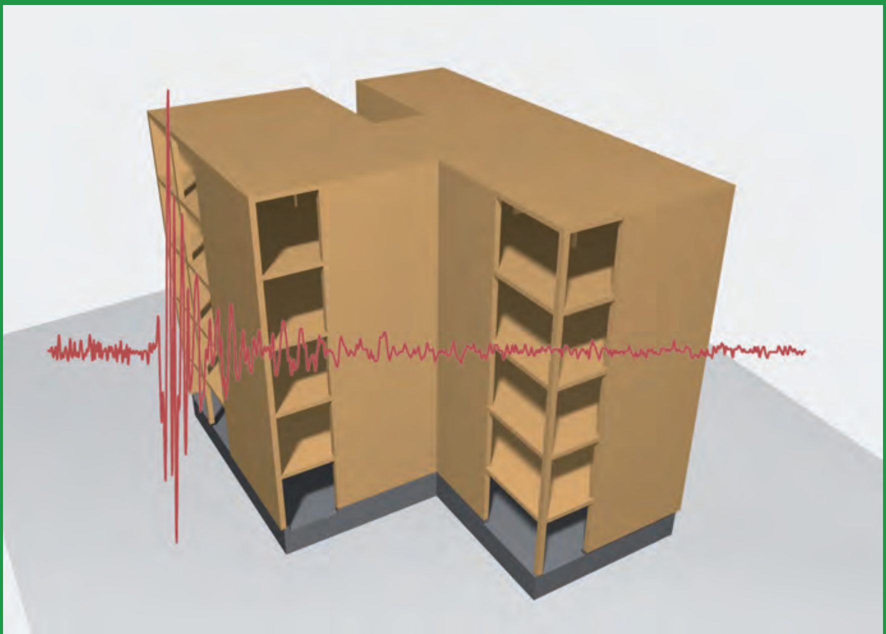
The seismic behaviour of buildings erected in Solid Timber Construction

Seismic design according to EN 1998 for a 5-storey reference building in CLT



Institute of Timber Engineering and Wood Technology
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Seismic design according to EN 1998 for a 5-storey reference building in CLT

ordered by

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Chapter **1**

Introduction and aim of the report

1 Introduction

1.1 Catastrophic earthquakes in the last years and their consequence for buildings

After having experienced the worldwide catastrophic natural disasters in the first months of the year 2011, it is believed that this year will make a gloomy entry in the history books. When on 11th March 2011 the surface of the earth of the Pacific Ocean just a few kilometres away from the Japanese archipelago began to shake measuring 9.0 on the Richter scale, no one was able to imagine the disastrous consequences [22]. However, the effect was detrimental: A tsunami devastated an area of 470 km^2 on the whole and caused thousands of casualties while even a higher number of people were made homeless. Added to this, the long term effect was an ultimate MCA of the nuclear power station Fukushima 1 with far-reaching consequences [31].

About two weeks before this serious incident an earthquake measuring 6.3 on the Richter scale caused extensive damage of ca. 13 billion dollars in Christchurch, New Zealand. Throughout the course of this natural disaster more than 5000 houses were destroyed and approximately 200 people were killed [22]. However, due to its local damage limitation, relatively little attention was devoted to this earthquake by the European media compared to the tragic incident in Japan.

Nevertheless, because of the fact that this earthquake did not cause a tsunami, this incident is of particular interest, since the buildings of Christchurch suffered from severe damage. The measurement of the intensity of the seismic load, which is made in order to categorise the damage caused by earthquakes, is based on the Modified-Mercalli-Intensity-Scale (MMI) and resulted in level IX (of XII) for the Christchurch-earthquake [22]. This implies that even buildings with a type of construction resistant to earthquakes suffer from severe damage, which in the end might lead to their partial collapses. Added to this, it is highly possible that buildings are shifted from their foundations causing cracks in the surface and resulting in extensive damage to underground conduits [32]. The following illustrations should make the categorization clearer.

Fig. 1.1 total collapse of a r/c skeleton structure [32]

Fig. 1.2 partial collapse of a building in brick construction with timber ceilings [32]

Fig. 1.3 severe damage to a single family house in Light-Weight Timber Construction with non-bearing masonry between the frames and 'heavy' roofing [32]

The natural disaster 'earthquake' also struck terror in the heart of Central Europe on 6th April 2009, when the Italian town L'Aquila suffered from potential damage caused by one of the most powerful earthquakes of Europe in the last decades. It measured 5.8 on the Richter Scale and inflicted 297 casualties [25]. Several of the 67.500 victims, who have sought help from civil defence after having lost their homes, still wait for the reconstruction of their homes [33]. Added to the total devastation of the historic town centre, several villages surrounding the epicentre were destroyed. The following illustration shows the location of the epicentre and the distance within the surrounding residential areas.

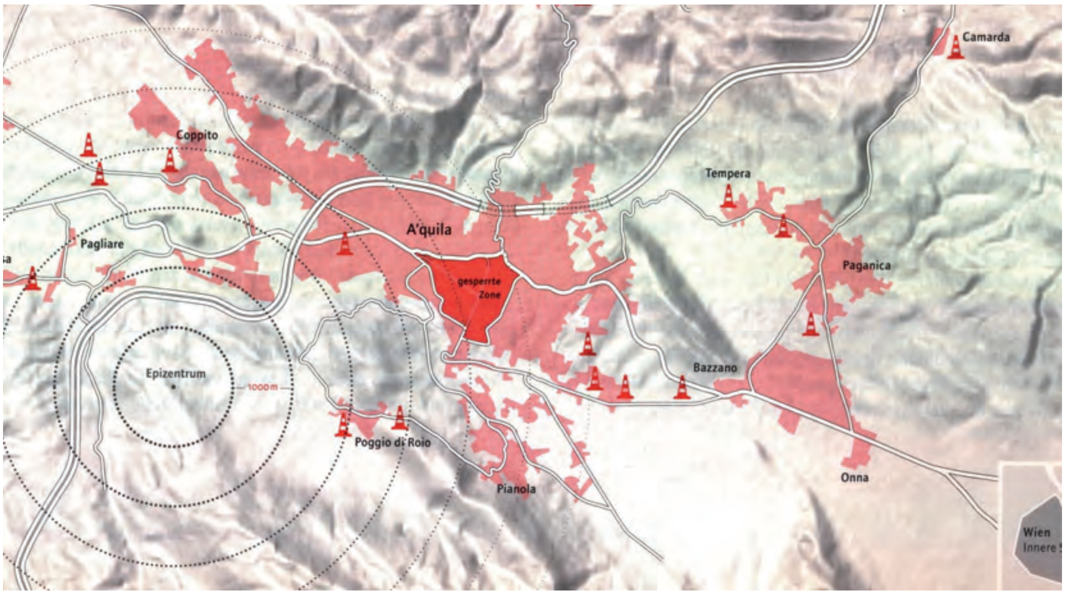

Fig. 1.4 location of the epicentre of L'Aquila and the affected areas [25]
(red-coloured area = residential area, traffic cone = affected village/town)

This earthquake was categorised as level IX based on the EMS-98 (European macroseismic scale) and was given the definition 'destroying'. Within this level of damage also a fundamental damage concerning the structures of buildings up to their total collapses is defined. The following illustration shows the devastated mountain village Onna, which was almost completely destroyed by the earthquake [25].

Fig. 1.5 aerial photograph of the mountain village Onna after the devastating earthquake [33]

To conclude this introduction, it is worth mentioning the earthquakes in Haiti in January 2010 and in Chile in February 2010, due to the fact that these natural disasters also inflicted a high number of casualties and made thousands of people homeless.

The list of devastating earthquakes and their tragic consequences for the resident population and buildings seems to be endless.

1. 2 The reconstruction of the disaster area based on the example of L'Aquila

The high number of homeless people as a consequence of the powerful earthquake in April 2009 provoked an immediate reaction of the Italian civil defence which lead to the call for bids for reconstructing 150 buildings in the very next month. It goes without saying that high criteria, such as lasting value, earthquake-resistance, environmental tolerance and especially construction time, were applied in the context of constructing these buildings. With regard to the latter criterion it is worth mentioning that the Italian civil defence intended to finish this ambitious construction project (including all the statutory periods concerning the calling for bids and the allocation of tasks) by the end of October 2009 [26].

All the previously mentioned important criteria were only able to be satisfied by using prefabricated types of construction. As a consequence, in more than the half of all these 150 buildings timber construction was used. In concrete terms this means that 15 buildings were erected in complete Solid Timber Construction, while either Light-Weight Timber Constructions or combinations of both designs were used with the rest. The following illustrations show several stages of construction in the context of erecting a building in complete Solid Timber Design and its completion [26].

About 11.000 m3 of Cross Laminated Timber elements manufactured in Austria were used altogether in this ambitious project.

Fig. 1.6 assembly of a staircase with a central lift core shaft [34]

Fig. 1.7 close-up of the buildings erected in complete Solid Timber Construction [34]

Fig. 1.8 building in Solid Timber Construction in completion [34]

In order to summarise this section, this illustrative chart should make the extremely short construction time clearer. It begins with the day of the devastating earthquake and ends with a completely erected building [26].

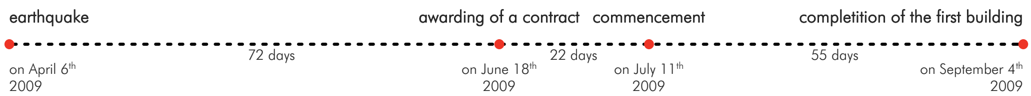

Fig. 1.9 illustrative chart beginning with the earthquake and ending with the completed reconstruction, according to [26]

1. 3 Conclusion

Based on the previously mentioned information the planning civil engineer, the executive building trade and the building industry are able to draw two fundamental conclusions in the context of erecting buildings in earthquake regions:

- The first aspect focuses on the development of an earthquake resistant philosophy of construction in earthquake regions aiming at minimising casualties and limiting damage *to fabrics* of buildings.

- The second aspect pays attention to the reconstruction of buildings and to the re-establishment of physical infrastructure. With regard to this element, it is the attempt to provide resistance to following earthquakes (aftershocks and other future earthquakes) as well as to significantly shorten the construction time under harsh conditions.

The combination of the foresaid aims strongly supports types of construction which have already proven to be robust, have the ability to resist imposed loads and have a short construction time.

The Solid Timber Construction making use of Cross Laminated Timber elements loaded in and out of plane and useable for walls and ceilings up to a height of 10 floors unites all these positive features. Added to this, due to the fact that the elements are made of sustainable raw material, Solid Timber Construction is an extremely environmentally friendly type of construction.

2 Aim of this report

The overall aim of this report is to assess the earthquake resistance on the basis of calculating the seismic design of a (residentual) building errected in Solid Timber Construction.

Thus, a detailed analysis of the sample building focusing on the instantaneous seismic design situation and using the currently valid ÖNORM EN 1998-1:2005 [12] as well as the related national appendix, ÖNORM B 1998-1:2006 [13], will be conducted.

In the context of assessing the earthquake resistance of a building, it is also highly interesting to analyse the control of the regular criteria in plane and evaluation due to their significant importance to the calculation.

The final determination of the primary seismic components, in this example the focus is on the bracing panels, is also made by using the in Austria currently valid Eurocodes for Timber Design in combination with the guidelines taken from the previously mentioned European Standard for earthquakes EN 1998:

ÖNORM EN 1995-1-1:2009 [8]

ÖNORM B 1995-1-1:2010 [9]

Chapter 2

The sample building

1 Description of the structure

1. 1 General survey of the concept of the building

A five-storey residential building without a basement floor and erected on a jointless r/c foundation slab was used for the analyses in the following sections. The following illustration shows the walls, columns and floor beams of the ground floor in plan.

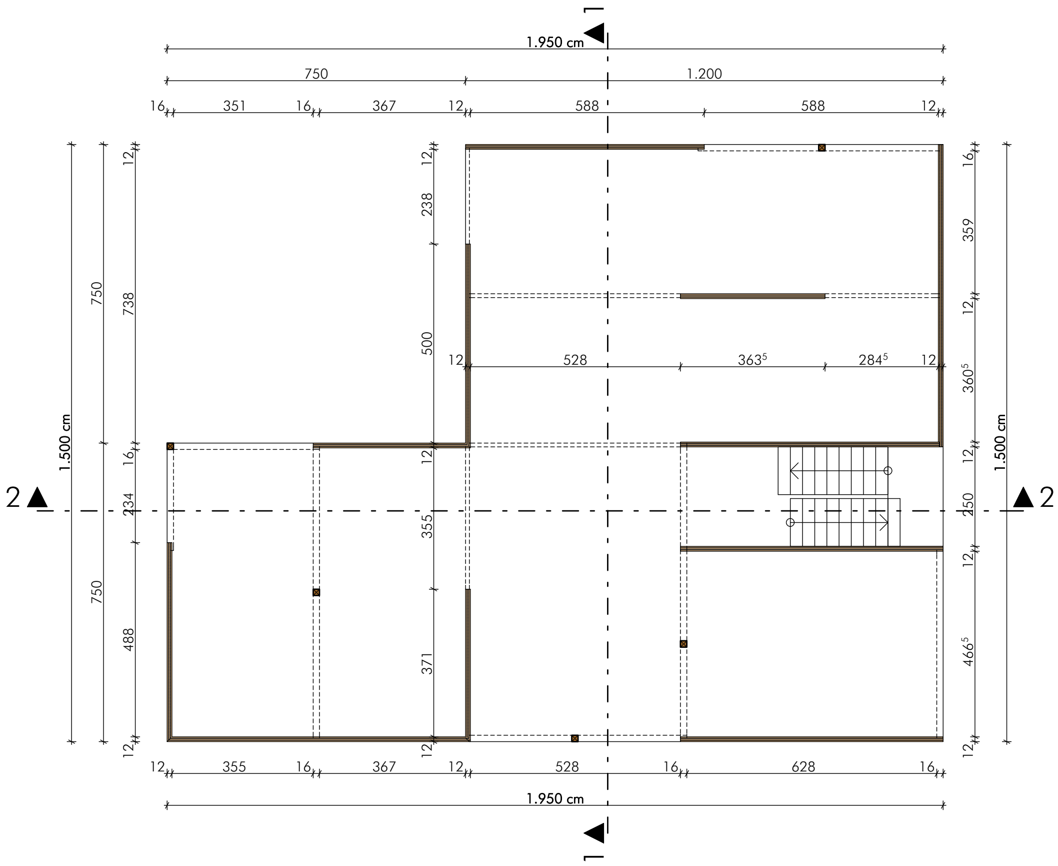

Fig. 1.1 ground floor of the sample building in plan

As it can be seen in fig. 1.1, the ground floor of the sample building is in the shape of a L, has a size of 19,50 x 15,00 m and is categorised into three areas (cf. section 2 for further information). Access is gained through the double run straight staircase, which is placed at the right external wall. Most of the vertical load-carrying structural members are panels in CLT, which in fulfilling their function as primary seismic members are used in order to pass on horizontal action.

note: According to ÖNORM EN 1998- 1 :2005 all members considered as part of the structural system that resists the seismic action, modelled in the analysis for the seimic design situation and fully designed and detailed for earthquake resistance in accordance with the rules of EN 1998 are regarded as primary seismic members.

The focus of interest in this report is the analysis of the bearing capacity of these panels made of Cross Laminated Timber. Added to the panels the illustrated floor beams (dotted) and columns contribute to the load distribution and, consequently, pass on vertical action. However, since the columns, in contrast to the panels, are defined as secondary seismic members, their contributions to passing on horizontal action are less significant.

The cover plates, as shown in fig. 1.1 as static single-span-, or rather multiple-span systems, are used in order to pass on vertical action to panels, floor beams and columns (out-of-plane panel behaviour). On the other hand, they are used as rigid shear panels in order to transfer horizontal action to the shear walls, which are regarded to act in the gravity centres of the prevailing ceilings (in-plane diaphragmatic behaviour). Therefore, also the ceiling plates need to be made of Cross Laminated Timber.

The following illustration shows the cross-section 1-1 of the structural system of the sample building.

Fig. 1.2 cross-section 1-1 of the sample building

As it can be seen in fig. 1.2, for all floors a height of ca. 3,00 m is chosen, which is a combination of a clearance height of 2,80 m and a ceiling thickness of ca. 0,20 m. By including an attic of 0,60 m in height the total height of the building H amounts to ca. 15,60 m. Added to this, with regard to further analyses, a vertical influencing zone of the ceiling h is calculated which is also 3,00 m in height for the floor slab.

note: The calculation of a vertical influencing zone h is essential in the context of seismic design, since the weight of a floor acts on the level of the prevailing floor slab.

The visualisation of the structural system in 3-D marks the end of this section.

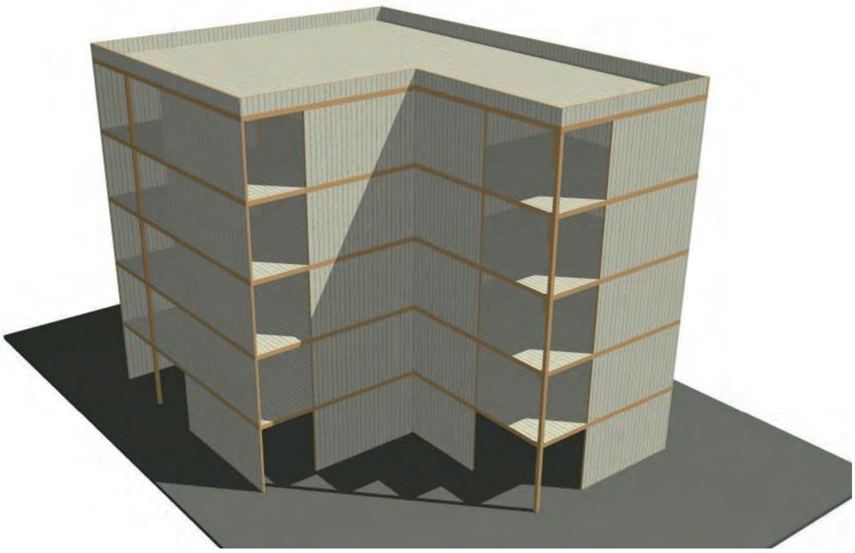

Fig. 1.3 structural system in 3-D of the sample building

1. 2 Assembling of the structural members and cross-section of the façade

The following tables and illustrations provide an overview of the assemblies used in this report.

note: The assemblies of the structural members are created in the context of this analysis model and should not be regarded as standard solutions.

- r/c base plate

run.Nr.	layer	thickness [mm]
1	glued parquet floor	10
2	final screed	60
3	PAE film	-
4	insulation of impact noise	30
5	EPS	100
6	filling material (stone chippings mixed with cement)	50
7	reinforced concrete slab	300
sum [mm]		550

Tab. 1.1 structural assembly „r/c base plate“

- ceilings

H01	floor slab	
run.Nr.	layer	thickness [mm]
1	glued parquet floor	10
2	final screed	60
3	PAE film	-
4	insulation of impact noise	30
5	filling material (stone chippings mixed with cement)	60
6	PAE film	-
7	cross laminated timber 196 mm, L5s	196
8	suspended ceiling (gypsum plaster)	95
sum [mm]		451

Tab. 1.2 structural assembly „floor slab“

H02	flat roof	
run.Nr.	layer	thickness [mm]
1	mineral plant substrate (layer of vegetation)	90
2	protective coat with filtering features	30
3	roof sheeting (PVC-free)	10
4	fleece	-
5	wooden formwork	20
6	thermal insulation with timber slats in between	250
7	vapour retarder / prov. roof sealing	-
8	cross laminated timber 196 mm, L5s	196
9	suspended ceiling (gypsum plaster)	95
sum [mm]		691

Tab. 1.3 structural assembly „flat roof“

- walls

H03	external wall load bearing	
run.Nr.	layer	thickness [mm]
1	façade plate	15
2	counter battens / ventilation zone	40
3	concealed façade insulating board with timber slats in between	160
4	cross laminated timber 95 mm, L5s	95
5	gypsum plaster plate	15
sum [mm]		325

Tab. 1.4 structural assembly „external wall load bearing“

H04	internal wall load bearing	
run.Nr.	layer	thickness [mm]
1	gypsum plaster plate	15
2	cross laminated timber 95 mm, L5s	95
3	gypsum plaster plate	15
sum [mm]		125

Tab. 1.5 structural assembly „internal wall load bearing“

	internal wall non-load bearing	
run.Nr.	layer	thickness [mm]
1	gypsum plaster plate	12,5
2	mineral wool with light metal posts	100
3	gypsum plaster plate	12,5
sum [mm]		125

Tab. 1.6 structural assembly „internal wall non-load bearing“

The cross section of the façade on the next page provides further inside in the combination of the previously mentioned make-ups of the structural members.

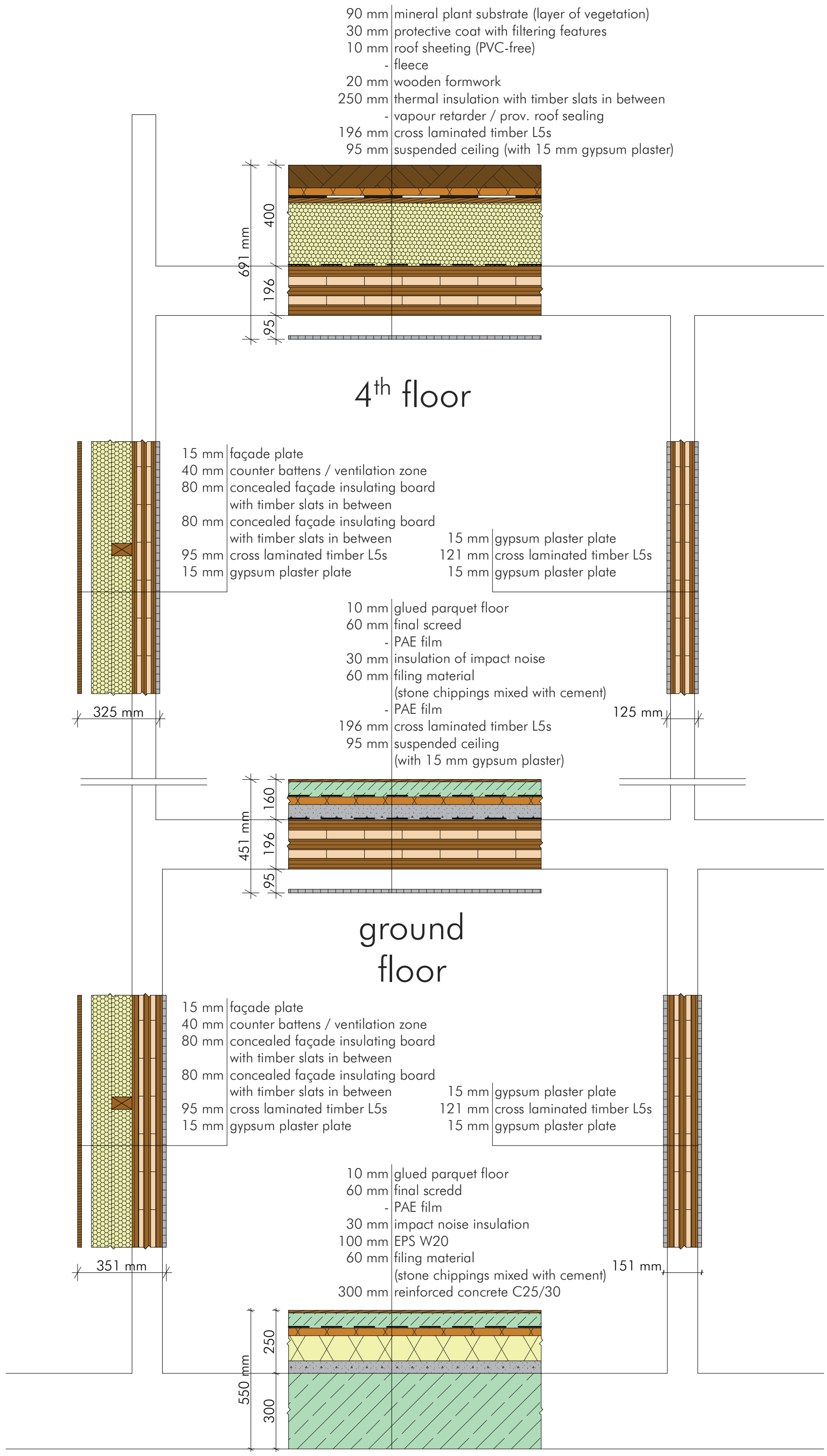

Fig. 1.4 combination of the structural assemblies shown in the cross section

2 Utilisation concept

Added to the dimensions of plan and elevation as well as to the definitions of the primary and secondary seismic structural members, it is of crucial importance to identify the utilisation concept in the context of analysing the earthquake resistance of a building. Hence, the sample building is defined as a residential building, has a floor area totalling 240 m^2 and is divided up in three flats per floor. The following illustration shows a possible arrangement of the flats of one floor of the sample building. All in all, this residential building with its five floors and three flats per floor provides 15 flats. Needless to say, a different utilisation concept of the sample building, such as an office building, would be also possible, but this idea is not pursued any further.

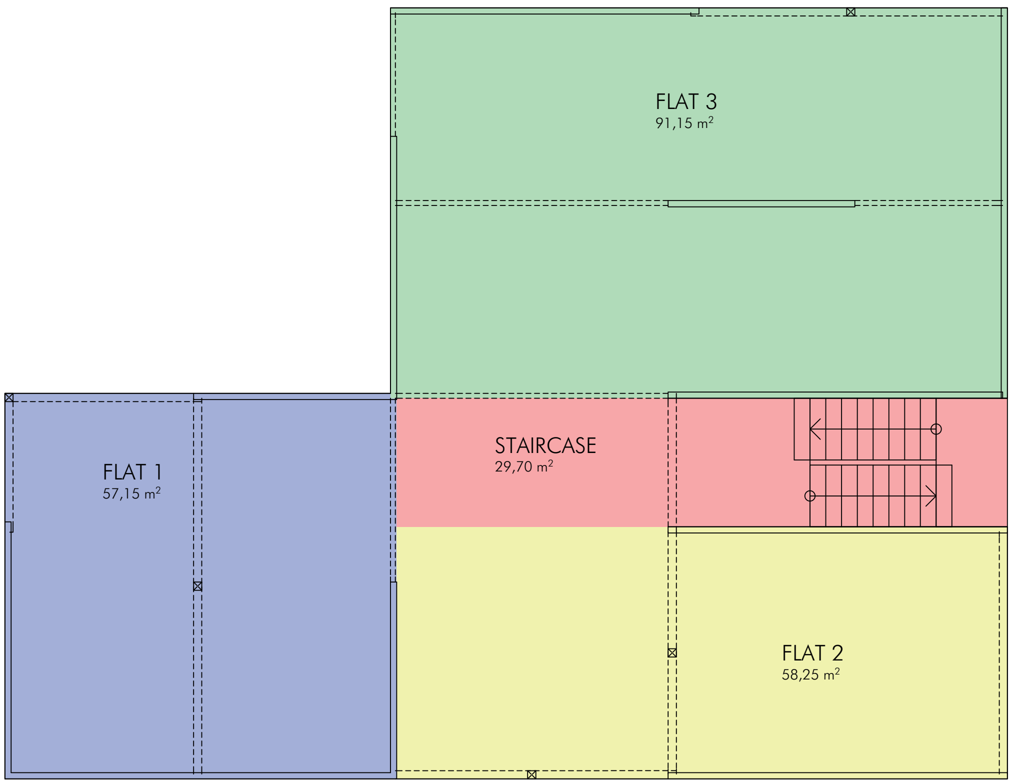

Fig. 2.1 utilisation concept with three flats per floor

Chapter 3

Pre-dimensioning of the load-bearing structure

1 Introduction

In the following sections the panels and ceiling elements, which have already been defined as primary seismic structural members in chapter 2, will be analysed in terms of their bearing capacities for vertical action. In concrete terms, they will be examined and, if necessary, modified in the context of their ultimate limit state (ULS) and their serviceability limit state (SLS). Due to the fact that the primary focus of this report is placed on assessing the earthquake resistance of buildings, the following calculations should be regarded as rough pre-dimensions.

In this case the pre-dimensioning of the bracing elements concerning the horizontal wind loading is not desirable, since the degree of the seismic load, which will be determined in the following chapter (cf. ch. 5), is significantly higher compared to the wind loading.

2 Determination of the vertical action

2. 1 Permanent action - dead weight of the structural assemblies

In this chapter the focus will be shifted to the permanent action of the structural assemblies, which have already been discussed in chapter 2. This determination is based on ÖNORM EN 1991-1-1:2003 [2] and ÖNORM B 1991-1-1:2006 [3].

2. 1. 1 Permanent actions

- ceilings

H01	floor slab			
run.Nr.	layer	thickness [mm]	γ [kN/m³]	d.γ [kN/m²]
1	glued parquet floor	10	8,00	0,08
2	final screed	60	22,00	1,32
3	PAE film			0,05
4	insulation of impact noise	30	1,40	0,04
5	filling material (stone chippings mixed with cement)	60	20,00	1,20
6	PAE film			0,05
7	cross laminated timber 196 mm, L5s	196	5,50	1,08
8	suspended ceiling (gypsum plaster)	95		0,33
sum	[mm]	451		4,15
	(without CLT layer)			3,07

Tab. 2.1 permanent action „floor slab“

H02	flat roof			
run.Nr.	layer	thickness [mm]	γ [kN/m³]	d.γ [kN/m²]
1	mineral plant substrate (layer of vegetation)	90		1,80
2	protective coat with filtering features	30		0,75
3	roof sheeting (PVC-free)	10		0,05
4	fleece	-		0,05
5	wooden formwork	20	5,50	0,11
6	thermal isnulation with slats in between	250	1,20	0,30
7	vapour retarder / prov. roof sealing	-		0,05
8	cross laminated timber 196 mm, L5s	196	5,50	1,08
9	suspended ceiling (gypsum plaster)	95		0,33
sum	[mm]	691		4,52
	(without CLT layer)			3,44

Tab. 2.2 permanent action „flat roof“

- walls

H03	external wall			
run.Nr.	layer	thickness [mm]	γ [kN/m³]	d.γ [kN/m²]
1	façade plate	15	8,00	0,12
2	counter battens / ventilation zone	40	0,44	0,02
3	concealed façade insulationg board with slats in between	160	1,93	0,31
4	cross laminated timber 95 (121) mm, L5s	95 (121)	5,50	0,52 (0,67)
5	gypsum plaster plate	15	-	0,15
sum	[mm]	325 (351)		1,12 (1,26)

Tab. 2.3 permanent action „external wall“

H04	internal wall			
run.Nr.	layer	thickness [mm]	γ [kN/m³]	d.γ [kN/m²]
1	gypsum plaster plate	15		0,15
2	cross laminated timber 95 (121) mm, L5s	95 (121)	5,50	0,52 (0,67)
3	gypsum plaster plate	15		0,15
sum	[mm]	125 (151)		0,82 (0,97)

Tab. 2.4 permanent action „internal wall"

note: The numbers in brackets in tab. 2.3 and tab. 2.4 are taken from an already conducted analysis and are valid for the walls in the ground floor and in the first floor. The reason for this is that due to the vertical action, it is necessary to extend the thickness of the external as well as of the internal walls to 121 mm using CLT-elements.

2. 2 Variable action - imposed loads

2. 2. 1 Imposed loads in general

According to ÖNORM EN 1991-1-1:2003 [2] and ÖNORM B 1991-1-1:2006 [3] the following imposed loads can be identified for a residential building (service class A1):

category		q_k [kN/m²]	Q_k [kN]
A1	ceilings	2,00	2,00
	staircases in residential buildings	3,00	2,00
	balconies	4,00	2,00

Tab. 2.5 imposed loads for residentual buildings according to [2] and [3]

2. 2. 2 Considering additional loads due to the self weight of movable partitions

According to ÖNORM EN 1991-1-1:2003 [2] the action resulting from the self weight of movable, non-load-bearing (internal) walls is added to the imposed loads in terms of surface load q_k (additional loads of movable partitions), if their self weight does not exceed an upper limit of 3,0 kN/m.

In the context of the movable partitions of the sample building (cf. ch.2) an additional load of q_k = 0,8 kN/m^2 is determined.

2. 3 Variable loads - snow loads

With regard to the fictitious position (cf. ch. 4) of the sample building, a vertical action based on snow loads of

s_k = 1,60 kN/m2

is determined.

3 Pre-dimensioning of the sample building

In order to be able to assess the necessary dimensions of the CLT elements a (timber-) strength class of C24, or rather GL24h, is used for calculation.

3. 1 Pre-dimensioning of the CLT-ceiling elements

Although there could be used a two-dimensional structural behaviour in order to create an economic structural design of floor systems out of Cross Laminated Timber, this pre-dimensioning is restricted to a one-dimensional girder system using a ceiling strip of 1m in width. In this context, the first step is to categorise the ceiling elements into various single span girders in plan. The following illustration shows a possible approach.

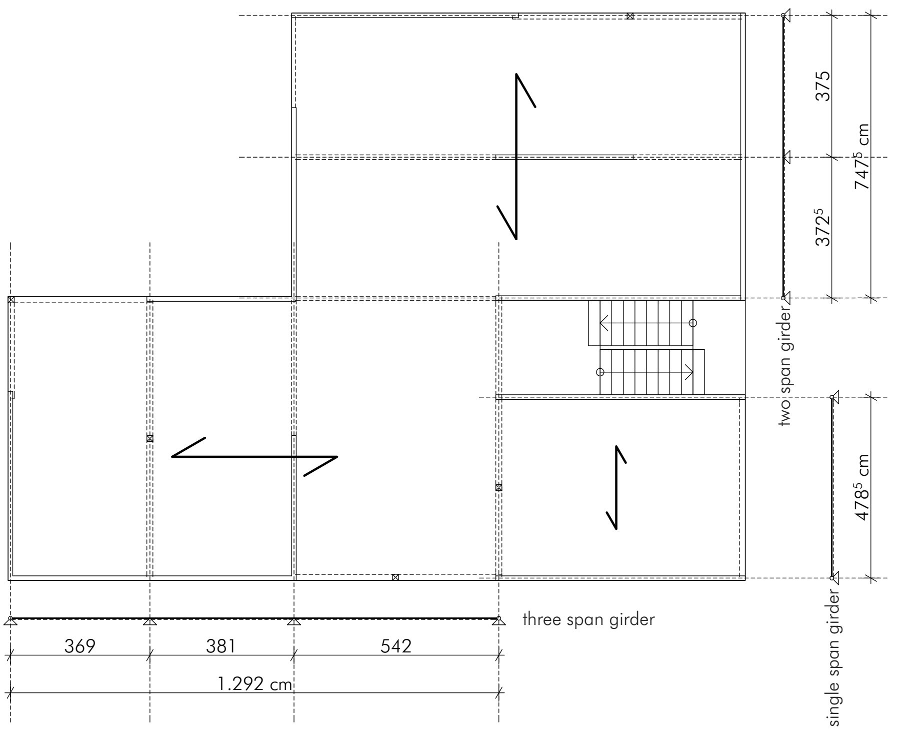

Fig. 3.1 possible approach to categorise the ceiling elements of a single spam girder system in plan

As shown in fig. 3.1, the ceiling is subdivided into three one-way slabs. In this regard, it is worth mentioning that the single span girder system is considered as the worst case in the context of the highly important service ability limit state (SLS) due to its wide span. However, since not only deflections, but also vibrations play a crucial role concerning the service ability limit state (SLS), the three span girder system is also analysed in terms of both limited states.

The determination of the CLT slab elements is made by entering the following data into the software application 'CLT designer' created at the Institute for Timber Engineering and Wood Technology of Graz University of Technology:

system	span(s) [m]	additional dead load $g_{2,k}$ [kN/m²]	imposed load q_k [kN/m²]	details to determine vibration
single span girder	4,79	3,07	2,80	damping coefficient ζ = 4%, final screed: d = 6 cm, E = 26.000 N/mm²
three span girder	3,69; 3,81; 5,42	3,07	2,80	damping coefficient ζ = 4%, final screed: d = 6 cm, E = 26.000 N/mm²

Tab. 3.1 input data to pre-determine the slab elements using the 'CLT designer'

The CLT elements, which are used for all slabs and the flat roof need to be finely balanced in terms of maximum deflection, natural frequency and efficiency. The following table shows the selected slab element (used in the single span girder system) with the utilisations in the previously mentioned limit states.

element	utilisation in ULS	utilisation in SLS	natural frequency
CLT 196 mm, L5s	33% (bending)	80% (deflection $w_{net,fin}$ according to ÖNORM EN 1995-1-1)	7,7 Hz

Tab. 3.2 utilisation of the selected CLT slab element in the chosen system (single span girder)

note: The estimated natural frequency of 7,7 Hz is below the standard specification of 8,0 Hz ([8]). This implies that the determination of the vibration of the system needs to be made more accurately (cf. calculating record in the appendix and 'BSPhandbuch Holz-Massivbauweise in Brettsperrholz' [17]). However, with regard to this report, the already determined natural frequency is sufficient.

3. 2 Pre-dimensioning of the wall panels

3. 2. 1 Dimensioning of the most highly stressed wall panel

In contrast to the determination of the ceiling elements, the wall panels are designed for a two-way structural behaviour of the ceiling elements. This means that panels which are situated perpendicularly to the main span girder axis are also influenced by the vertical action of the prevailing floors. The measurement used in this report is not explicitly necessary for determining the vertical action, but is highly advisable to be made in the context of seismic design. In order to determine the vertical forces of the walls with regard to the stress of the prevailing floor the 3-D Finite-Element program 'REFM' is used to model a whole floor of the sample building. In this model the floor slab has a unit load of

1 kN/m^2. The reaction forces which are given to the panels can be considered as influence coefficients in the context of further calculations. The following illustration shows a 3-D graphic of a floor of the sample building with the unit load of the slabs.

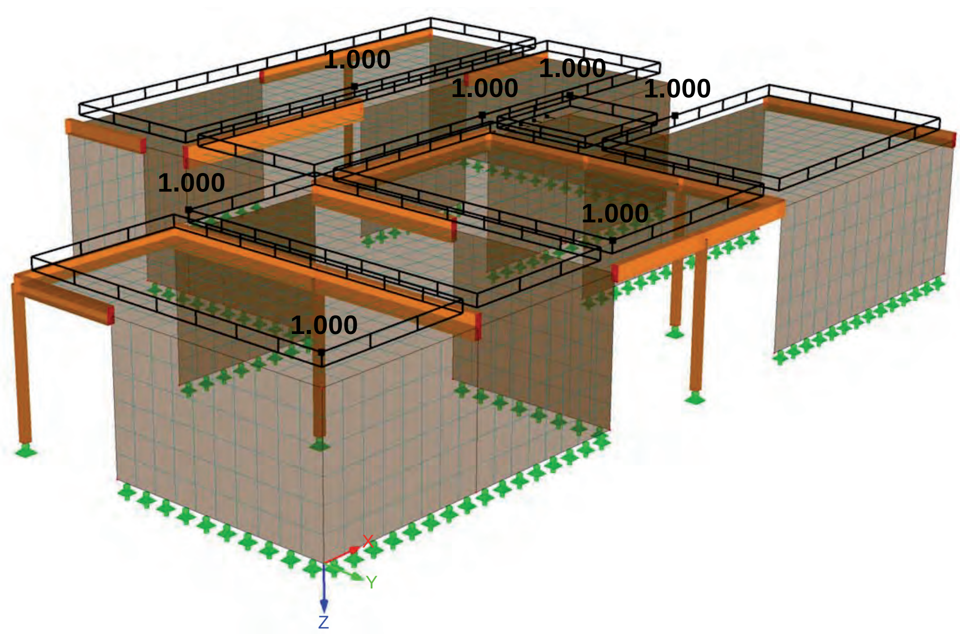

Fig. 3.2 3-D graphic of the REFM system to determine the influence coefficients of the panels

As it can be seen in fig. 3.2, the panels of the floor are fastened with linear bearings which have a certain load per meter as reaction force. Needless to say, this load is inconsistent (higher at free wall ends) due to the geometry of the floor. With regard to the pre-determination of the panels (following illustration) an average load per meter is used, which can be found in tab. 3.3 for all panels.

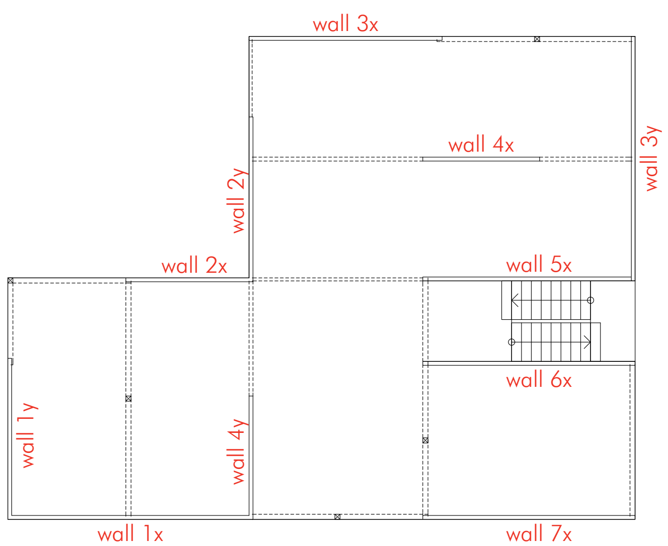

Fig. 3.3 definition of all wall panels for further calculations

x-direction		y-direction	
wall-nr.	influence coefficient e	wall-nr.	influence coefficient e
1x	1,07	1y	1,92
2x	3,20	2y	3,79
3x	1,96	3y	1,41
4x	**9,20**	4y	6,62
5x	2,71		
6x	3,98		
7x	2,21		

Tab. 3.3 determined influence coefficients for the prevailing panels

It can be said that wall '4x' with an influence coefficient of 9,20 has to cope with the greatest amount of slab stress. Hence, this wall is regarded as the archetype of all load-bearing walls of the sample building and is analysed in terms of vertical action in ultimate limit state.

3. 2. 2 Determining the internal design forces

As it can be seen in the cross section of chapter 2, the most powerful longitudinal force max $n_{y,dN}$ can be determined at the base point of the ground floor. The following calculations show the determination of this longitudinal force:

- loading combination
  load duration class 'medium term' (imposed load is decisive)

$$E_d = \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i \geq 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$$

with

action	weight [kN/m²]	permanent	variable
total weight flat roof $g_{1,k}$	4,52	x	
total weight floor slab $g_{2,k}$	4,15	x	
total weight internal wall $g_{3,k}$	0,97	x	
imposed loads incl. weight of moveable partition walls q_k	2,80		x
snow load s_k	1,60		x

Tab. 3.4 action on wall elements

leads to

$$\max \; n_{y,\,dN} = \gamma_G \cdot (g_{1,\,k} + 4 \cdot g_{2,\,k}) \cdot e + \gamma_G \cdot g_{3,\,k} \cdot 5 \cdot h + \gamma_Q \cdot (q_k \cdot 4 + \psi_{0,\,s} \cdot s_k) \cdot e$$

$$\max \; n_{y,\,dN} = 1,35 \cdot (4,52 + 4 \cdot 4,15) \cdot 9,20 + 1,35 \cdot 0,97 \cdot 5 \cdot 3,0 + 1,50 \cdot (2,80 \cdot 4 + 0,50 \cdot 1,60) \cdot 9,20 = 448 \;\; kN/m$$

3. 2. 3 Determining the elastic modal shapes in the context of the Euler-case II

Due to the fact that an ideal combination of singular span columns can be used for the existent system of internal walls, the buckling bar of the ground floor is regarded as a column simply-supported at both ends. This results in the Euler case II. The following calculations show the determination of the buckling length:

$$l_k = \beta \cdot h = 1,0 \cdot 2,80 = 2,80 \;\; m$$

with

l_k as buckling length of the system [m]

h as system length (corresponds with the height of the floor) [m]

β as column buckling factor [-]

3. 2. 4 Determining the ideal elastic buckling load

The following illustration shows the dimensions of the selected CLT wall element.

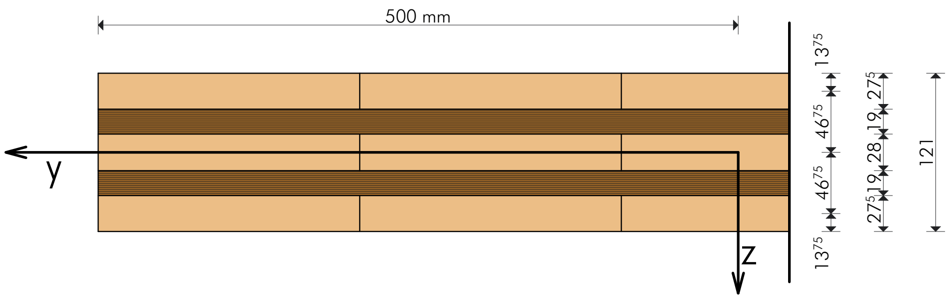

Fig. 3.4 cross section of the CLT panel

The bending stiffness EJ for one stripe of 1 m taken from this cross section is according to [17]:

$$EJ = E_{0,\,05} \cdot \left(2 \cdot \frac{0,0275^3}{12} + \frac{0,028^3}{12} + 2 \cdot 0,0275 \cdot 0,04675^2\right)$$

with

$$E_{0,05} = \frac{2}{3} \cdot E_{0,mean} = \frac{2}{3} \cdot 1,10 \cdot 10^7 = 7,33 \cdot 10^6 \ kN/m^2$$

leads to

$$EJ = 7,33 \cdot 10^6 \cdot \left(2 \cdot \frac{0,0275^3}{12} + \frac{0,028^3}{12} + 2 \cdot 0,0275 \cdot 0,04675^2\right) = 920 \ \ kNm^2$$

The shear stiffness S_{clt} for one stripe of 1 m of this cross section is according to [17]:

$$S_{clt} = \kappa \cdot \sum G_i \cdot A_i =$$
$$0,20 \cdot (2 \cdot 4,6 \cdot 10^5 \cdot 0,0275 + 4,6 \cdot 10^5 \cdot 0,028 + 2 \cdot 4,6 \cdot 10^4 \cdot 0,019) =$$
$$7986 \ \ kN$$

The ideal elastic buckling load n_{cr} can be determined as follows:

$$n_{cr} = \frac{EI \cdot \pi^2}{l_k^2 \cdot \left(1 + \frac{EI}{\kappa \cdot \sum G_i \cdot A_i \cdot l_k^2}\right)} = \frac{920 \cdot \pi^2}{2,80^2 \cdot \left(1 + \frac{920}{7986 \cdot 2,80^2}\right)} = 1141 kN$$

3. 2. 5 Stability verification

The verification of the effective area of cross section can be determined as follows:

$$A_{eff} = (2 \cdot 27,5 + 28,0) \cdot 1000 = 8,30 \cdot 10^4 \ \ mm^2.$$

The column buckling factor k_c of the system can be extracted as the smaller figure from:

$$k_c = \min \begin{bmatrix} 1,0 \\ \dfrac{1,0}{(k + \sqrt{k^2 - \lambda_{rel}^2})} \end{bmatrix}$$

with

$$k = 0,5 \cdot (1 + \beta_c \cdot (\lambda_{rel} - 0,30) + \lambda_{rel}^2)$$

and

$$\beta_c = 0,10.$$

$$\lambda_{rel} = \sqrt{\frac{n_c}{n_{cr}}} = \sqrt{\frac{A_{eff} \cdot f_{c,k}}{n_{cr}}} = \sqrt{\frac{83000 \cdot 21}{1141000}} = 1,236$$

$$k = 0,5 \cdot (1 + 0,10 \cdot (1,236 - 0,30) + 1,236^2) = 1,31$$

$$k_c = \min\begin{bmatrix} 1,0 \\ \dfrac{1,0}{(1,31 + \sqrt{1,31^2 - 1,236^2})} \end{bmatrix} = \min\begin{bmatrix} 1,00 \\ 0,57 \end{bmatrix} = 0,57$$

- Stability verification of load duration class 'medium term'

design value of the compressive strength in the direction of grain:

$$f_{c,d} = f_{c,k} \cdot \frac{k_{mod}}{\gamma_M} = 21,0 \cdot \frac{0,80}{1,25} = 13,44\ N/mm^2$$

verification:

$$\frac{\max\ n_{y,dN}}{k_c \cdot A_{eff} \cdot f_{c,d}} = \frac{447550}{0,57 \cdot 83000 \cdot 13,44} = 0,70 < 1,00$$

The verification is fullfilled, the degree of utlisation is **70%**.

As it can be seen in the verification of the selected wall, a relatively high level of utilisation is determined, although a CLT element with 5 layers and a thickness of 121mm is used. In order to consider the vertical stress of the walls as a consequence of the relatively heavy floor slabs, this CLT element (121 mm, L5s) is used for the ground and the first floor, whereas in the other storeys a CLT element with 5 layers and only 95 mm is used. As it will be shown in the following sections, the CLT wall elements share the same shear capacities (according to [17]) based on the verification of the Representative Volume Sub-Element (RVSE). Thus, this fact simplifies the verification in the context of seismic design.

3. 3 Summary of the CLT elements used for walls and slabs

The following table shall summarise the previously mentioned dimensions of the CLT elements for walls and slabs.

element	ground floor	first floor	second floor	third floor	fourth floor
walls	121 L5s	121 L5s	95 L5s	95 L5s	95 L5s
ceilings	196 L5s	196 L5s	196 L5s	196 L5s	196 L5s

Tab. 3.5 dimensions of the CLT elements used for walls and slabs

Chapter 4

Determining parameters relevant for seismic design

1 Introduction

In this chapter the focus of interest is on determining all the necessary parameters which are relevant for the seismic design used for the sample residential building. Based on its location, its material parameters as well as on the description of its soil class, in this chapter the quantity-surveying of the sample building and the examination of criteria of regularity in plan and elevation are conducted according to ÖNORM EN 1998-1 [12], which highly influence the extent as well as the mode regarding the seismic design.

2 Location of the sample building

2. 1 Explanation of the chosen ground acceleration a_g

When the rather low risk of earthquakes in Austria (just about 20% of the area of the country faces the possibility of being shaken by earthquakes) and the earthquake intensity of L'Aquila 2009 are taken under consideration, a fictitious location for the sample building needs to be generated. This location is supposed to fulfil two requirements: (a) ÖNORM EN 1998-1 [12], or rather ÖNORM B 1998-1 [13], need to be still valid and (b) the level of earthquake intensity needs to be at least as high as the one in L'Aquila of 2009. As a consequence, a ground acceleration, which can be hardly found in Austria, needs to be presupposed.

The intensity of ground acceleration is taken from the investigations in [18] and for a residential building in L'Aquila it amounts to

a_g = 3,34 m/s2.

note: If the building was located in a certain earthquake zone of Austria, the level of ground acceleration would be taken from ÖNORM B 1998-1, which can be found in the appendix (map of earthquakes and gazetteer). This determined ground acceleration a_g already includes the information of the importance factor γ_I, which results in 1,0 for a residential building according to ÖNORM EN 1998-1. Using this approach the maximum level of ground acceleration of a residential building in Austria according to [18] amounts to 1,42 m/s2 (zone IV).

2. 2 Soil quality of the fictitious location

Due to the fact that in the context of this report environmental parameters, such as the quality of the soil, play a subordinate role, the soil class A is chosen. According to EN 1998-1:2004 (E) [12] this class is defined as 'rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.'

3 Determining the relevant material parameters in the context of the instantaneous load case 'earthquake'

The determination of the material components is demonstrated on the basis of walls and slabs made of Cross Laminated Timber according to the guidelines of 'BSPhandbuch Holz-Massivbauweise in Brettsperrholz' [17]. The used parameters (spruce with the strength class C24) are allocated to the service class 1 and a load duration class 'instantaneous' (for accidental action). The modification factor for this design is according to ÖNORM EN 1998-1, section 8.6 [12] and ÖNORM EN 1995-1-1, table 3.1 [8]

$k_{mod} = 1{,}10.$

The conversion of the characteristic values of the construction material in design values is undertaken by using the equation

$$f_{xy,d} = k_{mod} \cdot \frac{f_{xy,k}}{\gamma_M}$$

with a partial safety factor γ_M of the construction material, which, with regard to this instantaneous load case 'earthquake', according to ÖNORM EN 1995-1-1, section 2.4 [8] results in

$\gamma_M = 1{,}0.$

According to EN 338 the material data sets for the base product 'board' are collected by

- Strength
  - compressive strength in grain direction:

    $$f_{c,clt,d} = k_{mod} \cdot \frac{f_{c,clt,k}}{\gamma_M} = 1,1 \cdot \frac{21,0}{1,0} = 23,1\ N/mm^2$$

  - compressive strength perpendicular to grain:

    $$f_{c,clt,90,d} = k_{mod} \cdot \frac{f_{c,clt,90,k}}{\gamma_M} = 1,1 \cdot \frac{2,50}{1,0} = 2,75\ N/mm^2$$

  - in-plane shear strength:

    $$f_{v,clt,d} = k_{mod} \cdot \frac{f_{v,clt,k}}{\gamma_M} = 1,1 \cdot \frac{5,0}{1,0} = 5,50\ N/mm^2$$

  - in-plane torsional strength:

    $$f_{tor,clt,d} = k_{mod} \cdot \frac{f_{tor,clt,k}}{\gamma_M} = 1,1 \cdot \frac{2,5}{1,0} = 2,75\ N/mm^2$$

- Stiffness
  - modulus of elasticity in grain direction:
    $E_{0,\,mean} = 11000\ N/mm^2$
  - modulus of elasticity perpendicular to grain:
    $E_{90,\,mean} = 370\ N/mm^2$
  - shear modulus:
    $G_{0,\,mean} = 690\ N/mm^2$
  - rolling shear modulus:
    $G_{90,\,mean} = 69\ N/mm^2$

4 Combination of actions in the context of the instantaneous load case 'earthquake'

The combination of actions with regard to the verification in the ultimate limit state in the context of the instantaneous load case 'earthquake' according to ÖNORM EN 1990:2003 [1] can be defined as follows:

$$\sum_{j \geq 1} G_{k,j} + A_{Ed} + \sum_{i \geq 1} \psi_{2,i} Q_{k,i}$$

with the combination coefficient for the quasi-permanent value of

- imposed loads effecting floor slabs

  $\psi_{2,floor.}$ = 0,3 (for category A1 according to ÖNORM EN 1990:2003 [1])

- imposed loads effecting the roof

  $\psi_{2,roof}$ = 0,0 (for category H according to ÖNORM EN 1990:2003 [1])

- snow loads

  $\psi_{2,snow}$ = 0,0 (for locations situated lower than 1000 m a. s. l. according to ÖNORM EN 1990:2003 [1])

- wind loads

  $\psi_{2,wind}$ = 0,0 (according to ÖNORM EN 1990:2003 [1])

The combination to determine the effective modal mass (cf. the following section) according to ÖNORM EN 1998-1:2005 [12] is defined as follows:

$$\sum_{j \geq 1} G_{k,j} + \sum_{i \geq 1} \psi_{E,i} Q_{k,i}$$

with the combination coefficient for variable action i, which is used in order to determine the stress values in the context of seismic design:

$\psi_{E,i} = \varphi \cdot \psi_{2,i}$ according to ÖNORM B 1998:2006 [13]

φ_{slabs} = 1,0 for categories A to C, which results in

$$\psi_{E,slabs} = 1,0 \cdot 0,3 = 0,3$$

5 Determining the effective modal mass

The effective modal mass of the prevailing structural members of the building are listed in the following table.

structural member		unit	lower part	ceiling over ground floor	ceiling over first floor	ceiling over second floor	ceiling over third floor	ceiling over fourth floor	sum
ceiling	H	m	1,40	3,00	3,00	3,00	3,00	1,60	
	A	m²	0,00	227	227	227	227	227	
	$\psi_{E,i} \cdot q_k$	kN/m²	0,00	0,84	0,84	0,84	0,84	0,00	
	$Q_{ceiling} = \psi_{E,i} \cdot q_k \cdot A$	kg	0,00	19039	19039	19039	19039	0,00	**76154**
	g_k	kN/m²	0,00	4,15	4,15	4,15	4,15	4,52	
	$G_{ceiling} = g_k \cdot A$	kg	0,00	94060	94060	940560	94060	102100	**478639**
external walls	l_W	m	41,71	41,71	41,71	41,71	41,71	41,71	
	A_W	m²	58	125	125	125	125	67	
	g_k	kN/m²	1,26	1,26	1,19	1,12	1,12	1,12	
	$G_{walls} = g_k \cdot A_W$	kg	7372	15796	14901	14007	14007	7470	**73553**
internal walls	l_W	m	20,39	20,39	20,39	20,39	20,39	20,39	
	A_W	m²	29	61	61	61	61	33	
	g_k	kN/m²	0,97	0,97	0,89	0,82	0,82	0,82	
	$G_{walls} = g_k \cdot A_W$	kg	2756	5906	5467	5031	5031	2683	**26876**
attic	l_W	m	0,00	0,00	0,00	0,00	0,00	69,00	
	A_W	m²	0,00	0,00	0,00	0,00	0,00	41	
	g_k	kN/m²	0,00	0,00	0,00	0,00	0,00	1,12	
	$G_{walls} = g_k \cdot A_W$	kg	0,00	0,00	0,00	0,00	0,00	4634	**4634**
sum		**kg**	**10128**	**134800**	**133468**	**132136**	**132136**	**117188**	**659858**

Tab. 5.1 determining the effective modal mass of the sample building in Solid Timber Construction

Legend:

H influence height of the effective modal mass (excl. slab thickness) [m]

A floor area [m²]

$\psi_{E,i}$ combination coefficient for a variable action i, to be used when determining the effects of the design seismic action [-]

q_k variable actions [kN/m²]

g_k permanent actions [kN/m²]

l_w length of the wall [m]

A_W area of the wall (length of the wall x influencing zone) [m²]

G_W mass of the walls [kg]

6 Verification of criteria for regularity in plan and elevation

In this chapter the criteria for regularity in plan and elevation of the sample building are analysed according to ÖNORM EN 1998-1, section 4.2.3 [12]. The verification or falsification of the criteria for regularity forms the basis of the approach taken in order to determine the seismic load according to ÖNORM EN 1998-1 [12].

6. 1 Criteria for regularity in plan

6. 1. 1 Compactness of the plan configuration

According to ÖNORM EN 1998-1, section 4.2.3.2(39) [12] the plan of each floor is supposed to be delimited by a polygonal convex line (in order to avoid setbacks). Occurring setbacks are accepted, if (a) they do not interfere with the lateral stiffness of the prevailing floor slabs (can be assumed in this case), and (b) the area of each setback between the delimitation of each floor and of the convex polygon, which acts as an enclosure, does not exceed more than 5% of the floor area. The following illustration shows the plan which is valid for all floors (regardless the type of construction) and includes all the marked setbacks as well as the areas which are essential for this process of verification.

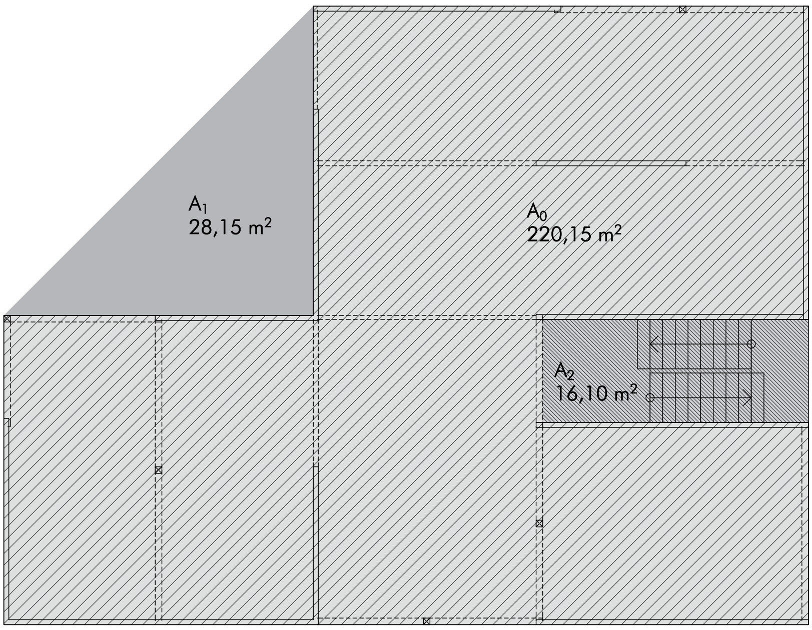

Fig. 6.1 areas of the setbacks in plan

As it becomes evident in fig. 6.1, two criteria need to be verified with regard to the compactness of the plan:

- criterion for the area ratio of the prevailing setbacks (re-entrant corner) to the net area of the floor

$$\frac{A_1}{A_0} \cdot 100 = \frac{28,15}{220,15} \cdot 100 = 12,8 > 5,0 \text{ criterion is not fulfilled}$$

- criterion for the area ratio of the prevailing setbacks (recess) to the net area of the floor

$$\frac{A_2}{A_0} \cdot 100 = \frac{16,10}{220,15} \cdot 100 = 7,3 > 5,0 \text{ criterion is not fulfilled}$$

As a consequence, it can be said that the criterion for compactness of the plan is **not fulfilled**. Hence, because of the fact that according to ÖNORM EN 1998-1, section 4.2.3.2(1)P [12] in a regular plan all criteria must be satisfied, the verification is stopped at this point and the plan is defined as **irregular**.

6. 2 Criteria for the verification of regularity in elevation

6. 2. 1 Lateral load resisting systems

All lateral load resisting systems run from their foundations to the top of the building without any interruptions.

According to ÖNORM EN 1998-1, section 4.2.3.3(2) [12] this criterion is **fulfilled**.

6. 2. 2 Avoidance of differences between the lateral stiffness and the mass of the individual storeys

The mass of the individual storeys and the lateral stiffness are reduced towards the top.

According to ÖNORM EN 1998-1, section 4.2.3.3(3) [12] this criterion is **fulfilled**.

6. 2. 3 Criterion for frame structures

According to ÖNORM EN 1998-1, section 4.2.3.3(4) [12] this criterion is **irrelevant**.

6. 2. 4 Criterion for setbacks

The sample building has no setbacks in elevation.

According to ÖNORM EN 1998-1, section 4.2.3.3(5) [12] this criterion is **fulfilled**.

To sum up, it can be said that all criteria for the constructive regularity in elevation are **satisfied**.

6. 3 Selection of the method of analysis

With regard to the previously mentioned criteria for regularity, the following table taken from ÖNORM EN 1998-1, section 4.2.3 [12] provides an overview of the relevant approaches concerning modelling and analysis.

The approach which is relevant to the determination of the seismic design discussed in this report is written in bold letters.

regularity		allowed simplifaction		behaviour factor
plan	elevation	model	method of analysis	for linear analysis
yes	no	planar	lateral force method	reference value
yes	no	planar	modal response spectrum	decreased value
no	**yes**	**spatial**	**lateral force method**	**reference value**
no	yes	spatial	modal response spectrum	decreased value

Tab. 6.1 consequences of the constructive regularity for seismic design according to [12]

In the following chapter of this report a detailed analysis of seismic design is conducted in the course of creating a spatial model by using 'RFEM'. The calculation of the first periods forms the basic input in order to be able to apply the lateral force method of analysis. The distribution of the horizontal seismic forces among the prevailing load-bearing walls of a floor is based on the stiffness of the walls and is created by using a MS EXCEL-calculation. Added to this approach, the prevailing internal forces of the walls are designed manually by conducting a modal analysis.

Chapter 5

Seismic design of the sample building in Solid Timber Construction with CLT

1 Introduction

In this chapter the suitability of the sample building in Solid Timber Construction in the context of a seismic design is analysed. This scrutiny is categorised into four steps:

- calculation of the first periods with an estimated number of connectors,
- application of the lateral force method of analysis- calculation and distribution of the horizontal seismic forces among several floors,
- calculation of the internal forces of the walls and connections as a result of the combination of action for the instantaneous seismic design situation
- shear verification of the decisive panel

2 Calculation of the first periods with a spatial member-plate-model

2. 1 General

As it was mentioned in chapter 4, the calculation of the first periods (x-direction and y-direction in plane) is performed by using a 3-D model. This model, which is designed by using the software application 'RFEM' takes all the primary seismic members into consideration. In this context it is worth mentioning that due to the limits of modelling, in the latest software version load-bearing walls are represented as one-dimensional members, while slabs are depicted as plates. In the next section a thorough definition of all the essential parameters concerning material data, section properties, supporting conditions, couplings of walls-walls and couplings of walls-slabs as well as effective masses is given.

2. 2 Determination of the essential parameters

The following illustration of wall 1x gives an insight into the various parameters (per wall panel) which need to be entered into the RFEM program.

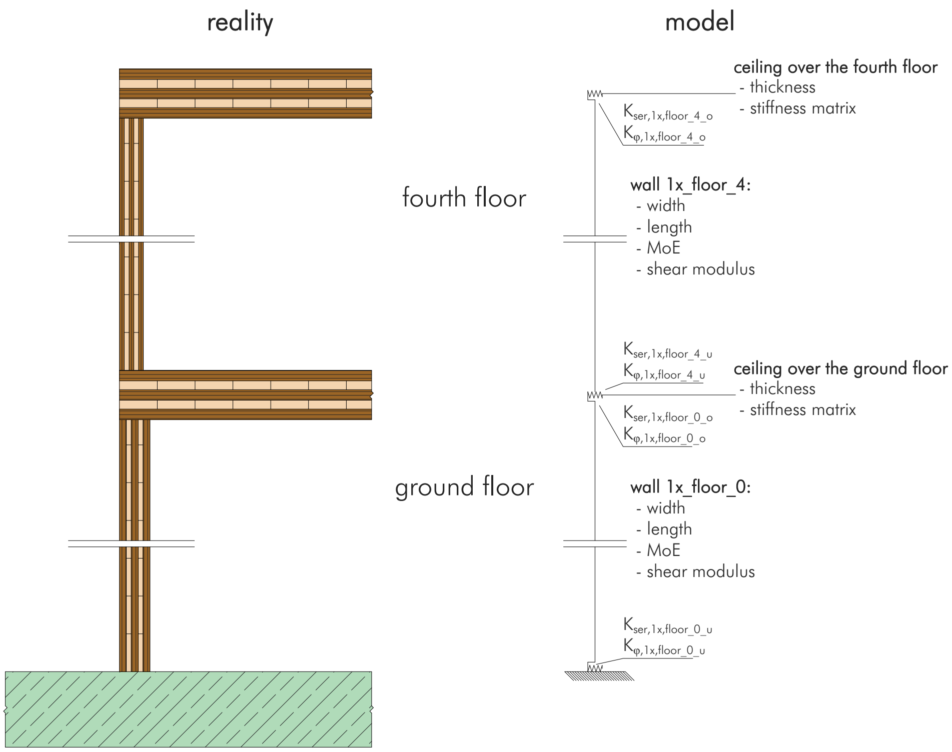

Fig. 2.1 parameters for wall 1x in the context of modelling designed by using the RFEM application

2. 2. 1 Entering the ceiling components as orthotropic panels

Added to the set of material data, which was mentioned in the chapter 4, further parameters describing the orthotropic behaviour of the laminar CLT-components are needed in order to model the structure. In order to determine the orthotropy of the used ceiling panels, the stiffness matrix, which according to the 'BSPhandbuch' [17] differs among bending performance and membrane action, needs to be entered.

- for bending performance

$$\begin{Bmatrix} m_x \\ m_y \\ m_{xy} \\ q_x \\ q_y \end{Bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & 0 \\ D_{21} & D_{22} & D_{23} & 0 & 0 \\ D_{31} & D_{32} & D_{33} & 0 & 0 \\ 0 & 0 & 0 & D_{44} & D_{45} \\ 0 & 0 & 0 & D_{54} & D_{55} \end{bmatrix} \cdot \begin{Bmatrix} \partial\varphi_y/\partial x \\ -\partial\varphi_x/\partial y \\ \partial\varphi_y/\partial y - \partial\varphi_x/\partial x \\ \partial w/\partial x + \varphi_y \\ \partial w/\partial y - \varphi_x \end{Bmatrix}$$

- for membrane action

$$\begin{Bmatrix} n_x \\ n_y \\ q_{xy} \end{Bmatrix} = \begin{bmatrix} d_{11} & d_{12} & d_{13} \\ d_{21} & d_{22} & d_{23} \\ d_{31} & d_{32} & d_{33} \end{bmatrix} \cdot \begin{Bmatrix} \partial u/\partial x \\ \partial v/\partial y \\ \partial u/\partial y - \partial v/\partial x \end{Bmatrix}$$

The prevailing coefficients D_{11} to D_{55} and d_{11} to d_{33} are defined as follows:

$$D_{11} = E_x \cdot J_{x,\,eff}$$

$$D_{22} = E_y \cdot J_{y,\,eff}$$

$$D_{12} = D_{21} = 0$$

$$D_{33} = \frac{\Phi}{100} \cdot \frac{G_{0,\,mean} \cdot h_{ges}^3}{12}$$

$$D_{44} = \frac{(G_{0,\,mean} \cdot h_x + G_{90,\,mean} \cdot h_y)}{\kappa}$$

$$D_{55} = \frac{(G_{90,\,mean} \cdot h_x + G_{0,\,mean} \cdot h_y)}{\kappa}$$

$$D_{45} = D_{54} = 0$$

$$d_{11} = E_x \cdot h_x$$

$$d_{22} = E_y \cdot h_y$$

$$d_{33} = h_{ges} \cdot \left[\frac{G_{0,\,mean}}{1 + 6 \cdot \left(0,32 \cdot \left(\frac{t}{a}\right)^{-0,77}\right) \cdot \left(\frac{t}{a}\right)^{2}} \right]$$

$$d_{12} = d_{21} = d_{13} = d_{31} = d_{23} = d_{32} = 0$$

with

E_x,E_y as modulus of elasticity, equals $E_{0,mean}$ according to chapter 4, section 3 [kN/m²]

$G_{0,mean}$ as shear modulus of the component according to chapter 4, section 3 [kN/m²]

h_{ges} as total thickness of the component [m]

h_x as thickness of all board layers as long as their direction of grain runs along the component (x-direction) [m]

h_y as thickness of all board layers as long as their direction of grain runs straight through the component (y-direction) [m]

κ as shear correction factor according to the ’BSPhandbuch’ [17], for 5 layered elements: $\kappa = 4{,}12$ [-]

t as average single layer thickness of the component [m]

a as width of the used boards, here a value of $a = 15$ cm is used

Φ as factor to consider a reduced torsional stiffness [-]

The following table shows all the determined coefficients of the stiffness matrix for the CLT element used for all ceilings.

- CLT element, h_{total} = 196 mm, 5 layers, „Stora Enso 196 L5s"

stiffness matrix for bending performance			
	[kNm]		[kNm]
D_{11}	5682	D_{22}	1220
D_{12}	0	D_{33}	173
	[kN/m]		[kN/m]
D_{44}	22274	D_{55}	13833
stiffness matrix for membrane action			
	[kN/m]		[kN/m]
d_{11}	1386000	d_{22}	770000
d_{12}	0	d_{33}	98823

Tab. 2.1 coefficients of the stiffness matrix of the used element

2. 2. 2 Entering the panels as bending members

In order to be able to enter the panels as vertical bending members it is necessary to enter the system parameters wall length, wall width and wall height as well as the material data modulus of elasticity and shear modulus into the FE application. Due to the fact that the panels are implemented as one-dimensional members, a reduction of the effective shear modulus is made according to the 'BSPhandbuch' [17]. Additionally, the modulus of elasticity, as it is defined in chapter 4, section 3, is taken into consideration (here only for the 5 layered CLT wall panel with a width of 95 mm).

$$G = \frac{G_{0,\,mean}}{1 + 6 \cdot \alpha_T \cdot \left(\frac{t_{mean}}{a}\right)^2} = \frac{690}{1 + 6 \cdot 1,57 \cdot \left(\frac{19}{150}\right)^2} = 599,4\ \text{N/mm}^2$$

with

$$\alpha_T = 0,32 \cdot \left(\frac{t_{mean}}{a}\right)^{-0,77} = 0,32 \cdot \left(\frac{19}{150}\right)^{-0,77} = 1,57$$

t_{clt} total thickness of the panel [mm]

t_{mean} average layer thickness of the panel [mm]

a board width [mm]

$G_{0,mean}$ shear modulus of the used boards (C24 according to EN 338, cf. chapter 4, section 3)

The previously mentioned system parameters and the distance between the gravity centre of the panels and the left position at the bottom of the plane (in the following sections defined as coordinate origin) are listed in the following table. In order to simplify matters, the positions of the gravity centres of the panels stay always at the same position, although the width of all the walls of the sample building is reduced from 121 mm to 95 mm.

wall i	width b_i	length l_i	height h_i	horizontal distance x_i	vertical distance y_i
[-]	[m]	[m]	[m]	[m]	[m]
in x-direction					
wall 1x	0,121 (0,095)	7,379	3,00	3,811	0,061
wall 2x	0,121 (0,095)	3,830	3,00	5,585	7,440
wall 3x	0,121 (0,095)	6,000	3,00	10,500	14,940
wall 4x	0,121 (0,095)	3,633	3,00	14,716	11,120
wall 5x	0,121 (0,095)	6,600	3,00	16,200	7,466
wall 6x	0,121 (0,095)	6,600	3,00	16,200	4,844
wall 7x	0,121 (0,095)	6,600	3,00	16,200	0,061
in y-direction					
wall 1y	0,121 (0,095)	5,000	3,00	0,061	2,500
wall 2y	0,121 (0,095)	5,121	3,00	7,561	9,940
wall 3y	0,121 (0,095)	7,474	3,00	19,440	11,263
wall 4y	0,121 (0,095)	3,830	3,00	7,561	1,915

Tab. 2.2 system parameters of the panels for all floors (the numbers in brackets define the width of all walls from the second to the fourth floor)

2. 2. 3 Entering the joint parameters

Added to the cross-sectional dimensions (length, width, height, thickness of the ceiling) and the material data (modulus of elasticity, shear modulus) the joints between the walls and the foundation (wall-foundation-joint) and the joints between walls and the slabs (wall-slab-joint, or rather wall-slab-wall-joint) play a crucial part in determining the first periods using the FEM program.
The mass of the building and the (lateral) stiffness of the load-bearing structure highly influence the dynamic characteristics of the building. Again in order to simplify matters, it can be said that the load-bearing structure of light-weight buildings is defined by a considerably high degree of deformation. Consequently, it is believed that these buildings share very high first periods and, thus, provide rather small seismic actions.

For the sample building in Solid Timber Construction the previously mentioned high degree of deformation is reached by using connectors in order to seal the joints. As it can be seen in the following section, lateral bending performance and shear strain of the panels have only a small impact on the total deformation of the load-bearing wall. There-

fore, in the context of developing the structural design, it is of crucial importance to arrange the connectors in a way which correlates with the frame structure. However, since the level of strain highly depends on the number, form and position of the connectors, it is simply impossible to be aware of the final number and position of the connectors at the beginning of the calculation.
As a consequence, an iterative calculation, which is influenced by further edge conditions (capacity design), concerning the amount of connectors is absolutely necessary. In order to clarify matters the following flow chart provides an insight into the process.

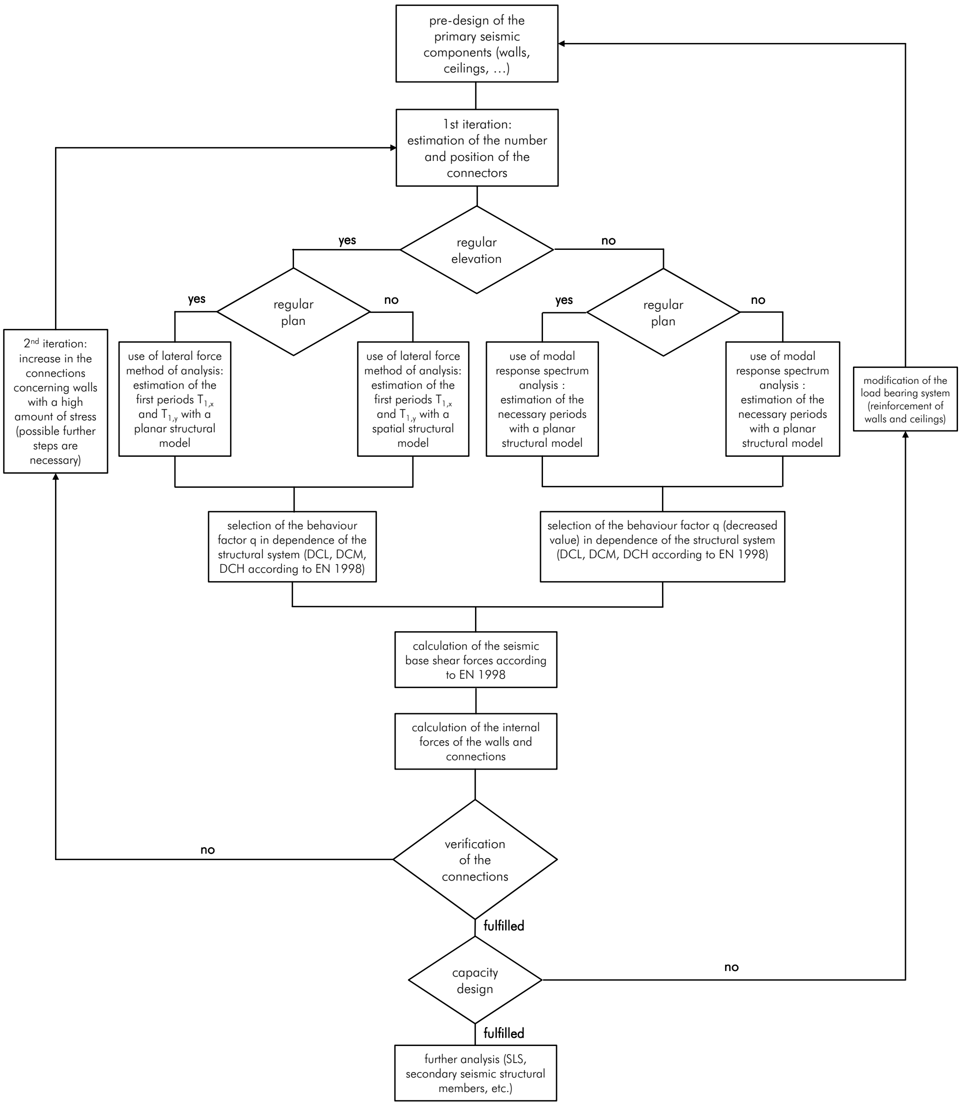

Fig. 2.2 flow chart to determine the first periods and conception of connectors

note: With regard to verification in fig. 2.2, a local capacity design is used. The idea is to analyse whether in the context of a dissipative structural resistance the collapse of a building is based on a ductile failure of the load-bearing structure. This calculation is done with the intention of preventing a collapse of brittle components. If the calculation of local capacities was unsuccessful, it would be highly advisable to significantly modify the load-bearing structure.

- Selection of the connectors

In order to transfer internal bending moments and shear forces along the wall axis, hold-downs and angle brackets are used in the context of Solid Timber Construction. In this model it is believed that the used hold-downs are exclusively able to transfer tensile forces (uplift), while the used sliding brackets are only able to transfer shear force (longitudinal force along the axis of wall). Based on the results of topical experiments conducted at the Institute for Timber Engineering and Wood Technology of Graz University of Technology [20] the following two types of sliding brackets are used:

connection joint	angle bracket	connection with timber	bearing capacity of shear $R_{xz,d}$	shear stiffness K_{ser} *
			[kN]	[MN/m]
wall-foundation	AE116 built-in at both sides	CNA ring-shank nails 4,0x60 mm	29,2 kN	5,80
wall-ceiling-wall	ABR90 built-in at both sides	CNA ring-shank nails 4,0x60 mm	11,8 kN	2,00
* results of a current experiment conducted at the Instiute for Timber Engineering and Wood Technology, Graz University of Technology				

Tab. 2.3 used angle brackets to transfer shear forces along the wall axis

The following illustrations show the used angle brackets.

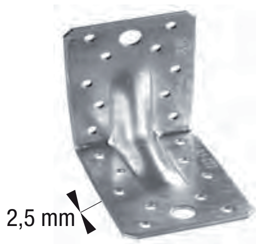

Fig. 2.3 angle brackets AE116 and ABR90 used in the context of verification, taken from [27]

While the sets of data concerning the stiffness of the used angle brackets are already known, the used hold downs HD 480-M20 [27] still need to be analysed. Based on the information taken from [27] a maximal anchor force of

$$R_{1,d} = 31,9 \text{ kN}$$

is estimated.

The calculation of the stiffness modulus K_{anchor} is done as follows:

$$K_{anchor} = \frac{1}{\frac{1}{K_{ser,1}} + \frac{1}{K_{ser,2}}}.$$

with

$K_{ser,1}$ as stiffness modulus of the nailed connection of the hold downs

$K_{ser,2}$ as stiffness modulus of the washer as anchor bold

The following illustration shows a hold down and the stiffness modulus of a wall-ceiling-wall connection joint.

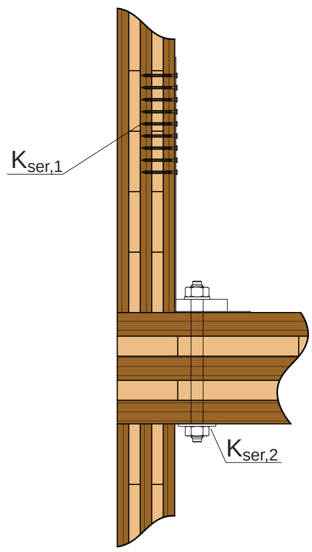

Fig. 2.4 wall-ceiling-wall connection joint with the hold down '480-M20' and the prevailing stiffness modulus

As it becomes evident in fig. 2.4, added to the already defined tension springs 'nail setup' and 'steel plate' further parts of the connection system 'hold down', such as the T-square, the fixing plate and the steel pin, are deformed by tensile stress. The degree of these deformations is relatively insignificant and, therefore, will not be considered any further.

The stiffness modulus of the nailed connection under tensile stress is determined as follows

$$K_{ser,\,1} = n_{nail} \cdot K_{ser,\,nail} = 15 \cdot \frac{1}{25} \cdot \rho_k^{1,5} \cdot d^{0,8} = .$$

$$15 \cdot \frac{1}{25} \cdot 350^{1,5} \cdot 4,0^{0,8} = 11909,71\ N/mm = 11,91\ MN/m$$

The stiffness modulus of the washer is defined by the flexibility of the CLT panel under compression perp. to grain. In this special context the applied formula is taken from the master thesis 'Historische Dachstühle' [19]:

$$K_{ser,\,2} = \frac{E_{90,\,mean} \cdot A_{90}}{H/2}$$

with

$E_{90,mean}$	as modulus of elasticity of the CLT element perp. to grain
A_{90}	as area under compression stress perp. to grain (correlates with the dimensions of the washer 180/70 mm)
H	as the thickness of the CLT panel

results in

$$K_{ser,\,2} = \frac{370 \cdot 0,18 \cdot 0,07}{0,196/2} = 47,57\ MN/m$$

Consequently, the stiffness modulus of a hold down is:

$$K_{anchor} = \frac{1}{\frac{1}{11,91} + \frac{1}{47,57}} = 9,53\ MN/m$$

This implies that when using it in the connection joint 'wall-foundation' the flexibility of the washer is eliminated and, hence, the stiffness of the anchor amounts to

$$K_{anchor,\,GF} = K_{ser,\,1} = 11,91\ MN/m$$

note: Based on [27] the determination of the anchor force and anchor stiffness is restricted to a singular hold down. As a consequence, in order to make use of both sides of the wall panel, the data needs to be duplicated (dual system).

- Estimation of number and position of connectors per connection joint

In this 1st iteration the estimation of number and position of the angle brackets and the hold downs per connection joint takes place. In the context of placing the angle brackets, 1 item per meter load-bearing wall (1 item = 2 angle joints due to making use of both sides) is used. Each hold down is situated at the beginning and the end of the wall.

- Determination of shear and rotation stiffness of the connection joints

The following illustration shows the step-by-step modelling on the basis of load-bearing wall 1x beginning with the previously described organisation of connections in combination with connection joints and ending with the model column to be entered into the FEM-application.

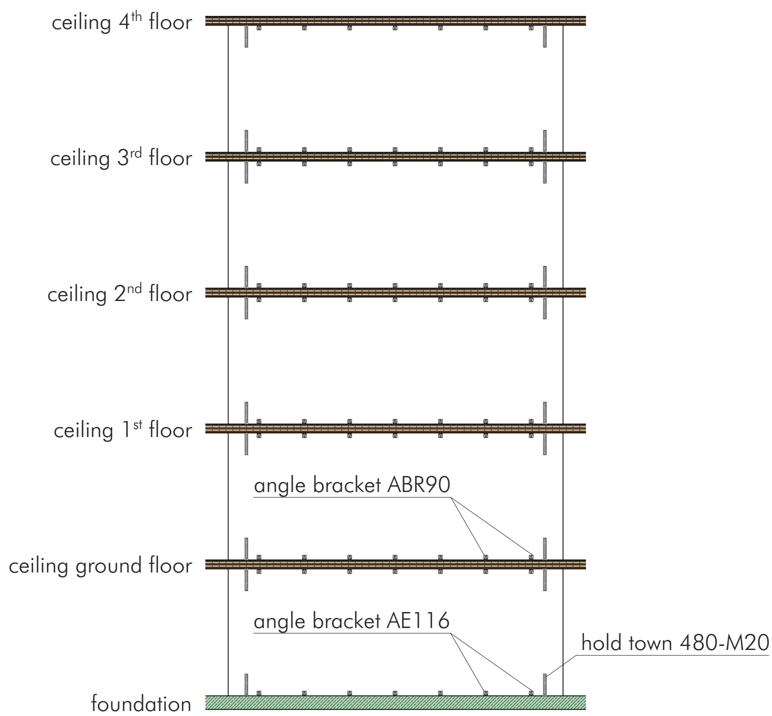

Fig. 2.5 organisation of connectors in the connection joints within the prevailing floors

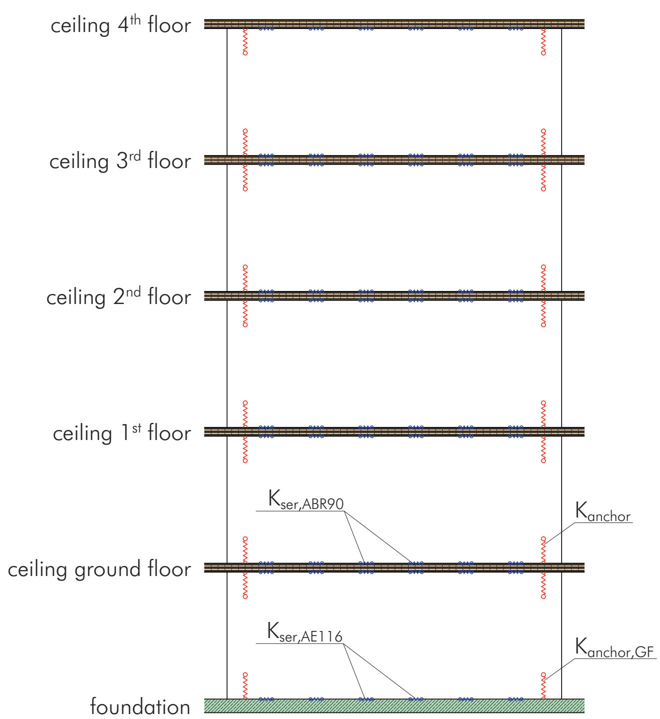

Fig. 2.6 organisation of connectors as a sequence of translational springs

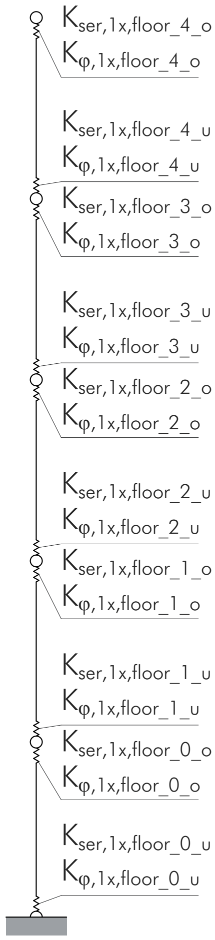

Fig. 2.7 summary of translational springs concerning rotational- and translational springs per connection joint

In order to determine a global shear stiffness (e.g. $K_{ser,GF,u}$) all the springs of the prevailing angle brackets need to be summarised. The shear springs of each connection work in a parallel system. Consequently, the shear springs of the prevailing connection joints can be determined using the following equation.

connection joint 'wall-foundation':

$$K_{ser,\,floor\_0\_u} = n \cdot K_{ser,\,AE116}$$

connection joint 'wall-ceiling', or rather 'ceiling-wall':

$$K_{ser,\,floor\_1\_u} = n \cdot K_{ser,\,ABR90}$$

with

n as number of angle brackets per connection joint

The following table includes the total shear stiffness of the connection joints of the prevailing wall-foundation connections and the wall-ceilings connections concerning the load-bearing walls 1x to 4y. Due to the fact that with regard to this 1st iteration the number of connectors per connection joint and floor remains the same, the shear stiffness of the connection 'joint wall-ceiling' is identical among all joints in the top floors.

wall i	length l_i	number n_i	$k_{ser,floor\_0\_u}$	$k_{ser,floor\_1-4,i}$
	[m]	[-]	[kN/m]	[kN/m]
in x-direction				
wall 1x	7,38	8	46400	16000
wall 2x	3,83	4	23200	8000
wall 3x	6,00	6	34800	12000
wall 4x	3,63	4	23200	8000
wall 5x	6,60	7	40600	14000
wall 6x	6,60	7	40600	14000
wall 7x	6,60	7	40600	14000
in y-direction				
wall 1y	5,00	5	29000	10000
wall 2y	5,12	6	34800	12000
wall 3y	7,47	8	46400	16000
wall 4y	3,83	4	23200	8000

Tab. 2.4 shear stiffness of the connection joints 'wall-foundation' and 'wall-ceiling'

In this context it is worth mentioning that determining the rotation stiffness $K_{\varphi,i}$ is more complex than calculating the shear stiffness. As it becomes evident in the following illustration, the rotation of a load-bearing wall is the result of an bending moment perpendicularly affecting the axis of wall by combining the wall curvature with the flexibility of the anchor (hold down) and the compressive deformation of the contact joint 'timber-concrete', or rather 'timber-timber'.

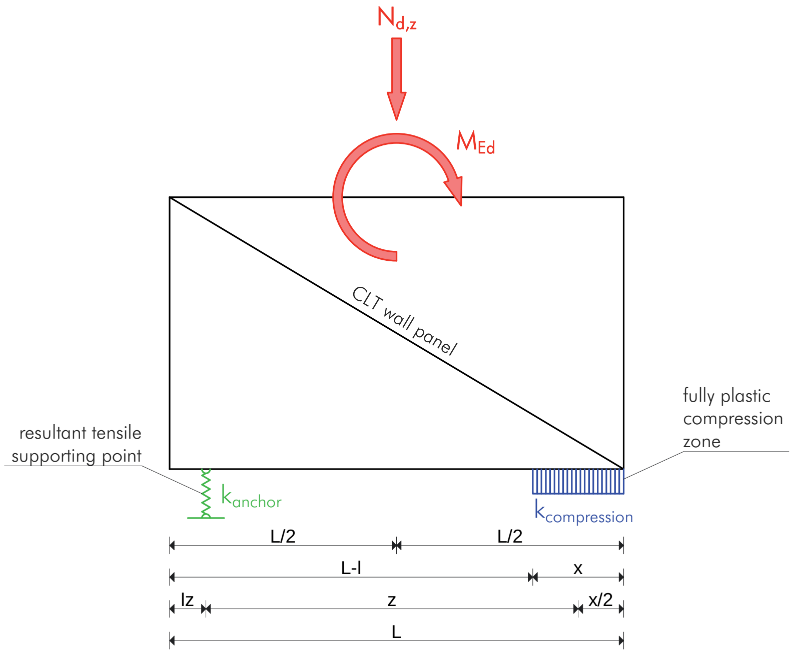

Fig. 2.8 model of a load-bearing wall being exposed to bending- and longitudinal force

While the wall curvature can be determined by cross-sectional characteristics of the model column, the prevailing flexibility of tension- and pressure springs is summarised in a rotation spring. By using a pair of hold downs at both ends of the wall, as it was explained in the 1st iteration, the tension spring stiffness results in the spring stiffness of the hold downs. The compressive spring stiffness, however, depends on the length of fully plastic compression zone x and can be estimated by being aware of the number of hold downs and the longitudinal force of the equilibrium system. As a consequence, the determination of the rotation spring is demonstrated step by step.

- 1st step
  Calculating the longitudinal force of all load-bearing walls and connection joints on the basis of the combination of actions according to
  ÖNORM EN 1998-1 :2005 [12]

This combination of actions has already been used in the context of determining the effective modal mass (cf. chapter 4) and is calculated as follows:

$$\sum_{j \geq 1} G_{k,j} + \sum_{i \geq 1} \psi_{E,i} Q_{k,i}$$

As a consequence, the determination of the longitudinal force is based on the dead weight of the load-bearing structure and the quasi permanent amount of imposed loads.

action	size [kN/m²]	permanent	variable
total load flat roof $g_{1,k}$	4,52	x	
total load floor slab $g_{2,k}$	4,15	x	
total load external wall $g_{3,k}$	1,26 (1,12)	x	
total load internal wall $g_{4,k}$	0,97 (0,82)	x	
imposed load incl. weight of the movable partitions q_k	0,84		x
note: the numbers in bracktes are valid for walls with 95 mm in thickness used in the top floors			

Tab. 2.5 action on the walls according to ÖNORM EN 1998-1:2005 [12]

The following table provides an insight into the determined longitudinal force $N_{d,z,i}$ of all load-bearing walls. In this context it is worth mentioning that the influence coefficient e of the wall, which has already been determined in chapter 3, is used again in order to calculate the distribution of the forces acting on the storey.

W	e	4th floor over	4th floor under	3rd floor over	3rd floor under	2nd floor over	2nd floor under	1st floor over	1st floor under	ground floor over	ground floor under
	[-]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]
x-direction											
1x	1,07	35,7	60,5	99,9	124,6	164,0	188,8	228,2	256,2	295,6	323,5
2x	3,20	55,4	68,2	129,4	142,3	203,4	216,3	277,4	291,9	353,1	367,6
3x	1,96	53,1	73,3	132,0	152,1	210,8	230,9	289,6	312,4	371,0	393,8
4x	9,20	151,0	160,0	326,7	335,7	502,4	511,4	678,2	668,7	855,4	866,0
5x	2,71	80,8	97,1	186,4	202,6	291,9	308,2	397,4	416,5	505,8	524,9
6x	3,98	118,7	135,0	266,0	282,3	413,4	429,7	560,8	579,9	711,0	730,1
7x	2,21	65,9	88,1	160,9	183,0	255,8	278,0	350,7	375,7	448,5	473,5
y-direction											
1y	1,92	43,4	60,2	108,1	124,9	172,8	189,6	237,5	256,4	304,3	323,2
2y	3,79	87,7	104,9	201,7	218,9	315,8	333,0	429,8	449,2	546,1	565,5
3y	1,41	47,6	72,7	125,3	150,4	203,0	228,1	280,7	309,0	361,6	389,9
4y	6,62	114,6	124,0	250,5	260,0	386,5	395,9	522,5	533,6	660,1	671,2

Tab. 2.6 longitudinal force of the connection joints of the walls according to ÖNORM EN 1998-1:2004 [12]

note: It is apparent that determining the seismic design of a building implies a significant amount of time necessary for computing. With regard to these calculations, spreadsheet programs, such as MS EXCEL, are extremely helpful.

- 2^{nd} step
  Estimation of the existent length of compression zone x based on the condition $\sum N = 0$

When examining the condition of equilibrium from a vertical perspective, it becomes evident that the existent compression force can be determined by making use of the total supporting capacity of n anchors

$$N_{c,d} = n \cdot R_{1,d} + N_{z,d}$$

The size of the necessary compression zone $x \cdot b_{eff}$ is estimated by transforming the verification of compression in grain direction (wall-foundation), or rather compression perp. to grain (wall-ceiling, ceiling-wall).

wall-foundation:

verification of compression in grain direction

$$\frac{\frac{N_{c,d}}{b_{eff} \cdot x}}{f_{c,0,d}} \leq 1,0$$

with

b_{eff} as effective width of the wall without considering the transverse layers. With regard to the wall component 121 5Ls, b_{eff} results up to
$b_{eff} = b - 2 \cdot 19 = 121 - 38 = 83\ \text{mm}$

resulting in

$$x = \frac{N_{c,d}}{b_{eff} \cdot f_{c,0,d}} = \frac{n \cdot R_{1,d} + N_{z,d}}{b_{eff} \cdot f_{c,0,d}}$$

wall-ceiling-wall:

verification of compression perp. to grain

$$\frac{\frac{N_{c,d}}{b_{eff} \cdot x}}{f_{c,90,d} \cdot k_{c,90}} \leq 1,0$$

with

b_{eff} as the width of the wall [mm]

$k_{c,90}$ as strength increasing lateral compressive coefficient, which in this context is regarded as 2,0. Due to this instantaneous seismic design situation, significant deformations are accepted [-].

resutling in

$$x = \frac{N_{c,d}}{b_{eff} \cdot k_{c,90} \cdot f_{c,90,d}} = \frac{n \cdot R_{1,d} + N_{z,d}}{b_{eff} \cdot k_{c,90} \cdot f_{c,90,d}}$$

- 3^{rd} step
  Determination of the compression spring $K_{compression}$

Depending on the connection joint the prevailing compression spring can be determined as follows

wall-ceiling:

$$K_{compression} = \frac{E_{0,mean} \cdot A_0(x)}{H_W}$$

with

A_0 as area of the fully plastic compression zone $x \cdot b_{eff}$ under compression parallel to grain [mm²]

$E_{0,mean}$ as MoE of the timber parallel to grain direction [N/mm²]

H_W as height of the load-bearing wall over the connection joint [mm]

wall-ceiling-wall:

$$K_{compression} = \frac{E_{90,mean} \cdot A_{90}(x)}{H/2}$$

with

A_{90} as area of the fully plastic compression zone $x \cdot b_{eff}$ under compression perp. to grain [mm²]

H as thickness of the CLT-slab component [mm], in this context $H = 196$ mm

- 4^{th} step
  Determination of the dimension of the internal moment arm z

According to fig. 2.8 the internal moment arm z as distance between the resultant pressure and tensile components is determined as follows:

$$z = L - l_z - \frac{x}{2},$$

with

l_z as distance between the hold town, or rather the resultant anchor force of n hold towns and the end of the wall. This distance is estimated to be 0,40 m for all connections [m].

- 5^{th} step
  Determination of the rotational spring / rotation stiffness of the connection joint

Estimation of the rotation stiffness of the connection is done by combining the already determined translational springs using the following equation:

$$K_{\varphi, i} = \frac{z_i^2}{\frac{1}{K_{anchor, i}} + \frac{1}{K_{compression, i}}}$$

The following table shows a summary of all rotational springs of the prevailing connection joints of the load-bearing walls 1x-4y.

W	4^{th} floor over	4^{th} floor under	3^{rd} floor over	3^{rd} floor under	2^{nd} floor over	2^{nd} floor under	1^{st} floor over	1^{st} floor under	ground floor over	ground floor under
	[kNm/rad]	[kNm/rad]	[kNm/rad]	[kNm/rad]	[kNm/rad]	[kNm/rad]	[kNm/rad]	[kNm/rad]	[kNm/rad]	[kNm/rad]
x-direction										
1x	704000	731000	757000	767000	776000	779000	793000	794000	794000	812000
2x	170000	172000	176000	176000	174000	174000	177000	176000	173000	194000
3x	463000	473000	489000	491000	493000	493000	502000	501000	497000	539000
4x	155000	155000	146000	146000	133000	132000	131000	130000	119000	183000
5x	586000	593000	609000	610000	608000	606000	614000	613000	604000	694000
6x	560000	603000	609000	608000	596000	593000	598000	596000	579000	720000
7x	578000	509000	607000	609000	609000	608000	618000	616000	610000	684000
y-direction										
1y	305000	312000	323000	324000	326000	326000	333000	332000	330000	347000
2y	337000	340000	345000	344000	338000	337000	341000	339000	330000	399000
3y	739000	762000	788000	795000	802000	803000	817000	817000	814000	866000
4y	175000	174000	172000	171000	162000	162000	162000	161000	153000	207000

Tab. 2.7 rotation stiffness of the connection joints of all load-bearing walls

This table gives an insight into all connection parameters which are relevant in the context of determining the first periods using the FEM-application. Entering these parameters into the program is done by defining the member releases in order to be able combine the prevailing model columns with translational- and rotational springs.

Based on the assumptions and calculations it is evident that the focus of interest is on the modelling (a) of the stiffness of translational springs in direction to the wall axis and (b) of the stiffness of rotational springs perpendicularly to the wall axis. Stiffness against displacement perpendicularly to the wall axis, or rather against rotation around the 'weak axis' of the wall, is set to zero when being entered into the program. In this context it needs to be mentioned that this assumption has to bee seen as a simplification, but the predominant amount of stress is placed on the wall axis, or rather on the 'strong axis'. Due to the fact that the z-axis of the wall is confronted with a very low degree of local torsional effects, the level of torsional stiffness of the wall connection is of secondary importance. Based on a conservative interpretation, the torsional influence is extended to infinity.

2. 2. 4 Estimation of the mass of the building

The estimation of the mass of the building is done by supplementing the CLT slab panels with additional masses in kg/m². These loads are created by dividing the prevailing total floor masses by the relevant floor areas (cf. chapter 4, tab. 5.1)

ceiling over	total mass	ceiling area	additional mass
	[kg]	[m²]	[kg/m²]
ground floor	134800	227	595
1st floor	133468	227	589
2nd floor	132136	227	583
3rd floor	132136	227	583
4th floor	117188	227	517

Tab. 2.8 additional masses of the prevailing ceiling areas

2. 3 First periods of the 1st iteration

After having entered all parameters into the FEM-application, as described in section 2. 2, the first periods in x- and y-directions can be determined by using the module 'RF-DYNAM (basic)'. Under these conditions the periods are defined as

$$T_{1,x} = 1,81 \text{ s}$$

and

$$T_{1,y} = 2,50 \text{ s}.$$

note: According to ÖNORM EN 1998-1:2004, section 4.3.3.2.1 (2)a [12] both first periods exceed the limit value. Therefore, it needs to be mentioned that the simplified response spectrum method might miscalculate the seismic load and, consequently, should not be applied. However, this approach is still used due to two aspects: First, it is the case that buildings in timber construction generally share high first periods (cf. 'Erdbebengerechte mehrgeschossige Holzbauten' [23]). Second, the calculation using the modal response spectrum method would be based exclusively on computing and, consequently, would no longer be comprehensible. Nevertheless, the actions and the internal forces of the prevailing load-bearing walls are compared with the results of a computerised modal response spectrum calculation later on.
Added to this, in the context of this iteration it might be possible that the stiffness of the building raises in the course of an increasing number of connectors within the connection joints. As a consequence, a new determination of the first periods would fulfill this boundary condition.

The following illustration shows a graphic of the 3-D model designed by using the RFEM application.

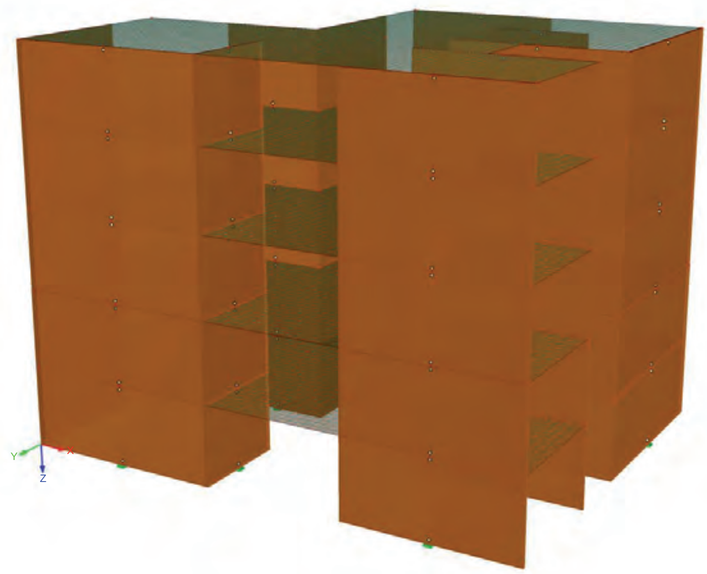

Fig. 2.9 3-D model of the load-bearing structure in RFEM

3 Calculation of the seismic base shear forces

3.1 Defining the parameters of the design response spectrum for the fictitious location

In order to be able to calculate the sample building, the soil class A is chosen. According to ÖNORM EN 1998-1:2005, section 3.2.2.2, table 3.2 [12], or rather ÖNORM B 1998-1:2006, section 4.3.4(2) [13], the following parameters can be defined for this building site in the context of this seismic design response spectrum (type 1):

$S = 1{,}00$

$T_B = 0{,}15$ s

$T_C = 0{,}40$ s

$T_D = 2{,}00$ s

3.2 Determining the behaviour factor q

According to the latest level of research (cf. 'New Technologies for Construction of Medium-Rise Buildings in Seismic Regions: The XLAM Case' [23]) for this type of timber structure (walls and slabs consist of CLT elements, usage of mechanical connectors, anchorage by using hold downs and angle brackets with nail setup, etc.) a **behaviour factor q of 3,0** is used. This parameter corresponds with a high capacity of energy dissipation and with the ductility class DCH according to ÖNORM EN 1998-1:2005 [12].

3.3 Graphic of the design response spectrum for linear analysis

The seismic design response spectrum for determining the seismic base shear force F_b is the result of the previously defined parameters in combination with the equations taken from ÖNORM EN 1998-1:2005, section 3.2.2.5(4) [12]:

$$0 \le T \le 0,15s \rightarrow S_d(T) = 3,34 \cdot 1,0 \cdot \left[\frac{2}{3} + \frac{T}{0,15} \cdot 0,167\right] \quad (1)$$

$$0,15s \le T \le 0,40s \rightarrow S_d(T) = 2,78 \quad (2)$$

$$0,40s \le T \le 2,0s \rightarrow S_d(T) = 3,34 \cdot 1,0 \cdot 0,83 \cdot \frac{0,40}{T} \quad (3)$$

$$2,0s \leq T \rightarrow S_d(T) = 3,34 \cdot 1,0 \cdot 0,83 \cdot \frac{0,8}{T^2} \quad (4)$$

for (3) and (4) it can be said:

$$S_d(T) \geq 0,2 \cdot 3,34 = 0,67 \quad m/s^2$$

The following illustration shows a graphic of the seismic design for the selected soil class and the existent ground acceleration.

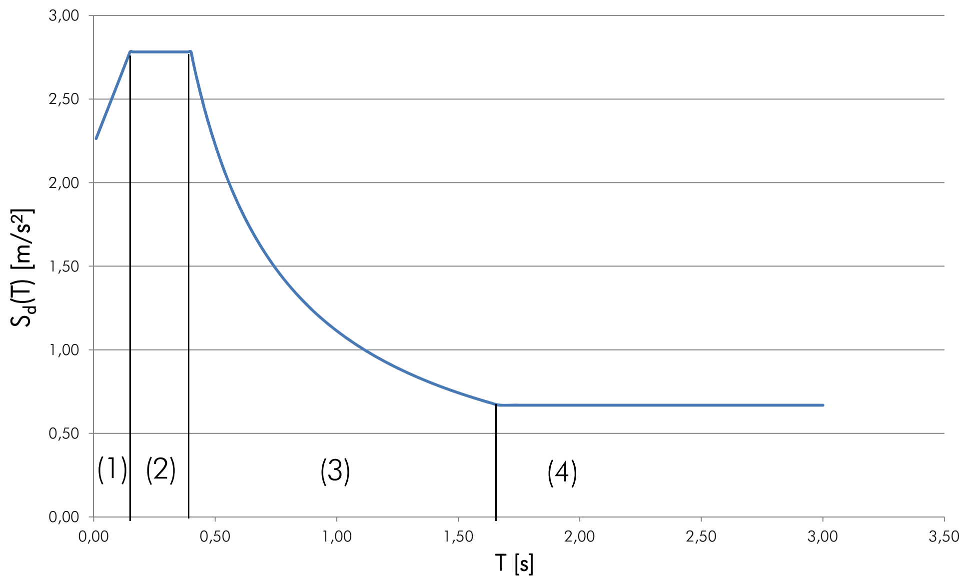

Fig. 3.1 design response spectrum for linear analysis (soil class A, a_g = 3,34 m/s²)

3. 4 Resultant seismic base shear force

According to ÖNORM EN 1998-1:2005, section 4.3.3.2.2 [12] the horizontal seismic base shear force F_b, which needs to be applied in the context of the lateral force method of analysis, is determined for both directions as follows:

$$F_b = S_d(T_1) \cdot m \cdot \lambda$$

with

$S_d(T_1)$ as ordinate of the design response spectrum with the first period T_1 [m/s²]

T_1 as fundamental period of vibration of the building [s]

m as aboveground total mass of the building [t], (according to chapter 4, tab. 5.1 it results in 660 t)

λ as correction factor [-], which depends on T_1. It can be said that

$\lambda = 0{,}85$ if $T_1 \leq 2 \cdot T_C$ and if the building has more than two floors

otherwise $\lambda = 1{,}00$.

Under these conditions the result is a seismic base shear force in x- and y-directions of

$$F_{b,x} = 0,668 \cdot 660000 \cdot 1,00 = 440880 \text{ N} = 441 \text{ kN}$$

and

$$F_{b,y} = 0,668 \cdot 660000 \cdot 1,00 = 440880 \text{ N} = 441 \text{ kN}$$

3. 5 Distribution of the seismic base shear force among the floors

The distribution of the seismic base shear force $F_{b,i}$ among each floor is based on ÖNORM EN 1998-1:2005, section 4.3.3.2.3(3) [12]. The point of application of the horizontal forces acting on the storey can be detected in the mass centre of the CLT slab panels of the prevailing floor slab. Consequently, it can be said

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_i \cdot m_i} .$$

The following table provides an insight into all results in the context of the distribution of the seismic base shear force.

floor	z_i	m_i	$z_i.m_i$	$F_{i,x}$	$F_{i,y}$
	[m]	[t]	[m.t]	[kN]	[kN]
ground floor	3,0	135	405	31	31
1st floor	6,0	133	798	62	62
2nd floor	9,0	132	1188	91	91
3rd floor	12,0	132	1584	122	122
4th floor	15,0	117	1755	135	135
sum				441	441

Tab. 3.1 resultant horizontal forces acting on the storey determined by using the lateral force method of analysis

4 Determination of the internal forces of the walls in the context of the instantaneous load case 'earthquake'

4. 1 Calculation of the coordinates of the centre point of stiffness in plan

With regard to dividing the seismic actions among the prevailing primary seismic structural members of each floor, it is of utmost importance to calculate the coordinates of the centre point of stiffness in plan. The coordinates are determined as follows:

$$x_s = \frac{\sum K_{y,i} \cdot x_i}{\sum K_{y,i}}$$

and

$$y_s = \frac{\sum K_{x,i} \cdot y_i}{\sum K_{x,i}}$$

with

$K_{x,i}$, $K_{y,i}$ as horizontal total stiffness of the load-bearing wall i in x- and y-directions [kN/m]

x_i, y_i as x- and y-distances between the mass centre of the load-bearing wall i and the coordinate origin [m]

While the distances of the mass centres of the walls can be already found in tab. 2.2, a calculation concerning deformation capacity following the principle of the virtual displacement is needed in order to determine the total stiffness of a load-bearing wall. This calculation is explained in the following section.

4. 1. 1 Determination of the total horizontal stiffness of the load-bearing walls

The total horizontal stiffness of a load-bearing wall is determined by loading the wall as model column with a horizontal unit load (cf. section 2. 2 for cross section and connection stiffness). In the course of loading, the total force acts at the height h' (cf. ÖNORM B 1998-1, appendix B [13]). The resulting stiffness is acquired by dividing the unit load by the already determined horizontal deformation of the top level of the wall. The height h' as point of application of the equivalent mass is determined as follows:

$$h' = \frac{\sum (m_i \cdot z_i^2)}{\sum (m_i \cdot z_i)}$$

with

i as the index for the number of floors

ceiling over	z_i	m_i	$z_i.m_i$	$z_i^2.m_i$
	[m]	[t]	[tm]	[tm²]
ground floor	3	135	405	1215
1st floor	6	133	798	4788
2nd floor	9	132	1188	10692
3rd floor	12	132	1584	19008
4th floor	15	117	1755	26325
sum			5730	62028

Tab. 4.1 parameters to determine h'

Consequently, h' results in

$$h' = \frac{62028}{5730} = 10,83 \quad m\,.$$

The load-bearing walls are entered as model columns into the software for designing framed structures and by taking into consideration all parameters deflected with a '1'- load. The resultant dimensions regarding the horizontal stiffness of walls are listed in the following table.

wall i	$K_{i,x}$	$K_{i,y}$	x_i	y_i	$K_{y,i} \cdot x_i$	$K_{x,i} \cdot y_i$
	[kN/m]	[kN/m]	[m]	[m]	[kN]	[kN]
1x	1536	0	3,81	0,06	0	92,91
2x	363	0	5,59	7,44	0	2699,62
3x	1000	0	10,5	14,9	0	14938,77
4x	289	0	14,7	11,1	0	3229,47
5x	1234	0	16,2	7,47	0	9215,34
6x	1224	0	16,2	4,84	0	5930,27
7x	1234	0	16,2	0,06	0	74,64
1y	0	665	0,06	2,50	40,25	0
2y	0	703	7,56	9,94	5318,18	0
3y	0	1595	19,4	11,3	31004,93	0
4y	0	348	7,56	1,92	2629,97	0
sum	6879	3311			38993	36181

Tab. 4.2 determined dimensions regarding the stiffness of walls $K_{i,x}$ in x-direction and $K_{i,y}$ in y-direction

As a consequence, the coordinates of the centre point of stiffness are

$$x_s = \frac{38993}{3311} = 11,78 \quad m$$

and

$$y_s = \frac{36181}{6879} = 5,26 \quad m\,.$$

In order to define the torsional influence based on the eccentric load (the seismic load applies to the gravity centre M of the slab, while the building turns around the centre point of stiffness) it is essential to estimate the coordinates of the gravity centre of the slab. These coordinates are taken from the FEM-application:

$$x_m = 11,02 \quad m$$

and

$$y_m = 6,73 \quad m\,.$$

The following illustration shows the positions of the centre point of stiffness and the gravity centre of the sample building in ground plan.

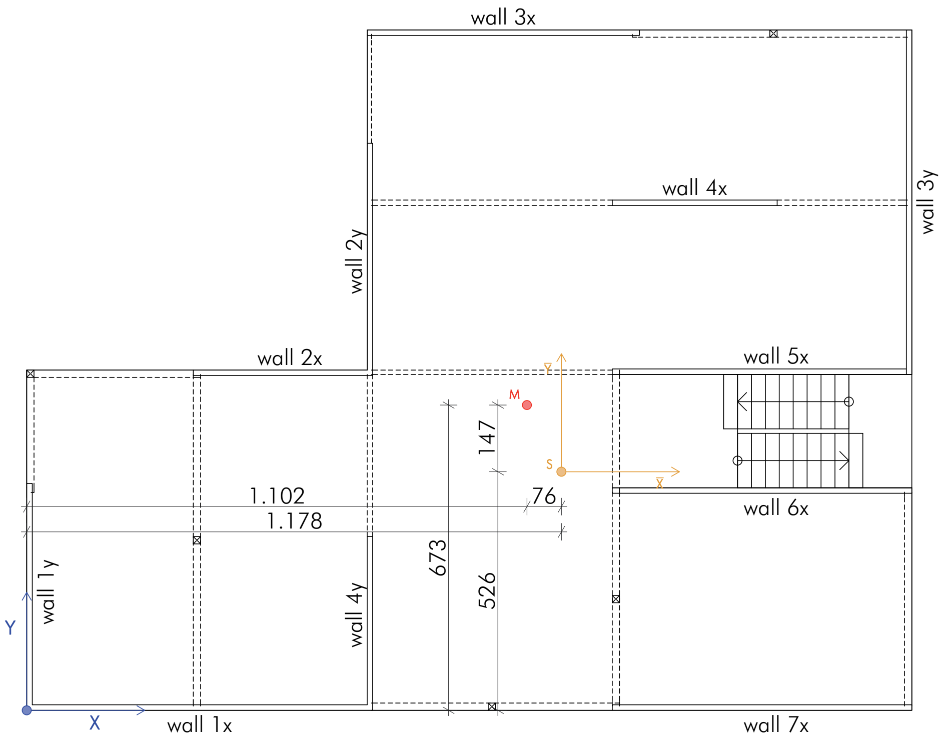

Fig. 4.1 in-plan positions of the centre point of stiffness and the gravity centre

4. 2 Considering the torsional influence - determining additional eccentricities

In order to take account of the realistic inhomogeneous distribution of mass and stiffness, two additional eccentricities need to be considered in addition to the distance between the centre point of stiffness and the gravity centre (eccentricity e_0) according to ÖNORM B 1998-1, section B [13].

The existent eccentricity e_0 is determined as follows:

$$e_{0x} = x_m - x_s = 11,02 - 11,78 = -0,76 \text{ m}$$

and

$$e_{0y} = y_m - y_s = 6,73 - 5,26 = 1,47 \text{ m}$$

In addition to e_0 the eccentricity e_1 is used to take the simplification of the analysis model according to appendix B consideration. The eccentricity e1 is determined as follows:

$$e_{1x} = \min\begin{bmatrix} 0,1 \cdot (l + b) \cdot \left(10 \cdot \frac{|e_{0x}|}{l}\right)^{0,5} \\ 0,1 \cdot (l + b) \end{bmatrix}$$

with

l as length of the building perpendicular to the direction of stress [m]

b as width of the building in direction of stress [m]

results in

$$e_{1x} = \min\begin{bmatrix} 0,1 \cdot (19,5 + 15,0) \cdot \left(10 \cdot \frac{|-0,76|}{19,5}\right)^{0,5} \\ 0,1 \cdot (19,5 + 15,0) \end{bmatrix} = -2,15 \text{ m}$$

note: The eccentricity e_1 is supposed to share the same sign as the eccentricity e_0.

$$e_{1y} = \min\begin{bmatrix} 0,1 \cdot (15,0 + 19,5) \cdot \left(10 \cdot \frac{|1,47|}{15,0}\right)^{0,5} \\ 0,1 \cdot (15,0 + 19,5) \end{bmatrix} = 3,42 \text{ m}$$

The second additional eccentricity e_2 is described in the master document, defined as 'accidental eccentricity' and determined as follows:

$$e_{2x} = 0,05 \cdot l$$

with

l as length of building perpendicular to the direction of stress [m]

b as width of the building in direction of stress [m]

results in

$$e_{2x} = 0,05 \cdot 19,5 = -0,98 \quad m$$

note: The eccentricity e_1 is supposed to share the same sign as the eccentricity e_0.

and

$$e_{2y} = 0,05 \cdot 15,0 = 0,75 \quad m$$

In addition to taking account of these two additional eccentricities, a case-by-case analysis needs to be considered in the context of determining rotational components of wall stress. This analysis is conducted by creating a maximum eccentricity e_{max} and a minimum eccentricity e_{min}. Generally, this calculation can be regarded as an analysis of the bound values, which makes it possible to consider the whole spectrum of potential eccentricities.

With

$$e_{max} = e_0 + e_1 + e_2$$

$$e_{min} = e_0 - e_2$$

following

$$e_{max,\,x} = -0,76 - 2,15 - 0,98 = -3,89 \quad m$$

$$e_{min,\,x} = -0,76 - (-0,98) = 0,22 \quad m$$

and

$$e_{max,\,y} = 1,47 + 3,42 + 0,75 = 5,64 \quad m$$

$$e_{min,\,y} = 1,47 - 0,75 = 0,72 \quad m$$

4. 3 Distribution of the seismic base shear forces among load-bearing walls

The division of the horizontal seismic base shear forces among load-bearing walls in plan is based on a rigid hierarchy within the walls according to their dimensions of stiffness in axial direction (translational part) and their distance to the centre point of stiffness (incl. distinction of cases e_{max} - e_{min}, rotational part).
According to ÖNORM B 1998-1, appendix B [13] a further subdivision is made into 'earthquake in y-direction' and 'earthquake in x-direction':

earthquake in y-direction

stress of the wall k in y-direction:

$$F_k = F_{by} \cdot \frac{K_{y,k}}{\sum K_{y,i}} + F_{by} \cdot e_{max,x}(e_{min,x}) \cdot \frac{K_{y,k} \cdot \bar{x}_k}{\sum(K_{y,i} \cdot \bar{x}_i^2) + \sum(K_{x,i} \cdot \bar{y}_i^2)}$$

stress of the wall r in x-direction:

$$F_r = -F_{by} \cdot e_{max,x}(e_{min,x}) \cdot \frac{K_{x,r} \cdot \bar{y}_r}{\sum(K_{y,i} \cdot \bar{x}_i^2) + \sum(K_{x,i} \cdot \bar{y}_i^2)}$$

earthquake in x-direction

stress of the wall r in x-direction:

$$F_r = F_{bx} \cdot \frac{K_{x,r}}{\sum K_{x,i}} + F_{bx} \cdot e_{max,y}(e_{min,y}) \cdot \frac{K_{x,r} \cdot \bar{y}_r}{\sum(K_{y,i} \cdot \bar{x}_i^2) + \sum(K_{x,i} \cdot \bar{y}_i^2)}$$

stress of the wall k in y-direction:

$$F_k = -F_{bx} \cdot e_{max,y}(e_{min,y}) \cdot \frac{K_{y,k} \cdot \bar{x}_k}{\sum(K_{y,i} \cdot \bar{x}_i^2) + \sum(K_{x,i} \cdot \bar{y}_i^2)}$$

with

$\bar{x}$, $\bar{y}$ as x- and y-distances between the prevailing wall and the centre point of stiffness [m]

These equations are applied to all 5 floors with 11 load-bearing walls each. Additionally, it needs to be mentioned that the seismic load is considered in y- and x-directions as well as the case-by-case analysis is conducted. This implies a total number of 220 equations, which need to be solved with regard to this example. As a consequence, the use of a spreadsheet program is highly advisable.

4. 4 Determination of the internal forces of walls as a result of the distribution of the seismic base shear forces

The determination of the internal forces M_{Ed} and V_{Ed} of all load-bearing walls is based on a cantilever-wall model (plate model) as it is shown in the following illustration.

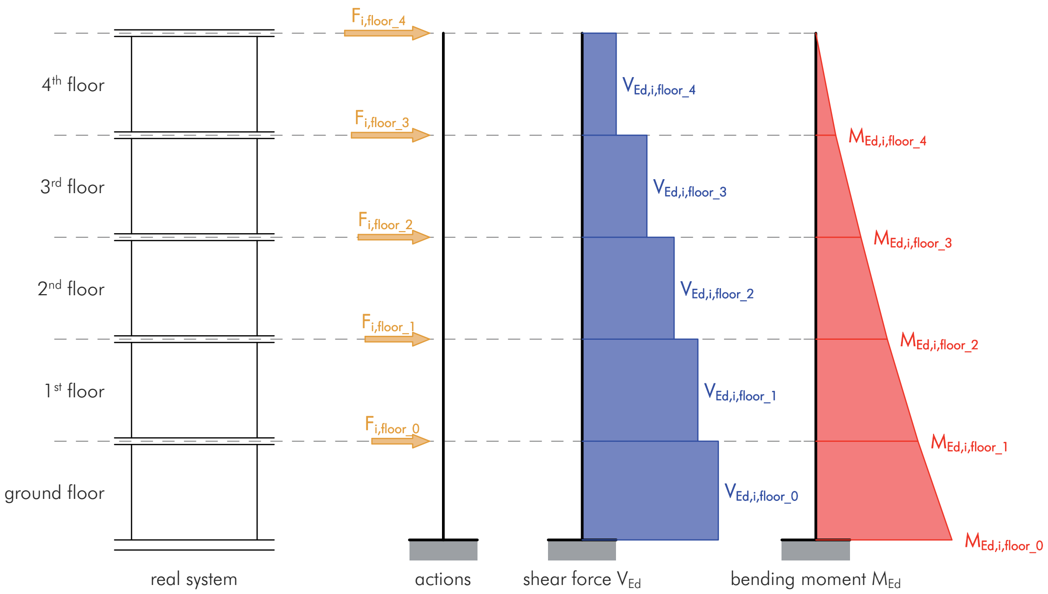

Fig. 4.2 cantilever-wall model with actions and internal forces

The determination of the internal forces of wall 1x at the bearing point, or rather at the connection joint 'wall-foundation', of the ground floor is regarded as the archetype of all load-bearing walls and floors for the following analysis.

4. 4. 1 System parameters for the calculation

y-distance between the centre point of stiffness and wall 1x:

$$\bar{y}_{1x} = -5,20 \text{ m}$$

stiffness of wall 1x in axial direction (x-direction):

$$K_{x,1x} = 1536 \text{ kN/m}$$

sum of the dimensions of wall stiffness in x-direction:

$$\sum K_{x,i} = 6879 \text{ kN/m}$$

$$\sum(K_{y,i} \cdot \bar{x}_i^2) + \sum(K_{x,i} \cdot \bar{y}_i^2) = 390303 \text{ kNm}$$

4. 4. 2 Determination of wall stress of each floor

earthquake in x-direction

- wall stress of the 4th floor

1st case - determination using $e_{max,y}$:

$$F_{1x,4OG} = 135 \cdot \frac{1536}{6879} + 135 \cdot 5,64 \cdot \frac{1536 \cdot (-5,20)}{390303} = 14,6 \text{ kN}$$

2nd case - determination using $e_{min,y}$:

$$F_{1x,4OG} = 135 \cdot \frac{1536}{6879} + 135 \cdot 0,72 \cdot \frac{1536 \cdot (-5,20)}{390303} = 28,2 \text{ kN}$$

It can be said that the 2nd case is decisive for the load-bearing wall 1x. Hence, the focus is shifted to this case.

- wall stress of the 3rd floor

2nd case - determination using $e_{min,y}$:

$$F_{1x,3OG} = 122 \cdot \frac{1536}{6879} + 122 \cdot 0,72 \cdot \frac{1536 \cdot (-5,20)}{390303} = 25,4 \text{ kN}$$

- wall stress of the 2nd floor

2nd case - determination using $e_{min,y}$:

$$F_{1x,2OG} = 91 \cdot \frac{1536}{6879} + 91 \cdot 0,72 \cdot \frac{1536 \cdot (-5,20)}{390303} = 19,0 \text{ kN}$$

- wall stress of the 1st floor

2nd case - determination using $e_{min,y}$:

$$F_{1x,1OG} = 62 \cdot \frac{1536}{6879} + 62 \cdot 0,72 \cdot \frac{1536 \cdot (-5,20)}{390303} = 12,8 \text{ kN}$$

- wall stress of the ground floor

2nd case - determination using $e_{min,y}$:

$$F_{1x,EG} = 31 \cdot \frac{1536}{6879} + 31 \cdot 0,72 \cdot \frac{1536 \cdot (-5,20)}{390303} = 6,5 \text{ kN}$$

earthquake in y-direction

- wall stress of the 4th floor

1st case - determination using $e_{max,x}$:

$$F_{1x,\,4OG} = -135 \cdot (-3,89) \cdot \frac{1536 \cdot (-5,20)}{390303} = -10,8 \;\; kN$$

2nd case - determination using $e_{min,x}$:

$$F_{1x,\,4OG} = -135 \cdot 0,22 \cdot \frac{1536 \cdot (-5,20)}{390303} = 0,6 \;\; kN$$

It can be said that the 1st case is decisive for the load-bearing wall 1x. Hence, the focus is shifted to this case.

- wall stress of the 3rd floor

1st case - determination using $e_{max,x}$:

$$F_{1x,\,3OG} = -122 \cdot (-3,89) \cdot \frac{1536 \cdot (-5,20)}{390303} = -9,7 \;\; kN$$

- wall stress of the 2nd floor

1st case - determination using $e_{max,x}$:

$$F_{1x,\,2OG} = -91 \cdot (-3,89) \cdot \frac{1536 \cdot (-5,20)}{390303} = -7,2 \;\; kN$$

- wall stress of the 1st floor

1st case - determination using $e_{max,x}$:

$$F_{1x,\,1OG} = -62 \cdot (-3,89) \cdot \frac{1536 \cdot (-5,20)}{390303} = -4,9 \;\; kN$$

- wall stress of the ground floor

1st case - determination using $e_{max,x}$:

$$F_{1x,\,EG} = -31 \cdot (-3,89) \cdot \frac{1536 \cdot (-5,20)}{390303} = -2,5 \;\; kN$$

4. 4. 3 Determination of the decisive internal forces of load-bearing walls

- internal forces resulting from earthquake in x-direction

The design shear force $V_{Ed,x}$ at the base point of the load-bearing wall is determined by adding all wall stresses.

$$V_{Ed,\,x,\,1x} = \sum F_{1x,\,i} = 28,2 + 25,4 + 19,0 + 12,8 + 6,5 = 91,9 \ \ kN\,.$$

The design bending moment $M_{Ed,x}$ at the base point of the load-bearing wall is determined by multiplying the horizontal forces acting on the storey by the relevant z-distances.

$$M_{Ed,\,x,\,1x} = \sum F_{1x,\,i} \cdot z_i = 28,2 \cdot 15,0 + 25,4 \cdot 12,0 + 19,0 \cdot 9,0 + 12,8 \cdot 6,0 + 6,5 \cdot 3,0 = 995,1 \ \ kNm$$

- internal forces resulting from earthquake in y-direction

The design shear force of earthquakes in y-direction is:

$$V_{Ed,\,y,\,1x} = 34,9 \ \ kN$$

(absolute value)

The design bending moment is:

$$M_{Ed,\,y,\,1x} = 378,1 \ \ kNm$$

(absolute value)

- combination of the internal forces according to ÖNORM EN 1998-1, section 4.3.3.5.1(2b) [12]

According to ÖNORM EN 1998-1 [12] the two horizontal components of seismic design can be combined with each other in x- and y-directions by using the square-root-sum-of-squares (SRSS) - model:

$$V_{Ed,\,1x} = \sqrt{V^2_{Ed,\,x,\,1x} + V^2_{Ed,\,y,\,1x}} = \sqrt{91,9^2 + 34,9^2} = 98,3 \ \ kN$$

and

$$M_{Ed,\,1x} = \sqrt{M^2_{Ed,\,x,\,1x} + M^2_{Ed,\,y,\,1x}} = \sqrt{995,1^2 + 378,1^2} = 1064,5 \ \ kNm$$

Due to the high number of results (5 floors, each with 2 connection joints and 11 load-bearing walls), the ground floor is regarded as the archetype of all internal forces of the load-bearing walls. Consequently, the following two tables just provide an insight into the internal forces of the load-bearing walls of the ground floor based on the SRSS principle of superposition.

wall i	$V_{Ed,i}$	$M_{Ed,i}$
	[kN]	[kNm]
x-direction		
1x	98	769
2x	29	223
3x	133	1038
4x	30	237
5x	97	761
6x	78	611
7x	79	618
y-direction		
1y	132	1035
2y	108	846
3y	229	1790
4y	53	418

Tab. 4.3 internal forces of the connection joint 'ground floor over'

wall i	$V_{Ed,i}$	$M_{Ed,i}$
	[kN]	[kNm]
x-direction		
1x	98	1064
2x	29	309
3x	133	1436
4x	30	328
5x	97	1052
6x	78	845
7x	79	855
y-direction		
1y	132	1431
2y	108	1171
3y	229	2476
4y	53	579

Tab. 4.4 internal forces of the connection joint 'ground floor under'

4. 5 Verification of the load-bearing capacity of the connectors

The end of the 1st iteration is marked by an inspection of the load-bearing capacity of the connectors regarding the previously determined actions in order to be able to finalise this seismic design. Thus, it is of crucial importance to be aware of the load-bearing capacities of the connectors of all connection joints. In the following two sections, shear- and bending moment load-bearing capacities are determined.

4. 5. 1 Shear capacity of the connection joints

The determination of the shear capacity is the result of the number of used angle brackets per connection joint multiplied by the structural resistance of a pair of angle brackets:

$$R_{v,\,d,\,i} = n_i \cdot R_{xz,\,d,\,i}$$

with

n_i as number of the used angle brackets per connection joint

$R_{xz,i}$ as structural resistance of a pair of angle brackets, according to tab. 2.3

Due to the fact that the estimation of the number of angle brackets on the basis of a chosen distance of ca. 1 m was performed preliminary to the actual calculations, the usage of various connectors applied to the joints 'wall-foundation' results in a significant difference among the shear capacities per connection joint. The following table gives an insight into the shear capacities of the connection joints with regard to the selected conception of connectors.

wall i	number ni	$R_{v,d,i,GF}$	$R_{v,d,i,floor\_1}$
	[-]	[kN]	[kN]
x-direction			
wall 1x	8	233,20	94,40
wall 2x	4	116,60	47,20
wall 3x	6	174,90	70,80
wall 4x	4	116,60	47,20
wall 5x	7	204,05	82,60
wall 6x	7	204,05	82,60
wall 7x	7	204,05	82,60

Tab. 4.5 dimensions of shear stifness of the connection joints ‘wall-foundation‘ and ‘wall-ceiling-wall‘

y-direction			
wall 1y	5	145,75	59,00
wall 2y	6	174,90	70,80
wall 3y	8	233,20	94,40
wall 4y	4	116,60	47,20

Tab. 4.5 dimensions of shear stifness of the connection joints 'wall-foundation' and 'wall-ceiling-wall'

4. 5. 2 Bending moment capacity of the connection joints

After defining the number of hold downs, the dimensions of the compression zone and the features of the internal moment arm as well as of the vertical design load, the stress resulting of a bending moment of the prevailing connection joints can be determined using the following equations:

$$M_{Rd} = N_c \cdot \left(L - l_z - \frac{x}{2}\right) - N_{z,d} \cdot \left(\frac{L}{2} - l_z\right).$$

connection joint 'wall-foundation'

$$x = \frac{N_{c,d}}{b_{eff} \cdot f_{c,0,d}} = \frac{n \cdot R_{1,d} + N_{z,d}}{b_{eff} \cdot f_{c,0,d}} \text{ und } N_c = x \cdot b_{eff} \cdot f_{c,0,d}$$

connection joint 'wall-ceiling' and 'ceiling-wall'

$$x = \frac{N_{c,d}}{b_{eff} \cdot k_{c,90} \cdot f_{c,90,d}} = \frac{n \cdot R_{1,d} + N_{z,d}}{b_{eff} \cdot k_{c,90} \cdot f_{c,90,d}} \text{ und } N_c = x \cdot b \cdot k_{c,90} \cdot f_{c,90,d}$$

Again the load-bearing capacities of the connection joints 'ground floor under' and 'ground floor over' can be regarded as the archetypes of all connection joints. Hence, the following tables show these capacities and those of the previously determined stresses (cf. tab. 4.3 and tab. 4.4).

wall i	$V_{Ed,i}$	$R_{v,d,i,floor\_1}$	utilisation	$M_{Ed,i}$	$M_{Rd,i}$	utilisation
	[kN]	[kN]	[-]	[kNm]	[kNm]	[-]
x-direction						
1x	**98**	**94**	**1,04**	769	1439	0,53
2x	29	47	0,60	223	764	0,29
3x	**133**	**71**	**1,87**	1038	1328	0,78
4x	30	47	0,64	237	1125	0,21
5x	**97**	**83**	**1,18**	761	1821	0,42
6x	78	83	0,95	611	2291	0,27
7x	79	83	0,96	618	1678	0,37
y-direction						
1y	**132**	**59**	**2,24**	**1035**	**952**	**1,09**
2y	**108**	**71**	**1,53**	846	1420	0,60
3y	**229**	**94**	**2,42**	**1790**	**1667**	**1,07**
4y	**53**	**47**	**1,13**	418	1089	0,38

Tab. 4.6 comparison of the internal forces with the load-bearing capacities of the connection joint 'ground floor over'

wall i	$V_{Ed,i}$	$R_{v,d,i,GF}$	utilisation	$M_{Ed,i}$	$M_{Rd,i}$	utilisation
	[kN]	[kN]	[-]	[kNm]	[kNm]	[-]
x-direction						
1x	98	233	0,42	1064	1600	0,67
2x	29	117	0,24	309	874	0,35
3x	133	175	0,76	1436	1484	0,97
4x	30	117	0,26	328	1554	0,21
5x	97	204	0,48	1052	2037	0,52
6x	78	204	0,38	845	2640	0,32
7x	79	204	0,39	855	1883	0,45
y-direction						
1y	132	146	0,91	**1431**	**1062**	**1,35**
2y	108	175	0,62	1171	1646	0,71
3y	229	233	0,98	**2476**	**1855**	**1,34**
4y	53	117	0,46	579	1363	0,42

Tab. 4.7 comparison of the internal forces with the load-bearing capacities of the connection joint 'ground floor under'

As it becomes apparent in both tables, with regard to this particular seismic design the existent load-bearing capacity is insufficient for several walls because of the selected distance of the connectors between the load-bearing walls with the maximum stress level, 1y and 3y. In concrete terms, this means that within the connection joint 'ground floor over' the degree of shear action of several load-bearing walls is considerably high due to the usage of the rather 'small' angle joints ABR90. Additionally, the connection joint 'ground floor under' exceeds the bending capacity of the walls 1y and 3y by far.

As a consequence, a significant increase in the number of tension anchors and angle joints is absolutely necessary. Based on the results of the spreadsheet program it is evident that the shear capacity of several walls up to the connection joint '3rd floor over' breaks the upper limit. Hence, it would be highly advisable to use angle brackets with higher load-bearing capacities. With regard to this seismic design, however, a higher number of tension anchors and angle brackets is used with the result of a recalculation of stiffness, first periods and seismic load in the context of a 2nd iteration.

5 Recalculation of the 2nd step of iteration

In this section the results in the context of the recalculation of the 2nd step of iteration are presented. Generally, it can be said that the increasing number of connectors leads to a growth in stiffness of the connection joints and to a reduction of first periods. A significant increase in the seismic base shear loads is the possible consequence.

The process of

- determining the stiffness
- calculating the first periods using the FEM application
- estimating the seismic base shear loads
- transferring these loads to each floor and to the prevailing load-bearing walls
- determining the internal forces of walls
- verifying the connection joints

has already been described in detail in section 2, section 3 and section 4 of this chapter. Therefore, the following sections exclusively provide a fundamental insight into essential results in the context of the 2nd step of iteration.

5. 1 Recalculation of the first periods

After revising the parameters of stiffness according to section 2. 2, the first two first periods in x- and y-directions are recalculated.

$T_{1,x} = 1,74\ \text{s}$

and

$T_{1,y} = 1,94\ \text{s}.$

note: In comparison with the first periods of the 1st iteration, it is evident that the first period $T_{1,x}$ has decreased insignificantly, whereas the recalculation of the first period $T_{1,y}$ has caused a drastic reduction. This drop is the result of reinforcing the walls 1y and 3y, which are particularly stressed by the seismic shear base forces. According to ÖNORM EN 1998-1:2005, section 4.3.3.2.1(2)a [12] both first periods are placed within the limit of 2,0 s now. Because of the fact that the second limit (4,0 x TD) highly depends on the parameters of subsoil, the two first periods still fall within the range of tolerance, although they exceed this cut-off point.

5. 2 Seismic shear base forces of the 2nd iteration

Due to the fact that the first periods are still placed below the limit of 0,2 x a_g, the seismic shear base forces do not increase within the 2nd iteration. Hence, a detailed analysis of the total amount of stress, the horizontal forces acting on the storey and the distributed wall loads is unnecessary.

5. 3 Considering second order effects

According to ÖNORM EN 1998-1:2005, section 4.4.2.2(2) [12] it is of crucial importance to consider possible second order effects to be able to calculate the internal forces of load-bearing walls. In this context, the following boundary condition needs to be verified.

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0,10$$

with

θ as the interstorey drift sesitivity coefficient [-]

P_{tot} as the total gravity load at and above the storey considered in the seismic design situation [kN]

d_r is the design interstorey drift [m]

V_{tot} as the total seismic storey shear [kN]

h as the interstorey height [m]

The design value d_r is defined as follows

$$d_r = q_d \cdot d_e .$$

With

q_d as displacement behaviour factor, to be equated with q [-]

d_e as displacement, determined by a linear analysis based on the design response spectrum of the examined point [m]

The examination of the wall with the maximum stress level, wall 3y (concerning deflection), is regarded as the archetype of analysing the mutual shifting of the whole floor. With regard to this scrutiny, it is worth mentioning that the horizontal deformation of this load-bearing wall is determined by using software for designing framed structures. Instead of the weight of the whole floor, just the weight which is allocated by the influence coefficient e to the wall 3y is taken into consideration. The results of this examination can be found in the following table.

floor	z_i	shear force V_d	allocated weight according to comb. N_d	horizontal deflection d_e	design value of the deflection d_d	**interstorey drift d_r**
	[m]	[kN]	[kN]	[mm]	[mm]	**[mm]**
ground floor	3,0	254	390	15,8	47,4	**47,4**
1st floor	6,0	236	309	39,6	118,8	**71,4**
2nd floor	9,0	200	228	70,8	212,4	**93,6**
3rd floor	12,0	148	150	106,8	320,4	**108,0**
4th floor	15,0	78	73	142,5	427,5	**107,1**

Tab. 5.1 interstorey drift d_r of the wall 3y

Consequently, the sensitivity coefficients of the interstorey drift are

4th floor:

$$\theta_{4OG} = \frac{73,0 \cdot 107,1}{78 \cdot 15000} = 0,001 < 0,10$$

3rd floor:

$$\theta_{3OG} = \frac{150,0 \cdot 108,0}{148 \cdot 12000} = 0,01 < 0,10$$

2nd floor:

$$\theta_{2OG} = \frac{228,0 \cdot 93,6}{200 \cdot 9000} = 0,01 < 0,10$$

1st floor:

$$\theta_{1OG} = \frac{309,0 \cdot 71,4}{236 \cdot 6000} = 0,02 < 0,10$$

ground floor:

$$\theta_{EG} = \frac{390,0 \cdot 47,4}{254 \cdot 3000} = 0,02 < 0,10$$

As a result, it is evident that the second order effects can be ignored at least in the context of this load-bearing wall.

5. 4 Verification of the Ultimate Limit State

Again the recalculation of the load-bearing capacities of the connection joints 'ground floor under' and 'ground floor over' can be regarded as the archetype of estimating all connection joints. The following two tables provide a fundamental insight into these load-bearing capacities and compare them with the existent stresses.

wall i	$V_{Ed,i}$	$R_{v,d,i,floor\_1}$	utilisation	$M_{Ed,i}$	$M_{Rd,i}$	utilisation
	[kN]	[kN]	[-]	[kNm]	[kNm]	[-]
x-direction						
1x	97	233	0,41	1046	1600	0,65
2x	27	117	0,23	290	874	0,33
3x	109	175	0,63	1184	1484	0,80
4x	26	117	0,23	285	1554	0,18
5x	91	204	0,45	987	2037	0,48
6x	78	204	0,38	846	2640	0,32
7x	78	204	0,38	840	1883	0,45
y-direction						
1y	187	204	0,92	2030	2168	0,94
2y	67	175	0,38	727	1646	0,44
3y	254	262	0,97	2747	3154	0,87
4y	33	117	0,28	360	1363	0,26

Tab. 5.2 comparison of internal forces and load bearing capacities of the connection joint 'ground floor under'

wall i	$V_{Ed,i}$	$R_{v,d,i,floor\_1}$	utilisation	$M_{Ed,i}$	$M_{Rd,i}$	utilisation
	[kN]	[kN]	[-]	[kNm]	[kNm]	[-]
x-direction						
1x	97	106	0,91	756	1439	0,53
2x	27	47	0,57	209	764	0,27
3x	109	130	0,84	856	1328	0,64
4x	26	47	0,56	206	1125	0,18
5x	91	94	0,97	714	1821	0,39
6x	78	83	0,95	612	2291	0,27
7x	78	83	0,94	607	1678	0,36
y-direction						
1y	187	189	0,99	1467	1699	0,86
2y	67	94	0,71	526	1420	0,37
3y	254	260	0,98	1985	2074	0,96
4y	33	47	0,70	260	1089	0,24

Tab. 5.3 comparison of internal forces and load bearing capacities of the connection joint ‘ground floor over‘

Both tables show that the verification of the connection joints is fulfilled for the instantaneous design situation ‘earthquake‘ (in ULS).

5. 5 Comparison with the results of the modal response spectrum analysis

As it was mentioned in section 2. 3, the first periods T_{1x} and T_{1y} exceed the limit of $4 \cdot T_C$ according to ÖNORM EN 1998-1, section 4.3.3.2.1(2a) [12]. Hence, the internal forces of this load-bearing structure in Solid Timber Construction are also determined by applying the modal response spectrum analysis (using the RFEM application with the additional module II ‘RF-DYNAM‘).

According to ÖNORM EN 1998-1, section 4.3.3.3.1 [12] the values of the first 8 modal responses need to be taken into consideration in order to fulfil the requirements of the modal response spectrum analysis. The combination of these values is based on the SRSS-principle of superposition, which has also been used to combine the internal forces in section 4. 4. It shall be mentioned that torsional influences are taken into account according to ÖNORM EN 1998-1, section 4.3.3.3.3 [12]. The following table shows all these internal forces, which are determined by using this process, and compares them with those of the connection joint 'ground floor under' of section 5. 4.

wall i	lateral force method of analysis		modal response spectrum analysis	
	$V_{Ed,i}$	$M_{Ed,i}$	$V_{Ed,i}$	$M_{Ed,i}$
	[kN]	[kNm]	[kN]	[kNm]
x-direction				
1x	97	1046	68	599
2x	**27**	290	**32**	219
3x	109	1184	82	651
4x	**26**	285	**35**	204
5x	91	987	70	601
6x	78	846	63	548
7x	78	840	56	494
y-direction				
1y	187	2030	114	1083
2y	**67**	727	**70**	502
3y	254	2747	179	1677
4y	**33**	360	**39**	262

Tab. 5.4 comparison of the internal forces as the results of both mentioned methods of the connection joint 'ground floor under'

As it becomes evident in tab. 5.4, the decisive shear forces $V_{Ed,i}$ of the walls 2x, 4x, 2y and 4y, which are defined by using the modal response-spectrum analysis, insignificantly exceed the results produced by using the lateral force method of analysis.

Due to the fact that also the estimated bending moments of these walls just differ insignificantly from those produced by manual calculation, it appears as if in the context of the modal method the horizontal actions are simply transferred to all walls in a more homogeneous way. This effect is based on ÖNORM B 1998-1, section B [13] regarding the sensitivity analysis of torsional influences. With regard to this standard, it is stated that walls with a distant centre point of stiffness deal with a great amount of stress. However, this approach is inconsistent with the formulation of torsional influences according to ÖNORM EN 1998-1, section 4.3.3.3.3(1) [12], which is applied to the modal response spectrum analysis.

To summarise, it can be said that with regard to this design the calculation of the seismic base shear forces based on the lateral force method of analysis is successfully completed, although some values exceed the limits according to ÖNORM EN 1998-1, section 4.3.3.2.1(2a) [12]. However, it needs to be mentioned that this scenario should not be generalised due to the fact that essential dimensions, such as structural system, mass of the building and design, vary in different cases.

6 Shear verification of the decisive panel

To conclude this seismic design an exemplary shear verification of the most stressed load-bearing wall of the ground floor is conducted.

Added to this, a verification of the hierarchy of the bearing capacities is conducted by comparing the reserve of brittle failures (walls) with the ductile failures of the connections.

6. 1 Shear verification of wall 1y of the ground floor

According to tab. 5.2 the design shear force V_{Ed} of this load-bearing wall amounts to

$$V_{Ed,\,1y,\,GF} = 187 \ \text{kN}\,.$$

With regard to this design, the shear verification is based on internal line forces which are the result of dividing the estimated shear force by the length of the wall 1y and amount to:

$$n_{xy,\,d} = \frac{V_{Ed,\,1y,\,GF}}{L_{1y}} = \frac{187}{5,00} = 37,4 \ \text{kN/m}\,.$$

The existent CLT-panel is analysed by using the software program 'CLTdesigner' written at the Institute of Timber Engineering and Wood Technology of Graz University of Technology and is limited by a shear capacity of

$$r_{xy,\,d} = 210 \ \text{kN/m}\,.$$

- verification of the shear stress

$$\eta = \frac{n_{xy,\,d}}{r_{xy,\,d}} = \frac{37,4}{210,0} = 0,18$$

To conclude this section, it can be said that the verification of the shear stress is successful and the utilisation amounts to 18%.

6. 2 Hierarchy within the structural resistance

The verification of the hierarchy within the structural resistance in the context of capacity design is of vital importance, since this examination establishes the proof of ductile failure of structural members, or rather of the building as a whole. When taking the sections of the previous chapter into consideration, it becomes evident that the dissipative structural resistance of the sample building with regard to seismic load is highly influenced by the features of the connectors within the connection joints. In other words, this means that connectors can be regarded as the weakest, but ductile, links of a chain of load-bearing capacities. Therefore, their failure is expected because of the formation of yield hinges (energy dissipation), while the possibility of brittle failures of other structural members is ruled out due to the fact that the load-bearing capacities of the components are compared with each other and their differences are determined as a multiple of the ductile bearing capacity.

Nevertheless, a modification of the global structural system is necessary, if the factor is below 1,2. This factor is determined as follows:

- Determination of the connection joint with the highest load-bearing capacity

The connection joint with the highest load-bearing capacity is identified as the one of wall 1y of the ground floor (wall-foundation, angle bracket AE116). The shear capacity (per meter) of this joint amounts to

$$r_{xy,\,d,\,bracket} = 40,81 \ \ kN/m\,.$$

Added to this, based on a conservative interpretation of this condition, a friction resistance depending on the longitudinal force (per meter) among the wall and the reinforced concrete slab is determined:

$$r_{xy,\,d,\,friction} = \mu \cdot n_{dN}$$

with

μ as friction coefficient, which is used regarding the contact of timber with reinforced concrete [-]

n_{dN} as longitudinal force per meter [kN/m], which is determined by dividing the longitudinal force per load-bearing wall by the length of wall:

$$n_{dN} = \frac{N_{Ed}}{L} = \frac{323,23}{5,00} = 64,65 \ \ kN/m$$

results in

$$r_{xy,\,d,\,friction} = 0,40 \cdot 64,65 = 25,86 \ \ kN/m$$

As a consequence, the maximal load-bearing capacity of the connection joint of shear stress amounts to

$$r_{xy,\,d,\,joint} = r_{xy,\,d,\,bracket} + r_{xy,\,d,\,friction} = 40,81 + 25,86 = 66,67 \ \ kN/m$$

- selection of the to-be-avoided mode of failure and determination of the load-bearing capacity

In contrast to a failure of angle connectors within the connection joints, the failure of the corresponding load-bearing wall per shear force is defined by a relatively unfavourable mode. Hence, it is highly advisable to avoid the latter one. With regard to the load-bearing capacity, the capacity of the corresponding load-bearing wall 1y has already been determined in section 6. 1 and amounts to

$r_{xy,d} = 210 \ \text{kN/m}$.

- comparison of both load-bearing capacities

As mentioned in the introduction of this section, both load-bearing capacities are compared with each other. As a result, the factor 'overcapacity' is defined:

$$\eta = \frac{r_{xy,d}}{r_{xy,d,joint}} = \frac{210,00}{66,67} = 3,15 > 1,20$$

To conclude, it can be said that between the failure of the connection joints and the unfavourable shear failure there exists a sufficient capacity. Additionally, the selected structural system satisfies the requirements of a high dissipative system.

Appendix

1 Appendix A - CLT-Desinger transcripts

1. 1 Transcript of the pre-design of the single-span girder system

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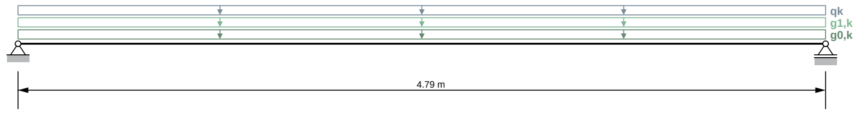
2 Structural system

Continuous beam with 1 spans

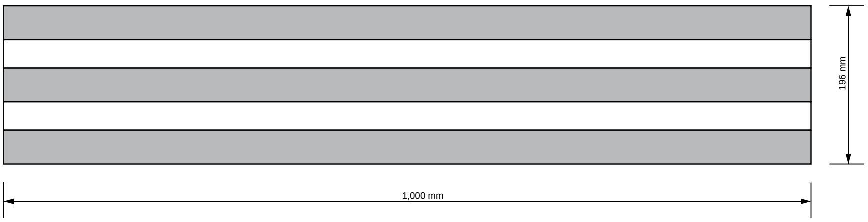


2.1 Width of supports

Support	x	Width
A	0.0 m	0.06 m
B	4.79 m	0.06 m

3 Cross section

CLT-Product with technical approval of the company StoraEnso: 196 L5s
5 layers (width: 1,000 mm / thickness: 196 mm)



3.1 Layer composition

Layer	Thickness	Orientation	Material
# 1	42 mm	0	C24-STORA ENSO
# 2	35 mm	90	C24-STORA ENSO
# 3	42 mm	0	C24-STORA ENSO
# 4	35 mm	90	C24-STORA ENSO
# 5	42 mm	0	C24-STORA ENSO

3.2 Material parameters

Partial safety factor $\gamma_M = 1.25$

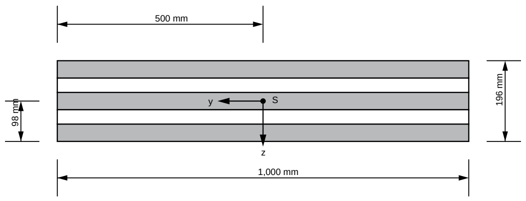
Material parameters for C24-STORA ENSO	
bending strength	24.0 N/mm²
tensile strength parallel	14.0 N/mm²
tensile strength perpendicular	0.4 N/mm²

Projekt
Summary of results

Material parameters for C24-STORA ENSO	
compressive strength parallel	21.0 N/mm²
compressive strength perpendicular	2.5 N/mm²
shear strength	4.0 N/mm²
rolling shear strength	1.25 N/mm²
Youngs modulus parallel	11,000.0 N/mm²
5%-quantile from Youngs modulus parallel	7,400.0 N/mm²
Youngs modulus perpendicular	370.0 N/mm²
shear modulus	690.0 N/mm²
rolling shear modulus	50.0 N/mm²
density	350.0 kg/m³
density mean value	420.0 kg/m³
in plane shear strength	5.0 N/mm²
torsional strength	2.5 N/mm²

3.3 Cross-sectional values

EA_{ef}	1.412E9 N
EI_{ef}	5.723E12 N·mm²
GA_{ef}	1.683E7 N



4 Loads

Field	$g_{0,k}$	$g_{1,k}$	q_k	Category	s_k	Altitude/Region	w_k
1	1.078 kN/m	3.07 kN/m²	2.8 kN/m²	A			

Load position:

5 Specification concerning structural fire design

No specifications are available

6 Information concerning vibrations

Damping factor: 4.0 %
The transfer of vibrations to neighbouring fields is perceived as a disturbance.
The vibrational design is carried out taking into consideration the stiffness of the screed (concrete topping).
Thickness of the screed (concrete topping): 6.0 cm
Youngs-Modulus of screed (concrete topping): 26,000.0 N/mm²
Bending stiffness of screed (concrete topping): 468.0 kNm²/m
Width perpendicular to the main load bearing direction: 1.0 m

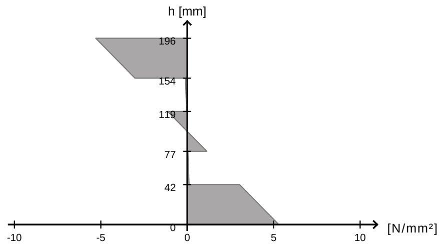
7 Results

Referenced standards: ON EN 1995-1-1:2009

7.1 ULS

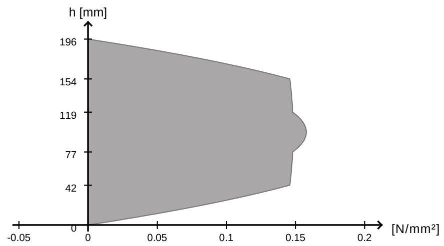
7.1.1 Bending

Utilisation ratio	31.3 %
k_{mod}	0.8
at x	2.395 m
Fundamental combination	$1.35*g_{0,k}$ + $1.35*g_{1,k}$ + $1.50*1.00*q_k$



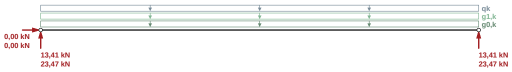
7.1.2 Shear

Utilisation ratio	18.5 %
k_{mod}	0.8
at x	0.0 m
Fundamental combination	$1.35*g_{0,k}$ + $1.35*g_{1,k}$ + $1.50*1.00*q_k$



7.1.3 Bearing pressure

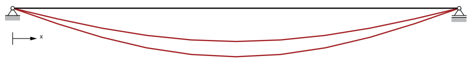
Utilisation ratio	14.0 %
k_{mod}	0.8
at x	0.0 m
Fundamental combination	$1.35*g_{0,k}$ + $1.35*g_{1,k}$ + $1.50*1.00*q_k$



7.2 SLS

7.2.1 Deflection

Utilisation ratio	79.9 %
w_{max}	15.3 mm
k_{def}	0.85
at x	2.395 m
Final deformation $w_{net,fin}$	t = inf (l/250)



7.2.2 Vibration

The verification is only valid for residential ceilings!

7.2.2.1 Verification corresponding to EN 1995-1-1

Eigenfrequency: f_1 = 7.7 Hz < 8.0 Hz
Stiffness: w_{1kN} = 0.4 mm < 4.0 mm
Velocity/Unit impuls: v = 1.8 mm/s < 31.3 mm/s
---> More accurate vibration verification is needed!

7.2.2.2 Verification corresponding to DIN 1052

w_{perm} = 6.8 mm > 6,0 mm ---> Vibration verification is not fulfilled or more accurate verification is needed!

1. 2 Transcript of the pre-design of the three-span girder system

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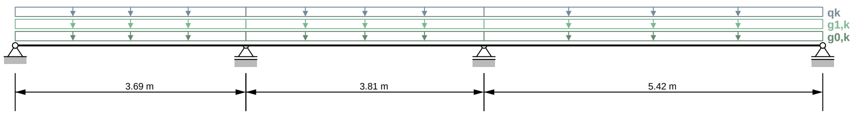
Service class 1

2 Structural system

Continuous beam with 3 spans



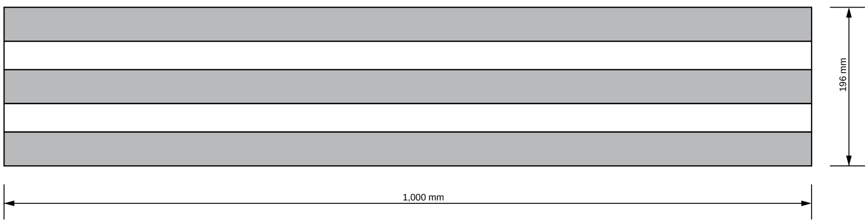
2.1 Width of supports

Support	x	Width
A	0.0 m	0.06 m
B	3.69 m	0.06 m
C	7.5 m	0.06 m
D	12.92 m	0.06 m

Projekt
Summary of results

3 Cross section

CLT-Product with technical approval of the company StoraEnso: 196 L5s
5 layers (width: 1,000 mm / thickness: 196 mm)



3.1 Layer composition

Layer	Thickness	Orientation	Material
# 1	42 mm	0	C24-STORA ENSO
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# 3	42 mm	0	C24-STORA ENSO
# 4	35 mm	90	C24-STORA ENSO
# 5	42 mm	0	C24-STORA ENSO

3.2 Material parameters

Partial safety factor γ_M = 1.3

Material parameters for C24-STORA ENSO	
bending strength	24.0 N/mm²
tensile strength parallel	14.0 N/mm²
tensile strength perpendicular	0.4 N/mm²

Projekt

Summary of results

Material parameters for C24-STORA ENSO	
compressive strength parallel	21.0 N/mm²
compressive strength perpendicular	2.5 N/mm²
shear strength	4.0 N/mm²
rolling shear strength	1.25 N/mm²
Youngs modulus parallel	11,000.0 N/mm²
5%-quantile from Youngs modulus parallel	7,400.0 N/mm²
Youngs modulus perpendicular	370.0 N/mm²
shear modulus	690.0 N/mm²
rolling shear modulus	50.0 N/mm²
density	350.0 kg/m³
density mean value	420.0 kg/m³
in plane shear strength	5.0 N/mm²
torsional strength	2.5 N/mm²

3.3 Cross-sectional values

EA_{ef}	1.412E9 N
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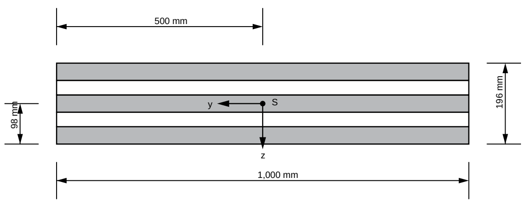
GA_{ef}	1.683E7 N



4 Loads

Field	$g_{0,k}$	$g_{1,k}$	q_k	Category	s_k	Altitude/Region	w_k
1	1.078 kN/m	3.07 kN/m²	2.8 kN/m²	A			
2	1.078 kN/m	3.07 kN/m²	2.8 kN/m²	A			
3	1.078 kN/m	3.07 kN/m²	2.8 kN/m²	A			

Projekt
Summary of results

5 Specification concerning structural fire design

No specifications are available

6 Information concerning vibrations

Damping factor: 4.0 %
The transfer of vibrations to neighbouring fields is perceived as a disturbance.
The vibrational design is carried out taking into consideration the stiffness of the screed (concrete topping).
Thickness of the screed (concrete topping): 6.0 cm
Youngs-Modulus of screed (concrete topping): 26,000.0 N/mm²
Bending stiffness of screed (concrete topping): 468.0 kNm²/m
Width perpendicular to the main load bearing direction: 1.0 m

Projekt

Summary of results

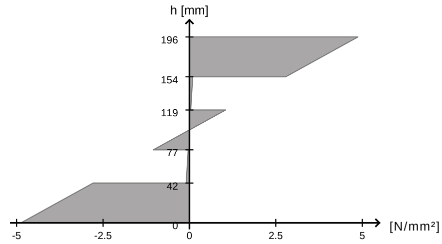
7 Results

Referenced standards: ON EN 1995-1-1:2009

7.1 ULS

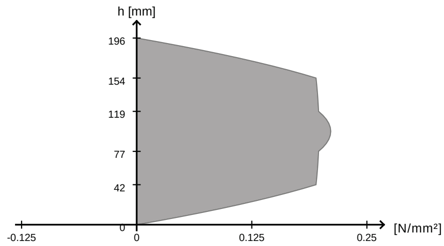
7.1.1 Bending

Utilisation ratio	30.0 %
k_{mod}	0.8
at x	7.5 m
Fundamental combination	$1.35 \cdot g_{0,k}$ + $1.35 \cdot g_{1,k}$ + $1.50 \cdot 1.00 \cdot q_k$



7.1.2 Shear

Utilisation ratio	25.7 %
k_{mod}	0.8
at x	7.5 m
Fundamental combination	$1.35 \cdot g_{0,k}$ + $1.35 \cdot g_{1,k}$ + $1.50 \cdot 1.00 \cdot q_k$



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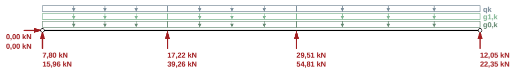
7.1.3 Bearing pressure

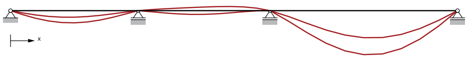
Utilisation ratio	33.9 %
k_{mod}	0.8
at x	7.5 m
Fundamental combination	$1.35 \cdot g_{0,k}$ + $1.35 \cdot g_{1,k}$ + $1.50 \cdot 1.00 \cdot q_k$



7.2 SLS

7.2.1 Deflection

Utilisation ratio	72.9 %
w_{max}	15.8 mm
k_{def}	0.85
at x	10.21 m
Final deformation $w_{net,fin}$	t = inf (l/250)



7.2.2 Vibration

The verification is only valid for residential ceilings!

7.2.2.1 Verification corresponding to EN 1995-1-1

Eigenfrequency: f_1 = 7.7 Hz < 8.0 Hz
Stiffness: w_{1kN} = 0.1 mm < 4.0 mm
Velocity/Unit impuls: v = 2.3 mm/s < 31.4 mm/s
---> More accurate vibration verification is needed!

7.2.2.2 Verification corresponding to DIN 1052

w_{perm} = 6.9 mm > 6,0 mm ---> Vibration verification is not fulfilled or more accurate verification is needed!

1. 3 Transcript of the determination of the in-plane shear capacity of the CLT element

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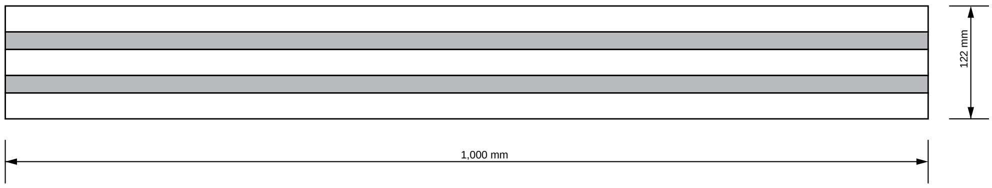
1 Cross section

User-defined cross section
5 layers (width: 1,000 mm / thickness: 122 mm)
Horizontal cross section



Vertical cross section

1.1 Layer composition

Layer	Thickness	Orientation	Material
# 1	28 mm	0	C24-STORA ENSO
# 2	19 mm	90	C24-STORA ENSO
# 3	28 mm	0	C24-STORA ENSO
# 4	19 mm	90	C24-STORA ENSO

# 5	28 mm	0	C24-STORA ENSO

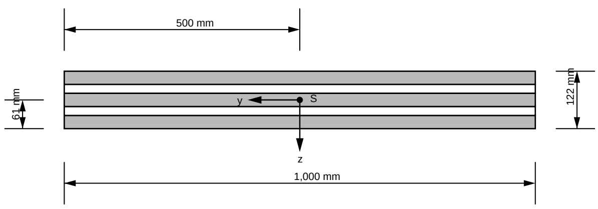
1.2 Material parameters

Partial safety factor γ_M = 1.25

Material parameters for C24-STORA ENSO	
bending strength	24.0 N/mm²
tensile strength parallel	14.0 N/mm²
tensile strength perpendicular	0.4 N/mm²
compressive strength parallel	21.0 N/mm²
compressive strength perpendicular	2.5 N/mm²
shear strength	4.0 N/mm²
rolling shear strength	1.25 N/mm²
Youngs modulus parallel	11,000.0 N/mm²
5%-quantile from Youngs modulus parallel	7,400.0 N/mm²
Youngs modulus perpendicular	370.0 N/mm²
shear modulus	690.0 N/mm²
rolling shear modulus	50.0 N/mm²
density	350.0 kg/m³
density mean value	420.0 kg/m³
in plane shear strength	5.0 N/mm²
torsional strength	2.5 N/mm²

1.3 Cross-sectional values

D_x	4.18E8 N/m
D_y	9.24E8 N/m
D_{xy}	6.982E7 N/m



Projekt
Summary of results

3 Internal forces, design values and results

Shear force per unit length $n_{xy,d} = 210.0$ kN/m
Modification factor $k_{mod} = 1.1$
Partial safety factor $\gamma_M = 1.0$

Mechanism I - shear	100.5 %
Mechanism II - torsion	38.2 %
Mechanism I - shear following ETA-09/0036	100.5 %
Mechanism II - torsion following ETA-08/0242	38.2 %

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