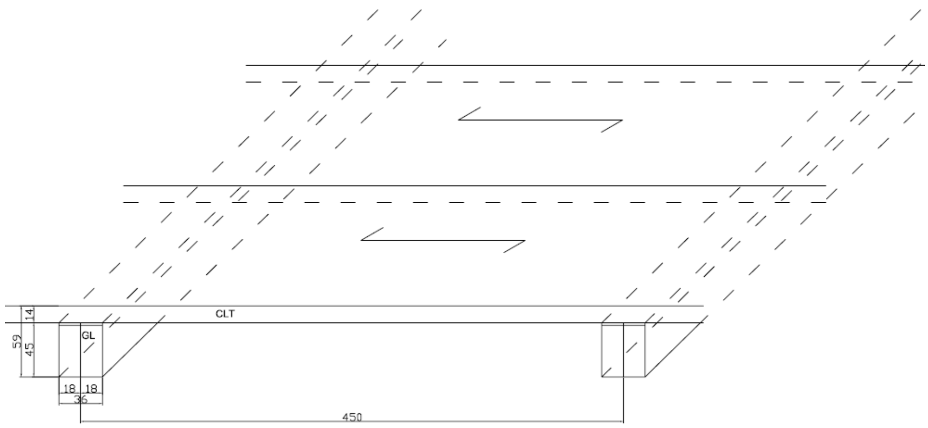


figure 5.17: Classic floor structure with 4,5 m column spacing and 45 cm height

The glulam beams have a cross section of b/h:36/45 cm and the CLT panel has a thickness of 14 cm. The overall height of this structure is 59 cm (see figure 5.18).
The total cubage needed is 0,18 m³/m².

0,14 m³/m² for the CLT panels
0,036 m³/m² for the GL beams
0,176 m³/m² in total

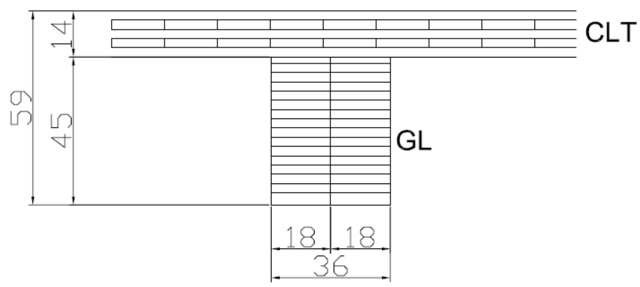


figure 5.18: Setup of classic floor structure with 4,5 m column spacing and 45 cm height

The second one is as well a classic floor structure with glulam beams with a span of 7,5 m and a CLT slab with a span of 4,5 m. In this case the overall height was tried to be reduced to 45 cm (see figure 5.19).

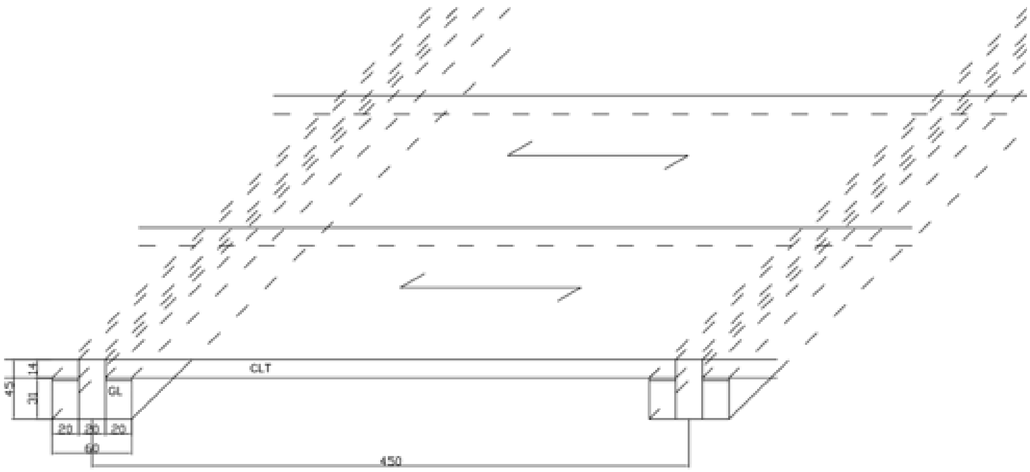


figure 5.19: Floor structure with 4,5 m column spacing and reduced height of 31 cm

The glulam beams have a cross section of b/h:60/31 and the CLT panel remains with 14 cm thickness (see figure 5.20).
The total cubage needed is 0,19 m³/m².

0,14 m³/m²	for the CLT panels
<u>0,047 m³/m²</u>	for the GL beams
0,187 m³/m	in total

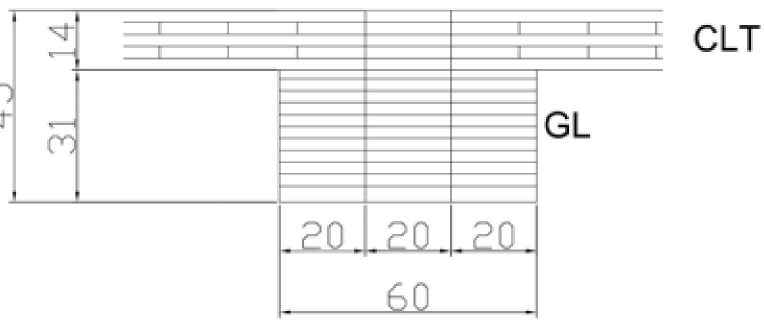


figure 5.20: Setup of floor structure with 4,5 m column spacing and reduced height of 31 cm

The third one is a floor structure with wood concrete composite elements with a span of 7,5 m supported by glulam edge beams with 4,5 m span (see figure 5.21).

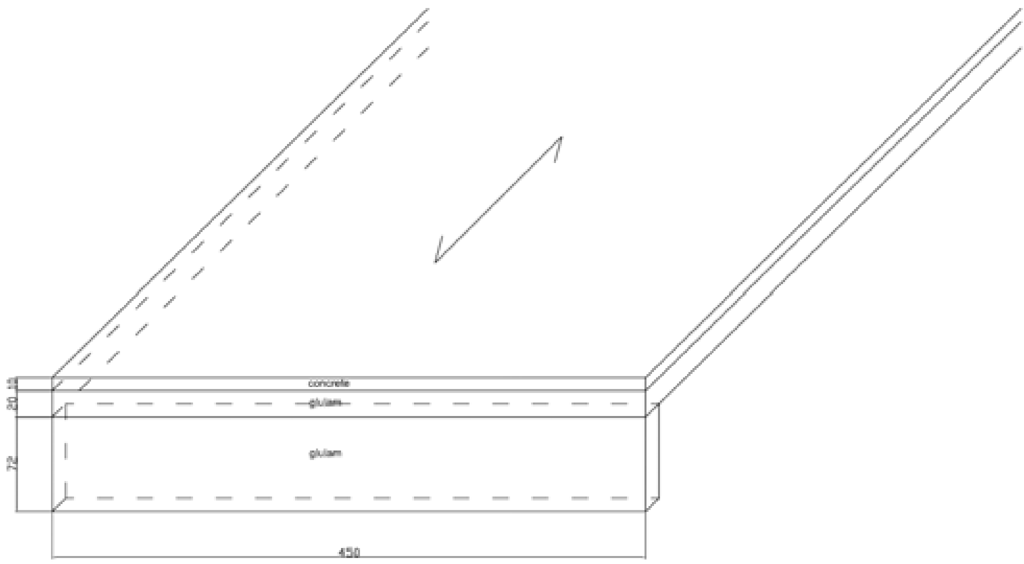


figure 5.21: Floor structure with 4,5 m column spacing made out of wood concrete composite

The composite structure consists in a 10 cm thick concrete slab and a 20 cm thick glulam panel (see figure 5.22). The glulam edge beams have a cross section of b/h:10/72 cm.

The total concrete cubage needed is 0,1 m³/m². The total glulam cubage needed is 0,22 m³/m².

0,1 m³/m² for the concrete slab

0,2 m³/m² for the GL panels

<u>0,0192 m³/m²</u> for the GL edge beams

0,2192m³/m² in total for GL

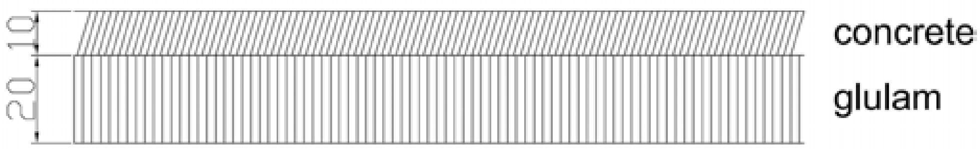


figure 5.22: Setup of floor structure with 4,5 m column spacing made out of wood concrete composite

For the column spacing of 2,25 m following variants are worked out.

The first one is a floor structure made out of prefabricated timber composite sections with a span of 7,5 m (see figure 5.23). The glulam beams and the CLT panels are shear connected.

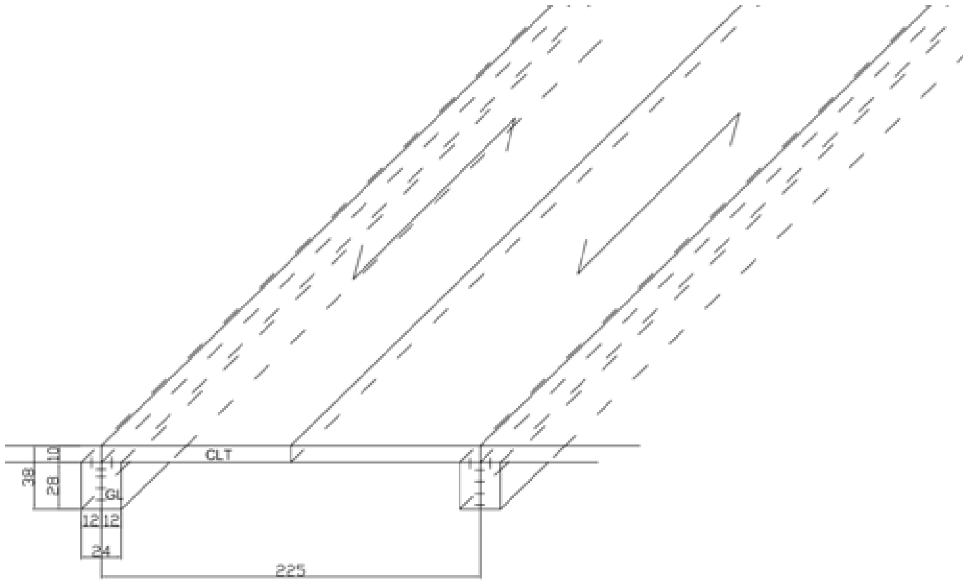


figure 5.23: Floor structure with 2,25 m column spacing made out of prefabricated timber composite section

The glulam beams have a cross section of b/h:24/28 cm and the CLT panel is 10 cm thick (see figure 5.24).

The total cubage needed is 0,13 m³/m².

0,1 m³/m²	for the CLT panels
<u>0,03 m³/m²</u>	for the GL beams
0,13 m³/m²	in total

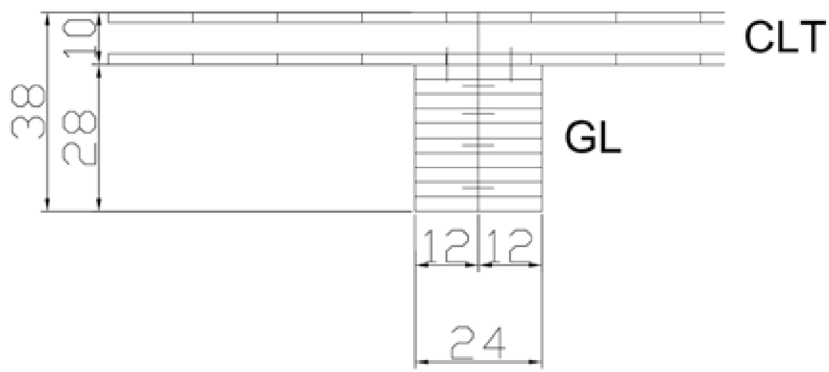


figure 5.24: Setup of floor structure with 2,25 m column spacing made out of prefabricated timber composite section

The second one is a floor structure made out of prefabricated wood concrete composite elements with a span of 7,5 m (see figure 5.25).

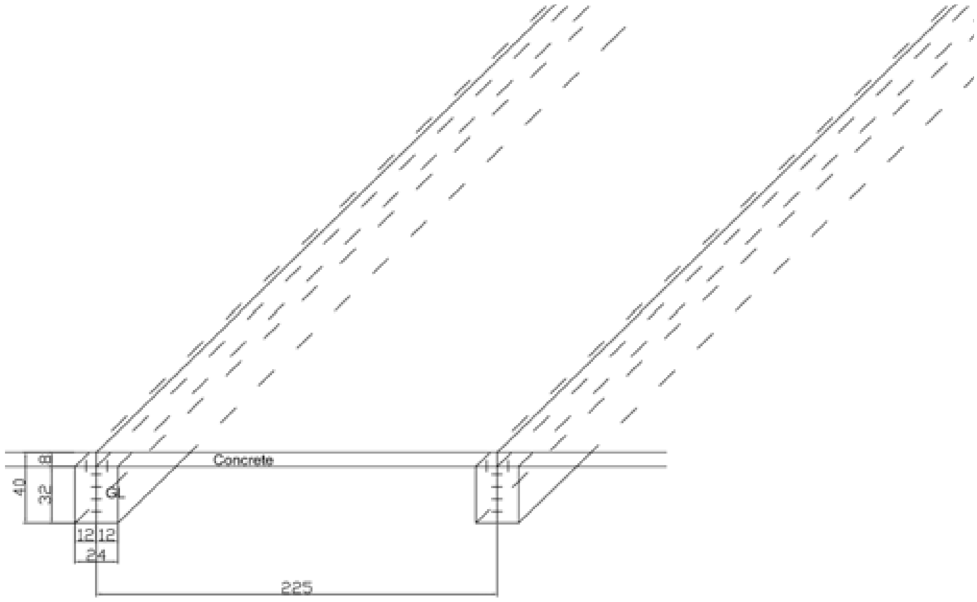


figure 5.25: Floor structure with 2,25 m column spacing made out of prefabricated wood concrete composite

The glulam beams have a cross section of b/h:24/32 cm and the concrete slab is 8 cm thick (see figure 5.26). The glulam beams and the concrete slab are shear connected.
The total cubage for concrete is 0,08 m³/m². The total cubage for glulam is 0,034 m³/m².

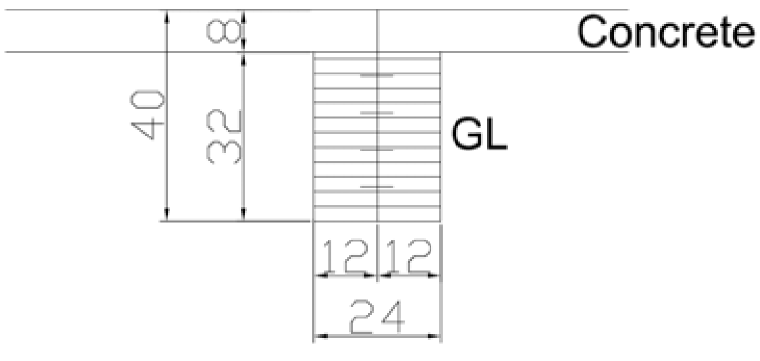
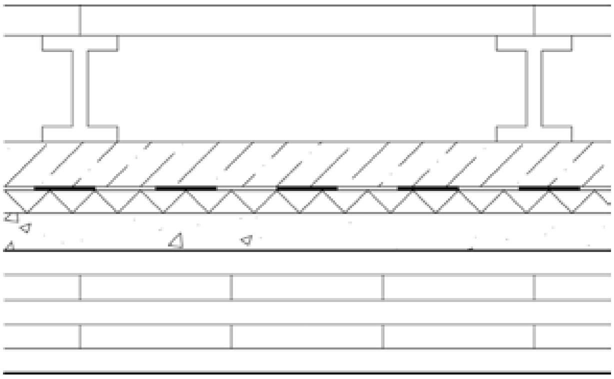


figure 5.26: Setup of floor structure with 2,25 m column spacing made out of prefabricated wood concrete composite

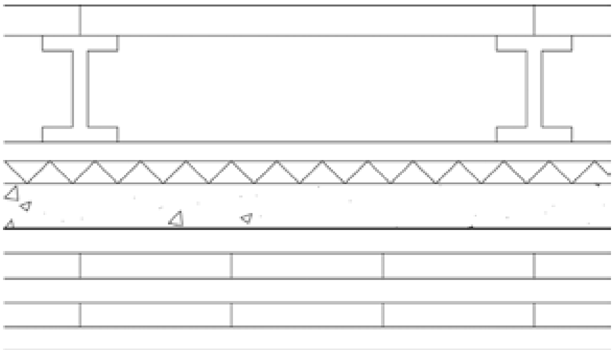
The following floor structures are investigated according to impact sound insulation by DI Dr. Martin Teibinger.

A double floor system will in any case be applied.



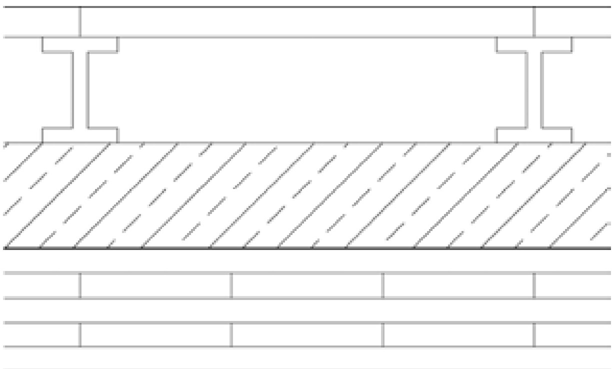
200mm Double floor Nortec
60mm cement screed
Separation layer
28mm Floorrock HP30-2
50mm grit fill, bulk density rd.1300kg/m³
162mm CLT

figure 5.27: Floor structure Nr. 1: CLT (screed)



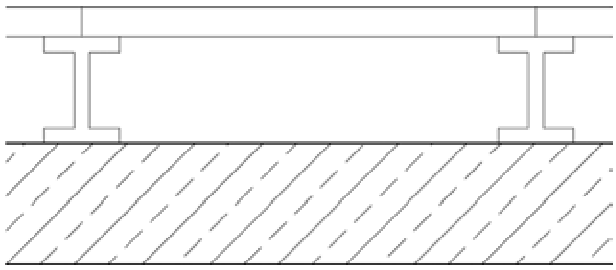
200mm double floor Nortec
25mm Rigidur screed element
28mm Floorrock HP30-1
50mm grit fill 4/8
162mm CLT

figure 5.28: Floor structure Nr. 2: CLT (Dry screed)



200mm double floor Nortec
140mm concrete
160mm CLT

figure 5.29: Floor structure Nr. 3: wood concrete composite



200mm double floor Nortec

160mm concrete

figure 5.30: Floor structure Nr. 4: concrete

In table 5.2 the results of the investigation are displayed. The required value for the impact sound is $L_{n,w} < 48$ dB. The results point out the better performance in terms of less weight per unit area and less construction height of the floor structure number 2 with the CLT panel and the dry screed.

table 5.2: Possible floor structures depending on the span

Nr.	Structure	Span [m]	Weight per unit area [kg/m²]	Construction height [mm]	$L_{n,w}$ [dB]
1	CLT (screed)	4,5	325	500	< 42 dB
2	**CLT (dry screed)**	**4,5**	**240**	**477**	**< 43 dB**
3	Wood concrete composite	7,5	430	500	46
4	concrete	7,5	410	360	47

5.3 Column

The figure 5.31 shows the results of the investigation for the dimensions of columns needed depending on form and load.

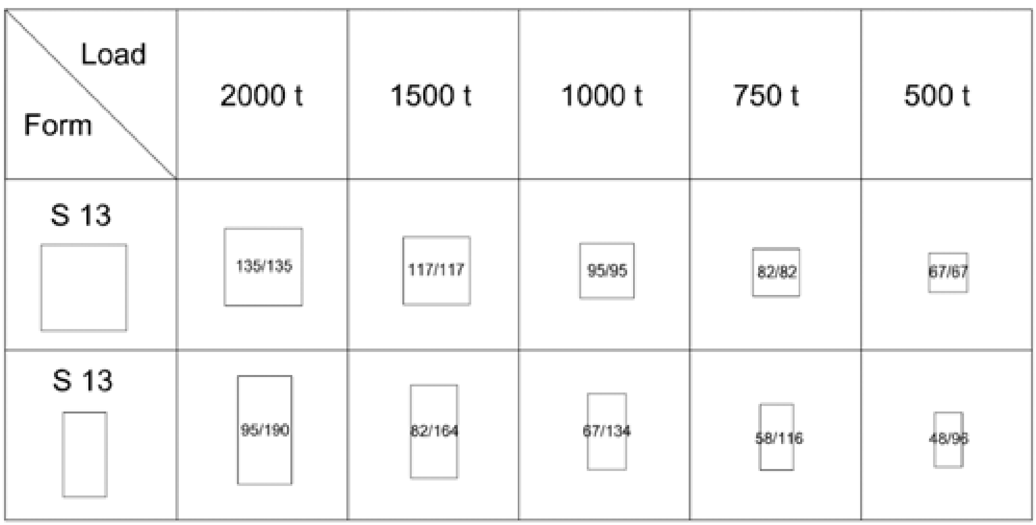


figure 5.31: Column dimensions depending on form and load

5.4 Conclusion of research approach

The investigations for the bracing systems show that massif CLT panels used in the core for bracing systems do not show the estimated performance. The frame systems in the building show the same performance. Therefore the bracing system in the facade is more efficient and should be utilised.
The results also prove that as well the rigid connections as the diagonal bracing are possible systems. To fix the supports could decrease the displacement of a multiple. If it is reasonable in economical and static terms must be investigated.

For the floor structure the results show that the span should be limited to 9 m or less and the column spacing should be limited to 4,5 m or less. The significant parameters for these advisements are the construction height and the cubage needed. Good solutions to reduce the construction height and the cubage are prefabricated timber composite elements.
The sound insulation has as well an important role for the floor structure.
On one hand the more mass the better works the insulation and on the other the structure should be light regarding the earthquake and the construction height should be minimised regarding the floor to floor height and the related costs

Regarding the number of storeys and the related vertical loads applied on the columns the limitation of span and column spacing will be necessary not to get enormous dimensions for the columns.

6. Investigation and optimizing of 2 systems

According to the results of the research approach two systems are designed for being investigated and optimized. The first one is a system with linear elements and diagonal bracing. The second one is braced by partial rigid connections of CLT elements.

For the investigation the first three concrete storeys are estimated as very stiff and therefore not taken into account for the calculations and the graphics.

The connections are assumed to be stiff. The partial rigidity of the connections must be evaluated in further researches.

6.1 Square type partial crosswise stiffened

The first type has a square shape with a plan area of 729 m² with 27m by 27m (see figure 6.1). The core of 9m by 9m is not stiffening, just used for vertical load transfer. For the finite element analyses the supports of the core are put cloth together to avoid a big lever arm for bracing (see figure 6.1, plan view) such as the lateral forces are picked up just by the supports located in the façade. The bracing diagonals are located on the corners of the building and every fifth storey is overall crosswise stiffened. The column spacing in the façade is 4,5 m. The beams have a span of 9 m. The floor is supposed to be shearing stiff. The reduced stiffness and the additional deformation due to the connection of the diagonals are not taken into account for the finite element analyses.

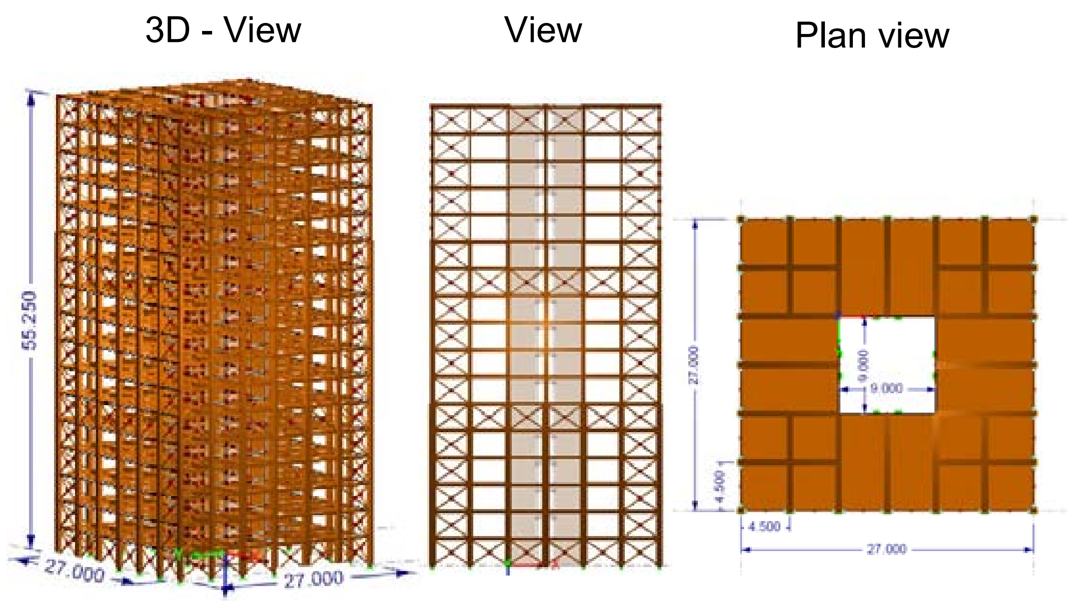


figure 6.1: Views of square type partial crosswise stiffened, screen shot RFEM

The materials and the cross sections used are displayed in figure 6.2. The panels used for floor and wall construction are made out of CLT. The beams are glulam beams and the bracing diagonals are softwood. The floor is stressed with a span of 4,5 m. For a first optimising there is a grading of the cross sections of 20% for every six storeys related to the increasing vertical load.

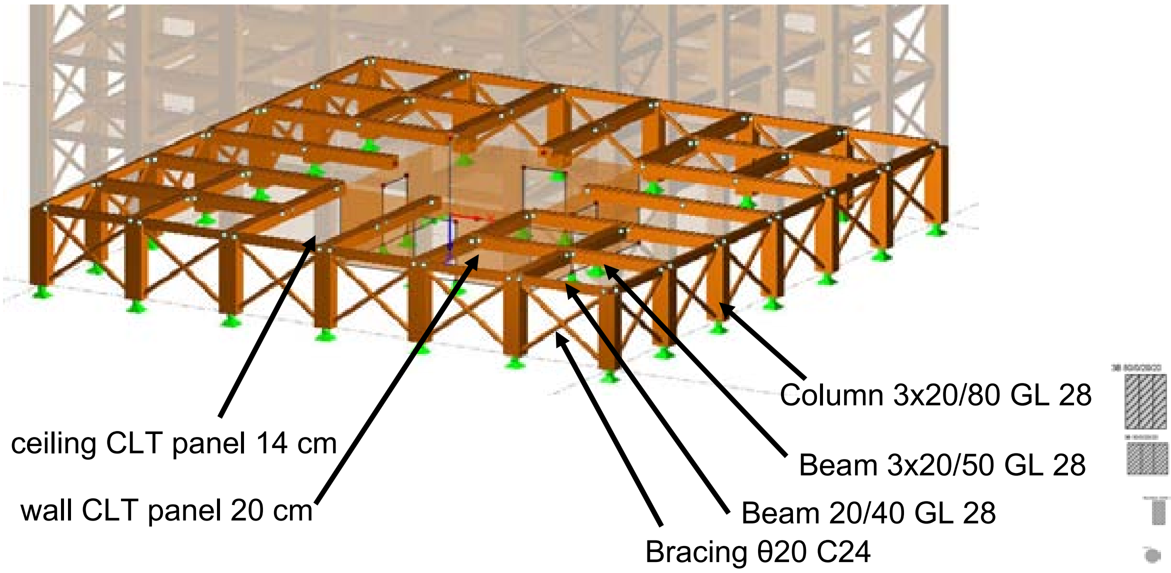


figure 6.2: One storey with cross sections of square type partial crosswise stiffened, screen shot RFEM

The diversified cubage for glulam, cross laminated timber and softwood is shown in table 6.1. The most part of the material is located in the floor. The highest amount of wood is used for the CLT panels in the floor.

table 6.1: Cubage in cubic meter for square type partial crosswise stiffened

cubage in m³	GL	CLT	softwood	Σ
floor	1065 (9m span)	1542 (4,5m span)	-	2607
columns	435	-	-	435
walls	-	328	-	328
bracing	-	-	49	49

The total cubage needed for the square type partial crosswise stiffened is 3419 m³. Applied on the area of the building the material used is about 0,31 m³/m². In terms of weight it amounts to 108,5 kg/m².

In table 6.2 the lateral forces resulting from the different load case combinations are compared. As earlier on mentioned the wind loads are depending on the first natural frequency of the building and therefore vary with the different load case combinations. For the ultimate limit states the maximum tension is chosen to be relevant. The significant load case combination is LK3: 1g+1,5w even though the value of the wind load applied is not the maximal one. Due to the softness of the structure the earthquake is not that significant (see also figure 6.3). For the serviceability limit states load case combination LK5: 1g+1w+0,5p is significant.

table 6.2: Lateral forces applied on the square type partial crosswise stiffened

	Vertical Load	natural frequency [Hz]	natural period [sec]	Wind static [kN/m²	Wind dynamic [kN/m²]	Earth quake [kN/m²]	ULS (break)	SLS (displacement)
LK1	1,35g+1,5p	0,267	3,75	0,78	w1=1,71	-		
LK2	1,35g+1,05p	0,293	3,41	0,78	w2=1,63	-		
LK3	**1g**	**0,445**	**2,25**	**0,78**	**w3=1,33**	**-**	**X tension**	
LK4	1g+0,3p	0,384	2,60	-	-	1,76*		
LK5	**1g+0,5p**	**0,358**	**2,79**	**0,78**	**w5=1,47**	**-**		**X**
LK6	1g+1p	0,310	3,22	0,78	w6=1,59	-		

* calculation based on the 1. natural frequency

In figure 6.3 the applied and resulting lateral forces due to static wind load, dynamic wind load and equivalent earthquake load are displayed. The factor for load case combinations is just mentioned in parenthesis but not totalled up. The load distribution, the factor applied on the lateral forces and the softness of the building avoid the load case earthquake to become significant. The load distribution for the load case earthquake is simplified for regular buildings according to EC 8 and DIN 4149.

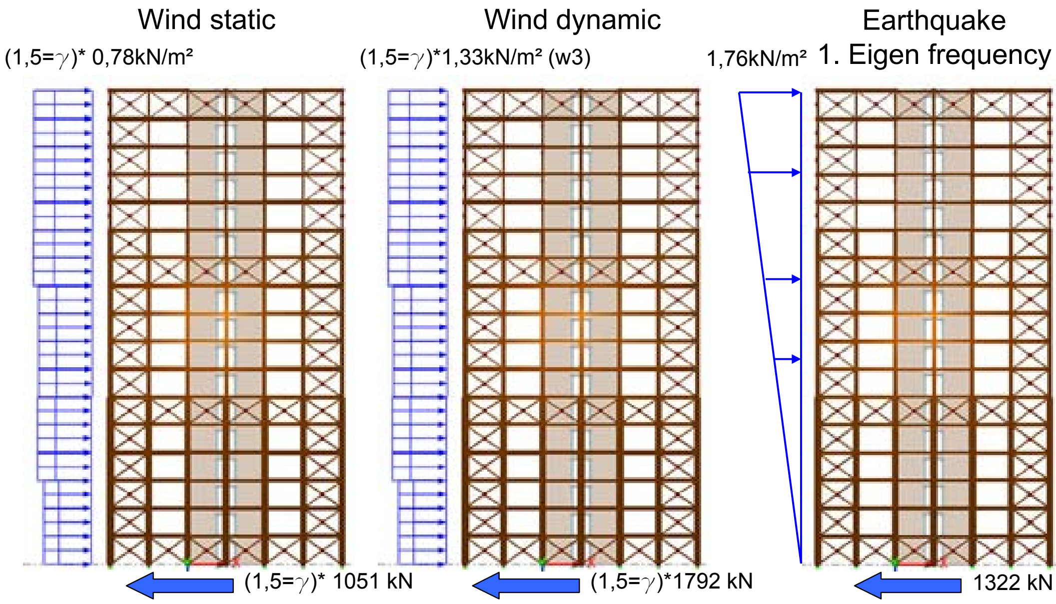


figure 6.3: Comparison lateral forces due to wind load and earthquake for square type partial crosswise stiffened, screen shot RFEM

For the connection of the diagonals a maximum load of about 215 kN needs to be transmitted (see figure 6.4). The difficulties are the pushing of the column (see figure 6.4, 1) and the local indentation of the beam (see figure 6.4, 2). Possible solutions could be the embedding of steel bearing plates or the planking with e.g. OSB plates with 15 mm thickness.

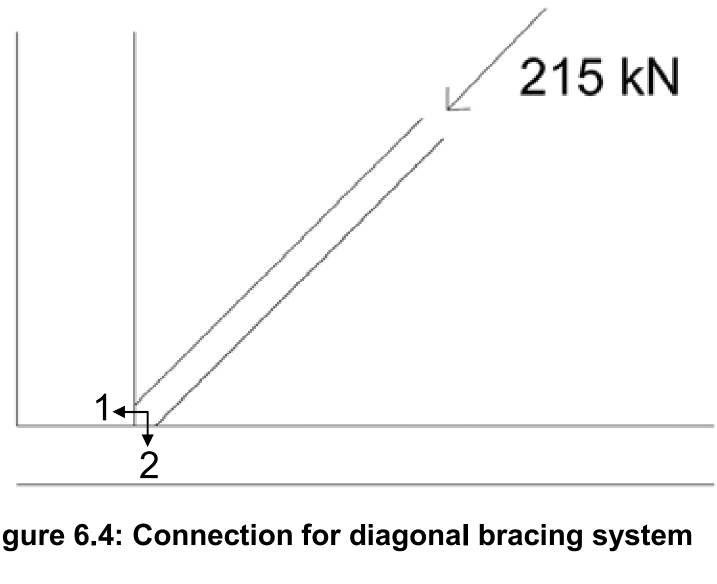


figure 6.4: Connection for diagonal bracing system

6.2 Rectangular type with partial rigid connection

The second type has a rectangular shape with a plan area of 742,5 m² with 45m by 16,5m (see figure 6.5). The vertical load transmission is effected by CLT panels located in the façade and columns forming a corridor in longitudinal direction located in the central part of the building (see figure 6.7). The CLT panels in the façade do as well work for the bracing system. They are joined with vertical CLT elements to a partial rigid connection (see figure 6.9). The entire bracing is done in the façade. The floor is supposed to be shearing stiff. The reduced stiffness and the additional deformation due to the partial rigid connection of the CLT elements are not taken into account for the finite element analyses.

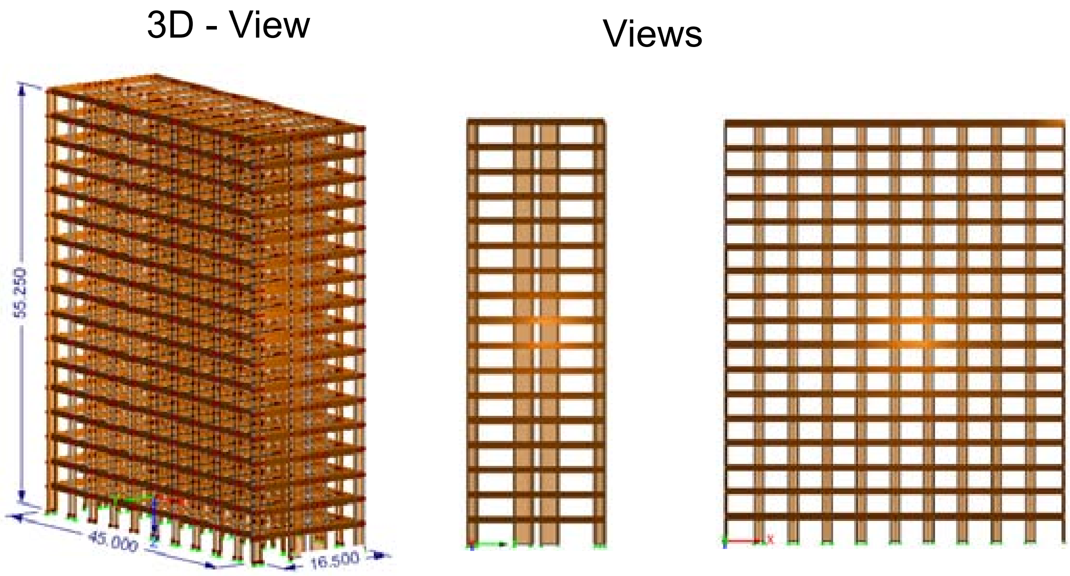


figure 6.5: Views of rectangular type with partial rigid connections, screen shot RFEM

The beams are oriented in transverse direction (see figure 6.6). They have on both sides of the corridor a span of 7,5 m. The beam over the corridor has a 1,5 m span.

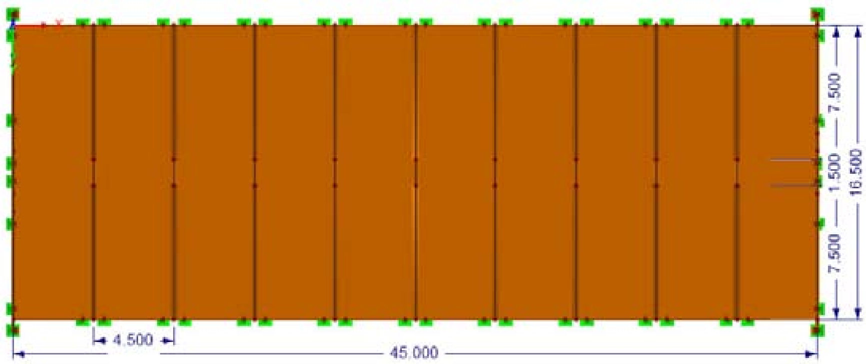


figure 6.6: Plan view of rectangular type with partial rigid connections, screen shot RFEM

The CLT wall panels in transverse direction are in the middle 2,4 m broad and on the sides 1,2 m broad. In longitudinal direction all CLT wall elements are 1,2 m broad (see figure 6.7). The column spacing is 4,5 m.

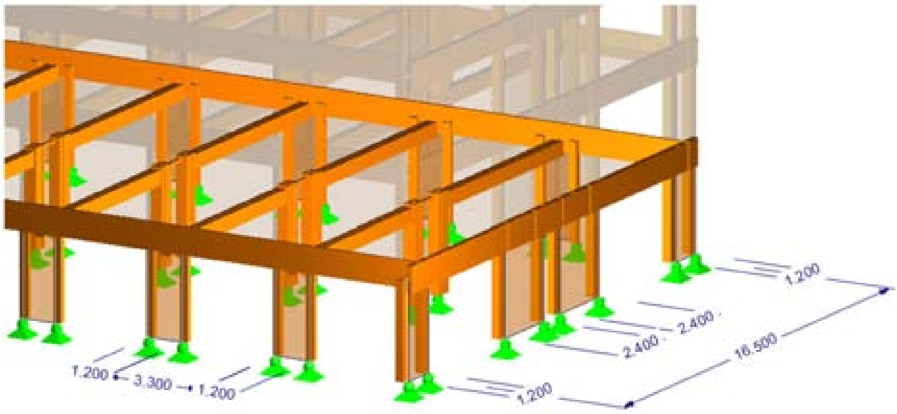


figure 6.7: One storey of rectangular type with partial rigid connections, screen shot RFEM

The materials and the cross sections used are displayed in figure 6.8: One storey with cross sections of rectangular type with partial rigid connections, screen shot RFEM. The panels used for floor and wall construction are made out of CLT. The beams are glulam beams. The cross sections used in transverse direction are twice the dimension used for the cross sections in longitudinal direction due to the larger area for wind acting load and the smaller amount of stiffening elements. The floor is stressed in longitudinal direction with a span of 4,5 m.
For a first optimising there is a grading of the cross sections of 20% for every six storeys related to the increasing vertical load.

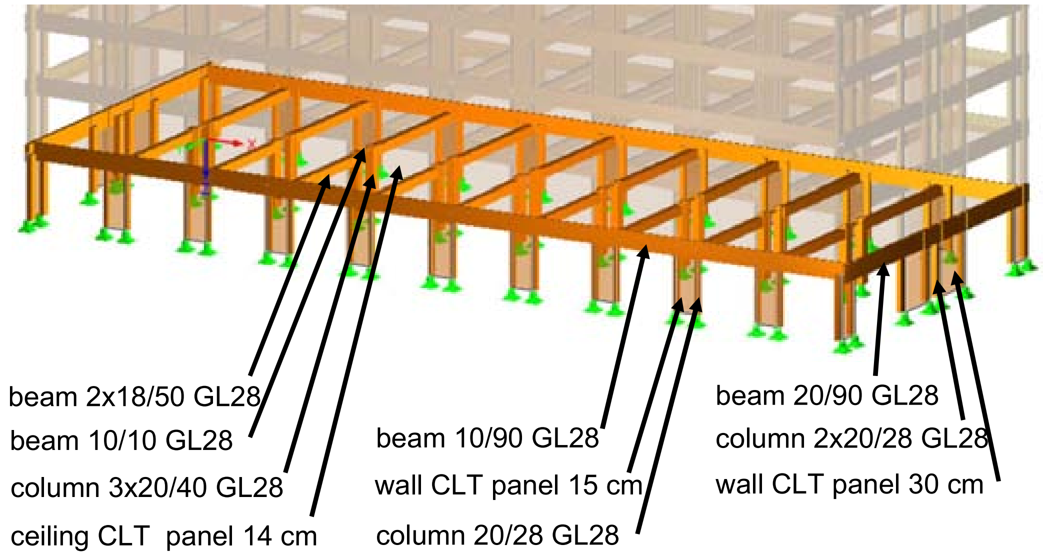


figure 6.8: One storey with cross sections of rectangular type with partial rigid connections, screen shot RFEM

The diversified cubage for glulam, cross laminated timber and softwood is shown in table 6.1. As for the square type the most part of the material is located in the floor. The highest amount of wood is used for the CLT panels in the floor.

table 6.3: Cubage in cubic meter for rectangular type with partial rigid connections

cubage in m³	GL	CLT	Σ
floor	415 (7,5 m span)	1768 (4,5 m span)	2183
Columns	164	-	164
Bracing vertical	172	330	502
Bracing horizontal	194	-	194

The total cubage needed for the square type partial crosswise stiffened is 3043 m³. Applied on the area of the building the material used is about 0,24m³/m². In terms of weight it amounts to 84 kg/m².

The partial moment rigid connection can be done with an angle iron. The moment is transmitted just by compression (see figure 6.9).

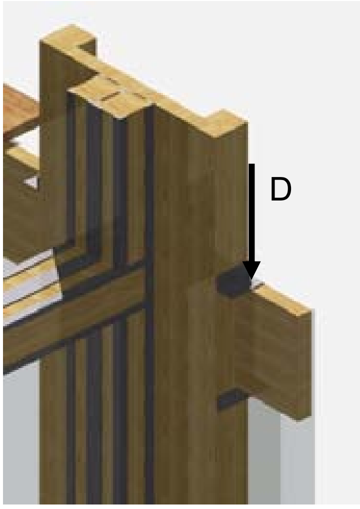


figure 6.9: Detail of partial rigid connection

For an average moment of M=100 KNm and a lever arm of a=1,3 m, the resulting compression load is D~80 KN.
With 0,5KN/cm² admissible compression the required area is $A_{necessary}$=160cm².
For a broad of b=10cm a length of $L_{necessary}$=16cm is relevant.

The anchorage of high tensile forces in CLT elements can be done by Hilti-nails connection (see figure 6.10). Steel nails are shot through two steel plates which have to be connected. The result is a mixture of a nailed and welded connection. The performance of these connections was investigated by the Institute of Architectural Sciences, Structural Design and Timber Engineering leaded by Prof. Winter of Vienna University of technology at the “Holzforschung Austria”. DI Georg Neubauer and I (Tamir Pixner) executed the tests. Tensile forces up to 470 kN were anchored with this connection.

figure 6.10: Connection for high tensile force anchorage made by Hilti-nails

(Source: Institute of Architectural Sciences, Structural Design and Timber Engineering, Vienna University of Technology)

7. Results of investigation

To compare the two investigated systems the maximum displacement of the building is relevant.

The applied wind load for the serviceability limit states results of the load case combination LK5: 1g+1w+0,5p for type 1 is 1,5 kN/m² (see figure 7.1).

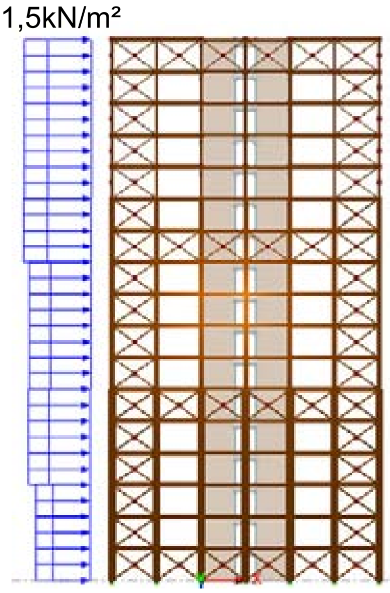


figure 7.1: Applied wind load for square type with diagonal bracing for serviceability limit states, screen shot RFEM

The maximum displacement is 9,3 cm (see figure 7.2). As the square type is complete regular the maximum displacement is in both axes identical.

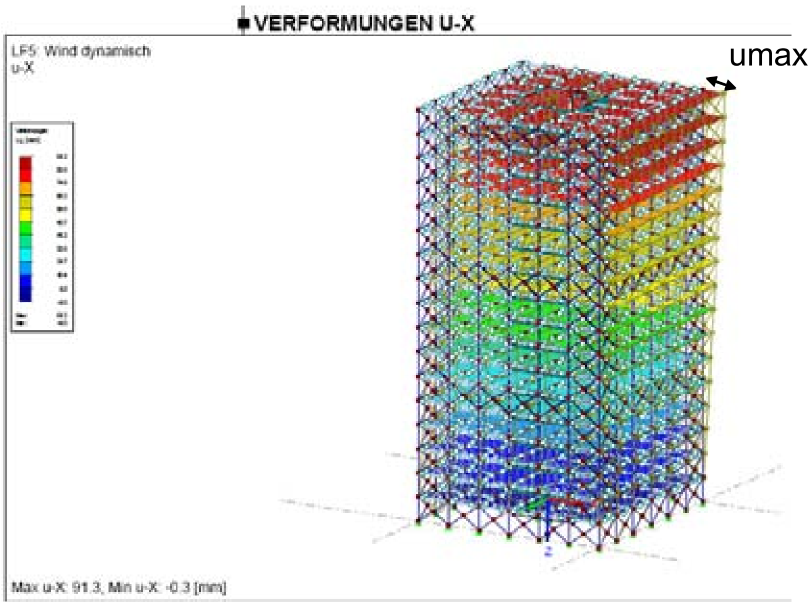


figure 7.2: Displacement of square type with diagonal bracing according to SLS LK5: 1g+1w+0,5p, screen shot RFEM

The applied wind load for the serviceability limit states results of the load case combination LK5: 1g+1w+0,5p for type 2 is 1,4 kN/m² (see figure 7.3).

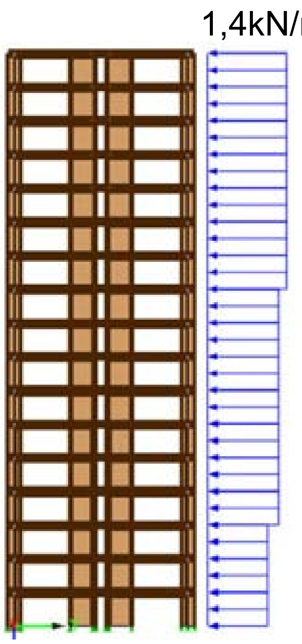


figure 7.3: Applied wind load for rectangular type with rigid connection for serviceability limit states, screen shot RFEM

The maximum displacement is 13 cm (see figure 7.4). The maximum displacement occurs in transversal direction where the structure is less stiff and the wind load acting area much greater.

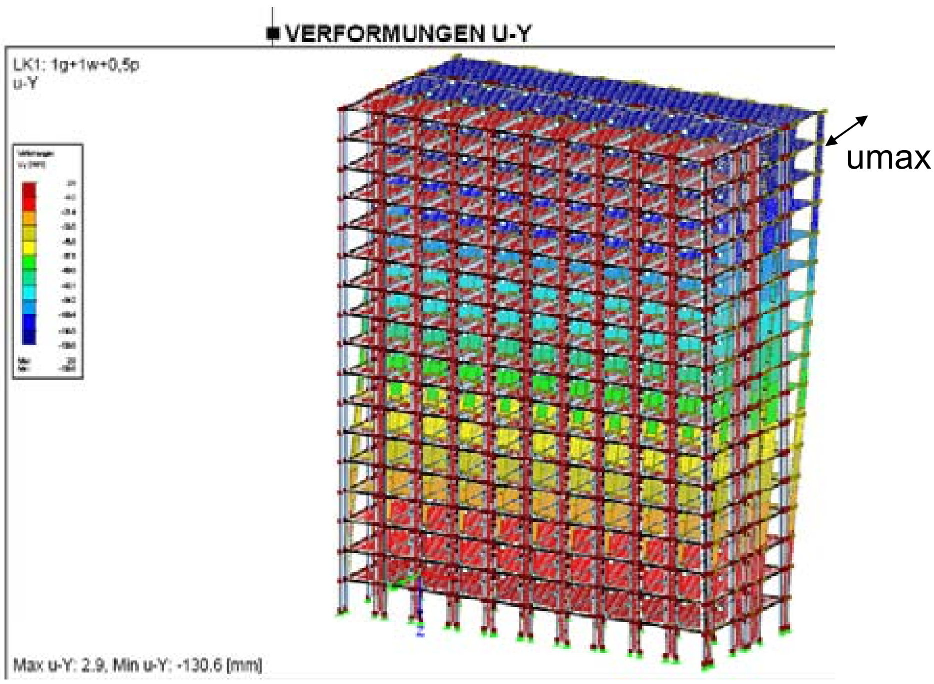


figure 7.4: Displacement of rectangular type with rigid connection according to SLS LK5: 1g+1w+0,5p, screen shot RFEM

In table 7.1 the main results of the comparison between the square type with diagonal bracing and the rectangular type with rigid connections are shown. Speaking in terms of used material the rectangular type with rigid connections needs less cubage. However the structure is less stiff resulting in greater displacement and anchorage tensile forces.

table 7.1: Results of comparison

		Structure square plan	Structure rectangular plan
	Total cubage	3419 m³	3043 m³
	cubage per square meter useable area	0,31m³/m²	0,24 m³/m²
Wind dynamic	Support reaction ULS LK3: 1g+1,5w	tension 928 kN compression 2241 kN	tension 1987 kN compression 2775 kN
	Displacement SLS LK5: 1g+1w+0,5p	U=91,3 mm<h/500	U=130,6 mm<h/500

For having a more meaningful comparison it will be necessary in further researches to compare two systems with the same amount of cubage or the same construction costs or the same stiffness or even more equal parameters.

8. Conclusion

This masters thesis present just a first step for the research work for multi storey timber buildings up to 20 storeys as several assumptions have been taken like the rigidity of the connections. Nevertheless such buildings should be static practicable. The main statements of this research work are followings:

Concerning the two types of structural systems these remarks can be given.
The bracing with linear diagonal elements should be adjusted in the way that the numbers of joints are reduced to avoid higher deformation. A reasonable way is the diagonal bracing over more than one storey. The analysed type 1 should just gives an impression about the stiffness of diagonal braced systems.
The bracing with partial rigid connections of CLT panels seems to be an intelligent system. These panels have multiple functions. First they have a high resistance against lateral loading. Furthermore the cross section assures the great vertical load transmission. Additionally the panels are located in the façade, that means less area is used inside the building and they are used as façade components. Still the behaviour of such partial rigid connections has to be investigated.
The floor structure is playing an important role. It influences the structural setup and as it is shearing stiff also the bracing system. The contribution to the bracing must be analysed as well as the setup in terms of sound insulation and weight.
The grade of prefabrication will be crucial for high rise timber buildings and the optimized usage of steel elements needs to be investigated.

Concerning the load cases following remarks can be given.
Light and soft timber structures show good behaviour in case of earthquake. The behaviour under short and long wave loading has to be investigated with different time-history analyses. Slender timber buildings need sufficient stiffness against wind. The stiffness of the structure can be varied depending on the earthquake and wind zone for optimal behaviour. The different requirements for the displacement limits for wind and earthquake loading must be taken into account.

Concerning the building regulations for timber constructions Austria is a far step back compared to other European countries. It will be necessary to work on the belief in this material not only of people who are in charge of the building regulations but of every one.

9. Figures

10. Tables