

Terraced housing projects: feasibility and advantages of CLT

A multi-criteria analysis of the feasibility of application of cross-laminated timber (CLT) as the primary building material for low-rise, terraced housing projects

Timo Van Ancum





Terraced housing projects: feasibility and advantages of CLT

by

T.S.J Van Ancum

to obtain the degree of Master of Science
at the Delft University of Technology

Author

T.S.J. Van Ancum

timovanancum@hotmail.com

+31 6 20103423

Student no.: 4035224

Graduation committee

Prof.dr.ir. J.W.G. van de Kuilen Delft University of Technology, Chairman

Dr.ir. G.J.P. Ravenshorst Delft University of Technology

Dr.ir. P.C.J. Hoogenboom Delft University of Technology

Ir. C.M. van der Zanden Woodteq houtconstructies bv, Holten

"This page is intentionally left blank"



Preface

Before u lies the report of my thesis research on the feasibility and advantages of CLT in terraced housing projects, to obtain the Master's degree in Structural Engineering. The past educational year has driven me to delve further into the world of timber structures, increasing my already present interest in product development and sustainability of the building industry.

I would like to thank my graduation committee; Jan-Willem van de Kuilen, Geert Ravenshorst and Pierre Hoogenboom, for the continuous assistance throughout the lengthy process of this research. Firstly, I would like to thank Jan-Willem for the assistance with regard to CLT-information and for having challenged my knowledge and creativity during our meetings to push me to find the right paths within this work to reach the desired goals. Next, I would like to thank Geert for supporting me as my main supervisor, for the quick and pleasant communication when I had questions to keep my productivity at the desired level, for asking me questions with the aim of staying critical of my own work to create a substantiated report, and for giving me the opportunity to get more familiar with timber structures by joining the Houtdag, organized by Het Houtblad, and the CLT course in November 2017, organized by Centrum Hout and Rothoblaas. Lastly, I would like to thank Pierre for initially critically examining the structural engineering objective of this research, but subsequently helping me restructure my work and evaluate the structural engineering output to vastly increase the quality of the result of the research. Additional thanks go out to Wolfgang Gard, for joining in at a critical moment in my research to give greater insight in the sustainability analysis.

I would like to thank Woodteq bv., and especially Evert Laarman and Caspar van der Zanden, for giving me the opportunity to join in on their pilot project and investigate solutions which may presently not yet be economically feasible, but will likely get more value towards the future, if and when more concern is put towards environmental issues. Thanks for incorporating me in a stable working environment and during meetings to get a taste of what my professional career will possibly look like after graduation.

Lastly, I would like to thank family and friends for your support and understanding over the course of writing this thesis report, as well as during my whole education. Special thanks to my sister, Jeanine Van Ancum, who spent this period in a similar situation, as you have been finishing your PhD. Thank you for constantly motivating me, for being my working partner and friend and for helping me through the rougher times, both medically and mentally.

Timo Van Ancum

Hilversum, April 2019

"This page is intentionally left blank"



Abstract

Due to the current environmental issues, sustainability gets more emphasis in the building industry. The large CO₂-emission due to concrete production is decreasing the viability of this material, and alternatives are sought in many applications. In large-scale, repetitive, terraced housing projects, concrete is one of the most applied materials due to its relatively low cost. With a view to the future, Woodteq got the request to search for sustainable timber alternatives to a prefabricated concrete structure for these projects. Due to the way timber grows in trees, it has good strength characteristics in one direction, but is weaker in the other directions. An engineered slab is required to replicate the characteristics of concrete slabs. Cross-laminated timber (CLT) is the obvious replacement in that sense. Qualitatively comparing timber to concrete as a building material, important differences are noticeable in terms of relatively better carbon absorbance, CO₂-emission mitigation and renewability of the material. Analysing material properties in terms of building physics performance gives a comparable result, as both materials have their strengths and weaknesses.

In the design of the timber structure, many building criteria need to be taken into account, and unity checks point out critical aspects of a design. Timber is considered a lightweight material with good strength-to-weight ratios, and therefore performs differently to relatively heavy concrete. Building criteria include safety, functionality, ventilation and acoustic performance. The safety criterion includes the requirement for a stable structure. In the CLT-design, assuming the external facades provide the stability, this proves to be critical, as these facades require the application of holes for doors and windows. Finite element analyses (FEA) is a suitable method to investigate the stabilizing functionality of a CLT member, but requires precise input of material characteristics, geometries and boundary conditions to sufficiently replicate reality.

The material properties of timber boards allows for many other engineered slab configurations besides CLT, and an alternative in the shape of hollow core CLT (HCCLT) slabs may be more material efficient in fulfilling the different building criteria. These two variants, CLT and HCCLT, are judged in a multi-criteria analysis, which results in favour of CLT due to easier, automated production without additional challenges in the engineering of the slabs.

An economical comparison between constructing a three storey terraced house with CLT and concrete shows the CLT structure is approximately 30% or €10.000,- more expensive than concrete. This difference needs to be overcome before the CLT-structure actually becomes a viable option for large-scale application in these types of projects. Plans for a CO₂-tax could significantly increase the concrete price, creating opportunity for CLT, but until that time, governmental subsidy for sustainable housing is the best method of compensating for the cost difference.

"This page is intentionally left blank"



Table of contents

Preface	iii
Abstract	v
Table of contents	vii
List of tables.....	ix
List of figures.....	xi
1. General introduction	1
1.1. Rationale.....	1
1.2. Research topic.....	4
1.3. Research objective	6
1.4. Division of theoretical chapters and working method	7
2. Literature study – How to perform a multi-criteria analysis?	8
3. Literature study - Timber vs. concrete: analysis of sustainability	9
3.1. Introduction to sustainability	9
3.2. Sustainability strategies in building projects	12
3.3. Source of raw materials.....	15
3.4. From material to member: production of structural elements	20
3.5. Application of elements: creating a structure	23
3.6. MPG (MilieuPrestatie Gebouwen)(Environmental performance buildings)	25
3.7. Environmental performance during exploitation	26
3.8. Demolition phase	29
3.9. Subjective criteria	35
3.10. Results.....	40
3.11. Quantifying sustainability through EPD's	42
4. Guideline analysis – identifying terraced housing building criteria.....	45
4.1. Introduction	45
4.2. Platform framing method	46
4.3. Limit states and load situations	47
4.4. Fire safety	58

List of tables



4.5.	Sustainability of structure	62
4.6.	Breathability and ventilation	65
4.7.	Acoustics	66
4.8.	Durability	67
4.9.	Building method.....	68
4.10.	Aesthetics	69
5.	CLT-house of the Future: A structural overview of the variants.....	70
5.1.	Minimum CLT design	70
5.2.	Variant: Hollow core CLT.....	143
6.	Results	148
6.1.	Multi criteria analysis.....	148
6.2.	Cost estimate ideal design	158
7.	Discussion.....	162
8.	Conclusion.....	165
	References.....	167
Appendix A:	Calculations minimum CLT design	A1
A.1	Assumptions	A1
A.2	Structural geometry	A2
A.3	Structural checks	A2
A.4	Additional fire safety measures	A60
A.5	Thermal insulation measures	A62
A.6	Air tightness measures	A63
A.7	External cladding	A63
A.8	Acoustic measures	A65
A.9	Ventilation measures	A70
Appendix B:	Variant – Hollow core CLT application	B1
B.1	Assumptions	B1
B.2	Principle of structural performance.....	B1
B.3	Hollow core CLT structural design	B2



List of tables

Main report

Table 1 - Material comparison	40
Table 2 - EPD eco-profile CLT vs. precast concrete	43
Table 3 - Building criteria	45
Table 4 - Reduction factors, source: NEN-EN 1990 NB Table NB.2 - A1.1	48
Table 5 - Reaction to fire classification.....	60
Table 6 - Sound insulation requirements	66
Table 7 – Strength check vertical rod.....	75
Table 8 – Strength check horizontal rod.....	75
Table 9 – Material input ANSYS workbench.....	81
Table 10 - FEM-results	133
Table 11 – MCA result.....	157
Table 12 - Area, thickness, volume and mass per CLT member-type.....	158
Table 13 - Cost comparison CLT and prefab concrete.....	161

Appendices

Table A1 – $C_{pe,10}$ -values, source: NEN-EN 1991-1-4 Table NB.7-7.2	A3
Table A2 – $C_{pe,10}$ -values, source: NEN-EN 1991-1-4 Table NB.6-7.1	A3
Table A3 – Strength check roof slab.....	A9
Table A4 – ψ -factors for buildings	A11
Table A5 – Strength check under fire conditions	A12
Table A6 – Instantaneous deflection of roof slab	A13
Table A7 – Long-term and maximum deflection of roof slab	A14
Table A8 – Strength check floor slab.....	A19
Table A9 – Strength check floor slab under fire conditions.....	A21
Table A10 – Instantaneous deflection of floor slabs	A22
Table A11 – Long-term and maximum deflection of floor slabs	A22
Table A12 – Floor vibration criteria; source: subtask 7 for prEN16351 Table 9.2.....	A23
Table A13 – Factor $k_{c,1}$ for fundamental frequency of two-span floor; source: subtask 7 for prEN16351 Table 9.2	A23
Table A14 – Fundamental frequency floor slab.....	A24
Table A15 – Deflection due to 1kN point load halfway the span	A24
Table A16 – Floor vibration velocity.....	A25

List of tables



Table A17 – Additional vibration control design method	A25
Table A18 – Load-bearing wall tension check	A35
Table A19 – Load-bearing wall compression check	A35
Table A20 – Slenderness and k-factors for load-bearing wall	A36
Table A21 – Buckling strength check for load-bearing wall	A37
Table A22 – Strength check for load-bearing wall under fire conditions	A39
Table A23 – Spacing requirements VGS fasteners, source: Rothoblaas	A51
Table B1 – Cross-sectional properties hollow core CLT	B3
Table B2 – Strength check characteristic load situation, hollow core CLT	B4
Table B3 – Deflection check characteristic load situation, hollow core CLT	B4
Table B4 – Strength check under fire conditions, hollow core CLT	B5



List of figures

Main report

Figure 1 - a. Manufacturing of CLT, b. Anisotropy in CLT, source: http://www.structuremag.org/?p=10054	3
Figure 2 - Pilot project Woodteq bv.	5
Figure 3 - a. Forest, b. Aggregate mine, c. Pumpjack at an oil well, sources: https://www.geospatialworld.net/news/mda-provide-operational-forest-change-detection-alerts-app/ , https://www.armofmn.com/mining , http://www.imeche.org/news/news-article/8-questions-fossil-fuels	10
Figure 4 - Optimal material choice regarding fuel consumption	14
Figure 5 - Photosynthesis and respiration, source: https://lifescitpjhs.wikispaces.com/Photosynthesis+%26+Respiration	16
Figure 6 – Timber material flowchart	21
Figure 7 - Aggregates in concrete	22
Figure 8 - Material life cycle	33
Figure 9 - Aesthetics, a. concrete, b. CLT; sources: https://cdn.trendir.com/wp-content/uploads/old/house-design/assets_c/2015/07/house-with-raw-concrete-living-room-6-thumb-1200xauto-54404.jpg , https://i.pinimg.com/originals/98/f4/38/98f43898c1cb6654a10b2f2b60fee105.jpg ...	36
Figure 10 - Link between nature and health; source: (Health Council of the Netherlands and Dutch Advisory Council for Research on Spatial Planning Nature and the Environment, 2004)	37
Figure 11 - Perceived attributes of wood versus other natural furnishing materials (1 = material possesses attribute; -1 = material lacks attribute); source: (Rice et al., 2006)	39
Figure 12 - Life cycle analysis stages, source: (Egoïn, 2018)	42
Figure 13 - a. Platform framing, b. Balloon framing	46
Figure 14 - Variable loads on platform framed structure	49
Figure 15 - Simply supported beam including shear- (V) and moment line (M)	50
Figure 16 – wall-floor-connection; a. under compression, b. under tension	51
Figure 17 – a. wind load distribution over height of building, b. wind load on floor slab	52
Figure 18 - Horizontal load on stabilizing wall	53
Figure 19 - Vertical deflections; source: NEN-EN 1990	55
Figure 20 - Horizontal displacements	56
Figure 21 - Fire triangle. source: https://www.elitefire.co.uk/help-advice/basics-fire-triangle/	59
Figure 22 - Loads per row of terraced houses	71
Figure 23 – Framing approach of shear wall with hole	71
Figure 24 – a. Internal and external forces in framework, b. axial force, shear force and bending moment diagram	73

List of figures



Figure 25 – Bending moment diagram (Dr. Frame).....	73
Figure 26 – Isolated members in frame (Dr. Frame).....	74
Figure 27 - CLT-node schematization.....	76
Figure 28 - In-plane shear failure modes; a. gross cross section failure parallel to grain, b. net cross section failure perpendicular to grain, c. torsional failure in lamination layers. Source: (Flaig & Blaß, 2013), edited by author	77
Figure 29 - Contact area; loads and stresses	80
Figure 30 - Layered input SpaceClaim, ANSYS.....	82
Figure 31 - Hole-less CLT slab ANSYS.....	82
Figure 32 - Material assignment ANSYS.....	83
Figure 33 - Contact areas ANSYS	84
Figure 34 - Merged contact areas.....	84
Figure 35 - Mesh input ANSYS.....	85
Figure 36 - Boundary conditions ANSYS.....	86
Figure 37 - Axial stress in x-direction in the horizontal (middle) layer, no hole.....	87
Figure 38 - Axial stress in z-direction in the vertical (edge) layers, no hole	87
Figure 39 – Shear stress in x-direction on the z-face in the horizontal (middle) layer, no hole	89
Figure 40 – Shear stress in x-direction on the z-face in the vertical (edge) layers, no hole	89
Figure 41 – Frictional stress in lamination layer, no hole	90
Figure 42 - Shear stress in y-direction on the z-face in the horizontal layer, no hole	90
Figure 43 - Shear stress in y-direction on the x-face in the vertical layer, no hole	91
Figure 44 - Deformation, multiplied 150x, no hole.....	91
Figure 45 - Geometry slab with hole.....	92
Figure 46 - Axial stress in x-direction in the horizontal layer, 3900x1500mm hole.....	93
Figure 47 - Axial stress in z-direction in the vertical layers, 3900x1500mm hole	93
Figure 48 - Local stress peaks, a. tension, b. compression, 3900x1500mm hole.....	94
Figure 49 - Shear stress in x-direction on the z-face in the horizontal layer, 3900x1500mm hole.....	94
Figure 50 - Shear stress in x-direction on the z-face in the vertical layer, 3900x1500mm hole	95
Figure 51 - Detail shear stress, 3900x1500mm hole.....	95
Figure 52 - Frictional stress in lamination layer, 3900x1500mm hole	96
Figure 53 - Frictional stress detail, 3900x1500mm hole	97
Figure 54 - Shear stress in y-direction on the z-face in the horizontal layer, 3900x1500mm hole.....	97
Figure 55 - Shear stress in y-direction on the x-face in the vertical layer, 3900x1500mm hole.....	98
Figure 56 - Deformation, multiplied 24x, 3900x1500mm hole	98
Figure 57 - Dr. Frame calculation for hole at bottom of the slab	99
Figure 58 - SpaceClaim geometry, 3900x1500mm hole at bottom of slab	100
Figure 59 - Axial stress in x-direction in the horizontal layer, 3900x1500mm hole at bottom of slab	101



Figure 60 - Axial stress in z-direction in the vertical layers, 3900x1500mm hole at bottom of slab	101
Figure 61 - Shear stress in x-direction on the z-face in the horizontal layer, 3900x1500mm hole at bottom of slab	102
Figure 62 - Shear stress in x-direction on the z-face in the vertical layer, 3900x1500mm hole at bottom of slab	102
Figure 63 - Detail shear stress, 3900x1500mm hole at bottom of slab	103
Figure 64 - Frictional stress in lamination layer, 3900x1500mm hole at bottom of slab	103
Figure 65 - Frictional stress detail, 3900x1500mm hole at bottom of slab	104
Figure 66 - Shear stress in y-direction on the z-face in the horizontal layer, 3900x1500mm hole at bottom of slab	104
Figure 67 - Shear stress in y-direction on the x-face in the vertical layer, 3900x1500mm hole at bottom of slab	105
Figure 68 - Deformation, multiplied 41x, 3900x1500mm hole at bottom of slab	105
Figure 69 - Geometry, centered door hole.....	106
Figure 70 - Deformation, multiplied 44x, centered door hole	107
Figure 71 - Deformation, multiplied 55x, centered door hole, "compression only support" inserted ...	107
Figure 72 - Axial stress in x-direction in the horizontal layer, centered door hole, "compression only support" inserted.....	108
Figure 73 - Axial stress in z-direction in the vertical layers, centered door hole, "compression only support" inserted.....	109
Figure 74 - Shear stress in x-direction on the z-face in the horizontal layer, centered door hole, "compression only support" inserted.....	109
Figure 75 - Shear stress in x-direction on the z-face in the vertical layer, centered door hole, "compression only support" inserted.....	110
Figure 76 - Shear stress detail, a. horizontal layer, b. vertical layer; centered door hole, "compression only support" inserted.....	110
Figure 77 - Frictional stress in lamination layer, centered door hole, "compression only support" inserted.....	111
Figure 78 - Frictional stress detail, centered door hole, "compression only support" inserted	111
Figure 79 - Shear stress in y-direction on the z-face in the horizontal layer, centered door hole, "compression only support" inserted.....	112
Figure 80 - Shear stress in y-direction on the x-face in the vertical layer, centered door hole, "compression only support" inserted.....	112
Figure 81 - Geometry, "door + window" variant.....	113
Figure 82 - Deformation, multiplied times 53, "door + window" variant, wind from left	114
Figure 83 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from left.....	114
Figure 84 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from left.....	115

List of figures



Figure 85 - Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from left.....115

Figure 86 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from left.....116

Figure 87 - Frictional stress in lamination layer, "door + window" variant, wind from left116

Figure 88 - Frictional stress detail, "door + window" variant, wind from left117

Figure 89 - Shear stress in y-direction on the z-face in the horizontal layer, "door + window" variant, wind from left117

Figure 90 – Shear stress in y-direction on the x-face in the vertical layer, "door + window" variant, wind from left.....118

Figure 91 - Deformation multiplied times 33, "door + window" variant, wind from right119

Figure 92 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from right..120

Figure 93 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from right.....120

Figure 94 - Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from right.....121

Figure 95 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from right.....121

Figure 96 - Shear stress detail in vertical layer, "door + window" variant, wind from right122

Figure 97 - Frictional stress in lamination layer, "door + window" variant, wind from right122

Figure 98 - Frictional stress detail at the top of the door, "door + window" variant, wind from right ...123

Figure 99 – Deformation multiplied times 55, "door + window" variant, wind from right, incl. extra tensile connector124

Figure 100 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from right, incl. extra tensile connector125

Figure 101 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from right, incl. extra tensile connector125

Figure 102 - Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from right, incl. extra tensile connector126

Figure 103 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from right, incl. extra tensile connector126

Figure 104 - Frictional stress in lamination layer, "door + window" variant, wind from right, incl. extra tensile connector127

Figure 105 - Frictional stress detail at top of window, "door + window" variant, wind from right, incl. extra tensile connector127

Figure 106 - Shear stress in y-direction on the z-face in the horizontal layer, "door + window" variant, wind from right, incl. extra tensile connector.....128



Figure 107 - Shear stress in y-direction on the x-face in the vertical layer, "door + window" variant, wind from right, incl. extra tensile connector	128
Figure 108 - Deformation, multiplied by 24, "door + window", wind from right, elastic supports and stiffness factors inserted	129
Figure 109 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted	130
Figure 110 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted	130
Figure 111 - Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted	131
Figure 112 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted	131
Figure 113 - frictional stress, "door + window" variant, wind from right, elastic supports and stiffness factors inserted	132
Figure 114 - Shear stress in y-direction on the z-face in the horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted	132
Figure 115 - Shear stress in y-direction on the x-face in the horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted	133
Figure 116 - Disregarded contact areas in hand-calculation	135
Figure 117 - 3d overview of two terraced houses	136
Figure 118 - Suggested area plan for ground floor, including section denotation	137
Figure 119 - 3d view of sectioned structures, including detailing denotation	137
Figure 120 - Suggested area plan for 1 st and 2 nd floor.	138
Figure 121 - Detail 1 in Figure 116	139
Figure 122 - Detail 2 in Figure 116	140
Figure 123 - Detail 3 in Figure 116	140
Figure 124 - Detail 4 in Figure 116	141
Figure 125 - Detail 5 in Figure 116	141
Figure 126 - Detail 6 in Figure 116	142
Figure 127 - Detail 7 in Figure 116	142
Figure 128 - Additional measures in intermediate floor/load-bearing wall connection	143
Figure 129 - Hollow core CLT design for floor- and roof slabs	144
Figure 130 - HCCLT roof slab incl. insulation and vapour barrier	146

Appendices

Figure A1 - 3d sketch of structure	A1
Figure A2 - Loads in structural members of terraced house	A4

List of figures



Figure A3 – Wind zones roof slab, source: NEN-EN 1991-1-4 Figure 7.6	A5
Figure A4 – Wind zones walls, source: NEN-EN 1991-1-4 Figure 7.5	A5
Figure A5 – a. self-weight + constant live load, b. self-weight + wind load.....	A6
Figure A6 – Diaphragm action of roof; a. wind on side façade, b. wind on front-/back facade.....	A15
Figure A7 – Load distribution in floor diaphragm, source: prEN16351.....	A16
Figure A8 – Internal stresses per contact area in CLT	A16
Figure A9 – Stress orientation in CLT, source: prEN16351	A17
Figure A10 – Holes in floor slabs.....	A20
Figure A11 – Diaphragm action floor; a. wind on side façade, b. wind on front-/back facade.....	A27
Figure A12 – FE-models of beams and walls, source: (Shahnewaz et al., 2015).....	A29
Figure A13 – Diaphragm with holes in slab; a. wind on side façade, b. wind on front-/back facade ...	A30
Figure A14 – Floor diaphragm detail.....	A31
Figure A15 – Detail roof/wall connection	A33
Figure A16 – Detail wall/foundation connection	A34
Figure A17 – Considered fire situation	A38
Figure A18 – Wind load on shear wall.....	A40
Figure A19 – Detail roof/wall connection 2	A42
Figure A20 – Load distribution width and dispersion length, source: prEN16351	A43
Figure A21 – Detail intermediate floor/wall connection.....	A44
Figure A22 – Longitudinal joint options in CLT; a. leaf joint, b. tenon joint, c. half-lapped joint.....	A47
Figure A23 – Longitudinal joint stresses; a. due to sliding (shear), b. due to in-plane moment.....	A49
Figure A24 – Longitudinal leaf joint between two horizontal (floor- or roof) slabs, dimensions in mm	A50
Figure A25 – Dowelled leaf joint.....	A52
Figure A26 – a. Screwed roof/wall connection, b. view A-A.....	A53
Figure A27 – a. Dowelled roof/wall connection, b. view A-A.....	A54
Figure A28 – a. Side view screwed wall/floor connection, b. front view.....	A56
Figure A29 – a. Side view dowelled wall/floor connection, b. view A-A in a.....	A57
Figure A30 – a. Side view of screwed floor/front facade connection, b. view A-A in a., c. view B-B in a.	A58
Figure A31 – a. Side view of dowelled floor/front facade connection, b. view A-A in a., c. view B-B in a.	A59
Figure A32 – Transmission loss for 160mm CLT	A65
Figure A33 – Transmission loss for 98mm CLT	A67
Figure A34 – Contact noise propagation through house-separating wall composition.....	A69
Figure B1 – Hollow core CLT composition.....	B2
Figure B2 – Hollow core cross-section under fire conditions.....	B5

"This page is intentionally left blank"



1. General introduction

1.1. Rationale

In 2016, Dutch residential real estate had a total estimated value of 1.7 billion Euros, which is more than three times the gross domestic product (GDP) (Government of the Netherlands, 2015). This total value is expected to rise even more, since the forecasted number of households should expand to reach almost 8.400.000 by 2030, whereas this number was approximately 7.750.000 in 2016. This development is confirmed when consulting available statistics in the Statline database of the CBS (Centraal Bureau Statistiek) (statistics institute in the Netherlands).

With the increased demand for housing, residential investment will become a growing contributor to the GDP for years to come, and cities will most likely need to expand in both area and height to make room for the expanding number of households. This will lead to an increase in land value, additionally causing house prices to rise. The scale of building projects and their corresponding material usage will grow alongside the growing demand for new residences, as mass production (of houses, apartments, etc.) is often economically friendlier than designing and building multiple smaller projects. This should drive the prices of houses in cities and suburban areas back down towards a more respectable level. During the past couple of decades, environmental issues have become a bigger cause for concern around the globe, with global warming being the leading topic in this discussion. Intergovernmental organizations like the United Nations have continuously been setting new goals to push the sustainable development of member states forward, with the approach that trying to change the world must start by making many small (local) changes which together will eventually lead to a bigger, more global goal. These changes can be subdivided into three categories; individual (affecting households), governmental (affecting streets and other public places) and corporate. This means they will have a certain impact in almost all aspects of daily life, and no individual will be able to shy away from the issue. The new global goals regarding sustainability and the following corporate changes also have their effect on the building sector. Developers need to pay more attention to which materials and how much of these materials they use. The final design of a project including a phasing/transportation schedule can give a clear insight in the total impact the project has on the environment, but some pieces

1. General introduction



of knowledge still need to be put together to create a clear overview of what building method should work best in a certain case.

In the Netherlands, concrete is the most used- and therefore most important building material for these kinds of projects. Because most projects are executed with this material, this also leads to much research being done to improve the performance and application, which further strengthens its leading position in the market compared to other materials. Concrete has many advantages when considering the structural performance and building physics (thermal mass, durability, acoustics and water resilience), and the development of higher strength classes of concrete allows for bigger spans and more slender designs. However, the environmental impact of concrete is more complex. Concrete looks like a homogeneous, stone-like material, but it consists of various components with different characteristics, production processes and natural resources, with cement being one of the primary ingredients. Back in 2002 it was the second most consumed substance on earth after water, with almost three tons used per inhabitant on this planet (World Business Council for Sustainable Development, 2002). As cement is the primary ingredient of concrete this means it is produced in enormous amounts, accounting for approximately 5% of the global carbon dioxide (CO₂) emission. Producing a ton of cement generally generates 900 kg of CO₂ (Rubenstein, 2012). Besides the emissions, the production process of cement is energy intensive, as extreme heat is required, leading to a relatively high value of the embodied energy (energy used to transform raw materials to building products). On the other hand, the aggregates in concrete can be excavated from earth directly and don't require an extra production process (Struble & Godfrey, 2004). Mining is required to execute this excavation, demanding more energy, and the aggregates are not renewable meaning depletion of the mines might occur at some point in time.

To save time in big terraced housing projects, contractors often opt for prefabricated concrete instead of in situ. The repetition factor is high because all houses consist of the same structure, meaning the production of slabs in a factory is a lot more efficient than pouring the concrete on site and having to wait for the concrete to set before moving on with the building process.

When looking for more sustainable and renewable building materials compared to concrete, timber is often first mentioned. If harvested correctly with sustainable forest management, timber can fulfil the material requirements where concrete has its shortcomings. Timber is lightweight meaning it will not



require very heavy machinery on the building site. It does however require a number of production steps before wood becomes a basic timber building product like boards or columns. These products can then be processed further to create engineered products like cross laminated timber (CLT) for an even wider field of application. CLT consists (as its name indicates) of boards of timber which are cross-wise laminated on top of each other (Figure 1a). They always contain an odd number of layers, so there is always a stronger and a weaker direction within a CLT-slab (Figure 1b). This timber product, its structural characteristics and its application method come closest to that of prefabricated concrete and should therefore be a very suitable choice to build terraced houses with. However, timber is an organic material, which might lead to problems regarding building physics. One should investigate whether a CLT house can meet requirements regarding i.a. fire safety, thermal insulation, sound insulation and natural/biological hazards.

Both concrete and timber have advantages and disadvantages, and a complete overview of their performance in housing projects seems to not be present in the current literature. Such an overview could persuade contractors to choose one building material over the other, and therefore may have a positive influence on the current environmental issues.

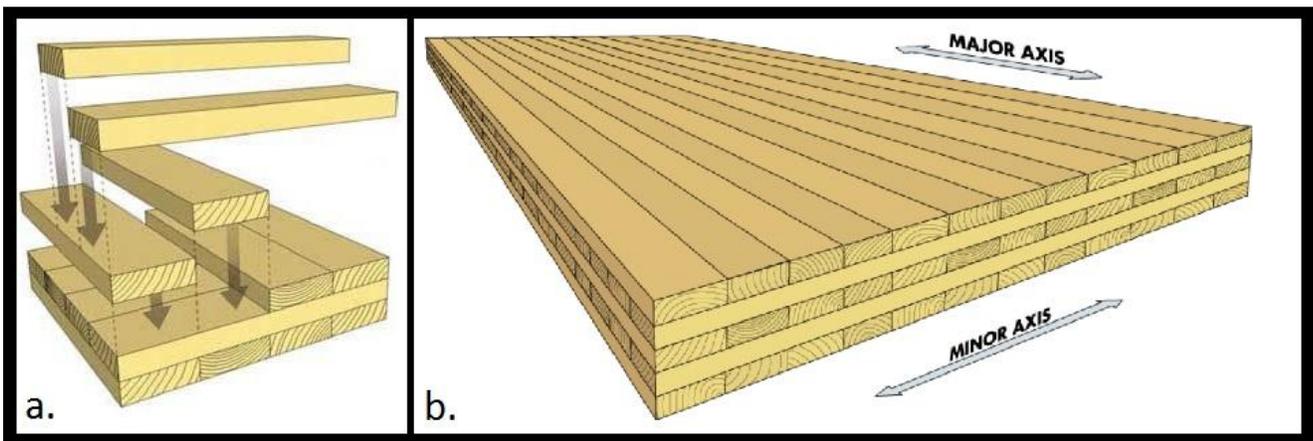


Figure 1 - a. Manufacturing of CLT, b. Anisotropy in CLT, source: <http://www.structuremag.org/?p=10054>

1. General introduction



1.2. Research topic

For special projects, initiators and contractors have started to shift to timber building as the primary building method with the goal of building in a more sustainable way. Examples of such projects are Patch 22 and Hotel Jakarta, both located in Amsterdam, the Netherlands. If timber really is a more sustainable material than concrete, the choice for timber as the primary material for more generic, large-scale projects should become the standard. In the Netherlands, most of these projects revolve around the building of low-rise terraced houses. These houses form most of the suburban areas of the bigger cities and villages. Till now these projects are executed with prefabricated reinforced concrete slabs as the primary structural material. The most obvious timber product to replace these concrete slabs is CLT. In this thesis the best methods to implement timber in common building projects will be investigated with the main goal to create a more sustainable solution for each structural challenge. This is done through a multi-criteria analysis (MCA) of a CLT terraced housing structure compared to the generic prefabricated concrete structure.

One critical point in need of mentioning prior to performing the research is that the current CLT price cannot compete with that of concrete. Additionally, compared to the regular CLT structure the aim is to economically optimize it to create a very attractive alternative to prefabricated concrete structures, both pricewise as regarding sustainability. If we succeed, and the solution can be applied widely throughout the Netherlands, or even the world, this would hopefully mean the total CO₂ footprint of the entire building sector decreases significantly.

The research will consist of an analysis of the sustainability characteristics of timber compared to concrete, and secondly a multi-criteria analysis of how these materials form a structure which meets all requirements from Eurocodes and the Dutch Building Decree. Investigated criteria will include safety, manufacturability and durability, amongst others.

So, the first step in this thesis is a qualitative analysis of the sustainability over the total life cycle of timber and concrete to quantify the differences between these materials. This analysis is performed qualitatively, since it is either too complex to quantify a sustainability characteristic or it requires too extensive research within the scope of this thesis. In this part all steps in the life cycle of these materials will need to be examined, from the origin of components till the degradation and demolition of the



material. After completion of the analysis for both materials a clear visual should be created of how much more sustainable timber is than concrete.

Afterwards, the most economical method to apply timber as the primary material of the structure of a low-rise house is inspected (3 storeys). The structural optimization will be done on the design for a pilot project by Woodteq bv (Figure 2), but the principle requirements and calculation methods are also applicable for many other designs of terraced houses. Solutions are examined with existing and new timber products to limit the total usage of timber and additional (maybe less sustainable) materials. An important aspect in this optimization is the anisotropic behaviour of timber, which creates challenges regarding stability, strength and stiffness in multiple directions. Examples of existing timber products are glued laminated timber, cross laminated timber and laminated veneer lumber, as well as conventional solid timber boards, columns and beams. Depending on the design the existing products may need adaptation to improve design to its fullest potential.



Figure 2 - Pilot project Woodteq bv.

1. General introduction



1.3. Research objective

The main objective of this thesis is to investigate the feasibility of the application of cross-laminated timber (CLT) in repetitive, low-rise terraced housing projects. In this research, this feasibility is split into three parts; sustainability, structural safety and functionality. Conventionally, these different parts in projects are administered individually. However, an integral, multidisciplinary approach incorporates the characteristics of every structural element in different building criteria to create an ideal, minimal design. An improvement of the sustainability of a prefabricated concrete terraced house is the starting point of this project for Woodteq bv. First, the sustainability of CLT must be (qualitatively) compared to the sustainability of prefabricated concrete to substantiate a better performance. This thesis is aimed towards the future of the building industry, meaning an expectation for forthcoming changes in assumptions and building criteria could alter the outcome between the present optimal solution and a future solution. This expectation is complex to formulate as it is never certainly known. Assumptions with regard to the future can only be made with sufficient substantiation. The main objective is reached through the answering of the main research question:

What is the feasibility of application of cross-laminated timber (CLT) in repetitive, low-rise housing projects?

Sub-questions are defined to show the division of the main objective into the described parts:

1. *How does the sustainability of CLT compare to the sustainability of prefabricated concrete?*
2. *Which structural requirements does a CLT design have to meet to meet safety and functionality standards?*
 - 2.1. *How do these requirements influence the minimal CLT design?*
 - 2.2. *Which structural members provide the stability for the system in case of horizontal loads? What does the design of these stabilizing members look like?*
3. *Which additional measures are required according to building physics requirements to create a fully functional terraced house?*
4. *Can an alternative engineered timber slab allow a saving of material compared to the CLT design when designed according to safety requirements? How does this alternative perform considering other building criteria?*



1.4. Division of theoretical chapters and working method

The second chapter contains a concise introduction and description of a multi-criteria analysis (MCA). It is important to understand the different steps of this decision-making method before focussing on the theory of the research topic. Guidelines on how to perform an MCA are widely available in various sources dedicated to the description of working methods.

The third chapter, on sustainability of timber and concrete, consists of a literature study. A part of this study covers biological topics, like the natural production of wood (growth in forests), the forming of aggregates (stones, sand), and the influences of such processes on the environment. Another part of this chapter covers the production of structural elements from raw materials, including the addition of additional materials to e.g. connect elements to form a structure. Information for this literature study is gathered from books, articles, governmental reports and datasheets. If additional information is required which cannot be acquired through readings, experts in the concerning field of study will be contacted for further help.

The fourth, fifth and sixth chapters regard the main structural engineering part of this thesis. Chapter four is made up of a guideline and standard analysis for the design of a low-rise, terraced housing structure. The most important sources for this analysis are provided by NEN and Rijkswaterstaat, in the form of Eurocodes and the Building Decree, respectively.

Chapter five contains the overall structural design including an elaboration of the critical structural engineering (SE) problems. This chapter is supplemented by Appendices A and B, and created by applying the authors knowledge on SE, gathered throughout the MSc Structural Engineering at the Technical University in Delft. As an aid in the SE-research, software package ANSYS will be used to perform finite element analyses (FEA).

Lastly, chapter six involves the application of the MCA in the decision making for the “CLT-House of the Future”, by following the working method described in Ch. 2.

2. Literature study – How to perform a multi-criteria analysis?



2. Literature study – How to perform a multi-criteria analysis?

Before executing a multi-criteria analysis (MCA), research is required on what an MCA actually is. In which situations is this type of analysis best applied? With which approach or method can a goal be achieved? What exactly is the goal of an MCA?

Multi-criteria analysis provides a method to appraise different options in a decision-making problem (Department for Communities and Local Government, 2009). In a structural problem, MCA is a way to choose between multiple variants for the same type of structural member. E.g. it provides a method to choose between two different floor types, based on criteria like material type, amount of material used and building physics. Thus the two conditions to perform an MCA in a structural problem are the existence of multiple building criteria to base a decision upon, and the presence of multiple variants to allow for a comparison which eventually results in the most suitable solution.

The overall goal of an MCA is to choose the “best” option for a problem. This “best” choice may be partly based on objective criteria, but subjective weight factors need to be attached to these criteria to enable an accumulation of the criteria into a quantified comparison. The result is a weighted outcome, desirably expressed as a singular entity. To successfully achieve the goal of an MCA, the following steps should be fulfilled:

1. Identify objectives (Ch.0)
2. Identify the criteria for the comparison of variants (Ch.4)
3. Identify variants to solve the objectives (Ch.5)
4. Identify weight factors for the criteria (Ch.6.1)
5. Analyse the variants on each criteria (Ch.6.1)
6. Make decision with substantiated feedback (Ch.6.1)



3. Literature study - Timber vs. concrete: analysis of sustainability

3.1. Introduction to sustainability

Sustainability is the principle of preservation of the planet by harmonizing the balance between the exploitation of- and the investment in or preservation of natural resources. This balance can also be described as environmental homeostasis. A distinction can be made between three types of resources based on the time it takes for the source to "restock" after exploitation:

- Short refill time; considered as renewable and infinitely available if managed correctly
- Long refill time; finite resources, are susceptible to depletion
- Near-infinite refill time; non-renewable materials, totally unsuitable for extraction from nature

In a utopia, humans would invest just as much into natural resources as they would exploit it, leading to circularity regarding material- and energy consumption. This is the basis of a recent development called "circular economy", where total reduction of natural extraction is the main goal, through the reuse of products. This term will be elaborated in Ch.3.8.2.

The investment of humans in the natural resources requires both space and time, which directly composes the main issue in this economically driven society where most individuals are constantly competing with one another to create a profit for themselves. Land is more often reserved for human activities and exploitation than for resources to restock, and not enough people have the time or refunds to be able to give back to nature. In this regard an individual might consider the impact he/she has on the environment as very small, but the total impact of humankind is undeniable, so where does one need to start to address the issue?

For resources with short refill times, investing as much as is exploited can be done relatively simply. Consider a material like wood (Figure 3.a): the time for trees to reach canopy height is 30-40 years at most for slow growing trees. One of the main functions of wood is the use in building projects as structural timber. These projects are often designed for 50 to 100 years, but the real structural age before demolition may and probably will differ from this design life span, dependent on several factors. The design life only guarantees a certain reliability for the calculations performed on the basis of the Eurocode for this period of time. The factors which eventually affect the real age of a structure include changes in the land-use plan for that area, a change in requirements from the owner or whether

3. Literature study – Timber vs. Concrete: analysis of sustainability



the structure gathers some type of monumental value during its lifespan. It is difficult to predict the age of a structure before it is built and exploited, but for economic reasons the assumption can be made that it exceeds the maximum of 40 years it takes for trees to grow to canopy height. Therefore, new trees can grow at the location where other trees were cut down for structural timber purposes, making the material renewable for the function of building material. The only condition is that new trees are actually planted after harvesting, since it is alluring to use an area of harvested forest for human activities like housing or leisure.

When the restocking time of a material becomes longer, much more investment is required to achieve the “restocking” of a source. E.g. stones (Figure 3.b) are mined from rock formations to i.e. be used as aggregate in concrete. The formation of different rock types takes thousands of years, meaning if excavation occurs at a certain location this needs to be compensated in another location or at a later stage in the same location, by letting these locations rest for the next centuries. However, thousands of years is a long time compared to a human life, and given the high demand for rocky materials, restocking seems very difficult, if not impossible. Managing of these types of resources is an even bigger challenge than for resources with shorter refilling times and needs to be handled on a bigger scale. This bigger scale spans both time and space, since the issue demands attention between multiple countries/continents and multiple generations.



Figure 3 - a. Forest, b. Aggregate mine, c. Pumpjack at an oil well, sources: <https://www.geospatialworld.net/news/mda-provide-operational-forest-change-detection-alerts-app/>, <https://www.armofmn.com/mining>, <http://www.imeche.org/news/news-article/8-questions-fossil-fuels>



Lastly, fossil fuels are an example of completely non-renewable materials (Figure 3.c). These fuels are formed through natural processes over millions of years. They literally leave a void in the earth which cannot be filled. However, the various uses (especially for energy) and the added economic value push people to still delve them from the soil.

To create a structure, it seems obvious to only make use of renewable materials. When looking at Figure 3, the question can be asked which image is most preferable. Whichever the answer, applying renewable materials is much easier said than done, since functionality is top priority. A clear image of the accumulated impact on environmental issues should be created for newly designed structures, which may lead to adaptations to create a more sustainable structure. This accumulated impact includes:

- Design and construction phase
 - Raw material extraction (Ch. 3.3)
 - Production of structural members (Ch.3.4)
 - Assembly of members (incl. transportation)(Ch.3.5)
- Exploitation phase (Ch.3.7)
- Demolition phase (Ch.3.8)

3. Literature study – Timber vs. Concrete: analysis of sustainability



3.2. Sustainability strategies in building projects

The sustainability during the lifespan of a building can be split into the three previously mentioned phases in which strategies can be applied to address the effects on climate change; the energy consumption and material usage required to build the structure (including building material production)(design phase), the energy consumption during the exploitation phase (total energy demand for the use of the structure over its entire lifespan) and the possibility of reusing or recycling structural members and materials in the demolition phase. The list of phases from the previous paragraph can be expanded with their respective influences on the climate:

- Design and construction phase
 - Raw material extraction
 - Energy consumption for extraction
 - Effect of material on climate
 - Production of structural members
 - Energy consumption for transportation
 - Effects of (harmful) additives
 - Assembly of members (incl. transportation)
 - Energy consumption for transportation
 - Energy consumption for lifting
- Exploitation phase
 - Energy consumption for maintenance
 - Energy consumption for lighting, heating, etc.
- Demolition phase
 - Energy consumption for replacing (transporting/lifting) elements
 - Energy consumption for breaking down elements to recycle
 - Cutback on material usage of the next project in which the materials are reused

The distinction in three categories is sensible because of the different aspects and effects of climate change that come into play in each phase. Energy consumption, especially fossil fuel consumption, has a direct negative effect on the environment through the emission of greenhouse gasses. This negative effect should preferably be reduced fully, or at least reduced to a minimum. However, a total reduction



of effects from building material production is not achievable, since certain dimensions of a structure are always required to meet the standards from Eurocodes and the Dutch Building Decree. There is some mitigation of material consumption possible by creating the “minimum design”; a design as efficient as possible within the scope of these requirements. The effects of reusing and recycling at the end of a structures lifespan are always positive, as it ensures less production of new elements is required to build a new structure. This effect is an important building block for the total sustainability of a project, and may be the deciding factor on whether this project is considered “sustainable”.

The choice of a certain building material has influence on all three previously mentioned phases of the building process. Every building material has its own characteristics regarding structural functionality, weight and environmental effects from production. The structural functionality in time and the possibility for disassembly without destruction eventually determines whether a material or element is suitable for reapplication.

In Ch.3.1, fossil fuels were described as non-renewable, meaning the consumption of them needs to be minimized, preferably to zero. The primary method to achieve this goal is the reduction of the total demand for energy. One of many strategies to achieve this is a decrease of the weight of- and distances travelled by transported materials and elements, by selecting building materials with lower densities and by selecting producers of structural elements that are located close to the building site (

Figure 4). Alongside the reduction strategy for the energy demand, the remaining energy demand from non-renewable sources should be replaced with sustainable and renewable forms of energy, like wind-, solar- and wave energy.

A share of the fossil fuel consumption depends on the distance of the material plant/factory from the building site and by the weight of the material (

Figure 4). A material that weighs less than an alternative also requires lighter lifting machinery and lighter means of transportation, consuming less energy to put the elements in their place. Additionally it provides an easier working method, as the elements are handled more easily.

3. Literature study – Timber vs. Concrete: analysis of sustainability

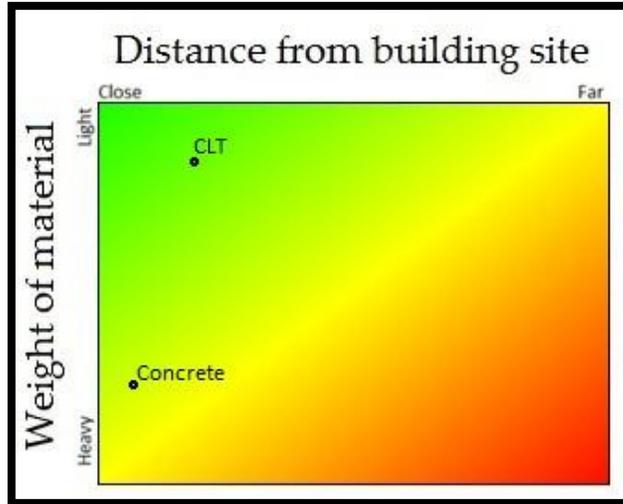


Figure 4 - Optimal material choice regarding fuel consumption

The goal of this chapter is to create a sustainability comparison between timber and concrete, since concrete is the most conventional material to build terraced houses with in the Netherlands, and the aim of this thesis is to create an alternative to the conventional building method. The weight by volume of structural timber (spruce) is approximately 500 kg/m^3 , which is relatively low compared to the weight of concrete of approximately 2400 kg/m^3 . The distance travelled by the elements will eventually lead to one material being beneficial in terms of additional fossil fuel consumption, but timber already has an edge over concrete considering their respective weights.

Current CLT factories are mostly located in Austria, Germany and Scandinavian countries, but the closest factory is located at a distance of 200km from Amsterdam, being relatively low compared to other industries where products and materials sometimes need to travel across the world. For Dutch projects this means the distance these elements need to travel is larger than for prefabricated concrete, as there are many concrete factories in the Netherlands, and a distance of 50km is a good assumption for maximum distance. In Figure 4, it shows that the relative difference in the distance from the building sites between CLT and concrete has a smaller effect on the sustainability of the material than the weight of the material, since the weight also influences how many truckloads are required, which may also increase the total distance travelled. If the Netherlands can realize a growth of the timber structure industry, and create better logistics for timber products including a CLT factory, CLT would



make a shift even further left and the advantage of the shorter distance of concrete would be diminished.

3.3. Source of raw materials

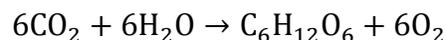
When selecting a building material, commonly chosen strategies to affect the climatic change include greenhouse gas emission reduction and creation/preservation of carbon sinks. Greenhouse gas emissions relate for the most part to the fossil fuel consumption mentioned in Ch.3.1 and Ch.3.2. A clear visual is required of the energy consumption during the production phase of building materials (see Ch.3.4) and of the impacts of the materials themselves to be able to accumulate the total environmental impact of the structure.

A carbon sink is defined as a storage which naturally absorbs carbon from the environment. Ideally, the intention is to create carbon sinks where carbon sequestration takes place, meaning carbon dioxide is absorbed from the atmosphere and converted into a non-gaseous form. Carbon sequestration is the counterpart to emission of carbon dioxide (CO₂) due to e.g. combustion of fossil fuels and thus mitigates global warming. The biggest natural carbon sinks across earth are the forests, soils and the oceans.

3.3.1. Timber

Timber as a structural material is naturally grown as wood in trees, which for a structure in the Netherlands mostly originate from European forests. Photosynthesis takes place within the cells of trees, causing the trees to grow (

Figure 5). In this process, carbon dioxide and water are converted into glucose and oxygen under the influence of light:



So, for the material to grow, carbon (dioxide) is taken from the air and “stored” in the wood as cellulose or glucose, which means forests are acknowledged as carbon sinks where sequestration occurs.

3. Literature study – Timber vs. Concrete: analysis of sustainability

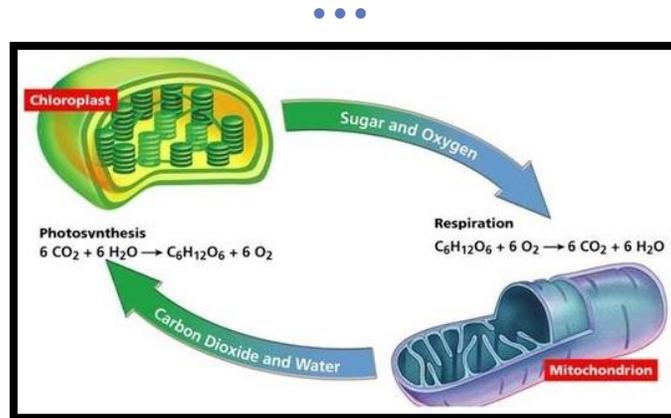


Figure 5 - Photosynthesis and respiration, source: <https://lifescitpjhs.wikispaces.com/Photosynthesis+%26+Respiration>

The mass of CO_2 absorbed from the air to produce 1 kg of tree mass is determined by considering the average chemical composition of a tree (spruce), made up out of cellulose, (types of) hemicellulose and lignin, and the volumetric mass density of the elements forming these components. The chemical composition of a tree can be converted into an elemental composition where wood contains about 50% carbon, 6% hydrogen, 44% oxygen and a trace of metal ions (Pettersen, 1984). So, to grow 1kg of wood, the tree needs to take 0,5kg C from its surroundings. Since a tree grows through the process of photosynthesis, it is assumed all this carbon is sequestered from the air in the form of CO_2 . Considering the molar masses of carbon and oxygen, this means 1,83kg CO_2 is required for the growth of 1kg of wood, accompanied by a release of 1,33kg O_2 .

At a certain stage in the life of a tree, the growth is reduced to a minimum. If a forest is left untouched for trees to age endlessly, this leads to a maximization of the total biomass stock and a saturation of the carbon sink of this forest. Trees, being living organisms, keep respiring after their growth (and therefore the photosynthesis) stagnates (Linsen, Karis, McPherson, & Hamann, 2005), resulting in a decrease of the stored CO_2 (

Figure 5). A method needs to be found to diminish the decrease and preserve the carbon stock after the stagnation of the growth of a tree.

The stored carbon in a living tree is preserved after it is harvested for its wood, and both photosynthesis and respiration come to a stop. Thus, the contribution to the total carbon sink of the tree remains intact after harvesting. To maximize the carbon sinks, the forests should be managed such that the accumulated growth will always be maximized (Gustavsson, Pingoud, & Sathre, 2006). This gives



an optimal age of trees to be cut down, under the condition that a new tree is planted in place of the harvested one. An extra challenge arises when considering the harvesting of trees for structural timber in building and infrastructure projects. Structures get designed for a certain design service life. If this design life does not coincide with the optimal tree age for harvesting with respect to the growth rate, the issue requires more optimization, because the replacement of a timber structure with a new timber structure is desired after its service life. This optimization is a research topic of its own and depends on the development of the timber market in the coming years, so this will not be further discussed in this thesis. Until this research leads to a better answer, a design service life of 50 years is generally assumed (unless stated otherwise), and thus a tree age of 50 years is considered at time of harvesting.

Lastly regarding forest management, it is essential to realize forests have other functions besides being a resource for human actions. They form a very important ecosystem for many species of plants and animals, meaning harvesting of a tree will most likely have a negative effect on flora or fauna in that area. This realization has made humankind more aware that sustainable forest management is necessary to keep the balance between three aspects; ecological, economic and cultural. If sustainable forest management is done correctly, a precise number of trees can be appointed to certain functions, leading to the preservation of all functions of the forest. Eventually, one should strive for the situation where 100% of the increment (growth) of the growing part of forests (growing stock) is harvested. This leads to a balance where ecosystems remain intact since the total forest area and volume remain constant while fulfilling all human demands at the same time.

To not waste any harvested wood, a function must be found for every part of the tree, e.g. as structural timber or as a source for biomass energy. Also, whenever the human demand increases (which can be expected if more timber projects are initiated), growing stock in European forests needs to increase equally. From 2005 till 2015 there was an annual increase of the growing stock of 1.29% (Forest Europe, 2015). The goal for this percentage should be to reach 0% by increasing the timber demand. This would create a perfect material cycle based on renewability for wood, as endless growth does not occur when forests are left untouched, and carbon sequestration comes to a halt. Increasing the demand for timber would imply the initiation of more- and bigger timber building projects. To eventually create a balance between supply and demand, a certain forest area and forest volume is required, maximizing both the

3. Literature study – Timber vs. Concrete: analysis of sustainability

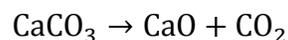


amount of harvested timber and the accumulated carbon sink of the forests and structural timber structures.



3.3.2. Concrete

Concrete is a composite material consisting of three components; aggregate (stones, sand), cement and water. The cement together with water reacts to bond the aggregates together to form a homogeneous stone-like material. This material has very favourable compressive strength, making it an ideal material for foundations of buildings. However, the tensile strength is low, meaning enhancements, like steel reinforcement bars, might be required to deal with tensile stresses and bending moments. The aggregates are extracted directly from the earth by mining, meaning there is a certain demand for (fossil) fuel to retrieve the materials, leading to emission of greenhouse gasses. The resources of aggregates are not endless, and their formation period takes around 1000 years, so they cannot be considered as renewable over the design life of our structures, causing a depletion over time. After the mining process, the aggregates only need to be sorted based on size, but no further altering is required. The aggregate is the component from which concrete gets its strength, but they require some sort of an adhesive to stick together, for which cement is used. Several types of cement are available, but Portland cement is the most common type. This cement type is a mixture of CaO, SiO₂, Al₂O₃, Fe₂O₃, H₂O and SO₃, in which CaO (calcium oxide) is the main ingredient, taking up 60% to 67% of the weight of Portland cement by the year 1996. The calcium oxide is obtained through heating calcium carbonate (CaCO₃). The heating process requires fuel combustion, emitting CO₂. The chemical reaction also releases CO₂ as a by-product:



When considering the Molar Mass of CO₂ and CaO and the fact that approximately 65% of cement consists of CaO, this results in a CO₂ emission of 0.5 tonne per tonne cement. (IPCC, 1996)

Much research is being done on ways to decrease the demand for CaO in cement to reduce CO₂ emissions. This is crucial if the concrete industry wants to stay relevant for years to come, because the concrete industry contributes a very significant part to the total global greenhouse gas emissions. In 2005 this contribution was 6% of the global CO₂ emissions (Zhang et al., 2014). When in the near future a CO₂ tax law is constituted, this could instantly push timber forward as the primary building material instead of the currently used concrete. There is much research and articles on the topic of a CO₂ tax, but whether the Dutch government actually will decide to constitute such a law remains to be seen.

3. Literature study – Timber vs. Concrete: analysis of sustainability



3.4. From material to member: production of structural elements

3.4.1. Timber

After harvesting trees from the forest, logs of wood are obtained and transported for further processing. The logs need to be converted into bigger elements like boards and planks or smaller particles and strands. These wooden elements can then be further processed into engineered timber products like glued laminated timber (GLT), cross laminated timber (CLT) and oriented strand board (OSB). These products have beneficial structural characteristics compared to regular boards, thus allowing more challenging designs.

A multiplication factor is drafted, converting the timber demand for a structure into a demand for tree biomass, considering two aspects; the moisture content difference between structural timber and wood from trees and the waste from the several harvesting steps like logging, sawing etc. This waste includes parts of the tree that cannot be harvested as logs, like tops and roots. It can be assumed that 53% of the mass of a tree (spruce) can be harvested as logs (roundwood). After the sawmill, 49% of this roundwood ends up as boards and planks, with 51% waste in the shape of chips and sawdust. Additionally, 10% of the timber elements is lost during construction and the wood mass decreases by 15% to achieve the desired moisture content (Gustavsson et al., 2006). Finally, the biomass demand will not be adjusted based on the proportion of timber to additional materials (glue) in GLT and CLT, since the percentage of glue is reduced to a minimum and therefore negligible. The total biomass needed to produce 1 kg of engineered timber product is approximately 5 kg (see Figure 6). However, the influence of by-products (waste) may not be incorporated in the sustainability of the material, and is therefore neglected in the following parts.

The overall importance of adhesives is becoming greater in timber engineering, as over two-thirds of all wood products were containing some sort of adhesive in 2011 (Pizzi & Mittal, 2011). There are several types of adhesives being applied in glued timber elements with different characteristics regarding environmental impact and structural functioning, like the resistance to heat or moisture and the shear strength of the glue layer. The biggest share of the structural adhesive market belongs to thermosetting synthetic adhesives. Historically, thermosetting glues were all based on formaldehyde. Four of the most common of these adhesives are: Urea Formaldehyde (UF), Melamine Urea Formaldehyde (MUF), Phenol Formaldehyde (PF) and Phenol Resorcinol Formaldehyde (PRF). These adhesives release



volatile organic compounds (VOC's), which are harmful and toxic to their environment (Transparency Market Research, 2014).

While these products are still available for purchase, because of the unsustainability and negative health impact, the formaldehyde content had to be lowered, resulting in an inferior performance. This eventually resulted in the development of adhesives which are free of formaldehyde. For structural timber this type of adhesive is called Polyurethane (PUR). Instead of formaldehyde, this adhesive contains isocyanate (Messmer, 2015).

With circular- and bio-economy (Ch.3.8.2 and Ch.3.8.3) in mind for the timber elements, an attempt is made to shift away from synthetic adhesives, to create a renewable and bio-degradable alternative. This has led to the emergence of soy-based adhesives, but their application remains questionable.

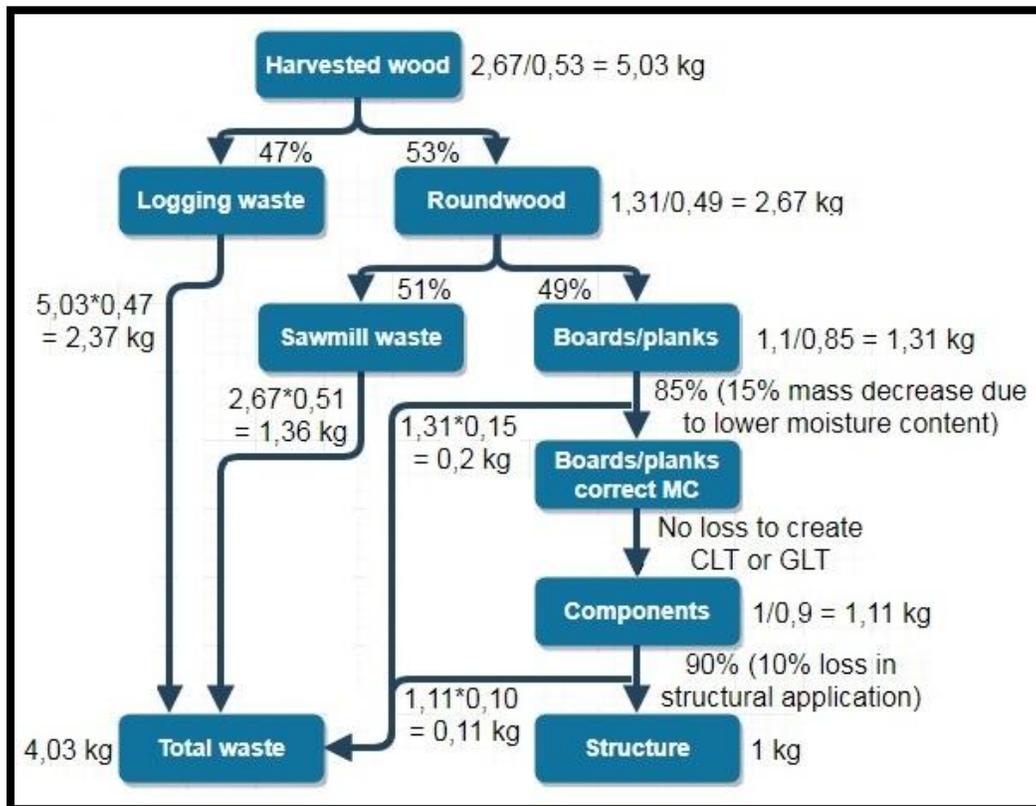


Figure 6 – Timber material flowchart

3.4.2. Concrete

Previously, the composition of concrete was described as consisting of several sizes aggregate, cement and water. The ratio between the mentioned components requires precision to create high-quality concrete, since the aggregates supply the (compressive) strength, but the cement bonds them together

3. Literature study – Timber vs. Concrete: analysis of sustainability



to create a “homogeneous” material. The word homogeneous is between quotation marks because it looks homogeneous and is considered to have homogeneous characteristics in all directions, but we now know it is actually built up out of multiple raw materials. Consider concrete with 3 different sizes of aggregate (1 being the biggest, 3 being the smallest)(Figure 7). Firstly, aggregate 1 (big stones) is distributed over the volume of the concrete. Secondly, the voids between aggregate 1 is filled with smaller aggregate 2 (e.g. smaller pebbles). Lastly, the remaining voids are filled with aggregate 3 (e.g. sand). The left-over space is filled with cement and water, bonding the aggregates together.

Generally, engineers opt to add steel reinforcement bars to the concrete mixture in places where tensile stresses or bending moments occur, to maximize the slenderness of the concrete elements. For projects with a high repetition-factor, like terraced housing projects, contractors often choose to build with prefabricated concrete slabs. These slabs are fabricated in a factory and transported to the building site where they are lifted in place. With correct preparation and phasing, this leads to a fast production of houses compared to in-situ structures.

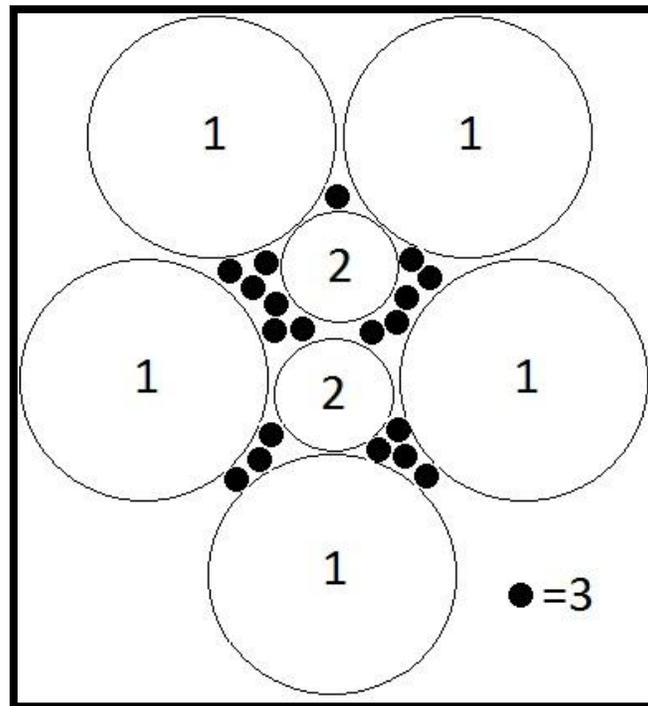


Figure 7 - Aggregates in concrete



3.5. Application of elements: creating a structure

When applying structural elements, multiple factors require attention. These are the transportation, split up into distance, complexity and weight of transport, and manufacturability on site, split up into putting elements in their place and creating the connections. Transportation and lifting has already been covered in Ch.3.2. This part focuses on the creation of correct connections, allowing the structural elements to cooperate so they can each fulfil their function. The design of the connections is therefore just as important as designing structural elements, because the weakest “link” in a structure will eventually determine structural failure.

3.5.1. Timber

Timber has a specific weight of approximately 500 kg/m^3 , which is relatively low compared to other building materials. This means the weight of transport and lifting is small, requiring less heavy machinery. Currently, most engineered timber products come from Scandinavian countries, Austria and Switzerland. This distance should be accounted for during the sustainability and economic analysis of a structure.

For the connections in timber structures in general, but also for CLT elements specifically, additional components are always required to secure the elements in place. Timber is a relatively light material compared to alternatives, meaning there may occur a resulting uplifting force on some structural elements when checking the structure in the extreme wind load situation. This vertical force tries to pull elements apart, causing tensile stresses in the connecting parts. Steel is therefore a favourable material for these connections, as its tensile strength is relatively high. Furthermore, steel connections increase the ductility of the system, making it more resistant to dynamic loads due to earthquakes or extreme fluctuating wind. The connections between the selected structural elements in the MCA need to be elaborated in a later stage. This is done in Ch.5. Amongst the most applied types of connections in CLT are (van de Kuilen et al., 2017):

- Traditional
 - Screwed connections
 - Steel hangers (concealed and in-sight)
 - Angle- and plate brackets

3. Literature study – Timber vs. Concrete: analysis of sustainability



- State of the art
 - Spider connectors
 - Bridge connectors
 - X-Rad

It is up to the structural engineer to determine which structures require the application of a certain type of connection. This depends on the desired circularity, price and structural requirements. The first step in the design of connections is to determine the type of connection; whether the connection acts as a hinge or as a rigid connection. The connection type affects the calculations for individual structural elements and the safety of the total structure. In CLT structures however, rigid connections are relatively complex to construct in the realization phase, and are therefore mostly avoided in CLT designs.

3.5.2. Concrete

Concrete weighs approximately 2400 kg/m^3 , which is approximately five times the weight of timber. However, sufficient concrete factories are established in multiple locations across the Netherlands, meaning the materials and structural members do not need to be transported over long distances, but do require heavy transportation means.

Connections in prefabricated concrete slabs are generally designed as steel anchors which extend the reinforcement from one element into the next. This way the tensile stresses can be transferred, while the compression is carried over by direct contact between the concrete parts. The rigidity of a concrete connection can also be raised with a correct composition of this extended reinforcement, as long as the reinforcement is located in the tensile zone of the bending stress distribution. Depending on the shear forces and the required rotational stiffness in the connections, the engineer may decide to thicken the concrete parts locally at the connections. The detailing for reinforced concrete connections is described in Eurocode 4 (NEN-EN 1994).



3.6. MPG (MilieuPrestatie Gebouwen)(Environmental performance buildings)

For any new building project for housing or industrial buildings larger than 100 m², the Dutch government requires an MPG with the request for an environmental permit. The MPG describes the environmental footprint of the materials needed to construct a building, expressed as a number. The lower this number, the more material efficient and sustainable the structure.

For the calculation of an MPG, first a life cycle analysis (LCA) needs to be performed on a single material. This is done by a qualified expert, resulting in 11 indicators of the environmental footprint. These indicators are multiplied by a weighted factor and accumulated into a single value which expresses the shadow price of that material per a chosen unit (kg, m³, etc.). This price is equal to the costs needed to be made to undo the damage done to the environment by extracting the material. This includes the costs of production and transportation. The results of LCA's within the Netherlands are collected in a national environmental database, controlled by the Stichting BouwKwaliteit (SBK, institution for building quality), but the Dutch norm is not up to date and has already been replaced by the European system of EPD's, which will be discussed in Ch.3.11. The accumulation of the shadow prices of all used materials within a structure gives the total shadow price. This includes the materials in need of replacement during the exploitation phase of the structure. Therefore a plausible prediction is necessary for the state of structural elements during the design life of the structure. Finally, The shadow price is divided by the design life and the floor area of a building.

The calculation method for the assessment of the environmental performance of buildings in terms of the MPG is defined in NEN-EN 15978. However, it is not necessary to use this extensive document to determine the MPG of a structure, as several simpler calculation tools have been created to designate a final value to the MPG. As of the 1st of January 2018, the MPG for new projects must be lower than 1,00 €/m².

Floors, walls and installations often contribute to 60% to 80% of the MPG of a building. When taking the MPG into account during the design of the structure, this means a significant portion of the MPG could possibly be mitigated. The MPG of a building is an integral part of important sustainability instruments and concepts. E.g. it is used as a reference value for the material usage in the BREEAM-assessment.



3.7. Environmental performance during exploitation

The previous parts have only analyzed the design- and construction phases. It is important to realize that a large part of the energy consumption of a building takes place in the exploitation phase. This realization has led to the draft of Building Decree chapter 5, which gives technical building instructions and regulations regarding energy-efficiency and environmental impact. This document refers to NEN7120, the Dutch norm to determine the energy performance of a building.

The first required action according to NEN7120 is to determine the characteristic energy usage. It is determined as the sum of the demand for primary energy, converted from the total fossil fuel consumption. The sum contains among others the fossil fuel consumption needed for heating and cooling, humidification and dehumidification, ventilating, lighting and hot water. This energy usage can be decreased by the amount of self-produced energy from e.g. solar panels. The sum of the total demand for energy is divided by the area of the building to eventually get a value with unity MJ/m². In the comparison of timber and concrete as the primary structural building material, it is necessary to determine how the choice of building material impacts the different aspects of the energy performance. Regarding heating and cooling, first an airtight envelope needs to be secured around the structure to avoid heat transfer through flowing air. For both building materials the slabs are connected to each other mechanically, requiring extra measures to seal the openings between those slabs to prevent air from penetrating the structure. Even though CLT and concrete are generally described as being air tight, test results have shown that CLT without glued board edges, which is the most common way to produce the product, does pass some air. Therefore it is advised to seal a CLT structure with a wind barrier (Skogstad, Gullbrekken, & Nore, 2011). Additionally, a continuous insulation layer should always be applied around the structural elements, such that the combination of structure and insulation is sufficient to create a “thermal envelope” which meets the requirements in the Building Decree. These requirements assure a sustainable desired living environment, regardless of high or low temperatures outside of the structure. This means the building stays cool in summer and warm in winter. Article 5.3, notes 1, 3 and 6 give the requirements for walls, roofs and ground floors respectively. First the thermal resistance is determined with NEN 1068, which must then be compared to and higher than the prescribed value. These prescribed values are given with unity [m²K/W] and for a building with a residential function they are 4,5 for external walls, 6 for roofs and 3,5 for ground



floors. The contributions of the structure to the thermal insulation depends on the thickness and the thermal conductivity of the material. The thickness depends on the structural requirements, but since timber has a beneficial thermal conductivity compared to concrete, it requires less added insulation in the composition of the external walls and roof.

In the previous part it was stated that a fully air tight envelop is desired around the structure. However, to regulate the (de-)humidification and ventilation of the structure, certain building materials need to be breathable as well. This breathability secures a healthy living environment for occupants inside the structure. Even though the term breathability suggests an aim for diffusion of air through the structure, it actually refers to three other concepts (May, 2005):

- Vapor permeability (ability of water to move through structure)
- Hygroscopicity (ability of material to absorb and release water as vapor)
- Capillarity (ability of material to absorb and release water as liquid)

The resistance to vapor permeability of a material (typical resistivity) is measured in MNs/gm. For solid timber this value is 200, for concrete it is 500. The glue between the timber layers in CLT likely raises this resistance. A value of 300 MNs/gm is assumed for CLT.

The hygroscopicity and capillarity of materials allows absorption water in different forms, meaning the material can store moisture from the interior climate to create a healthier living environment. The absorption speed needs to be high enough for the system to resist the development of moulds and bacteria, which can form within 45 minutes of the accumulation of moisture in certain areas. Furthermore, good capillarity allows the material to wick away condensation from its surface (May, 2005). Combining the requirements for air tightness and breathability proves to be complex, since the two criteria counteract each other. E.g. a breathable CLT structure and insulation layer may be abolished by the application of a non-breathable airtight layer. Nevertheless, a system needs to be realized where good insulation and ventilation are secured in a sustainable fashion. A waterproof, breathable membrane around the structure should be the ultimate solution, but these membranes have until now only been available at a relatively high environmental cost.

The desire for natural ventilation of a structure becomes more complex when considering no transportation of moisture is desired within the insulation layer. The air within a building is warmer and has a higher moisture content than outside the building. Full breathability of the external partition

3. Literature study – Timber vs. Concrete: analysis of sustainability



means warm air with a high moisture content moves through the insulation material and cools down to the external temperature. The moisture condensates, leading to wet insulation material which is undesired for health and safety reasons. Therefore a vapour-tight layer is required in a building on the warm side of the insulation layer.

The energy consumption for lighting and water usage do not depend on the structural design. These aspects can be addressed with the selection of sustainable additional installations. As of the 1st of January of 2020, the current sustainability requirements will become stricter with the introduction of the BENG-requirements (Bijna EnergieNeutraal Gebouw, translation: almost energy neutral building).

The requirements consist of three indicators:

1. A maximum energy demand in [kWh/m²] per year
2. A maximum primary fossil energy consumption in [kWh/m²] per year
3. A minimum percentage share renewable energy consumption of the total energy consumption

These new requirements will introduce stricter demands for e.g. insulation and generating green energy, and all disciplines within the scope of sustainability accumulated should ensure a structure which is as environmentally friendly as possible.



3.8. Demolition phase

To increase the sustainability and economic value of a material or a product, methods to reuse them after their application in a structure should always be investigated during the design phase, so one knows what to expect when the structure reaches its demolition phase. The reuse of materials started with actual reapplication of elements, and evolved into recycling. As of recently, the terms “circular economy” and “bio-based economy” have gained popularity. An endless material loop is the main goal of reusing materials and structural elements, because this decreases the material demand from natural resources, decreasing resource depletion and greenhouse gas emissions. Strategies to e.g. minimize required rework of an existing structural element or minimize onsite waste can help achieve these goals.

The next paragraphs will further elaborate the terms of recycling, circular economy and bio-based economy. Though a distinction is made, these terms do overlap, and more than one may be applicable for a certain material or structure.

3.8.1. Recycling

The first category is that of recycling. It has been a familiar strategy of waste management for most of human history, with maximization of the economic value of a product being the primary motive. In the early 1900’s, the phrase “Waste to Wealth” was adopted and popularized to describe the benefits to be achieved from recycling (Bradbury, 2017). This phrase is still applicable in the present, but the methods and reasoning to achieve benefits from waste management have changed drastically.

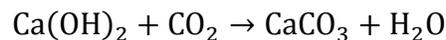
During WWI, an additional motive surfaced for recycling materials. Massive destruction, together with the shrinkage of the industrial capacity, led to a shortage of raw materials. This led to the initiation of applying waste management to refill the raw material stock, since the natural sources were mostly depleted or could not be excavated rapidly enough.

From WWI recycling slowly evolved, since the motives for waste management mostly remained unchanged during this period. Until the year 2000, when the Environmental Protection Agency (EPA) in the United States confirmed the connection between correct waste management (including recycling) and the decrease of greenhouse gas emissions, thus limiting the contribution of the building industry to global warming, provided that recycling is applied in an efficient manner.

3. Literature study – Timber vs. Concrete: analysis of sustainability



Recycling as a concept is applicable for both concrete and timber. Concrete, after demolition of a structure, has one function with respect to recycling, which is the reapplication of crushed concrete parts as recycled aggregate in newly produced concrete parts. This means new cement has to be added to the crushed concrete, leading to a certain energy consumption and to emission of CO₂. Depending on the quality of the recycled aggregate, a compressive strength can be obtained which is either higher, identical or lower than the natural aggregate concrete (Malešev, Radonjanin, & Broćeta, 2014; Tabsh & Abdelfatah, 2009). However, the modulus of elasticity is lower due to the presence of old mortar, therefore providing a lower resistance to shrinkage. Degradation and time-dependent factors in concrete can have many causes. External causes of degradation, like fire damage or physical damage, are ignored when considering long term quality of concrete. These causes are not related to the functionality of the material over time, and the structural elements are most likely renovated at the times these forms of degradation occur coincidentally. Internal causes however, like chemical damage and corrosion of reinforcements, are more complex. The progress of these processes is more challenging to monitor, and much research is still being done on the structural functionality of concrete under these influences. An inevitable time-dependent chemical process in concrete is carbonation, where calcium hydroxide reacts with carbon dioxide to form water and calcium carbonate:



The process starts on the edges of the concrete that are in contact with air (and thus with CO₂), but proceeds inwards over time. Although carbonation in unreinforced concrete actually improves the mechanical properties of this material, it is a common cause of corrosion in reinforced concrete (Lo, Tang, & Nadeem, 2008). Corrosion in its turn is the most common cause of overall deterioration of concrete structures, since the reinforcement steel area can get significantly decreased in these situations. This means the process of carbonation does not influence the potential of recycling a concrete slab, but it does increase the maintenance requirements of this slab and possibly leads to a quicker required replacement if issues become too severe.



Recycling of wood is not only applicable for the structurally applied timber elements, but as mentioned in Ch3.4.1, for every kilogram of glued timber produced, there is 4,03 kilograms of waste wood as a by-product. At the end of the life cycle of a structure, the applied timber elements will likely end up on the waste wood pile as well. Though the name waste wood insinuates differently, this wood may still have some functionality when a correct recycling strategy is applied. Structural examples are the application of waste wood in the form of strands or chips into engineered products like oriented strand board (OSB) and particle board. Addition of an adhesive is required to create these products, and a selection needs to possibly be made to separate the bark from the actual wooden parts. This will negatively influence the sustainability of the recycled timber products, but with the application of a bio-based adhesive and an energy-efficient separating method this negative effect can be decreased. An alternative to creating new products is to use the waste wood as a source for bio-energy. Technically spoken, it is called energy from wood biomass. This biomass is defined as any product suited for conversion into energy through direct combustion or gasification. In the consumption of bio-energy, CO₂ is emitted, similar to the consumption of fossil fuel. However, considering the renewability of raw materials from Ch.3.1, the carbon cycle for fossil fuel is substantially larger than the carbon cycle for bio-fuel, with the sequestration-times of both fuels of millions- and dozens of years respectively. The availability of waste wood will determine whether bio-fuel could actually be a suitable replacement to fossil fuel. Biomass has a lower energy density than fossil fuels, and therefore more mass is needed to generate the same amount of energy.

The SBK (Ch.3.6) gives fixed percentiles for the applied methods of waste management for different materials (Stichting Bouwkwiteit, 2019). 99% of concrete is being recycled and the other 1% is dumped as waste. For timber, assuming it is “clean”, only 10% is recycled. 5% is reused, 80% is burnt and 5% is dumped as waste. Clearly, at this moment in time concrete proves to be more favourable when it comes to recycling. Timber recycling needs to improve drastically to catch up with concrete. New applications to improve recycling of timber have already been mentioned, and with the forecasted growth of the timber building market, the availability of waste wood would grow simultaneously, meaning the future could look bright for all uses of timber and wood depending on innovations and research.

3. Literature study – Timber vs. Concrete: analysis of sustainability



3.8.2. “Circular economy”

The term “circular economy” can be interpreted in a number of ways. The consensus between the interpretations is the mitigation of waste, but the difference is mostly due to differences in the analysed timeframe. The difference can be expressed by distinguishing two cycles within the circular economy; the biological cycle and the technical cycle. The biological cycle relates to the renewability and biodegradability of the materials (bio-based economy, Ch.3.8.3), whereas the technical cycle relates to the reapplication of structural members, either without adaptation or through recycling, where members are demolished and parts used to rebuild new products. When analysing the full lifespan of a material, both cycles should be examined to create the full sustainability image of the project.

When someone speaks of the circular economy, most often this refers to part of the technical cycle; the reapplication of structural elements without any adjustments. This form of recycling requires no rework, and therefore no extra energy consumption or waste management are required. The elements must however be connected so they can be disassembled easily without any destruction to the element. This proves to be a challenge, because even simple connections like screwed connections in timber leave their mark after removal. In prefabricated concrete, the connections are often designed with continued reinforcement through the contact surface, which also creates a restriction for further application. Designers should be creative and inventive with their connecting systems to enhance the reusability of a structural element if existing systems do not satisfy.

For some, only considering methods to reapply structural components once is enough for a project to fit into the circular economy. The production of those components may however have resulted in waste or other byproducts, harmful or not. When considering the circularity of a product, it should include the circularity of the different materials and components over their entire lifespan used to create this product. Reapplication of a product is just one step, but it is just as important to think about what happens to the material afterwards. The blue box in

Figure 8 shows the cascading preferences with regard to the end-of-life management. Circularity over the full life span of a product should include the application of suitable waste management during production-, exploitation- and demolition phases. Unfortunately, a material like timber is susceptible to a decrease of material quality over time and elements cannot always be reapplied directly into similar structures. Before reapplication is possible, the current degradation/deterioration of the product



should be determined. This may for example be caused by extreme loading or exposure to biological influences like bacteria or fungi. After the quality control, an investigation of the safety and functionality in the new structure should be done. The Dutch building decree makes a distinction in requirements for new building projects and existing structures. Time-dependent influences on the timber may lead to lower strengths, meaning smaller spans may be achievable when reapplying beams or slabs. This decrease in functionality occurs with every step of recycling of structural elements, and at some moment in time there is no use left. Other applications may then still be a possibility, e.g. as smaller pieces like strands in OSB, or in non-structural functions like furniture, but these solutions require an addition of other materials and energy to create a new product.

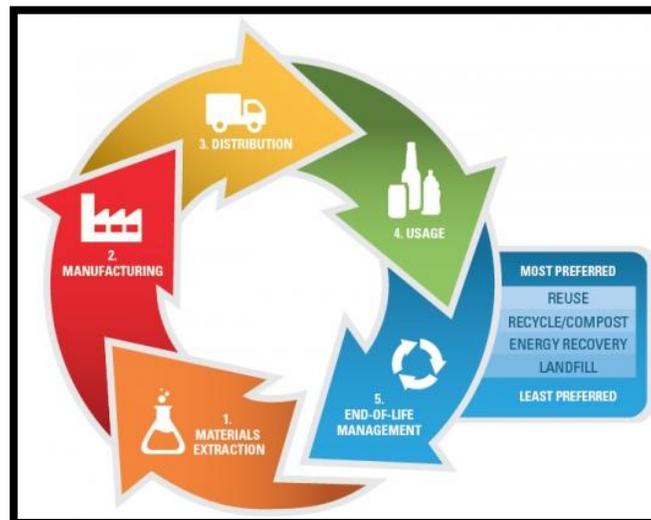


Figure 8 - Material life cycle

Prefabricated concrete elements, like timber products, can be reused in new structures if desired, altered or not. Again, the reusability of whole elements depends on the removability of connections and the condition due to the influence of time-dependent processes. An example of a time-dependent process is carbonation, described in Ch.3.8.1. If reuse of a concrete element is not possible, it may be recycled by crushing and reapplying the pieces as aggregate for new concrete. There is some loss in aggregate toughness, but it is still acceptable for structural applications. The compressive- and tensile strengths of concrete made with recycled concrete as aggregate is very dependent on the source of the aggregate. If the recycled concrete is of high strength, this high strength carries over to the new product, and there is less strength loss compared to concrete with lower strength recycled concrete as aggregate (Tabsh & Abdelfatah, 2009).

3. Literature study – Timber vs. Concrete: analysis of sustainability



3.8.3. “Bio-based economy”

At the end of a products design life, when the material cannot be recycled any further, with regard to the circular economy it is aspired to close the gap between waste and material source. Along with this aspiration comes the preference for bio-degradable and renewable materials. Ideally, a material can be extracted from a natural source to fulfil a function, and have the source replete itself during the exploitation of the material. The material should supplement this repletion after recycling.

Wood and timber might for instance act as a primary component for compostable organic products which is a way to “give” the timber back to nature, but some key requirements regarding purity, shape, size etc. are set in place for the elements. It is important to realize that the addition of adhesives might negatively affect the bio-degradability of the timber. This in turn affects the possibility of closing the material gap between recycling and material extraction (

Figure 8). Bio-based adhesives may provide a suitable solution for this issue. Soy-based adhesives were mentioned in Ch.3.4. Currently this is mostly applied in some particleboards, interior plywood and engineered wood flooring, but depending on future developments regarding its performance the field of application could expand to eventually replace synthetic adhesives in CLT(Vnuček, Kutnar, & Goršek, 2016). Even though soy adhesive still only accounts for 1% of the adhesive market, its growth is bigger than any of the other mentioned adhesives(Transparency Market Research, 2014). Additionally to the biodegradability and renewability of the soy adhesive, the environmental pollution- and volatile organic compound (VOC) levels are often low, which offers good perspective with respect to the overall sustainability of CLT.



3.9. Subjective criteria

Some subjective criteria might end up to be the deciding factor in the discussion whether timber or concrete is a “better” building material. These subjective criteria are related to human psychology, and may not always be quantifiable or noticeable. However, psychological research does prove the influence of these criteria on human well-being, which makes them worth mentioning.

3.9.1. *Indoor climate*

The indoor climate in a building is one of a number of factors which are collectively represented in the indoor environmental quality (IEQ). The indoor climate can be naturally controlled by sufficient ventilation through structural elements, meaning it is directly linked to the breathability of the structure. While creating a breathable, naturally ventilated structure, attention needs to be paid to the environmental performance. The requirements for breathability leads to a maximum thickness of elements whereas the thermal insulation means a minimum thickness is to be applied. The two counteract each other, and the design phase should cover this problem. Priority is to create an insulated structure, since ventilation can also be provided by the placement of installations. This is where the subjectivity of the criterion comes into play. Timber performs better than concrete when comparing the breathability and the insulation characteristics (Ch.3.6). Whenever a client prefers natural ventilation over mechanical ventilation, this means timber could provide a better solution than concrete.

3.9.2. *Acoustic comfort*

The effects of a building material on the acoustic performance of a structure are relatively complex, as the choice of a different building material results a different composition of structural elements and different connections. Additionally, specific material characteristics result in favourable circumstances with regard to either airborne- or structure-borne sound propagation.

The structural elements in this comparison are assumed to be composed as solid slabs (CLT and prefab concrete), without any filling materials or cavities. In reality, other compositions could be preferable over solid slabs, allowing measurements to improve the acoustic performance as well. The assumption of solid slabs does however create the possibility of a direct comparison between concrete and timber with regard to the acoustic behaviour of specific materials.

The mass-frequency law shows a higher mass is preferable for the insulation against airborne sound. For the resistance against transmission of airborne sound, a wall acts as a sound barrier which must

3. Literature study – Timber vs. Concrete: analysis of sustainability



first of all assure that there are no sound leaks like cracks or holes. Wall slabs can transfer vibrations of the air to the air on the other side of the slab, through vibration of the slab itself. . Heavy slabs do not vibrate as easily as relatively lighter slabs so the transmission of airborne sound is more difficult for these slabs as well. Since the density of concrete is approximately 5 times higher than the density of timber, its performance with regard to airborne sound is favourable.

The transmission of structure borne sound by a material is not as easily linked to a specific characteristic. Structure borne noise occurs whenever a source lets a structure vibrate in such a way that these vibrations couple with acoustic waves. Note: the structural vibrations are often in transverse direction, while acoustic waves are in longitudinal direction. To mitigate structure borne noise, structural elements should be acoustically uncoupled so vibrations in elements do not reach other compartments in the building. Considering a source like footfall, the uncoupling of floors and walls is easily executed. However, with the application of solid floor slabs for both concrete and timber, the transmission of structure borne sound to compartments below is not easily mitigated, especially considering the design of structural elements is focused on creating the thinnest possible elements. A solution is to apply a top layer with a damping function, so the structure is not put into motion by the source, and therefore does not transmit sound.

This criterion can be perceived as subjective because of the aesthetics. It could be desired by inhabitants of these houses to keep the structural materials in sight. The addition of extra layers could significantly increase the acoustic comfort within a structure, but this would then need to be preferred over the aesthetic function of the building material.



Figure 9 - Aesthetics, a. concrete, b. CLT; sources: https://cdn.trendir.com/wp-content/uploads/old/house-design/assets_c/2015/07/house-with-raw-concrete-living-room-6-thumb-1200xauto-54404.jpg, <https://i.pinimg.com/originals/98/f4/38/98f43898c1cb6654a10b2f2b60fee105.jpg>



3.9.3. Aesthetics

Besides the recent developments with regard to the sustainability of building materials, the relationship between inhabitant's well-being and building materials has been getting much attention as well. The psychological aspects of this relationship are fundamental in environmental psychology, which studies the transactions between individuals and their physical setting (Gifford, Steg, & Reser, 2011). Specifically, the aim is to distinguish the effects on human health of "natural" materials (timber) from "industrial" materials (concrete). The link between the natural environment and human health may be a first step in this distinction. Five mechanisms of action are identified (Figure 10)(Health Council of the Netherlands and Dutch Advisory Council for Research on Spatial Planning Nature and the Environment, 2004).

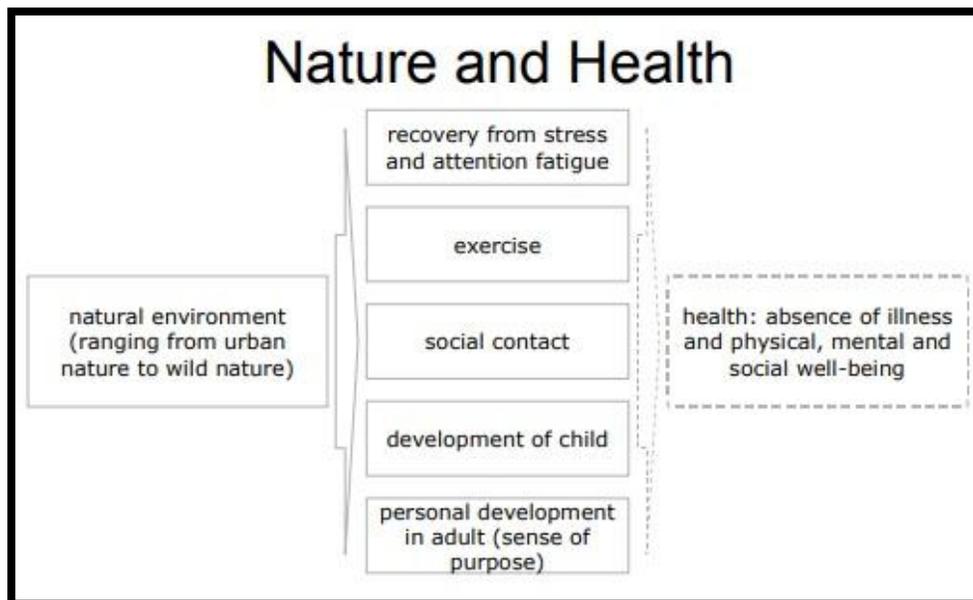


Figure 10 - Link between nature and health; source: (Health Council of the Netherlands and Dutch Advisory Council for Research on Spatial Planning Nature and the Environment, 2004)

More scientific research would be required for nature to start playing a serious role in current and future healthcare, but existing research provide plausible hypothesis and confirming conclusions to assume beneficial effects on the health of individuals when they get in contact with nature.

The next step is to determine whether the visual perception of structural timber is equal to the visual perception of nature, or at least creates a similar feeling and has similar effects on occupants. Apparently, many people see forests and wooden furniture at home in a similar, positive way. This positivity disappears with a small percentage of people who relate wooden interiors to the production

3. Literature study – Timber vs. Concrete: analysis of sustainability



process of wood and timber, which is often linked to the loss of forests and therefore a loss of nature. Evidence from surveys proves that the natural feeling of forests in individuals corresponds with the image of wood in interiors. Also, forests and nature are commonly used as synonyms, as forests make people feel most in touch with nature (Rametsteiner, Oberwimmer, & Gschwandtl, 2007).

Contrastingly to the use of wood and timber on a large scale, the general assumption that “forest needs to be protected by man” is very common. Previous chapters have already showed that with sustainable forest management, the contrary is true where humans should use more timber in various applications to make use of the (endless) supply due to renewability of forests. However it proves to be very complex to convince a layman of this statement. Two conflicting goals of sustainable forest management require more public attention to counter the common assumption of only preserving trees; on one side the focused production and harvesting of wood for industrial means, and on the other side the preservation of habitats and ecosystems.

Regardless of the opinion on the harvesting methods, timber is widely considered a beautiful, versatile and above all natural material. As suggested, this natural appearance positively affects the psychological well-being of occupants. The sense of naturalness is strengthened when individuals get exposed to the tactile sensation of wood and timber (Nyrud & Bringslimark, 2009). The majority of participants of an Austrian research preferred the feel of a natural timber floor over a lacquered or laminated surface, with the description that a natural floor feels “warm” and “fairly soft” (Berger, Kats, & Petutschnigg, 2006). The assumption can be made that natural timber walls have a similar effect. For the sake of this research, it is however more important to investigate the psychological consequences due to the aesthetical differences between timber and concrete. Not much research has been done on this exact difference, so forming a substantiated statement on the matter is complex. Figure 11 shows the results of a questionnaire in which test subjects had to judge a variety of materials (Rice, Kozak, Meitner, & Cohen, 2006). Wood was rated higher than stone in terms of being relaxing and natural, and lower on being industrial and artificial. Considering concrete is an artificial, stone-like material, it would rate even worse than natural stone in terms of being relaxing and natural.

...

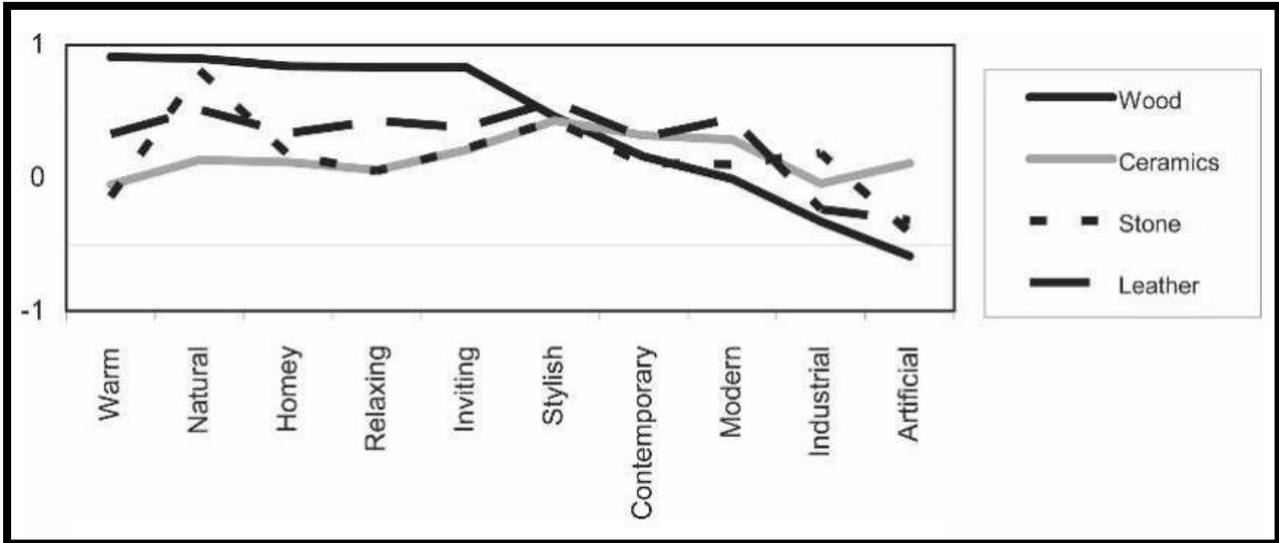


Figure 11 - Perceived attributes of wood versus other natural furnishing materials (1 = material possesses attribute; -1 = material lacks attribute); source: (Rice et al., 2006)

So, with timber being considered as having a more natural appearance than concrete, and nature having a positive influence on the well-being of individuals, this results in timber in sight within a building having a beneficial effect on occupants health compared to concrete. Though this effect would seem to be preferable for anyone, it remains up to individual inhabitants to decide in what type of house they would like to live. Therefore this criterion is classified as subjective.

3. Literature study – Timber vs. Concrete: analysis of sustainability



3.10. Results

To finish this chapter, the sustainability of CLT is compared to the sustainability of concrete. A quantitative analysis would be a suitable topic for further research but is too extensive within the scope of this thesis. A qualitative assessment is given in Table 1.

	Prefab concrete	Cross-Laminated Timber
<i>Carbon sink</i>	0	++
<i>CO₂ emission for production</i>	---	-
<i>CO₂ emissions for transporting/lifting</i>	-	-
<i>Renewability of resource</i>	--	++
<i>Harmful additives</i>	-	-
<i>Recyclability</i>	++	0
<i>Fit in the circular economy</i>	+	+
<i>Fit in the bio economy</i>	--	++
<i>Thermal resistance</i>	-	+
<i>Air tightness</i>	+	-
<i>Natural ventilation</i>	+	++
<i>Acoustics</i>	+	-
<i>Aesthetics</i>	-	+

Table 1 - Material comparison

Comparing carbon sinks of both materials is in favour of timber. The magnitude of carbon sequestration in carbonation of concrete is negligible compared to sequestration due to photosynthesis. CLT requires fossil fuel for production, customizing and transportation of the slabs, resulting in emission of carbon dioxide. Concrete is casted in the required shapes, and therefore requires less fossil fuel for production and customization of the products, but the production of the raw materials of concrete emit a lot of carbon dioxide. A reaction needs to take place to produce calcium oxide (for cement) with carbon dioxide as a by-product. Transportation of concrete is heavier, but fully depends on the distance between the building site and the factory. Generally, CLT needs to be transported further than concrete, so there is a trade-off between the distance and the weight of the transportation between the two materials. The renewability of the raw material for CLT is better than that of concrete. Trees can re-grow within the design life of a project, which is unachievable for the components of concrete. Both engineered products might require harmful additives for their performance; in prefab concrete steel reinforcement is applied, in CLT the addition of glue is required. How harmful these additions really are, is up for debate. Steel, once it's produced, is very recyclable, but the production



requires extreme heat (and therefore fossil fuel). Some glues are harmful for the environment, but research is leading to safe, bio-based innovative adhesives that might see broader application soon. CLT and prefab concrete are both recyclable, but also reach a certain point where their functionality decreases. Concrete can be recycled as aggregate in new concrete, but reapplication decreases the strength of the new concrete. How often this cycle can be repeated while still maintaining structural integrity should be researched. If a timber element cannot be reused as it is, it is also possible to break it down into strands or particles for application in other engineered products. After its use, timber can possibly be biodegraded to use as e.g. a main component in compostable products. The circularity of CLT and concrete is very similar, since the same specifications are required regarding functionality and disassembly of connections of elements. The thermal resistance and breathability of CLT are both better compared to those characteristics of concrete, resulting in less additional isolative material and less ventilation installations.

3. Literature study – Timber vs. Concrete: analysis of sustainability



3.11. Quantifying sustainability through EPD's

In an attempt to quantify the sustainability of CLT versus concrete, Environmental Product Declarations (EPD's) lead to a reliable comparison. These EPD's are created according to the European norm and have a set structure which allows for weighing different structures up against each other on the basis of the full life cycle of a product and its materials (Figure 12). Some EPD's do not include all the stages of the LCA, but the distinction allows to disregard or neglect certain stages and only make a comparison based on the available information. The EPD's are collected in an international database in an attempt to supply the most reliable information as possible.

Product stage			Construction process stage		Use stage							End-of-life stage				Resource recovery stage
Raw material	Transport	Manufacturing	Transport	Construction installation	Use	Maintenance	Repair	Replacement	Refurbishment	Operational energy use	Operational water use	De-construction demolition	Transport	Waste processing	Disposal	Reuse-Recovery-Recycling potential
A1	A2	A3	A4	A5	B1	B2	B3	B4	B5	B6	B7	C1	C2	C3	C4	D

Figure 12 - Life cycle analysis stages, source: (Egoïn, 2018)

There are many EPD's available for (precast) concrete products, and for this quantification an EPD for a general precast concrete product was selected with a declared unit expressed in weight (tonnes), to neglect the influence of design choices in the structure. The EPD of CLT has a declared unit of 1 m³. Assuming a terraced house requires approximately equal slab thicknesses for CLT and concrete (for both the floors and walls), expressing the environmental impact with the volume as the declared unit creates comparison-potential in terms of the application of the materials in terraced houses. Assume a specific weight of concrete of 2400 kg/m³. The environmental impact for CLT and concrete are found in In this report we will compare the global warming potential, the abiotic depletion potential (fossil resources), the embodied energy when used as raw materials and the net fresh water use. The EPD for precast concrete only incorporates stages A1 till A4 in its consideration. The materials are compared for these stages in Table 2.

• • •

	CLT					Precast concrete		
	A1	A2	A3	A4	Total	A1-A3	A4	Total
<i>GWP (kg CO₂)</i>	-716,18	27,45	3,2	47,82	-637,71	66,67	5,83	72,5
<i>Energy resources (MJ)</i>	29481,69	457,73	150,83	797,33	30887,58	839,17	79,17	918,34
<i>Net use of fresh water (m³)</i>	1,12	0,073	0,11	0,13	1,433	3,78	0,23	4,01

Table 2 - EPD eco-profile CLT vs. precast concrete

The interpretation of the results from EPD's specifically and LCA's in general is a complex task to fulfil. There is much debate on topics like the depletion of fossil fuels, embodied energy and the impact of CO₂ on the environment. Nevertheless, big differences are found for the LCA of the production stages of CLT and precast concrete. CLT scores well in terms of the global warming potential and the net water use, whereas its embodied energy is worse than that of precast concrete. These differences are explicable by considering the production process:

- While trees grow, a lot of carbon dioxide is absorbed for the photosynthesis process, causing a high negative value for CLT in stage A1
- The emission of carbon dioxide of precast concrete is relatively high due to the chemical process which is required to create cement
- The embodied energy is much higher for CLT, due to the necessity to lower the moisture content of the wood in a kiln.
- Concrete requires much less energy in the production, since it only requires a mixture of 3 components; cement, aggregates and water.
- More water is required for the production of water compared to CLT, since water is one of the main ingredients of concrete.

Based on these results only, a reliable quantification for the total structure cannot be made. In this report, the assumption has up till now been that the emission of CO₂ is the biggest problem in the discussion on environmental issues. However, the issue is too complex to only look at one aspect, and more research is necessary in an attempt to accumulate aspects based on different declared units.

It is clear that the high number embodied energy in timber has a different impact on the environment than the high carbon dioxide emission of concrete, but the magnitude may be similar. This mostly depends on the source of the energy used in the production of CLT. If this energy can be fully derived from biological sources, the negative environmental impact is mitigated and a fully sustainable production is realized.

3. Literature study – Timber vs. Concrete: analysis of sustainability



Unfortunately, the EPD's of precast concrete do not involve stages further than A4. This means vital information is lacking to create a full visual of the environmental impact across the life cycle of the product, which should be supplemented to be able to make a clear decision which building material is more sustainable. Stages A5 till D have however been discussed qualitatively in previous sections, with CLT receiving better scores than concrete.

Thus, considering the conclusion from Ch.3.10 and combining this with the results of the EPD's, CLT is proven to be a more sustainable option as a building material than precast concrete.



4. Guideline analysis – identifying terraced housing building criteria

4.1. Introduction

The requirements for structural elements within residential buildings depend on different characteristics of the materials used for creating these elements. Since the aim of this research is to create an MCA for a CLT-structure, the building requirements will serve as the criteria on which the performance of the structure will be judged.

The requirements are established in certain guidelines and standards. The creation of these documents is initiated internationally through the European Union, which leads to implementation in national standards by the Nederlands Normalisatie-instituut (Dutch normalisation institution) and Rijkswaterstaat.

The goal is to create a comprehensive structural comparison between the cross-laminated structure and the conventional structure for a terraced house, in which reinforced concrete is applied as the primary building material. The building criteria in Table 3 will be inspected by consulting the accompanying guidelines.

Building criteria	Guidelines and standards	
	General	Specific for timber
<i>Structural safety</i>	NEN-EN 1990 NEN-EN 1991	NEN-EN 1995 CLT draft for EN 1995 (prEN16351)
<i>Serviceability</i>	NEN-EN 1990 NEN-EN 1991	NEN-EN 1995
<i>Fire safety</i>	Building Decree (BD) sections 2.2, 2.8-2.12, 6.6, 6.7 EN 13501-1	
<i>Thermal insulation and sustainability</i>	BD section 5.1, article 5.3 NEN7120 NEN1068 EN-ISO 10456	
<i>Breathability</i>	BD section 3.6, article 3.29	
<i>Sound insulation</i>	BD section 3.4 NEN 5077	
<i>Durability</i>	NEN-EN 1990	NEN-EN 1995
<i>Building method</i>	-	
<i>Aesthetics</i>	-	

Table 3 - Building criteria

4. Guideline analysis – Identifying terraced housing building criteria



4.2. Platform framing method

The most commonly applied structural design for terraced houses in the Netherlands is relatively simple, to increase the repetition factor and building speed as much as achievable. The houses are attached to each other, so generally the walls that separate the houses carry the whole structure, as it is undesirable for the inhabitants to create holes in these walls. With the most commonly applied building material, (reinforced) concrete, this results in a slender wall slab which carries all imposed loads and self-weights of the floors and walls above. Two framing methods are distinguished for terraced housing structures; balloon framing and platform framing (Figure 13). In balloon framing, the walls cover the full height of the house, and intermediate floors are hung up inside the walls. Because the aim is to build a structure with CLT slabs without any protruding parts for support, it is impossible to apply the balloon framing method in this case. The other alternative is the platform framing method, meaning the structure is built up one storey at a time. First the load-carrying walls up to the first floor are placed. Then the first floor is put on top, which creates a working platform for the next storey, hence the name of the building method (de Graaf & Banga, 2002). The walls to the second storey are then placed on top of this platform, etcetera. With this method, the floors are directly supported by the walls, and the connections can be made relatively simple without additional, protruding elements. The downside of this method is that the horizontal stability needs to be ensured for every storey individually. This may lead to complex designs for stabilizing elements, as the total horizontal wind load on the building results into a large horizontal force on the bottom storey of the building.

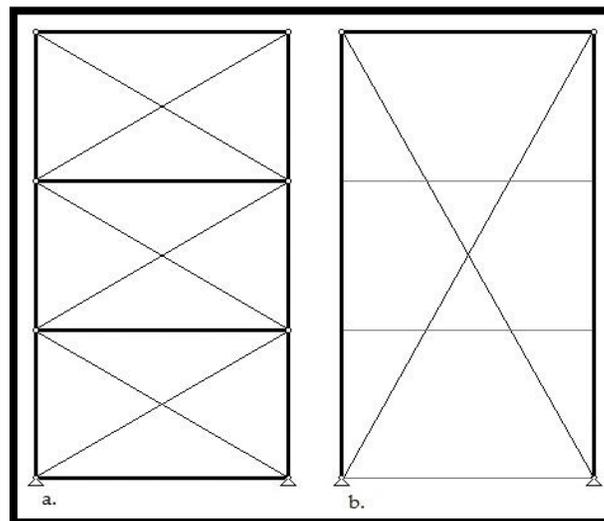


Figure 13 - a. Platform framing, b. Balloon framing



4.3. Limit states and load situations

The structure is designed according to previously mentioned European and national standards. Eurocode 0 (NEN-EN 1990) requires for structural elements to be checked in different limit states, leading to different critical loading situations. Prior to the calculations it is not known which situation is critical, so the calculations are executed on the structural elements for all possible combinations of the following loads, determined according to the relevant parts of the Eurocode:

- Permanent loads
 - Self-weight (NEN-EN 1991-1-1)
 - Additional permanent loads
- Variable loads (
 - Imposed (NEN-EN 1991-1-1)
 - Snow (NEN-EN 1991-1-3)
 - Wind (NEN-EN 1991-1-4)

The Eurocode gives insight in the different states, situations and load combinations to be examined. The applied loading state in which the calculation is done depends on the type of requirement. When determining structural failure of an element a load combination in the ultimate limit state(ULS) is used, whereas when checking the functionality of a structure a load combination in the serviceability limit state (SLS) is applied.

When comparing a cross-laminated timber structure to a concrete structure with regard to load combinations, only the self-weight differs. The other loads depend on dimensions and function of the structure, which are approximately equal for the application of both building materials. The self-weight of CLT (spruce) and concrete are assumed at respectively 500 kg/m³ and 2400 kg/m³.

4.3.1. Structural failure (ULS)

In case of structural failure, either an element within the structure fails due to an exceedance of strength properties (resistance) or the total structure forms a mechanism of plastic hinges where stability is lost. These two failure types refer to certain states within the ULS. Resistance of structural

4. Guideline analysis – Identifying terraced housing building criteria



elements is checked in the STR (structural) and FAT (fatigue) ultimate limit states, whereas stability of the overall structure is checked in the EQU (equilibrium) ultimate limit state.

The critical load combination in ULS is one of the following combinations (NEN-EN 1990):

- Fundamental combination for persistent and transient design situations:

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

With partial safety factors:

$$\gamma_G = 1,1 \text{ (unfavourable loads) or } \gamma_G = 0,9 \text{ (favourable loads)}$$

$$\gamma_Q = 1,5$$

Or for STR:

$$\left\{ \begin{array}{l} \sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \\ \sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \end{array} \right.$$

With partial safety factors:

$$\gamma_G = 1,35 \text{ (unfavourable loads) or } \gamma_G = 0,9 \text{ (favourable loads)}$$

$$\gamma_Q = 1,5$$

And reduction factor for unfavourable, persistent loads:

$$\xi = 0,89$$

- Combination for accidental design situations:

$$\sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ of } \psi_{2,1}) Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$$

- Combination for seismic design situations:

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

The reduction factors in previous formulae are:

Loads	ψ_0	ψ_1	ψ_2
<i>Imposed load residential compartments</i>	0,4	0,5	0,3
<i>Imposed load roofs</i>	0	0	0
<i>Snow load</i>	0	0,2	0
<i>Wind load</i>	0	0,2	0

Table 4 - Reduction factors, source: NEN-EN 1990 NB Table NB.2 - A1.1



Depending on the consequence class of the structure, the partial safety factors need to be multiplied by factor K_{FI} :

- CC1 $\rightarrow K_{FI} = 0,9$
- CC2 $\rightarrow K_{FI} = 1,0$
- CC3 $\rightarrow K_{FI} = 1,1$

In this structure in the ULS, only strength and stability of the structure and of the individual elements is of importance. The horizontal slabs (floors and roofs) carry their own weight (strength) and transfer the wind loads in horizontal direction towards the stabilizing elements (stability). The house-separating walls are load bearing, carrying the floors, roofs and walls above (strength), but additionally function as stabilizing elements for wind loads on the front- or back façade (stability). The front- and back façades have no function regarding vertical loads. They only provide the stability for wind loads on either side façade. An overview of the variable loads on a platform framed terraced house is given in Figure 14).

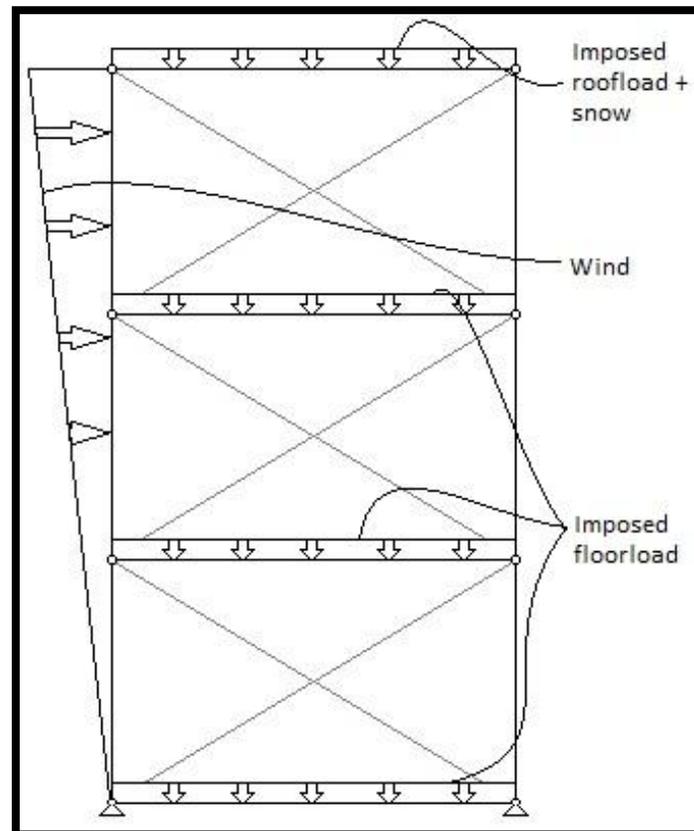


Figure 14 - Variable loads on platform framed structure

4. Guideline analysis – Identifying terraced housing building criteria



4.3.1.1. Strength

The horizontal slabs in this structure have the primary function in the ULS of supporting vertical loads over a span equal to the total distance between the load-bearing walls. This means they are schematized as simply supported beams with a standardized width of 1 meter (

Figure 15). The slabs are either in the category of floors or roofs. Floors inside the house need to carry their self-weight, an additional permanent load to account for the weight of interior walls and a variable imposed load. A reasonable assumption for CLT interior walls is $0,50\text{kN/m}^2$. The load combination for a roof is more difficult to determine, as multiple variable loads can work on the same slab. The roof needs to carry its self-weight, the weight of elements permanently placed on top of the roof and a combination value of three variable loads; an imposed load, a snow load and a wind load.

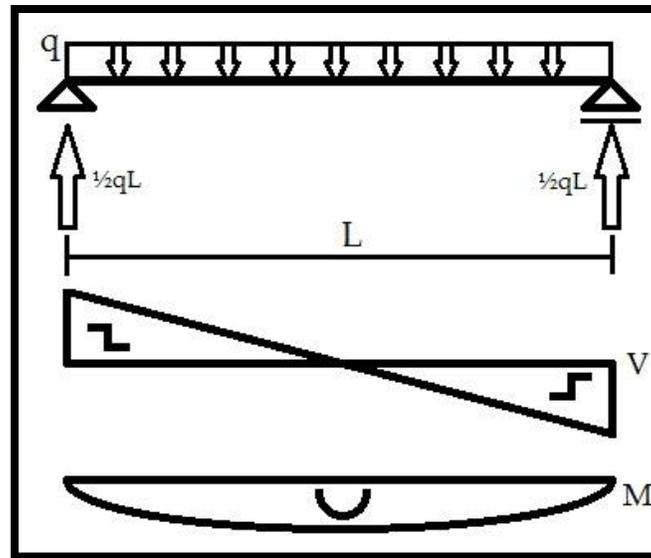


Figure 15 - Simply supported beam including shear- (V) and moment line (M)

The vertical loads on the horizontal slabs cause an out-of-plane bending moment in the slab which due to symmetry in the calculations reaches a maximum in the centre of the span (Figure 15). The maximum design bending stress is determined in the same location as the maximum bending moment, with the requirement of this stress being smaller than the bending strength.

The connection between the horizontal slabs and the supporting structure underneath is considered as a hinge. This consideration affects the calculation of our structural elements, and might lead to higher required thicknesses, but it also simplifies the realization of the connections in the construction phase.



Hinged connections mean no bending moments will be transferred through the connection from one slab to the other. However, normal- and shear forces require calculations to exclude local failure.

The house-separating walls and horizontal roof- and floor slabs form a stack of 3 “tubes” over the full depth of the house (Figure 14). Accumulating the self-weight of the structure and the governing vertical downward variable load leads to a maximum compressive force in the load-bearing wall at ground level. This force must be lower than the buckling load of this wall. Because the platform framing method is applied to construct this project, the floor slabs are supported between wall slabs of different storeys. The compressive forces in the walls cause local compressive stresses perpendicular to the grain in the floor between the walls, since this force is transferred through direct contact (Figure 16a). The stress needs to be smaller than the relevant compressive strength for this part, or other measures need to be taken to locally strengthen the parts of the floors situated between the walls.

In loading situations with extreme wind loads, when considering a lightweight structure (like timber), it might as well be possible to have a resulting tensile force in any of the wall-floor-connections (Figure 16b). In this case, the walls are to be checked on their tensile strength and extra connective elements are required to keep the system together and transfer the upward forces to the foundation of the building.

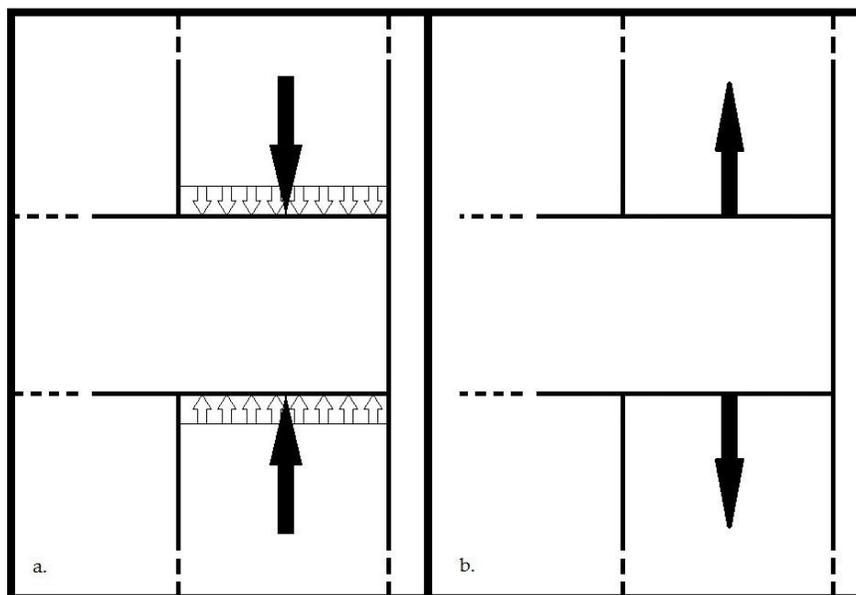


Figure 16 – wall-floor-connection; a. under compression, b. under tension

4. Guideline analysis – Identifying terraced housing building criteria



The vertical loads on the floor slabs lead to vertical reaction forces in the walls. In compression, these forces must remain small enough for the walls not to buckle. In tension, for example in the case of extreme wind loads, the walls must have enough tensile strength to keep the roof in its place.

4.3.1.2. Stability

The stability of the structure needs to be guaranteed for the wind load in two main directions, wind on the front- or back façade and wind on either side façade. In theory, the calculations are very similar, but since terraced houses are often much deeper than they are wide, the requirements may vary significantly. The stability system in a platform framed structure consists of a number of elements which should be sufficiently connected for the horizontal loads to be transferred to the foundation.

The first elements are the floors. Figure 14 shows a varying wind load across the height of the structure, which in reality is the case.. In NEN-EN1991-1-4 however, a constant distribution of the wind pressure is assumed for a building with smaller height than its width. Assuming there are always two houses next to each other when there is extreme wind, the height is always smaller than the width of the house in any direction.

The constant wind load on a facade is assumed to result in three (or four when including the load at the foundation) line loads on each storey (Figure 17a). For the roof, the value of this line load is equal to the wind pressure over half of the top storey. For the two middle floors, it is equal to the pressure over half of the storey above and half of the storey below, resulting in the height of one storey.

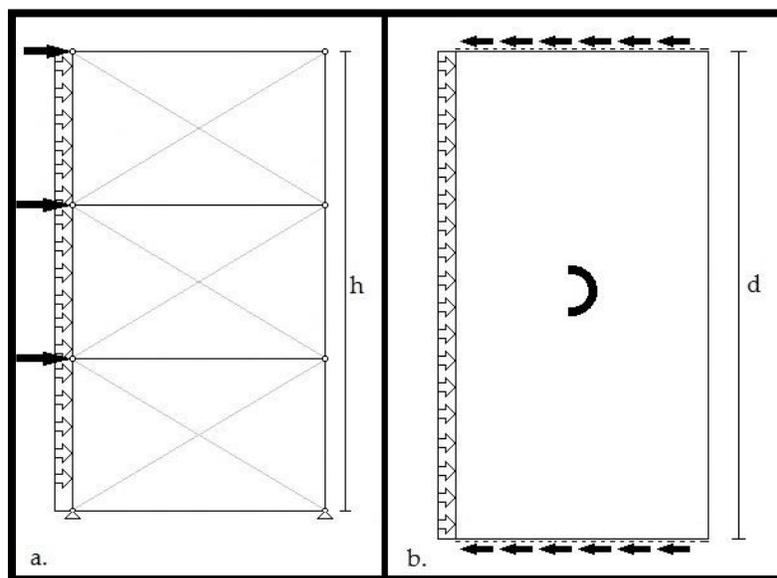


Figure 17 – a. wind load distribution over height of building, b. wind load on floor slab



Each horizontal slab is supported in horizontal direction by stabilizing walls at the edges (Figure 17b). These walls supply resistance depending on the applied connections, also schematized as a line load. The result is the slabs acting as diaphragms, where they can be schematized as deep beams supported at the edges.

As mentioned, all external walls of the structure are required to provide stability when the structure is loaded by horizontal wind loads. This results in requirements for the walls in terms of shear- and bending moment resistance (Figure 18). The design load in terms of shear and bending moment in these walls reaches a maximum in the bottom storey of the building, since it accumulates from the top downwards towards the foundation.

The resistances for shear and bending moments need to be sufficient for all floors, roofs and walls to withstand the relevant design loads per element. For the separating walls this should not be a problem, since holes in these slabs are never desired. For the floor slabs and the front-/back façades however, holes are required to create doors, windows, stairs, vents, etc. These holes significantly influence the resistance of the slabs, and much attention is needed to be paid to the calculations to assure enough safety regarding the stability of the structure. For the calculations of these slabs in this thesis, FEM analysis and the stringer panel model will be applied to analyse the differences of these calculation methods.

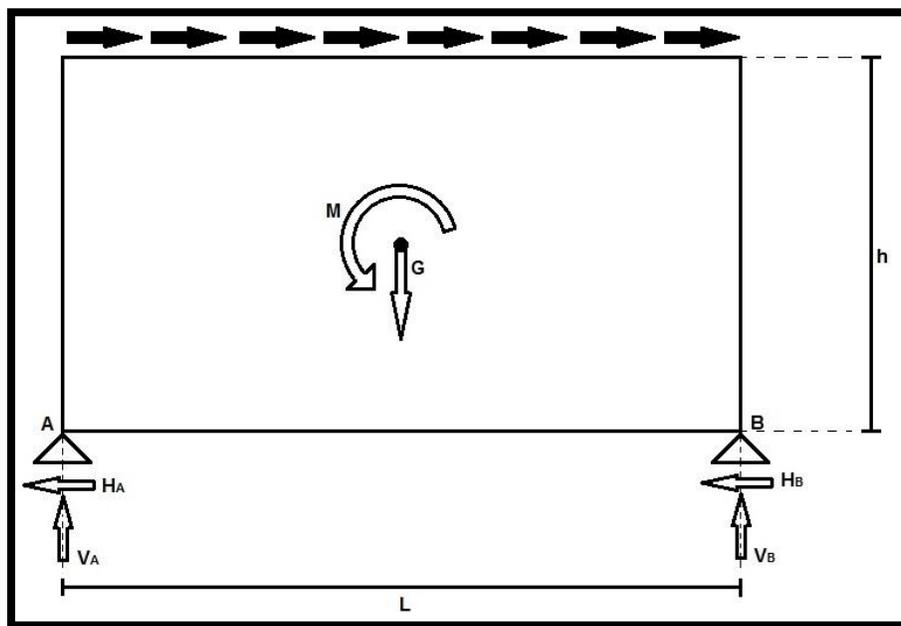


Figure 18 - Horizontal load on stabilizing wall

4. Guideline analysis – Identifying terraced housing building criteria



4.3.2. Functionality (SLS)

The requirements in the serviceability limit state need to ensure for the structure to remain useful for all loading combinations. This means the SLS is a limit state of structural properties like deflection, cracking and vibrations. In this state, the partial safety factor for the design value of persistent and transient loads is set to 1, since functionality is required for the exact critical loading condition. In case of occasional, less-expected exceedance of this load, structural safety has already been assured by applying safety factors in the ULS, but functionality is checked under more general circumstances.

The functional requirements, both static and dynamic, restrict the structural reaction to assure it can still fulfil its function in accordance with the design. This means for example that the floor of a bathroom cannot sag so much that water accumulates to form an indoor puddle at the lowest point, or that a floor cannot vibrate too much when an inhabitant walks across it. These requirements must ensure a feeling of safety for inhabitants and users. The emphasis in this sentence is feeling, because the ULS requirements ensure the actual safety, but a visible structural deformation may cause a feeling of discomfort for users.

4.3.2.1. Static deflections

Depending on the situation for which a design check is performed, three load combinations are applicable:

- Characteristic combination:

$$\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i > 1} \psi_{0,i} Q_{k,i}$$

- Frequent combination:

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

- Quasi-permanent combination:

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



For vertical deflections in the span of horizontal slabs, the national annex of NEN-EN1990 gives five requirements (Figure 19):

- Floors carrying partitioning walls prone to cracking:

$$(w_2 + w_3)_{frequent} \leq \frac{1}{500} l_{rep}$$

- Remaining floors and roofs intensively used by occupants:

$$(w_2 + w_3)_{frequent} \leq \frac{3}{1000} l_{rep}$$

- Remaining roofs:

$$(w_2 + w_3)_{characteristic} \leq \frac{1}{250} l_{rep}$$

- At floor separations located at a height difference:

$$(w_2 + w_3) \leq \frac{1}{150} l_{rep}$$

- If the appearance of the structure is of importance:

$$w_{max} \leq \frac{1}{250} l_{rep}$$

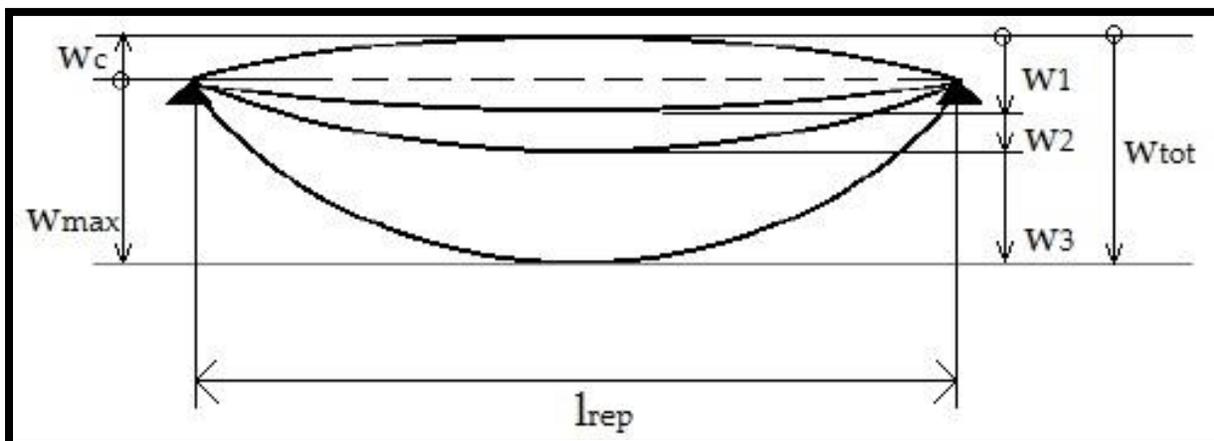


Figure 19 - Vertical deflections; source: NEN-EN 1990

4. Guideline analysis – Identifying terraced housing building criteria

• • •

Besides the requirements for vertical displacements, the national annex of NEN-EN 1990 also gives two requirements for the horizontal displacements of the characteristic load combination (Figure 20):

- Total horizontal displacement per storey:

$$u_i \leq \frac{H_i}{300}$$

- Total horizontal displacement of total building

$$u \leq \frac{H}{500}$$

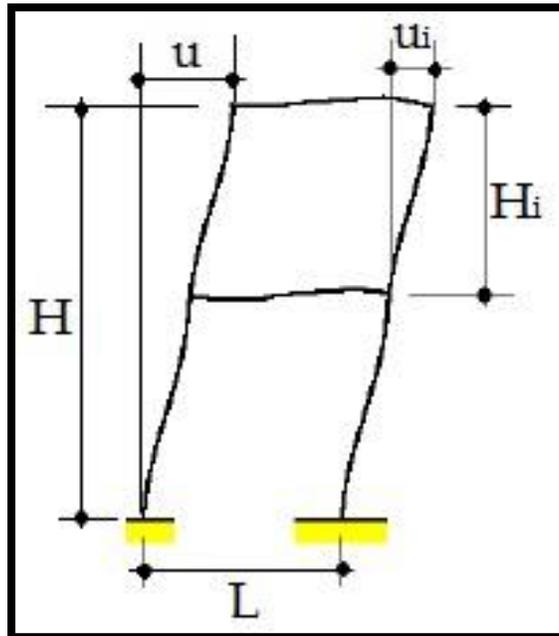


Figure 20 - Horizontal displacements



4.3.2.2. Vibrations (dynamic)

Besides the static loading requirements on the structure, dynamic loading cases are of equal importance for lightweight structures. Dynamic loads can cause bothersome vibrations for inhabitants and decrease the functionality of structural elements, thus influencing the serviceability of the total structure. For this reason, vibrations are checked in the serviceability limit state. The requirements aim to counteract the occurrence of resonance between the natural frequencies of structural elements and the frequencies of induced vibrations due to variable loads.

The Dutch national annex of NEN-EN 1990 gives the following basic requirements:

- The first natural frequency of a floor may not be lower than 3 Hz. This requirement may be neglected if the sum of the permanent and ψ_2 times the imposed load is bigger than 5 kN/m².
- For floors used for dancing or jumping, the first natural frequency may not be lower than 5 Hz.

The requirements above are related to the frequency that human actions can provoke on the structure. The frequencies correspond to a maximum short-term deflection for the quasi-static load combination of 34 mm for a frequency of 3 Hz and 12 mm for a frequency of 5 Hz. In the case of a load bigger than 5 kN/m², it can be assumed that the floor will not noticeably vibrate due to footfall.

Assuming terraced houses are always lower than 20 m high, the wind induced vibrations do not need to be checked separately.

4. Guideline analysis – Identifying terraced housing building criteria



4.4. Fire safety

Fire safety design has two main aims; mitigating human life loss due to a design fire, and preventing fire spread to any adjacent or nearby structure. An important distinction in fire safety is made between two aspects; reaction to fire and resistance to fire. The reaction to fire is a material characteristic. The resistance to fire describes the functionality of structural members in fire conditions. The following aspects of fire safety are considered in the Dutch building decree:

- Strength under fire conditions (section 2.2)
- Limitation of incensive conditions (section 2.8)
- Limitation of the development of fire and smoke (section 2.9)
- Limitation of expanding of fire (sections 2.10 and 2.11)
- Limitation of spreading of smoke (section 2.11)
- Escape paths (section 2.12 and 6.6)
- Firefighting and protection (section 6.7)

For terraced housing structures, mostly the strength under fire conditions and the limitation of expanding of the fire are of importance. The houses will mostly be regarded as consisting of one fire compartment. This compartment must be strong enough to give inhabitants enough time to escape in case of a fire. Additionally, a fire in one house must stay restricted to that house, instead of it leading to the whole row of houses burning down.

4.4.1. Reaction to fire

When designing a structure to satisfy the fire safety requirements, first the reaction to fire of the chosen materials needs to be determined. The reaction describes the combustibility and the behaviour over time of a certain material when it is exposed to extreme heat or direct flames.

Materials are heated in three ways; conduction, convection and radiation. In different fire scenarios, these three ways have different effects on the propagation of a fire. Conduction is the heat transfer occurring through physical contact. This can either take place within a structural member, or between connected members. Convection is heat transfer due to thermal energy carried by fluids. E.g. air will travel upwards when heated (absorbing thermal energy), and travel down when cooling down (releasing thermal energy). Radiation is a result of the emission of electromagnetic waves. All materials radiate thermal energy, but the amount of radiation is dependent on the temperature of that material.



The reaction to fire can be determined through calculations for simple and perfectly uniform materials, though it is quite complex. It is more often determined through testing under a selection of standardized fire scenarios, from which the reaction to all applications and scenarios can be derived. Additionally, the combustibility is tested. A material is considered combustible if it produces more heat than a permitted value, if it catches fire or if it loses mass at a temperature of 750°C.

To start a fire at a certain location, the presence of three components of the “fire triangle” is required. These components are: oxygen, a heat/energy source and a fuel. If any of these components are missing, it is impossible to start a fire. This knowledge is critical in the design of structures, and also shows the importance of the above distinction. The design influences the area of the material which is exposed to fire and air, and is therefore as important to the resistance of fire as the reaction from the material itself.

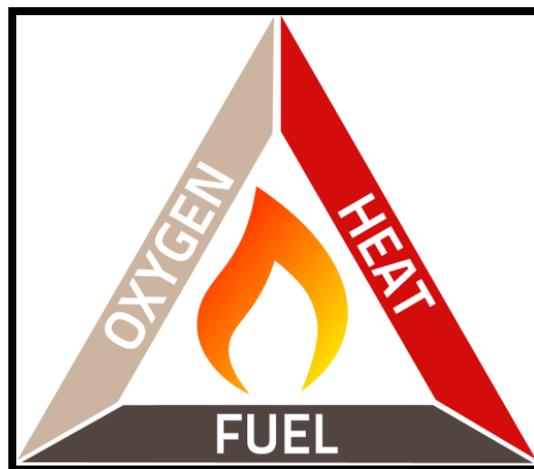


Figure 21 - Fire triangle. source: <https://www.elitefire.co.uk/help-advice/basics-fire-triangle/>

The reaction to fire itself is not a requirement for fire safety of a structure. When designing according to the fire safety requirements, first the reaction to fire of the structural element needs to be determined. With this reaction it can be determined which structural elements keep functioning for a set period of time, leading to an expression of the fire resistance of the overall structure. The classification of the reaction to fire is done through EN 13501-1, which gives the required tests which need to be passed to achieve a certain class (see Table 5).

4. Guideline analysis – Identifying terraced housing building criteria



Class	Required tests
A1	Non-combustibility (EN1182) and Combustion heat (EN1716)
A2	(EN1182) or (EN1716) and Single burning item (EN13823)
B,C,D	(EN13823) and Ignitability (EN11925-2)
E	(EN11925-2)
F	None

Table 5 - Reaction to fire classification

4.4.2. Resistance to fire

The resistance to fire is a structural characteristic rather than a material characteristic. The structure supplies enough strength to keep its shape for a certain required amount of time. With the known reaction to fire of the individual structural member, one can determine what the member will look like after that required time span. This allows for an additional strength check in the ULS, but now the structure is subject to the accidental design situation, given in NEN-EN 1990 as:

$$q_d = \sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \vee \psi_{2,1}) Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$$

The required time for which the structure must endure a fire is given in minutes in table 2.10.1 of the Dutch building decree. For a regular three storey terraced house, it can be assumed that the top floor is located at a height below 7m above ground level. This means the load-bearing elements of the structure need to remain functional for 60 minutes after the start of a fire. During this timeframe, the fire-affected structure is subject to the accidental load combination under fire conditions and must be checked in this situation. The actions on structures exposed to fire are given in NEN-EN 1991-1-2.

One needs to determine the reaction of the building material over the required time before collapse, to determine if the structure can actually withstand the loads. Examples of reaction to fire are the decrease of strength due to high temperatures or a decrease in cross-sectional properties like the area due to burning of the material. With these properties the load resistance is calculated and compared to the loads in fire conditions. When a residential area has no floor levels above 7 m high, and the fire load of the building is smaller than 500 MJ/m², the basic requirement may be lowered to 30 minutes.



The fire load is calculated with NEN6090. The fire load equals the total heat output of a fire compartment including content, divided by the floor area of that compartment:

$$q = \frac{1}{A} \cdot \sum (H_i \cdot m_i)$$

In this formula, q [MJ/m²] is the fire load, A [m²] is the usable area of the structure, H_i [MJ/kg] is the net calorific value of material i , and m_i [kg] is the applied mass of material i .

4.4.3. Fire- and smoke control through internal- and external walls

In the structural design, the spreading of fire from a certain fire compartment into other compartments of the building should be considered. This results in requirements regarding location and size of a fire compartment and regarding resistance against fire penetration and spread. Fire penetration is the expansion of fire through a structural element, whereas fire spread implies the expansion of fire through open air. Consulting the explanatory note of article 2.82 and clause 5 of article 2.83 of the Building Decree, it is stated that different rooms within a residential building are generally considered to be located in the same fire compartment, with the total area of this compartment limited to 1000 m².

Article 2.84 of the Building Decree gives the basic requirement for the expansion of fire from one fire compartment to either another fire compartment or to one of three specific areas which are not located within a fire compartment; a closed area containing an escape route, a non-closed safety exit route and to an elevator shaft containing a fire brigade elevator. This requirement is 60 minutes before expansion of the fire.

Besides the expansion of fire through a separating structural element, an additional requirement could be established with regard to the radiation/conduction from one compartment to another. The Building Decree does not include this topic in their fire safety requirements. A correct fire safety solution incorporates fire related phenomena of e.g. energy transmission. Through radiation and conduction (and to a lesser extent convection), the temperature of structural elements may rise, influencing structural characteristics like strength and stiffness. This aspect is often overlooked, but may be of crucial importance for the fire safety.

4. Guideline analysis – Identifying terraced housing building criteria



4.5. Sustainability of structure

Chapter 3 introduced a concise comparison in terms of the sustainability of timber and concrete. The goal was to give a clear image of the reasons why timber will likely become a more suitable and favorable building material than concrete in the near future. However, after this comparison based on material properties, sustainability remains an important criterion in the consideration between different structural variants within the scope of this MCA. Although these variants do not differ from each other with regard to the raw material they consist of, the different applications can vary significantly in their sustainability performance. This difference is quantifiable on two aspects of the sustainability; the environmental impact of the structure itself and the energy efficiency during the exploitation phase.

4.5.1. Environmental impact of structure

When the design of a structure is completed through the different ULS and SLS checks described in Ch.4.3, this means the total material demand of the structure can be quantified. Together with an estimate of the required energy consumption to put all elements in their place, this creates a full image of the environmental impact of the structure.

With respect to the reapplication of structural elements or materials after demolishing, as stated in Ch.3.8, the final design should give an idea of the possibilities. Which members contain additional materials, influencing the reuse or biodegradability of that member? How easily dismountable are the connections so the structure can be taken apart without damaging the members and thus allowing reapplication without any alteration? Through answering these questions and possibly applying adaptations to increase the performance, reapplication in any way, shape or form can have a very positive influence on the total environmental impact of the structure.

4.5.2. Energy efficiency during exploitation

Given in Ch.3.7 are the basic requirements for the thermal insulation of a building with residential purpose. This thermal insulation needs to assure a building is sufficiently energy efficient by limiting the thermal exchange through external structural members. A thermal resistance is required for the walls, roof and ground floor of 4,5; 6,0 and 3,5 m²K/W respectively.



In EN-ISO 10456 the values for the thermal conductivities of various building materials can be found. The timber processed into cross laminated timber slabs has an assumed density of 500 kg/m^3 . EN-ISO 10456 gives a thermal conductivity for this timber of $0,13 \text{ W/mK}$. With the calculated thicknesses fulfilling the structural requirements, a certain percentage of the requirements for thermal insulation from the Building Decree is reached. For example, if a CLT-slab is 20 cm thick, the thermal resistance is equal to $0,2/0,13=1,54 \text{ m}^2\text{K/W}$. Thus, this slab would fulfill $1,54/4,5*100=34\%$ of the insulation requirement for an outer wall. Considering the same thickness for a concrete slab of medium density ($2000 \text{ [kg/m}^3\text{]}$), with a thermal conductivity of $1,35 \text{ W/mK}$, this has a contribution to the total insulation of a wall slab of 3%. This is a significant difference, especially considering the thermal resistance of the structural element will be worsened even further by the addition of reinforcement in the concrete slab. Also stated in Ch.3.7, the requirements for energy efficiency are currently expressed in the EPC (energy performance coefficient). The EPC is indicated by a unitless value, and the lower this value, the better the energy performance. It is determined by the primary energy consumption for a building with the integration of some correction factor. From the 1st of January 2020, the EPC requirements for the total structure will be replaced as the new BENG requirements get their introduction on this date. BENG contains three requirements; maximum energy demand, maximum primary fossil fuel consumption and a minimum share of energy coming from a renewable source. The newly required maximum energy demand will set new goals regarding mitigation of the loss of energy (heat) through structural elements, possibly resulting in a demand for higher thermal resistances of the outer shell of a building. These requirements show that the sustainability considerations are only related to the energy demand in the exploitation phase of the building. However, the aspiration within the MCA in this research is to create the most sustainable solution for the structure, expressed as the sustainability criterion. This sustainability is not only dependent on the exploitation phase but will be integrated over the total lifecycle of the materials and building, since the environmental influence of a structural element is not only present during the period that it is in use.

4. Guideline analysis – Identifying terraced housing building criteria



4.5.3. *Integration of recyclability, circular- and bio economy*

Recycling has been a common phenomenon for the last couple of decades, receiving much attention in almost every part of human life. Ch.3.8 has additionally introduced the relatively newer terms “circular economy” and “bio economy”. The variants may differ significantly in composition and must be analysed with respect to the terms of waste management. E.g. the recyclability and fit in bio-economy may be negatively affected by the addition of non-degradable glues in one variant that may not be necessary in another variant. These additives may also influence the durability of an element and lead to the necessity of replacement of a structural member during the life of the structure.

The recyclability, fit in circular economy and fit in bio economy will be included in the sustainability criteria of the MCA. The accumulated environmental impact is therefore multiplied by a factor which takes into account the waste management of the structure.



4.6. Breathability and ventilation

The desire for breathable facades is an aspiration rather than a requirement in the building decree, since it is only one of multiple solutions for the same problem regarding the ventilation of a building. The actual requirement relates to the overall air conditioning within a structure to maintain a healthy indoor climate. Air supply and air drainage can be provided through natural- or mechanical systems, resulting in four possible combinations of these systems to meet the requirements. As discussed in 3.6, the breathability of a wall structure regulates the moisture within a space, whether in liquid or gaseous form. The challenge is to determine how the breathability of a façade benefits the purpose of creating a healthy environment in the building; i.e. supplying fresh air and draining exhaled air and moisture. Structural additions to e.g. improve the vapor- or air-tightness may also effect the overall breathability of a structure.

For the ventilation requirement, a distinction is made between a residential area and a residential room. Article 4.3 of the Dutch Building Decree states a residential area has a minimum floor area of 5 [m²], a minimum width of 1,8 [m] and a minimum height of 2,6 m. If these conditions are not met, the space may be considered as a residential room if it has a minimum width of 1,8 m and a minimum height of 2,6 [m]. Spaces with smaller dimensions are not considered to be habitable areas.

Residential areas have to meet a ventilation requirement of 0,9 dm³/s per m² of floor area, with a minimum of 7 dm³/s. For residential rooms this requirement is lowered to 0,7 dm³/s per m² of floor area, while the minimum for these rooms remains 7 dm³/s. For kitchens, toilets and bathrooms, the required ventilation is increased to a minimum of 21, 7 and 14 dm³/s, respectively (Dutch Building Decree art. 3.29). The combination of systems need to assure the requirements are met, meaning a natural ventilation could assure the requirement for regular residential areas, with the addition of a mechanical system to increase the ventilation in the kitchen.

4. Guideline analysis – Identifying terraced housing building criteria



4.7. ACOUSTICS

The propagation of sound is regulated in section 3.4 of the Building Decree, which means it is considered to be an issue related to the health of inhabitants. Distinction is made between propagation within a building and propagation through the external facades to and from other buildings as well as between airborne- and structure-borne sound. The requirements for airborne sound are stated in the BD as the airborne sound level difference, giving a minimum value for the separative structural elements. The structure-borne sound requirement is related to the maximum measured noise level in an adjacent compartment. See Table 6 for the required values.

	Airborne	Structure-borne
<i>External</i>	>52 dB	<54 dB
<i>Internal</i>	>32 dB	<79 dB

Table 6 - Sound insulation requirements

For the internal requirements, the Building Decree gives an exception in article 3.17a.3. which says the requirements do not need to be fulfilled if the space with the sound source is in direct open connection with another space, or if they are connected through a door opening. This statement may create confusion, since the article does not mention openings between spaces on different floors, or spaces connected by a hallway or corridor.

Sound can propagate from one room to another through direct transmission or through flanking paths of the structure. All paths must be considered to reach the desired goal. The characteristic sound insulation of separating members (wall, floor), is calculated with the use of NEN 5077. To reach the required insulation values, a certain damping of the sound vibrations is desired from and between structural members to cover all sound paths.



4.8. Durability

Eurocode 0 (EN 1990) gives the general requirements for structures regarding the durability. Depending on the applied building material and climate in which the structure is placed (e.g. expressed in the service class for a timber structure, exposure class for a concrete structure), this may lead to adaptations to increase long-term performance. Long-term effects are described and handled in the specific Eurocodes for each material. These effects have different causes like long-term loading on the structural members (creep) or harmful natural processes (deterioration or destruction of material).

When designing a structure, the durability must be considered for all structural members, since the different members work together to keep the structure from failing. When applying a certain material to connect members together, this connection might be well designed for short-term effects, but after passing of time may not be sufficient to keep fulfilling all structural requirements.

The following aspects need to be examined for an integrated approach of the durability of a structure:

- Intended exploitation of the structure
- Required design- and calculation criteria
- Expected environmental conditions
- Composition, characteristics and performance of materials and products
- Properties of the soil
- Selection of the structural system (platform framing in this case)
- Shape of members and their structural detailing
- Quality of craftsmanship and the level of supervision/control
- Special protective measures
- Planned maintenance during exploitation

4. Guideline analysis – Identifying terraced housing building criteria



4.9. Building method

4.9.1. *Manufacturability and connections*

The manufacturability of structural elements is possibly the most important criterion when considering whether or not a structure can be built. Structural engineers always strive to create solutions which use the least amount of material and are as cheap as possible. However, one should always consider that if the production process of one type of engineered product is fully automated with multiple manufacturers, it will likely be an economically friendlier option than a newly invented, material-saving solution which requires the setup of a new production line. This consideration is however complex, depending on the size of current and future projects constructible with these types of structural elements. Ultimately, if it is economically verifiable there would be a current and future market for a new building product which is significantly more material- and environmentally friendly than existing products, setting-up a new production line might become a possibility.

With the design of new structural elements, the types of connections should also be investigated, as they are just as important for the final structural design as the elements themselves. Additionally, complex connections may result in problems during the building phase. Creating a new building product should ideally involve equal or simpler connective methods than the existing product it is competing with. E.g. a new type of engineered timber slab should always allow for connections with screws, since CLT slabs can be connected with screws as well.

4.9.2. *Logistics and phasing*

Logistics, like transportation and lifting on site, only play a role if the factories of different variants are located at significantly varying distances from the building site, or if the weight of structural elements per variant differ substantially. This should be investigated in the design phase of a structure since the additional costs to get structural elements in their place could be considerable compared to only the material costs.

The logistics play an important role in the phasing of the project. All steps in the execution phase should be investigated so they are fully conformed to one another. Structural elements should preferably arrive on the building site at the exact moment when they can be lifted into their place in the structure. E.g. the walls at ground level should arrive before the floor slab for the first floor. The complexity of the logistics and phasing criterion depends predominantly on the amount of



prefabrication within the structural elements and the available space on the building site to potentially store materials and products in case of wrong deliveries by manufacturers. A well thought out phasing schedule contains enough risk management and pushes all parties involved in the project to fulfil their agreements and promises. This will guarantee a successful project, since the additional costs due to errors will be mitigated.

4.10. Aesthetics

Aesthetic differences between concrete and timber have already been discussed in Ch.3.9.3. In the decision-making in the MCA regarding the structural variants, aesthetics have an impact as well. Previous criteria may lead to the necessity of additional layers, resulting in the impossibility of leaving the structural material in sight. Depending on the possibilities with regard to the top layer, this affects the overall aesthetics and may lead to individuals choosing for one variant over another. Therefore, investigation is required to which structural elements actually need an additional topping. This can be necessary to e.g. deal with installations and acoustic requirements.



5. *CLT-house of the Future: A structural overview of the variants*

The structure of the terraced house in this report always has a similar composition, uninfluenced by the selection of a certain variant after this chapter. The platform framing method is applied, meaning load-bearing, house-separating walls carry the total structure. The floor and roof slabs are imposed on these wall slabs. The front- and back facades supply the stability in the direction perpendicular to the load-bearing walls.

The connections are desired to be dismountable and sustainable besides their regular function of keeping the structural components attached to each other. The ideal connection type would be by the application of timber connective elements, e.g. dowels. If this ideal connection is not possible due to a lack of strength, steel elements in combination with prefabricated slots in the slabs could be a solution. This will be elaborated for every variant, and additionally for connections between different variants in case one variant is ideal for floor slabs while another is more suitable for the load-bearing walls.

The requirements mentioned in Ch.4 have been elaborated in Appendix A and B for different variants. The structures will be summarized in the next paragraphs of this chapter.

5.1. Minimum CLT design

Most of the strength calculations and the elaboration of the structural performance to create a minimum CLT design for the terraced house structure according to the building criteria supplied by NEN and Rijkswaterstaat are done in Appendix A. A clear design code is missing for the stabilizing function of façades with significantly sized holes, like the front- and back walls of this house, and in current projects, the calculations are done very conservatively leading to overuse of material. With the desire of design freedom for the facades for the possibilities of e.g. big windows and doors, the stability becomes a critical requirement in the dimensioning of the structure. This problem needs to be analysed before the total CLT design can be finalized and visualized.

5.1.1. *Structural analysis of front- and back facades*

The analysis starts with an approximated hand calculation to increase the understanding of CLT shear walls. Due to the platform framing method, the façade consists of three parts with a height of 3 meters, together covering the total height of the building. The loading situation for a row of 6 terraced houses is shown in Figure 22. In Appendix A, this figure has been further elaborated, resulting in the internal

...

forces are shown in Figure A17. In this section, a hand calculation is provided for a shear wall without holes. To allow a hand calculation with the addition of holes, assumptions and simplifications need to be made to approach the real situation. To start, the first simplification of a wall with a single hole in the centre is schematized as a frame as shown in Figure 23.

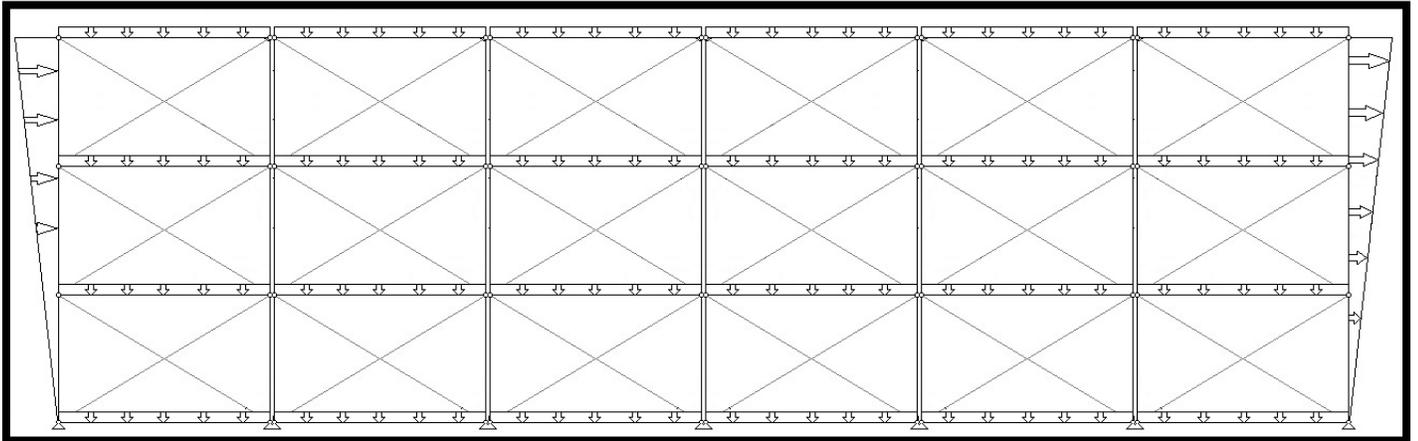


Figure 22 - Loads per row of terraced houses

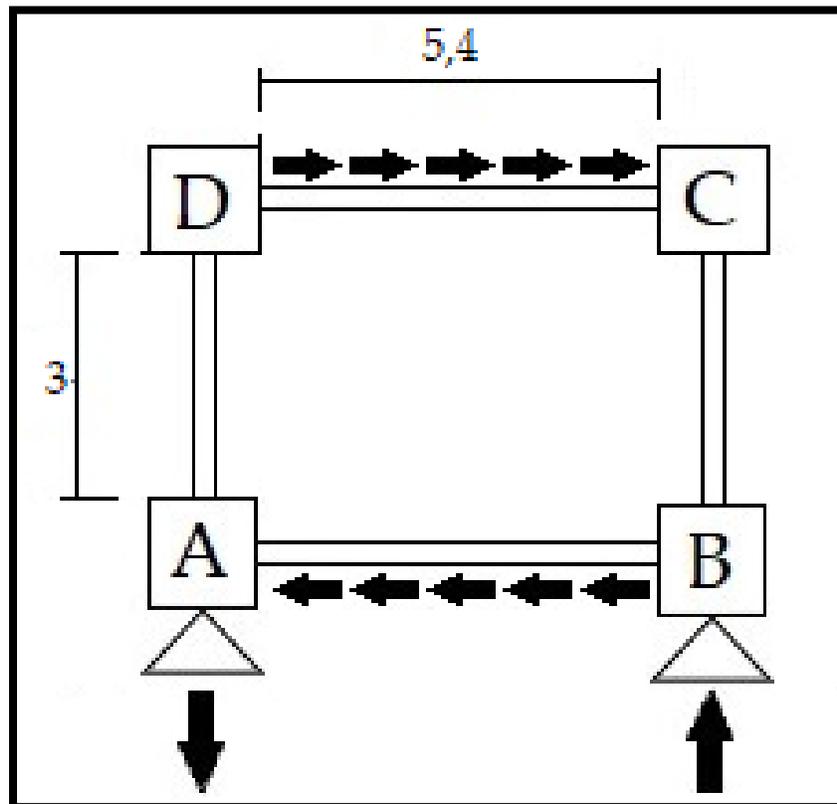


Figure 23 – Framing approach of shear wall with hole

5. CLT-house of the Future: A structural overview of the variants



The rods between nodes A,B,C and D have the cross-sectional properties of the layers with the grain parallel to the direction of the rods. Within the nodes, internal bending moments, axial and shear forces are transferred from layers in one direction to the other. The resistances to these internal loads will be discussed in a later part.

In Appendix A, the horizontal wind load on the top rod was determined as 12,53 kN/m. It causes an equal reactional distributed load over the length of the bottom rod, where it is connected to the foundation with shear connectors. Nodes A and B are supported in vertical direction by the foundation (compression) and with tensile plate connectors. The nodes and rods in the frame can be separated to show the (direction of) the internal forces and moments, as shown in Figure 24.a. The (external) reaction forces in Figure 24.a are calculated as follows:

$$\sum T_A = 12,53 \cdot 5,4 \cdot 3 - V_{B,ext} \cdot 5,4 = 0 \rightarrow V_{B,ext} = V_{A,ext} = 37,6 \text{ kN}$$

With the external loads known, the internal forces and moments can be determined. Considering the horizontal wind load distributes evenly over the two vertical rods cause of horizontal symmetry of the structure, this leads to:

$$H_C = H_D (= H_A = H_B) = \frac{1}{2} \cdot 12,53 \cdot 5,4 = 33,8 \text{ kN}$$

The horizontal internal forces in nodes C and D are conducted vertically through the rods so the horizontal internal forces in nodes A and B are also equal. Due to vertical symmetry of the structure, the internal bending moments are considered to be equal for the corners. Consider the isolated rod between A and D with the internal loads:

$$\begin{aligned} \sum T_D &= M_D + M_A + 3H_A \\ M_D = M_A (= M_B = M_C) &= \frac{1}{2} \cdot 3 \cdot H_A = 50,7 \text{ kNm} \end{aligned}$$

Considering the isolated rod between D and C:

$$\sum T_D = M_D + M_C + 5,4V_C \rightarrow V_C = V_A = V_B = V_D = 18,8 \text{ kN}$$

All internal forces are known, allowing the creation of the axial force-, shear force- and bending moment diagrams (Figure 24.b). This hand calculation can be verified with simple framing software like Dr. Frame, as shown in Figure 25 and Figure 26.

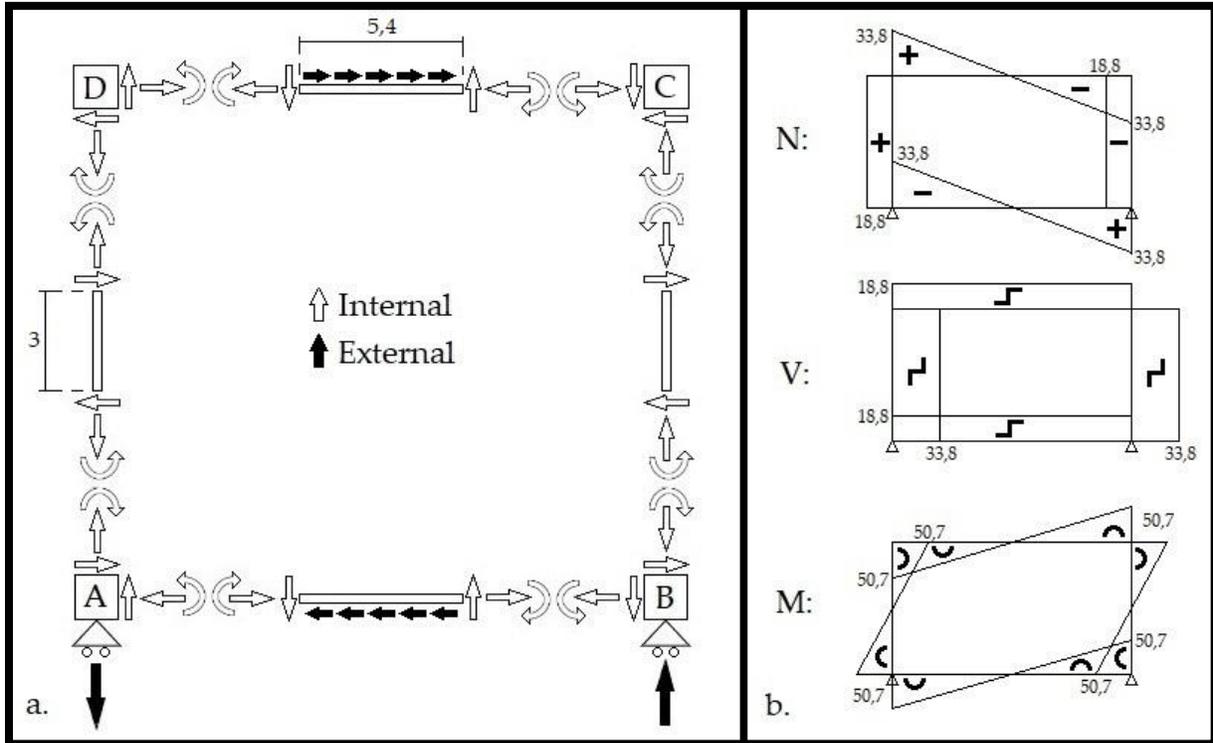


Figure 24 – a. Internal and external forces in framework, b. axial force, shear force and bending moment diagram

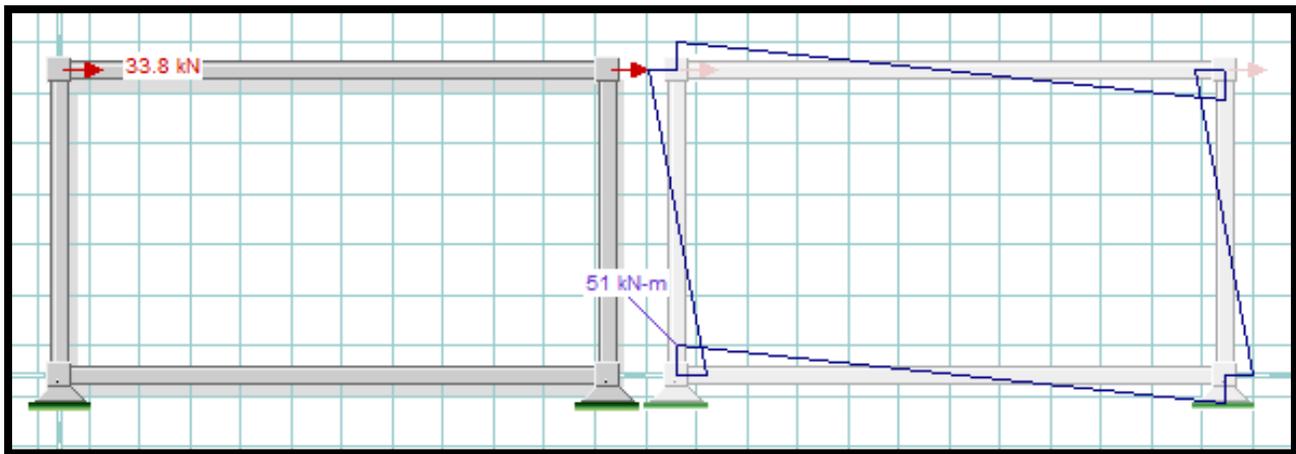


Figure 25 – Bending moment diagram (Dr. Frame)

5. CLT-house of the Future: A structural overview of the variants

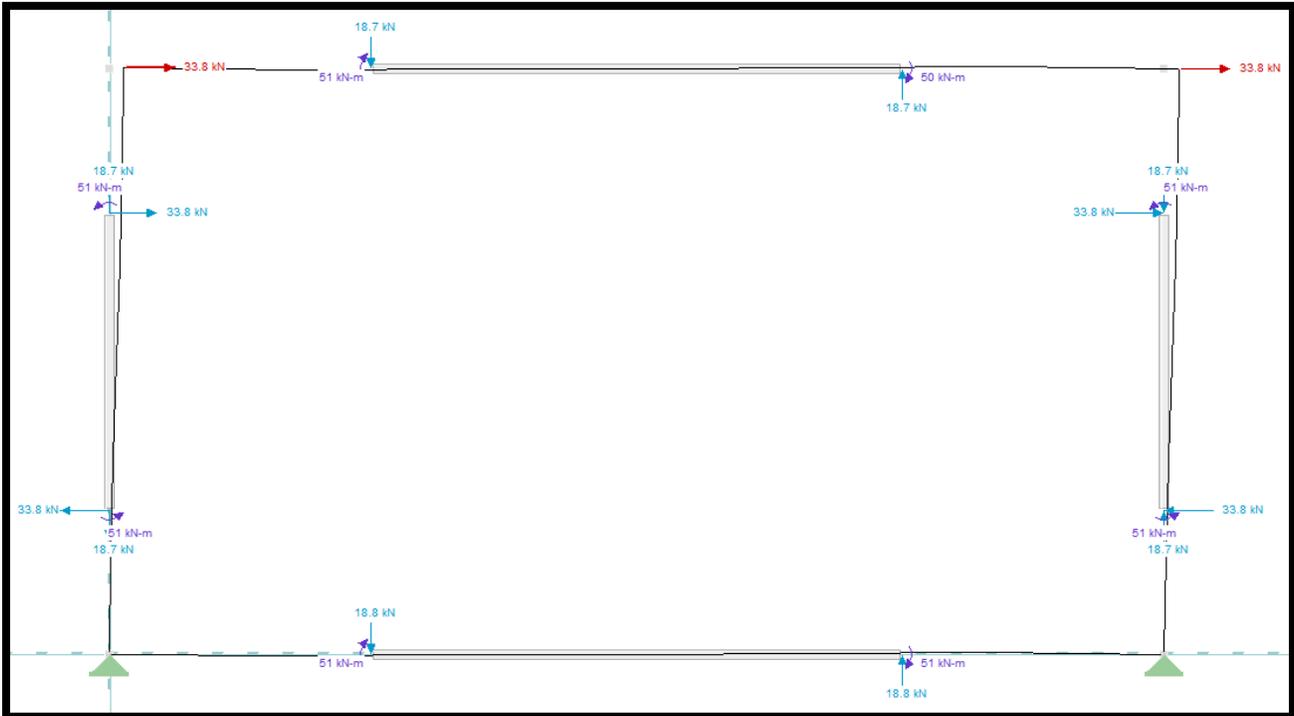


Figure 26 – Isolated members in frame (Dr. Frame)

For a 3 layered CLT slab of 98 mm thick, the vertical rods consist of 2 layers of 150mm high boards with a width of 29 mm. The rod is checked for internal axial- and shear stresses in the critical cross-section, located at the nodes where the maximum bending moment occurs. The internal bending moment between the rod and the node is:

$$M = 50,7 \cdot \frac{(1,5-0,45)}{1,5} = 35,5 \text{ kNm}$$

The design strengths of C24 timber are given in Appendix A as:

- Compression: $\sigma_{c,0,d} = 0,8 \cdot \frac{21,5}{1,25} = 13,76 \text{ N/mm}^2$
- Tension: $\sigma_{t,0,d} = 0,8 \cdot \frac{17}{1,25} = 10,88 \text{ N/mm}^2$
- Shear: $\sigma_{v,0,d} = 0,8 \cdot \frac{3,5}{1,25} = 2,24 \text{ N/mm}^2$

Due to the presence of cross layers, the internal moment of inertia is assumed to have an effective height of the total height of the vertical rod, since the cross boards act as shear connectors between the individual boards. The internal stresses in the cross-section of the vertical rod are determined for different numbers of boards (n), assuming a constant (shear) stress distribution (according to prEN16351):

$$\sigma_V = \frac{N}{A} + \frac{My}{I}, \tau_V = \frac{V}{A}$$

n	h_{total} [mm]	A_{net} [mm ²]	I_{net} [mm ⁴]	σ_V [N/mm ²]	UC	τ_V [N/mm ²]	UC
1	150	8,70E+03	1,63E+07	165,38	15,20	3,89	1,73
2	300	1,74E+04	1,31E+08	41,89	3,85	1,94	0,87
3	450	2,61E+04	4,40E+08	18,86	1,73	1,30	0,58
4	600	3,48E+04	1,04E+09	10,74	0,99	0,97	0,43
5	750	4,35E+04	2,04E+09	6,96	0,64	0,78	0,35
6	900	5,22E+04	3,52E+09	4,89	0,45	0,65	0,29

Table 7 – Strength check vertical rod

So the cross-section of the vertical rod for 4 boards satisfies the strength requirements.

The horizontal rods consist of 1 layer of 150mm high boards and a width of 40mm. Again the bending moment and axial force have a maximum value at the nodes. The bending moment and axial force in the critical location of the horizontal rod have values of:

$$N = 33,8 \cdot \frac{(2,7-0,9)}{2,7} = 22,5 \text{ kN}, M = 50,7 \cdot \frac{(2,7-0,9)}{2,7} = 33,8 \text{ kNm}$$

n	h_{total} [mm]	A_{net} [mm ²]	I_{net} [mm ⁴]	σ_H [N/mm ²]	UC	τ_H [N/mm ²]	UC
1	150	6,00E+03	1,13E+07	229,08	21,06	3,13	1,40
2	300	1,20E+04	9,00E+07	58,21	5,35	1,57	0,70
3	450	1,80E+04	3,04E+08	26,29	2,42	1,04	0,47
4	600	2,40E+04	7,20E+08	15,02	1,38	0,78	0,35
5	750	3,00E+04	1,41E+09	9,76	0,90	0,63	0,28
6	900	3,60E+04	2,43E+09	6,88	0,63	0,52	0,23

Table 8 – Strength check horizontal rod

The stress checks for the timber cross-sections in the nodes are similar to the previous calculations for the rods. Additionally, in the nodes, the stresses are conducted from one layer to another, leading to requirements for the contact area between the boards. The resistance of the connection is an accumulation of the resistances of all contact areas (Figure 27).

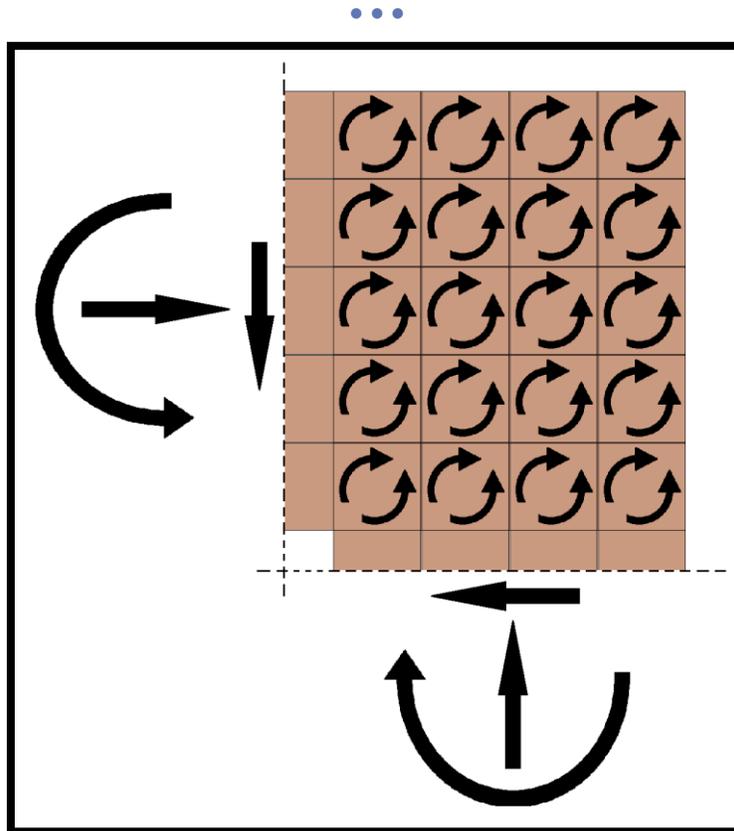


Figure 27 - CLT-node schematization

In case of proper production, the glue layer may be assumed to be stronger than the resistance to sliding of the timber. Three failure modes are distinguished for CLT when subjected to shear stresses (Flaig & Blaß, 2013). These failure modes are incorporated in the latest draft of the CLT guidelines for EN1995.

1. Shear failure parallel to the grain in the gross cross section; occurs between unglued joints with equal stresses in longitudinal- and transverse layers (Figure 28.a.)

For this failure mechanism to take place, layers in both directions need to shear off simultaneously in parallel direction to the grain. This means in our case that the shear stress at a certain location needs to remain lower than the shear strength of the five layers in that same location.

2. Shear failure perpendicular to the grain in the weak net cross section; occurs in lamellae perpendicular to the joints. (Figure 28.b.)

In this mechanism, only the net cross-section needs to shear off in perpendicular direction for the total slab to fail. In this case the boards can freely slide along each other.



3. Torsional shear failure within the crossing-areas (laminations) between orthogonally bonded lamellae (Figure 28.c.)

In this mechanism, failure in the contact areas between the board layers occurs such that the boards can freely rotate relative to each other. The strength is determined as a combination of the rolling shear strength and the torsional shear strength, since the two act simultaneously in this failure mechanism.

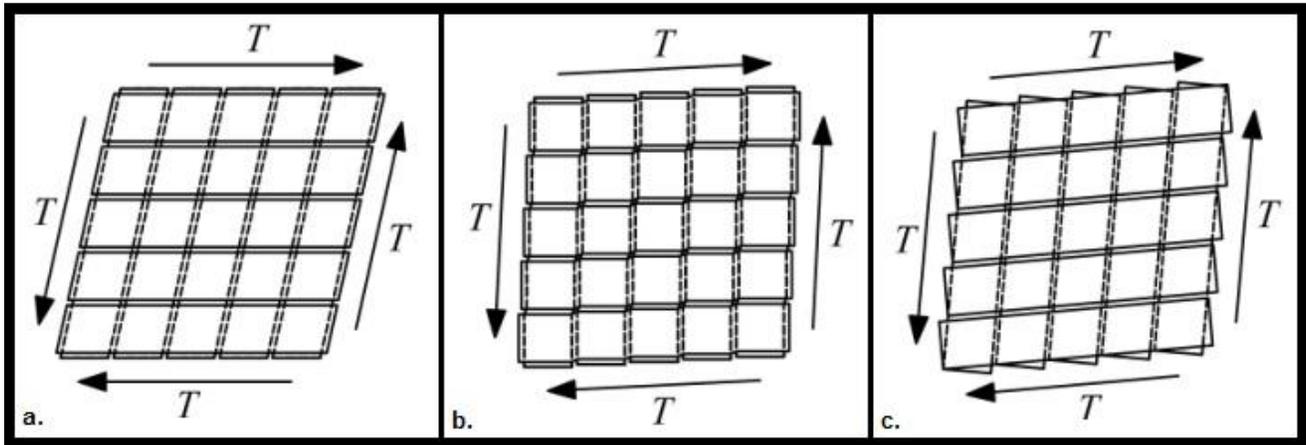


Figure 28 - In-plane shear failure modes; a. gross cross section failure parallel to grain, b. net cross section failure perpendicular to grain, c. torsional failure in lamination layers. Source: (Flaig & Blaß, 2013), edited by author

Formulae have been drafted for the three failure modes to determine the shear strength of a slab (Flaig & Blaß, 2013). Characteristic values for the shear strengths in different directions for CLT can be found in the latest draft of CLT guidelines for EN1995.

$$f_{v,CLT} = \min \left\{ \begin{array}{l} f_{v,lam,0} \\ f_{v,lam,90} \cdot \frac{t_{net}}{t_{gross}} \\ \frac{b \cdot n_{CA}}{2t_{gross}} \cdot \frac{1}{\frac{1}{f_{v,tor}} \cdot \left(1 - \frac{1}{m^2}\right) + \frac{2}{f_R} \cdot \left(\frac{1}{m} - \frac{1}{m^2}\right)} \end{array} \right.$$

Where t_{net} is the accumulated thickness of the boards in weak direction, t_{gross} is the total thickness of the CLT slab, n_{CA} is the number of laminations in the crossing area, b is the board width and m is the total number of boards in weak direction over the full length of the slab.

5. CLT-house of the Future: A structural overview of the variants



The formulae convert the net shear strength and rotational shear strength into a standardised value for the general shear strength in the cross-section of the CLT. Therefore the shear stress due to the wind load can directly be compared to the minimum value of these formulae.

The timber shear strengths corresponding to the failure modes are found in literature as:

1. The shear strength parallel to the grain of combined laminated spruce is found in the Eurocode and in literature as 3,5 [N/mm²] (Schickhofer, Brandner, & Bauer, 2016). The horizontal load is introduced in the slab through multiple connecting elements along the top of the slab. Therefore the resulting force must be divided over the total length and over the thickness of the wall.
2. The shear strength perpendicular to the grain in the net cross section is not given in the Eurocode. Literature gives a value of 5,5 N/mm² for this shear strength (Schickhofer et al., 2016). This should be a safe assumption considering our board thicknesses. In our slab the weak direction consists of 2 boards of 23 mm thick. We assume the gaps between the lamellas are smaller than 6 mm.
3. The torsional shear strength of timber is given in literature as 2,5 N/mm² (Schickhofer et al., 2016), the rolling shear strength is 0,8 N/mm² for C24, given in the latest draft of prEN16351.

Assume a thickness of 98 mm with a 29-40-29 layered composition. Following the strength checks of the rods, the number of boards is 4 in vertical direction and 5 in the horizontal direction for the node. The minimum expression for torsional failure strength is found for the minimal number of boards, being 4. This leads to the following expressions for the accumulated shear strength of the total CLT slab:

$$f_{v,CLT} = \min \left\{ \begin{array}{l} 3,5 \\ 5,5 \cdot \frac{58}{98} = 3,26 \vee 5,5 \cdot \frac{40}{98} = 2,24 \\ 150 \cdot 2 \cdot \frac{1}{2 \cdot 98 \cdot \left(\frac{1}{2,5} \cdot \left(1 - \frac{1}{4^2}\right) + \frac{2}{0,8} \cdot \left(\frac{1}{4} - \frac{1}{4^2}\right) \right)} = 1,81 \end{array} \right.$$

The governing shear failure mode is torsional failure in the timber at the contact areas between the boards, with a frictional shear strength of 1,81 N/mm² over the full thickness of the wall, since both the torsional shear and the rolling shear contribute in this strength formula. This minimum value insinuates that the shear strength of an in-plane loaded slab is always higher than pure rolling shear,



since the restricted rotation in each contact area between the timber layers strengthens the parts of the boards that would individually be subject to rolling shear. This means every contact area has a maximum frictional shear resistance independent on the direction of the stress.

Moments, horizontal and vertical loads on the node are distributed evenly over the number of contact areas in the node, with stress distributions of the different components shown in.

$$M_i = \frac{M_C}{n_{lam}} = \frac{50,7}{n_{lam}}$$

$$F_V = \frac{V_C}{n_{lam}} = \frac{18,8}{n_{lam}}$$

$$F_H = \frac{H_C}{n_{lam}} = \frac{33,8}{n_{lam}}$$

The bending moment stress distribution is equal for horizontal and vertical direction due to the equal board width ($h = b = 150\text{mm}$). This results in frictional stress expressions in vertical and in horizontal direction, which need to be smaller than the allowable frictional shear stress of $1,81 \text{ N/mm}^2$:

$$\sigma_V = \frac{F_V}{hb} + \frac{M_i}{\frac{1}{6}bh^2} = \frac{18,8 \cdot 10^3/n_{lam}}{150^2} + \frac{50,7 \cdot 10^6/n_{lam}}{\frac{1}{6} \cdot 150^3} \leq 1,81 \rightarrow n_{lam} \geq 50,26$$

$$\sigma_H = \frac{F_H}{hb} + \frac{M_i}{\frac{1}{6}hb^2} = \frac{33,8 \cdot 10^3/n_{lam}}{150^2} + \frac{50,7 \cdot 10^6/n_{lam}}{\frac{1}{6} \cdot 150^3} \leq 1,81 \rightarrow n_{lam} \geq 50,62$$

The contribution of the axial forces to the frictional shear in the contact area is negligible in comparison to that of the bending moment. The checks show a minimum number of contact areas of 51. Given some simplifications in terms of boundary conditions and load distributions have to be made to allow a hand calculation, this leads to an over dimensioning, and for now a composition of 5 vertical boards and 5 horizontal boards will be assumed, meaning we have 2 laminations of 5 times 5 contact areas, equals $n_{lam} = 50$.

Concluding this hand calculation, the vertical- and horizontal rods have a “depth” of 750mm. Up till now, only a change in the number of boards has been investigated, limiting the dimensions of the hole to approximately 3900 by 1500 mm. Changing the composition of the CLT from 29-40-29 to thicker layers leads to lower stress results, since the cross-sectional properties change, but the principle of the calculations remains unchanged.

5. CLT-house of the Future: A structural overview of the variants

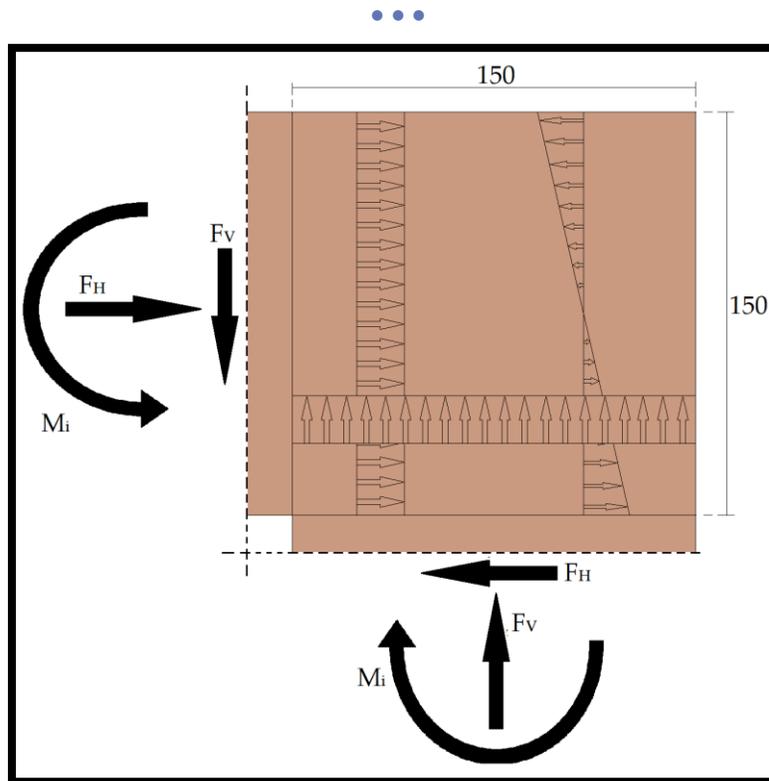


Figure 29 - Contact area; loads and stresses

Finite Element Analysis:

To further analyse the situation, other slab options in terms of hole locations and hole sizes need to be investigated, especially for door holes at the bottom of the walls. In case of terraced housing, the front- and back facades are not often symmetric. This leads to issues for the schematization of the slabs as frames, and makes the hand calculations very complex. To further investigate these slabs, the finite element method (FEM) could provide assistance in the calculations. This method is versatile and allows less need for simplification than the hand calculation. Special attention is needed to interpret and evaluate the results, as errors are easily overlooked in the different stages of finite element analysis (FEA). In this case, ANSYS workbench will be used since this software application is ideal for the analysis of multiple solids, meaning ideal for the analysis of boards with different characteristics due to their orientation. A static structural analysis (linear elastic) is performed on the wall, since the wind load is determined according to the Eurocode including dynamic effect factoring, meaning a dynamic analysis is not required by itself.



1. Material input

The first step in the process is the input of engineering data. This is where the material characteristics are entered for two different materials, as this simplifies the distinction between boards in horizontal and in vertical direction. C24 timber characteristics are entered for the grain direction in the x-direction and for the grain direction in the z-direction (Table 9).

C24	Grain direction		C24	Grain direction	
	x	z		x	z
$P [kg/m^3]$	500	500	Orthotropic Stress Limits		
Orthotropic Elasticity			$f_{t,x} [N/mm^2]$	14,5	0,4
$E_x [N/mm^2]$	11000	370	$f_{t,y} [N/mm^2]$	0,4	0,4
$E_y [N/mm^2]$	370	370	$f_{t,z} [N/mm^2]$	0,4	14,5
$E_z [N/mm^2]$	370	11000	$f_{c,x} [N/mm^2]$	-21	-2,5
$\nu_{xy} [-]$	0,03	0,9	$f_{c,y} [N/mm^2]$	-2,5	-2,5
$\nu_{yz} [-]$	0,9	0,03	$f_{c,z} [N/mm^2]$	-2,5	-21
$\nu_{xz} [-]$	0,03	0,03	$f_{v,xy} [N/mm^2]$	3,5	0,8
$G_{xy} [N/mm^2]$	690	69	$f_{v,yz} [N/mm^2]$	0,8	3,5
$G_{yz} [N/mm^2]$	69	690	$f_{v,xz} [N/mm^2]$	3,5	3,5
$G_{xz} [N/mm^2]$	690	690			

Table 9 – Material input ANSYS workbench

2. Geometry input

For the second step, Workbench redirects the user to SpaceClaim to enter the geometry. In this program, only the solids are created, without yet assigning material characteristics and boundary conditions. To assure a correct cooperation between the different boards inside a CLT slab, this distinction between boards is defined here by creating separate solids for each board. In an attempt to get sufficient overview of the structure and the individual boards, layers can be distinguished by creating separate components (Figure 30). In this model, a spacing of 1 mm is chosen between the boards to create space for deformations.

Another component is created called “Voids”, in which the holes are modelled as solids to “cut” the different layers with. This way, after modelling the regular, hole-less CLT slab (Figure 31), different files for hole compositions are easily created. In this model, the dimensions of the slab differ slightly from the real situation so the model consists only of whole boards. The holes will always be designed to totally cut through boards as well.

5. CLT-house of the Future: A structural overview of the variants

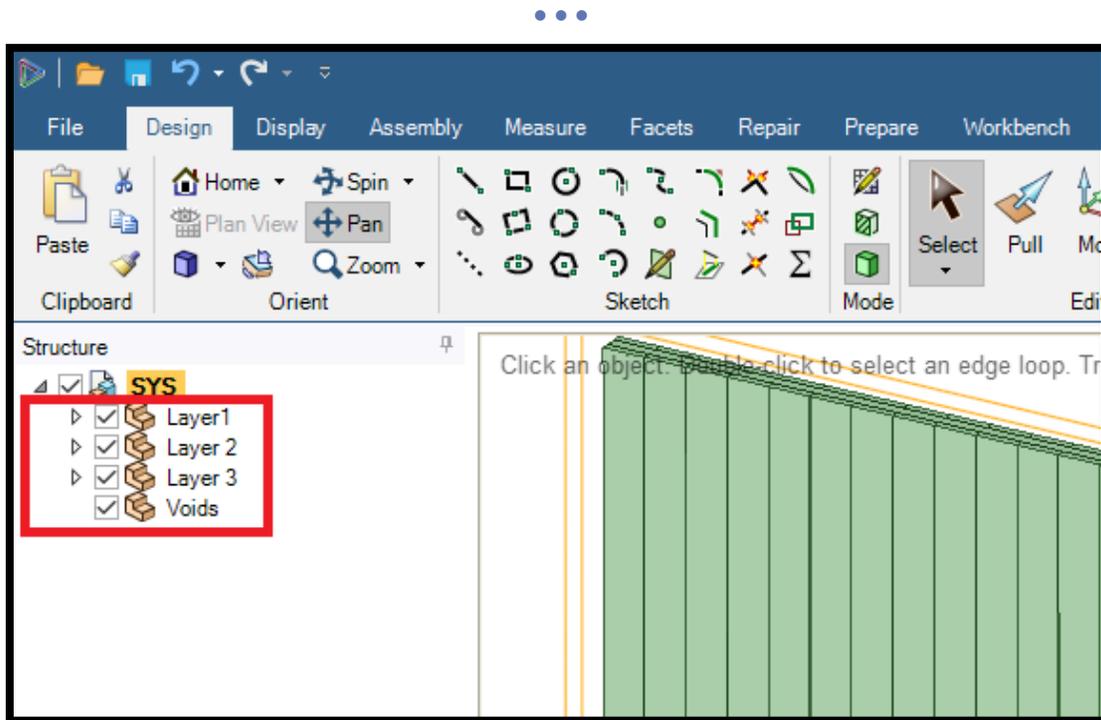


Figure 30 - Layered input SpaceClaim, ANSYS

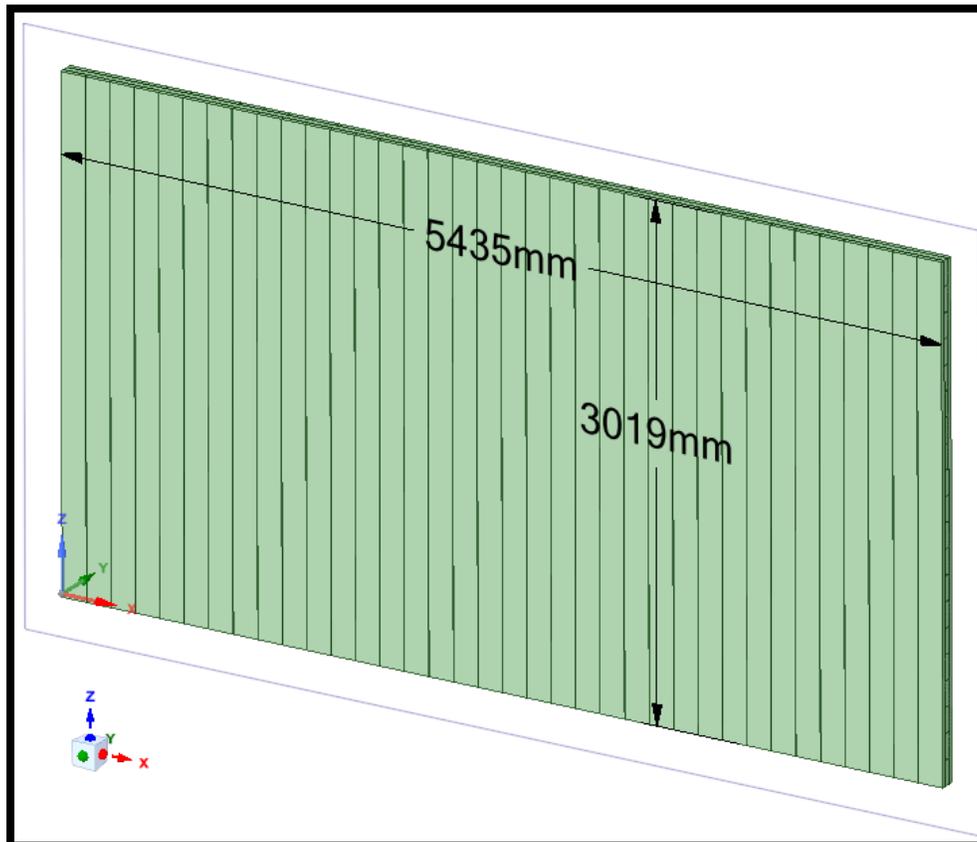


Figure 31 - Hole-less CLT slab ANSYS



3. Modelling CLT

The final steps of the finite element analysis are done in the Mechanical extension for ANSYS for the Static Structural analysis. In the third step, the previous steps are combined as the material characteristics are attached to the correct solids, and the solids are connected to properly resemble CLT. In the Outline window of the project, solids can be selected in the Geometry folder and in the detail window, the material selection is changed. As shown in Figure 32, solids in layer 1 in this model are directed in the vertical (z-)direction, and therefore get assigned the C24 properties that match this direction. Using the layered division as described in step two makes it easier in this step to select solids in the same layer to assign the same material, since the grain direction in these boards is similar.

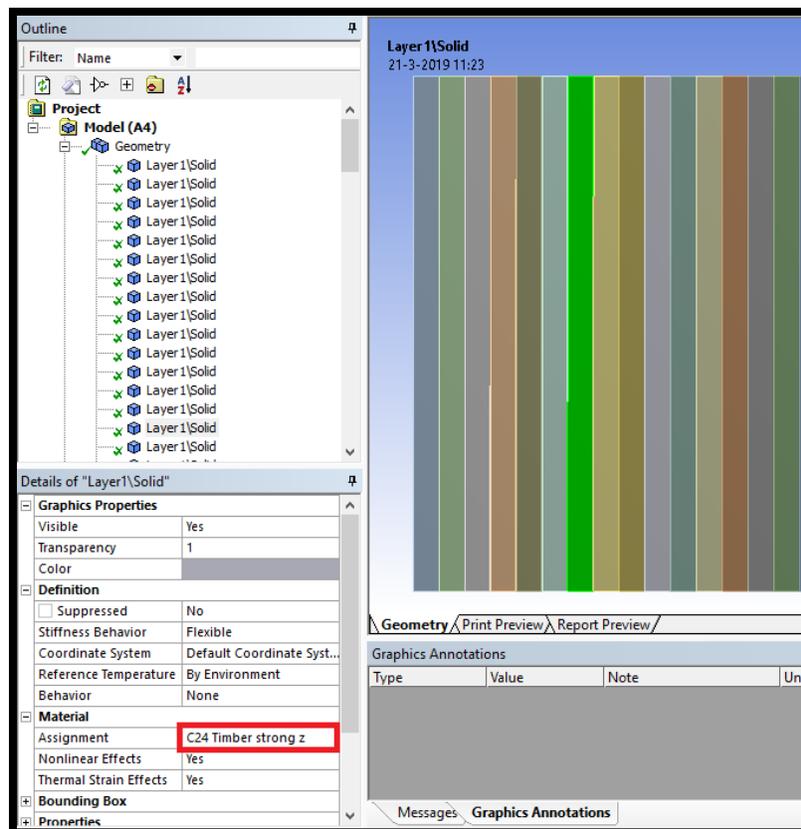


Figure 32 - Material assignment ANSYS

The second part of modelling correct CLT slabs is to assure correct contact areas between the boards. Glue is only present in the lamination layers between the cross-layered timber, and the board edges are non-bonded. The Mechanical extension automatically generates contact areas between the boards (including the board edges), even when applying a spacing of 1 mm between the boards.

5. CLT-house of the Future: A structural overview of the variants

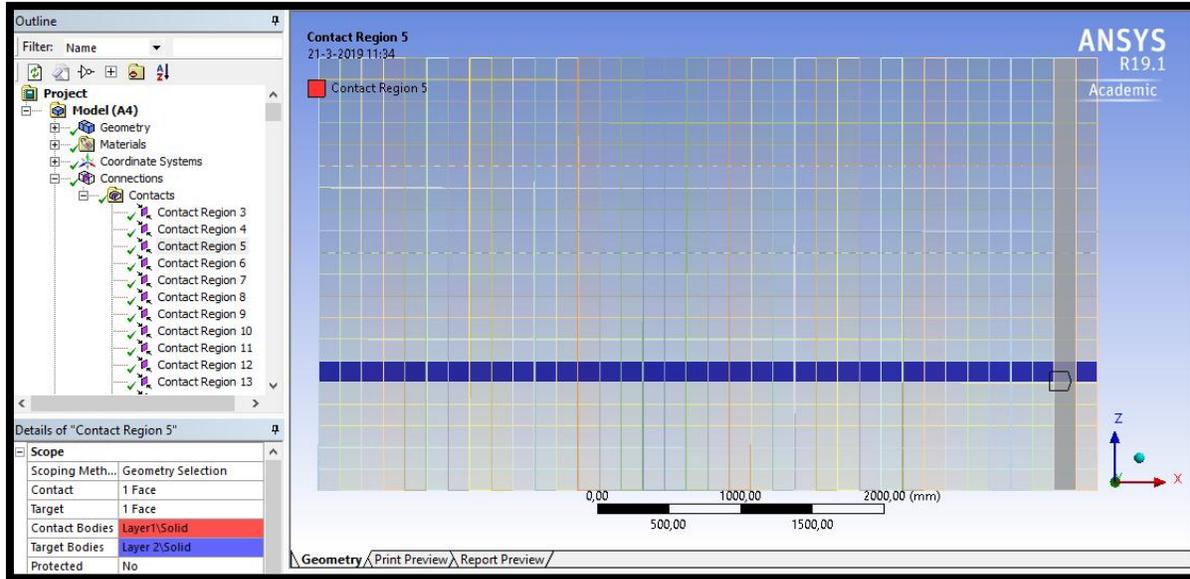


Figure 33 - Contact areas ANSYS

Under the Connections folder in the Outline window, contact regions can be selected and checked in the visualization and detail windows. This is an extensive task, since the user needs to run past every contact region to assure it connects boards in different layers. Contact regions between similar board layers need to be deleted from the list. Finally, the contact regions within one lamination layer are merged for easier analysis of the frictional stresses between the boards (Figure 34).

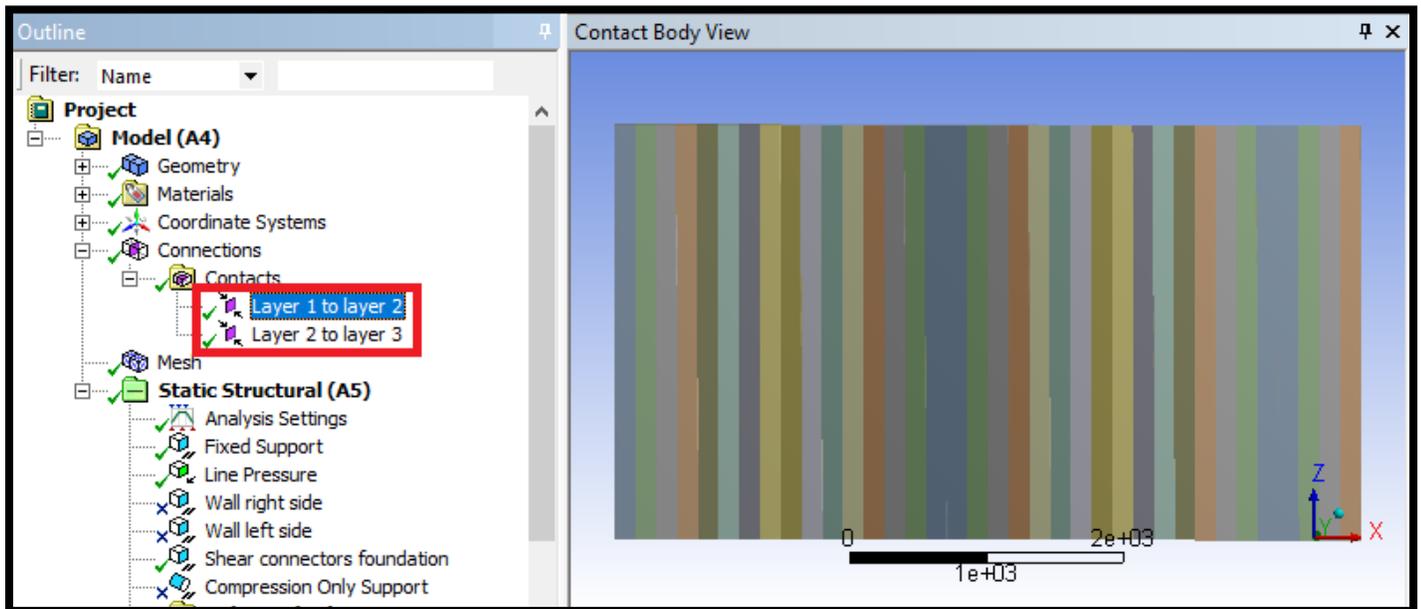


Figure 34 - Merged contact areas



4. Generating mesh

Mechanical gives a simple method to generate the mesh. Selecting Mesh in the Outline window allows the input of a mesh size in the details (Figure 35). Right-clicking on Mesh shows the option to automatically generate the mesh.

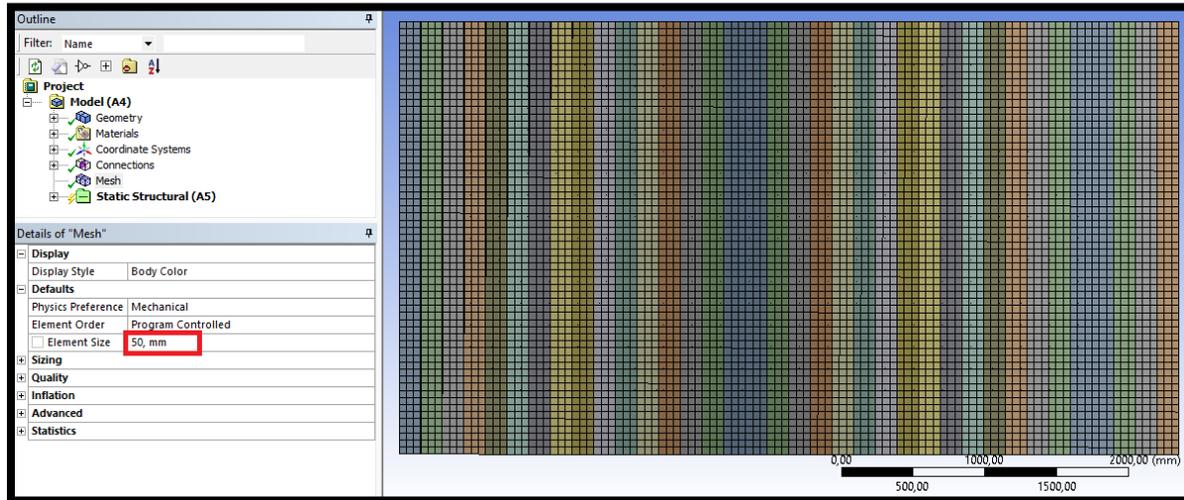


Figure 35 - Mesh input ANSYS

5. Boundary conditions

The boundary conditions are inserted by right-clicking the Static Structural folder in the Outline window. To recreate the situation from the hand calculation, the bottom of the slab is connected to the foundation with shear connectors over the full length, and fixed in the corners to conduct tension and compression due to the in-plane moment. The fixed part is assumed to be 5 boards wide, since this is the minimum width of the vertical rod according to the hand calculation. A line load is applied along the top of the slab with a total value of 12,53 kN/m, but introduced over the full width of the slab and therefore divided into two line loads of 6,27 kN/m. Additional boundary conditions can be inserted to better recreate the real situation, but this leads to a deviation from the solution produced with the hand calculation. The boundary conditions are shown in Figure 36.

5. CLT-house of the Future: A structural overview of the variants

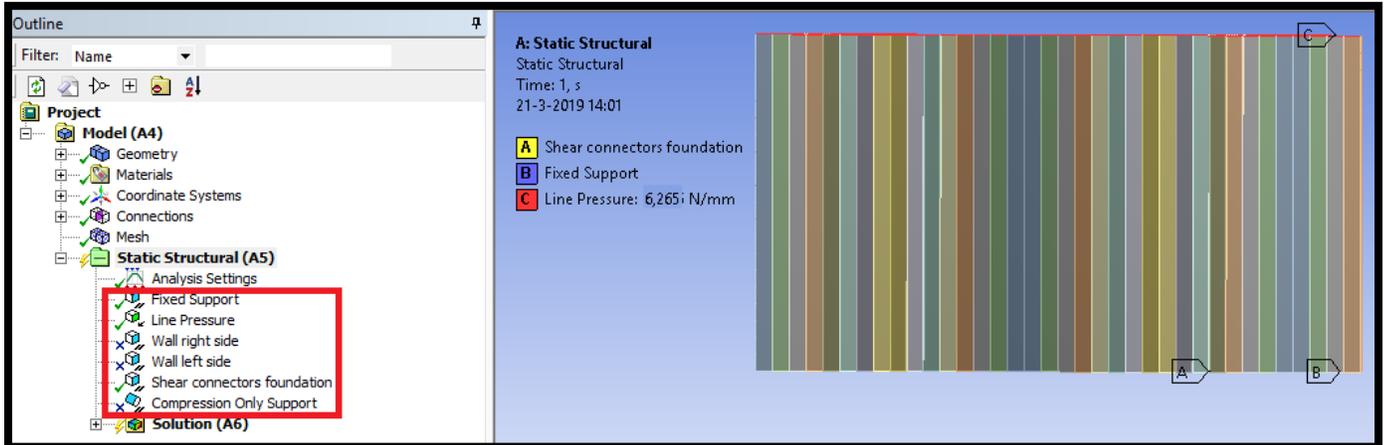


Figure 36 - Boundary conditions ANSYS

6. Solution

Mechanical provides many solutions for the static structural analysis which can be inserted by right-clicking the Solution folder in the Outline window. The user must determine which results are required for correct analysis and evaluation of the structure that is analysed. In the comparison with the hand calculation, the normal stresses in x- and z-direction are relevant, as well as the shear stress in the x-direction on the z-face of the element (or in the z-direction of the x-face)(perpendicular to the grain). Additionally, the deformation and the frictional stress in the contact areas are generated to gather all necessary information to determine whether the wall can provide the stability for the building. Right-click the Solution folder to solve.

7. Results

The first considered slab composition is the hole-less slab, to secure correct CLT modelling. In the results, stress distributions should be visible within individual boards. However, the total load in the CLT slab is assumed to be distributed equally over the total dimensions (CEN, 2018), and therefore the contribution per board should be approximately equal.

Figure 37 shows the normal stress in x-direction in the horizontal layer of the slab. Stress peaks are only apparent in the top corners of the slab due to the load introduction on this layer in this location. In this layer it is conducted towards the vertical layers, from where it is conducted towards the foundation through the shear resistance of these layers.

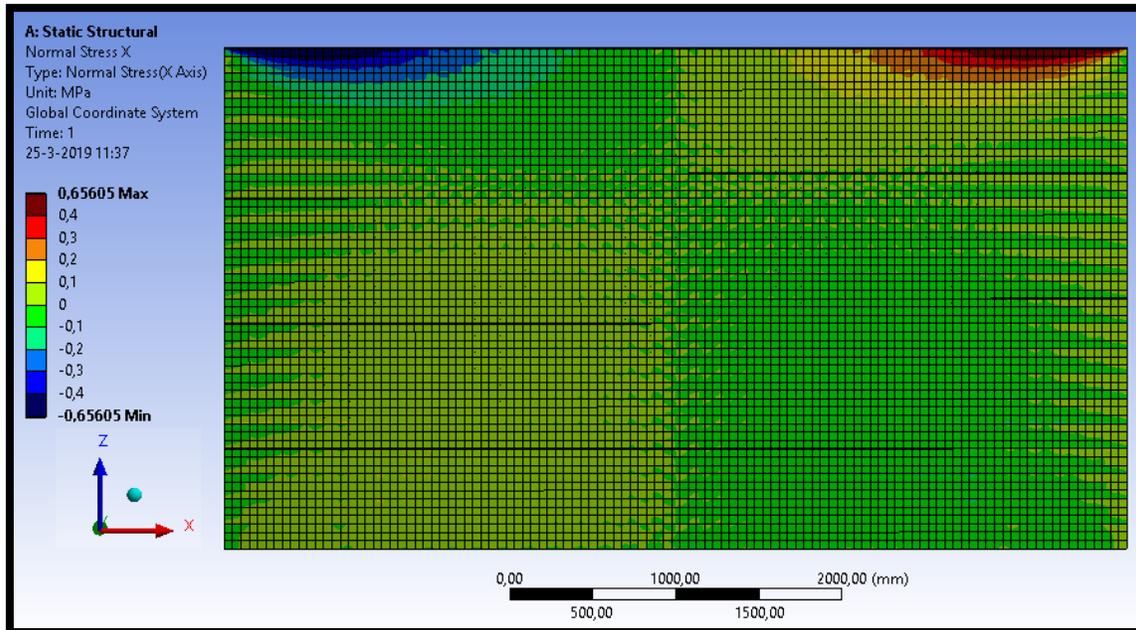


Figure 37 - Axial stress in x-direction in the horizontal (middle) layer, no hole

Figure 38 shows the normal stress in z-direction in the vertical layers of the slab. Peaks are visible in the bottom corners, at the location of the reaction forces in the fixed supports due to the in-plane moment. From the corners, the stress is distributed by the cross-layers, reducing the stress in the rest of the slab to a minimum. The max and min axial stress values in Figure 37 and Figure 38 are smaller than the compressive- and tensile strengths of C24 timber of 13,76 N/mm² and 10,88 N/mm², respectively.

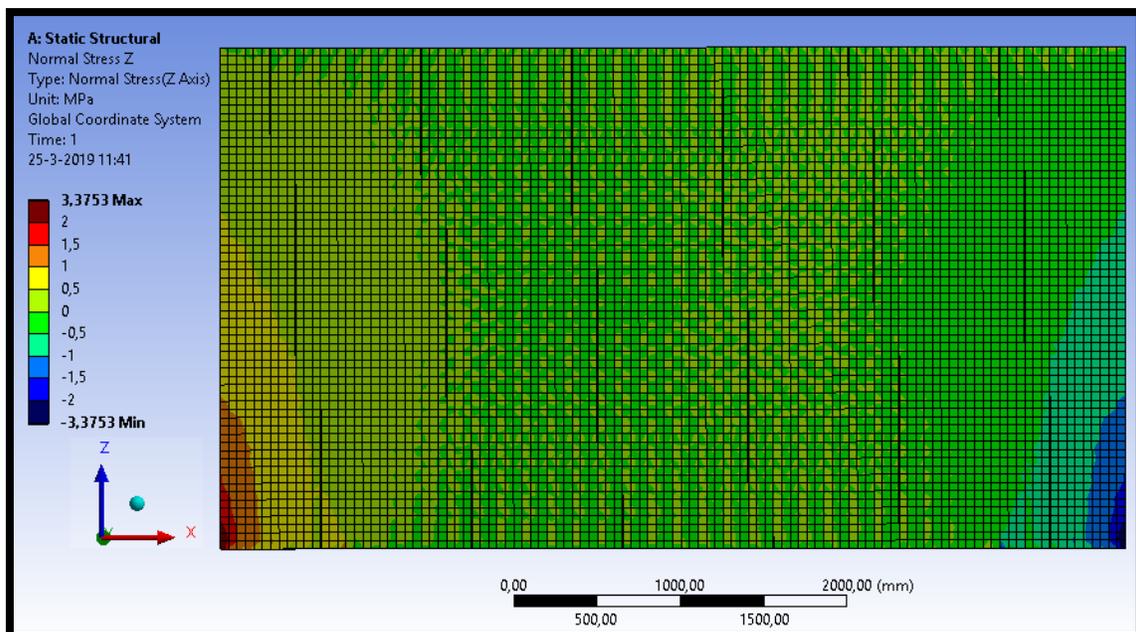


Figure 38 - Axial stress in z-direction in the vertical (edge) layers, no hole

5. CLT-house of the Future: A structural overview of the variants



Figure 39 and Figure 40 show the shear stresses in the x-direction on the z-face of the element in respectively the horizontal- and the vertical board layers. These figures confirm a correct modelling of the CLT, since stress peaks visible at the locations where the connection in the layer shifts from one board to the other. These peaks are caused by a local shift in cross-section from the gross- to the net cross-section in the considered direction. The shear stress peaks in one layer increase the shear stress in adjacent layers. This phenomenon is very apparent at the fixed supports, where the vertical reaction forces cause a shear stress peak in the horizontal layers between the supported part of the slab and the unsupported part. This stress is distributed to the timber area of the vertical boards, causing for a (smaller) peak in these boards as well. The shear stress values in Figure 39 and Figure 40 are smaller than the shear strength of C24 timber of 2,24 N/mm².

The frictional shear strength was previously determined at 1,81 N/mm². This is the strength of the timber at its contact (lamination layer) with a cross layer. ANSYS provides a contact tool to analyse the frictional stresses in the contact areas in the slab (Figure 41). The stress values are smaller than the frictional shear strength. Additionally, the shear stresses in the x-direction on the y-face of the element - and the shear stresses in the z-direction on the y-face of the element provide the directional components of this frictional stress (Figure 42 and Figure 43), which can be directly compared to this frictional shear strength.

An additional check can be done on the max and min values in Figure 42 and Figure 43. These stresses are directed in either x- or z- direction on the y-face (similar as vice versa; in y-direction on the x- or z-face), and therefore relate to pure local rolling shear. The values do not exceed the rolling shear strength. Explanation lies in the input of the material characteristics. Stresses in a certain direction within the CLT slab are always redistributed to the stronger parts (strong layers) of a cross-section. The horizontal layers for example are stronger (regular shear) than the vertical layers (rolling shear) when subjected to shear stresses in the x-direction on the y-face, and therefore transmit a larger share of the shear in that direction.

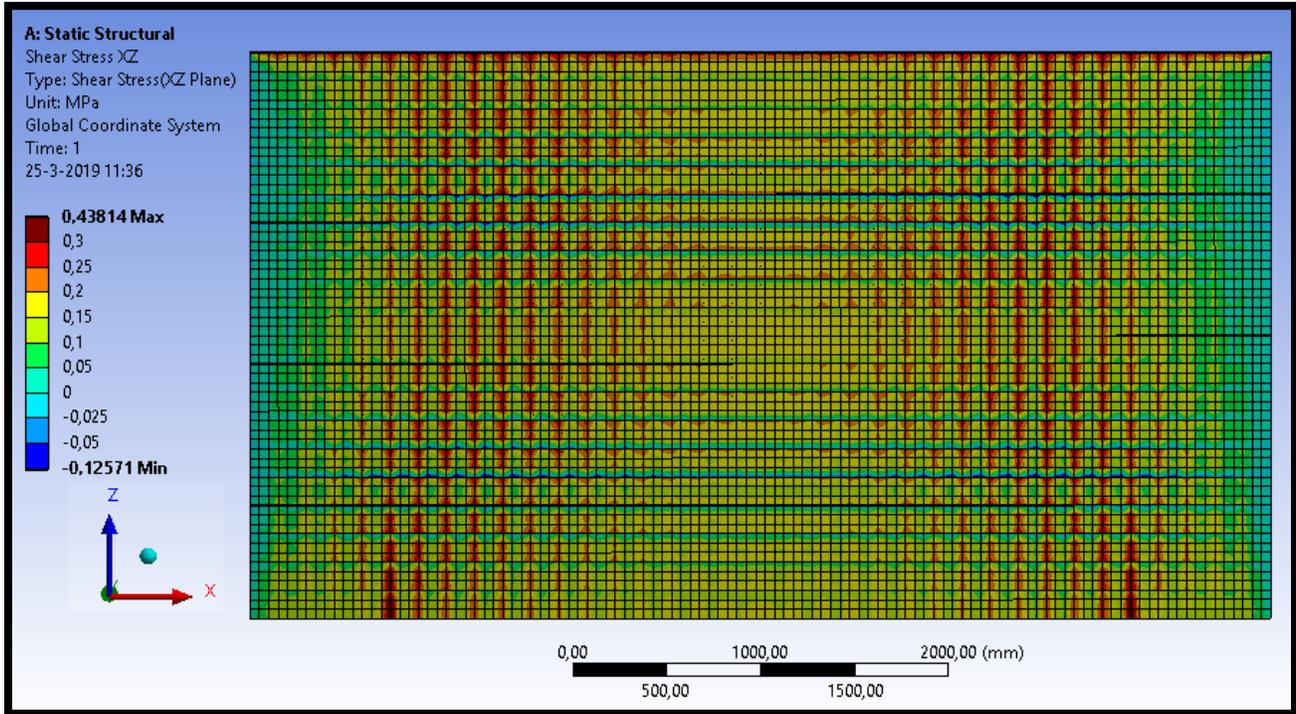


Figure 39 – Shear stress in x-direction on the z-face in the horizontal (middle) layer, no hole

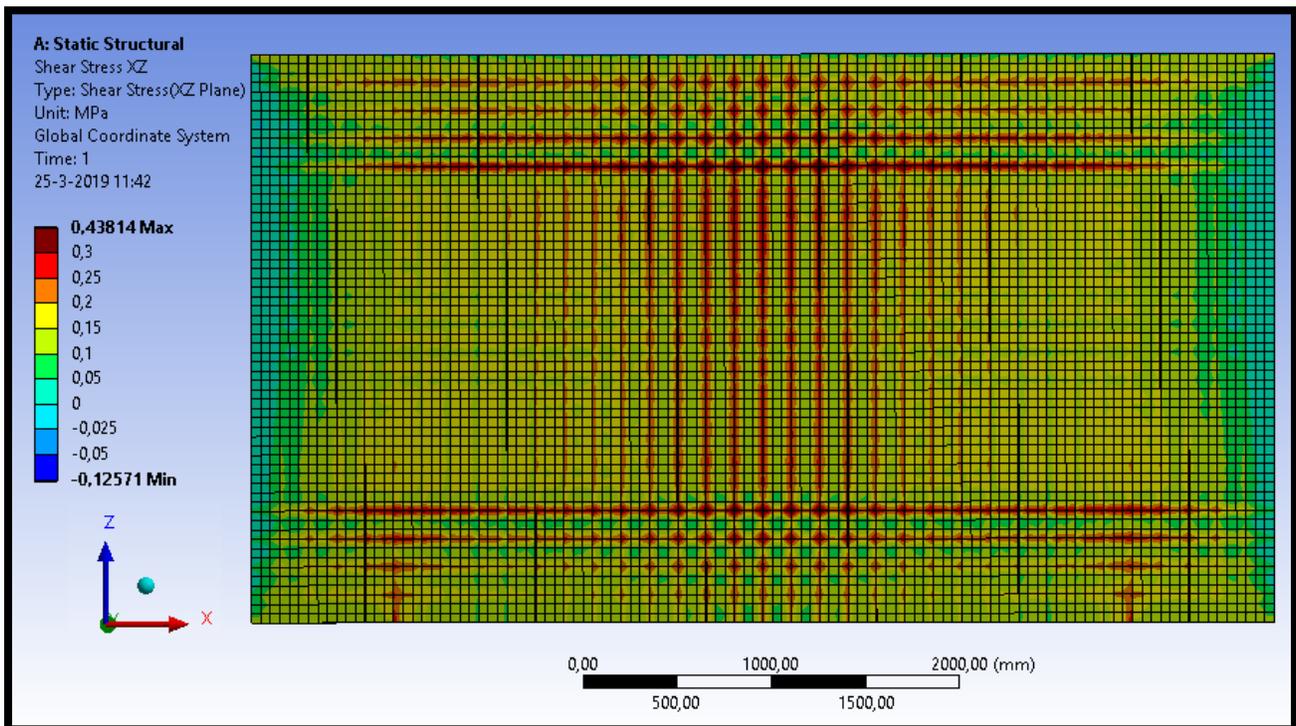


Figure 40 – Shear stress in x-direction on the z-face in the vertical (edge) layers, no hole

5. CLT-house of the Future: A structural overview of the variants

• • •

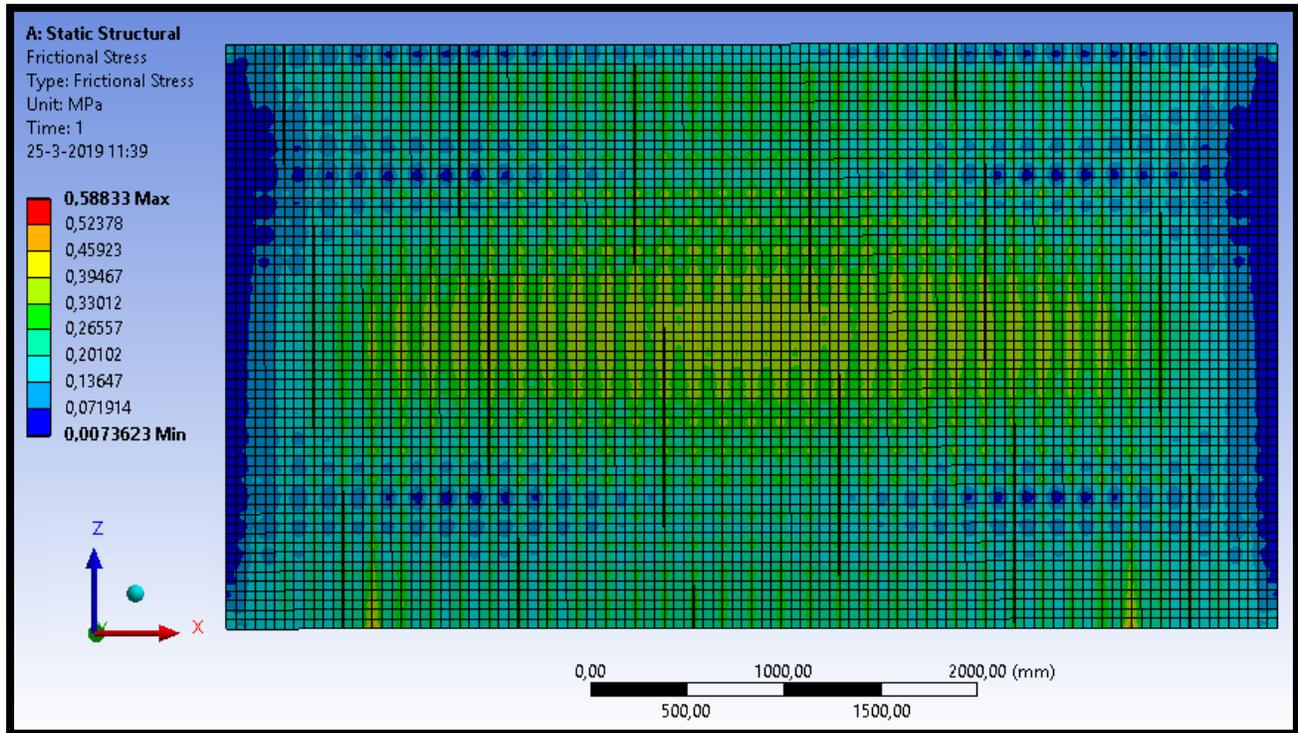


Figure 41 – Frictional stress in lamination layer, no hole

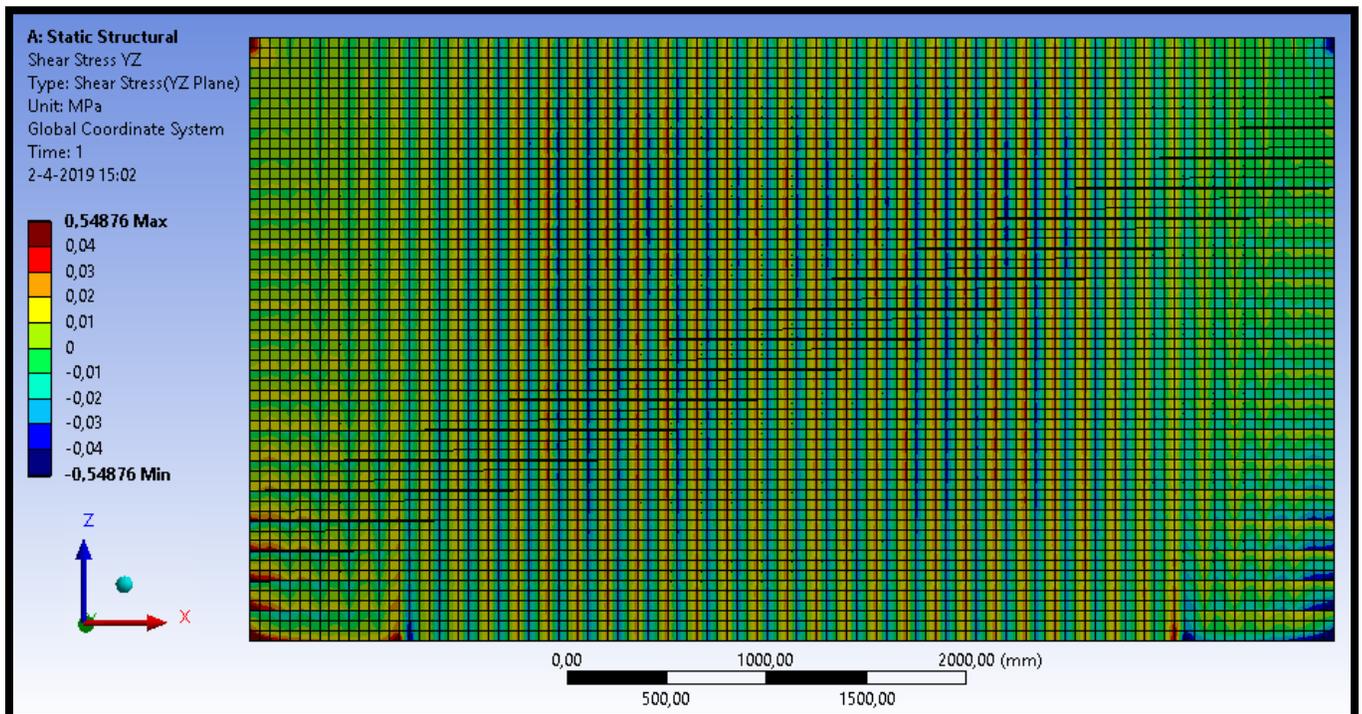


Figure 42 - Shear stress in y-direction on the z-face in the horizontal layer, no hole

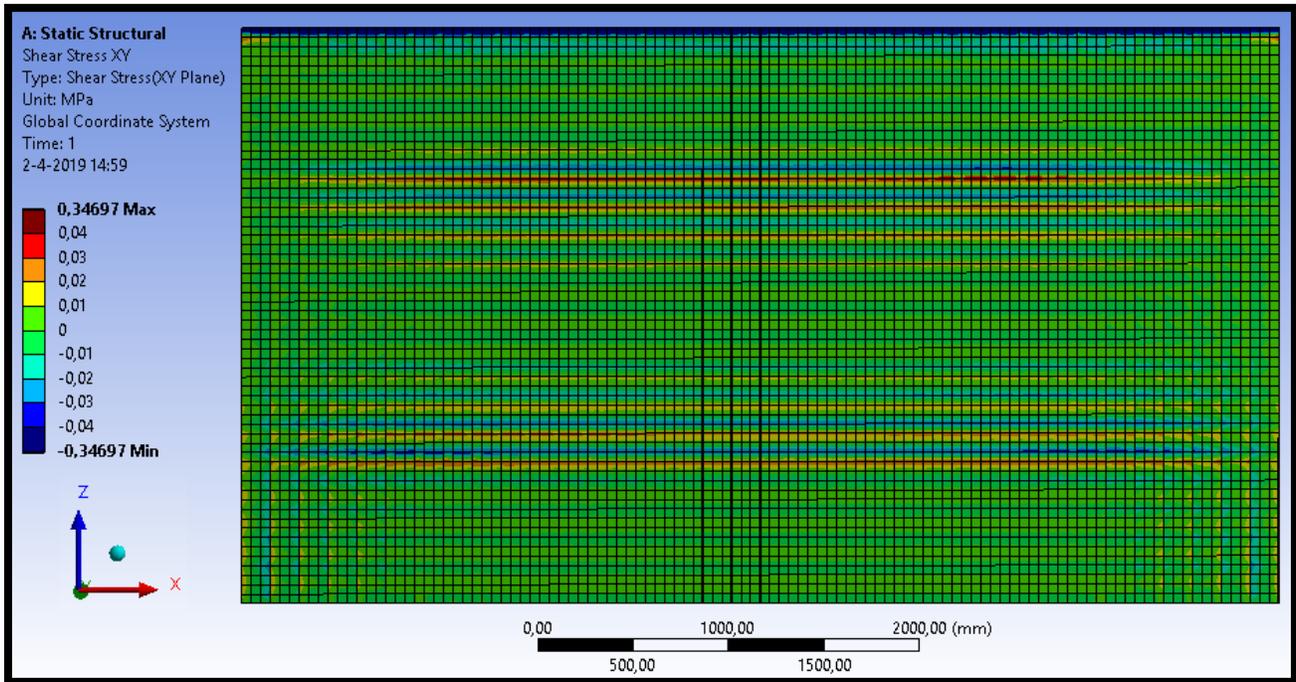


Figure 43 - Shear stress in y-direction on the x-face in the vertical layer, no hole

The deformation of the slab is shown in Figure 44, multiplied 150 times from the real deformation. The deformation per 3m high storey must be smaller than 10 mm (Ch.4.3.2), so the deformation in the finite element model satisfies the requirement.

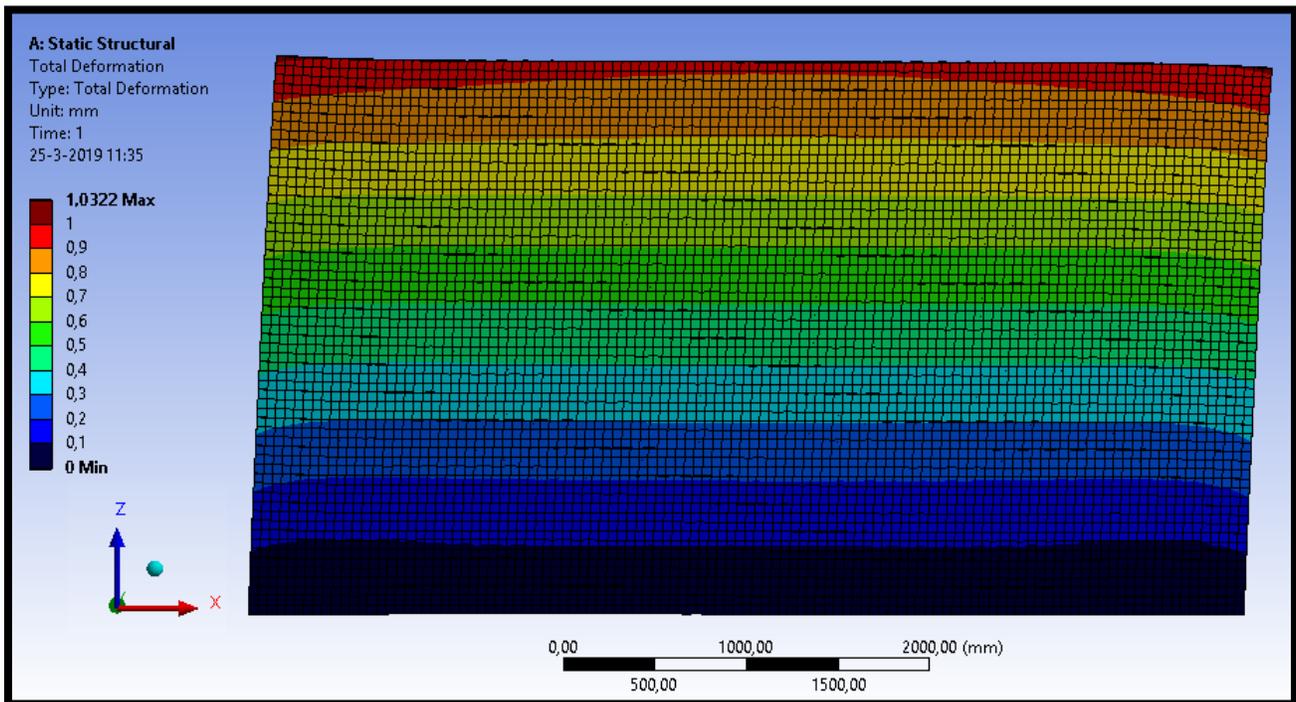


Figure 44 - Deformation, multiplied 150x, no hole

5. CLT-house of the Future: A structural overview of the variants



The second considered slab provides a direct comparison to the hand calculated composition. This means the horizontal- and vertical “rods” have a depth of 5 boards of 150 mm plus the spacing, which is used as input for the geometry in SpaceClaim (Figure 45). The modelling of CLT characteristics, boundary condition input and mesh generating remain similar to the hole-less slab.

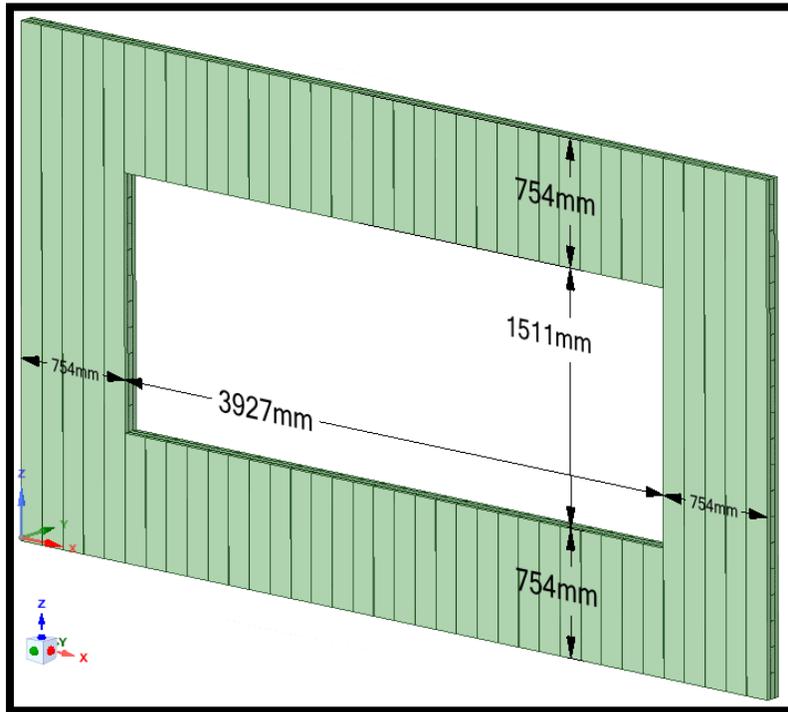


Figure 45 - Geometry slab with hole

Figure 46 and Figure 47 show the axial stresses in the x-direction in the horizontal layer and the z-direction in the vertical layers, respectively. Besides the stress peaks which have been interpreted in the hole-less model, new stress peaks occur at the corners of the hole due to the bending moment in the corners. The maximum (tensile) and minimum (compressive) stress values in the figures exceed the relevant strengths of the timber. These stress peak values are not noticeable in the overviews of the slab, but only in details (Figure 48), meaning the larger part of the stresses in the slab is lower than the axial strengths. Because the analysis is linear elastic, the local strength exceedance will cause a plastic deformations and lead to redistribution of the stress peaks over larger areas, decreasing the maximum and minimum stress values.

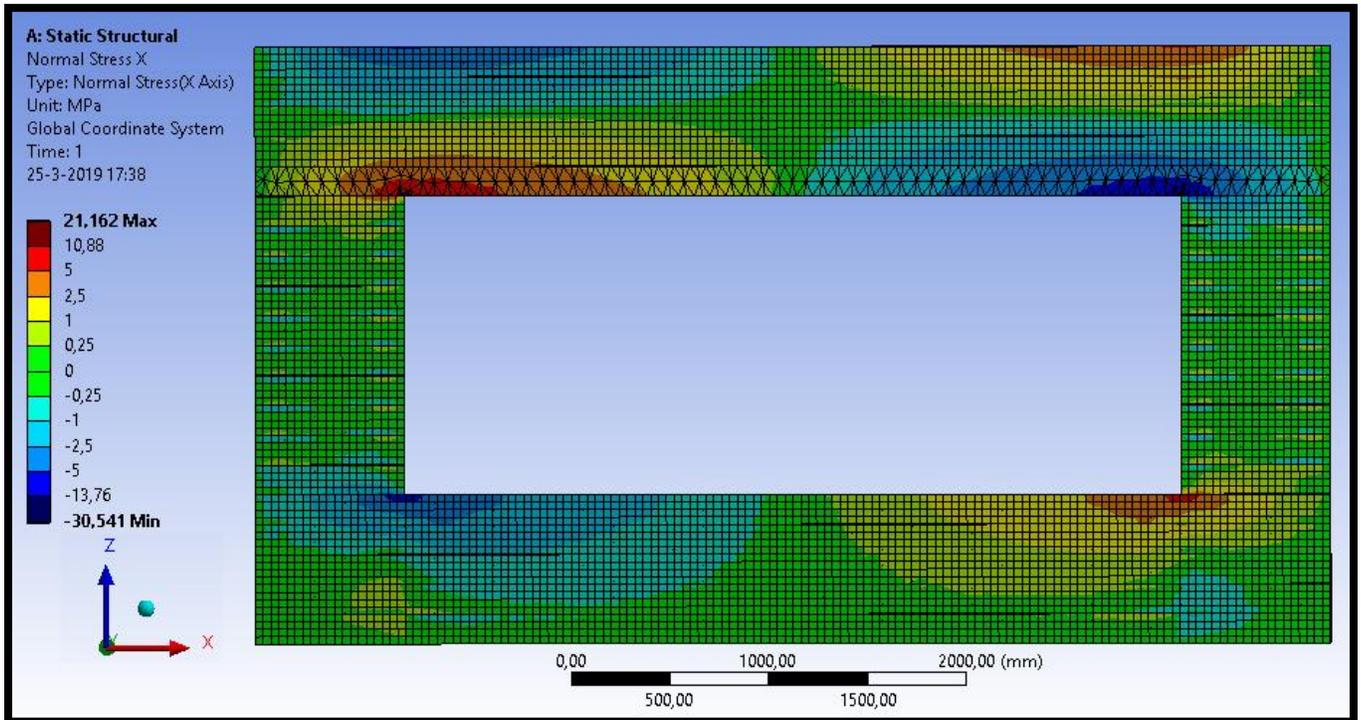


Figure 46 - Axial stress in x-direction in the horizontal layer, 3900x1500mm hole

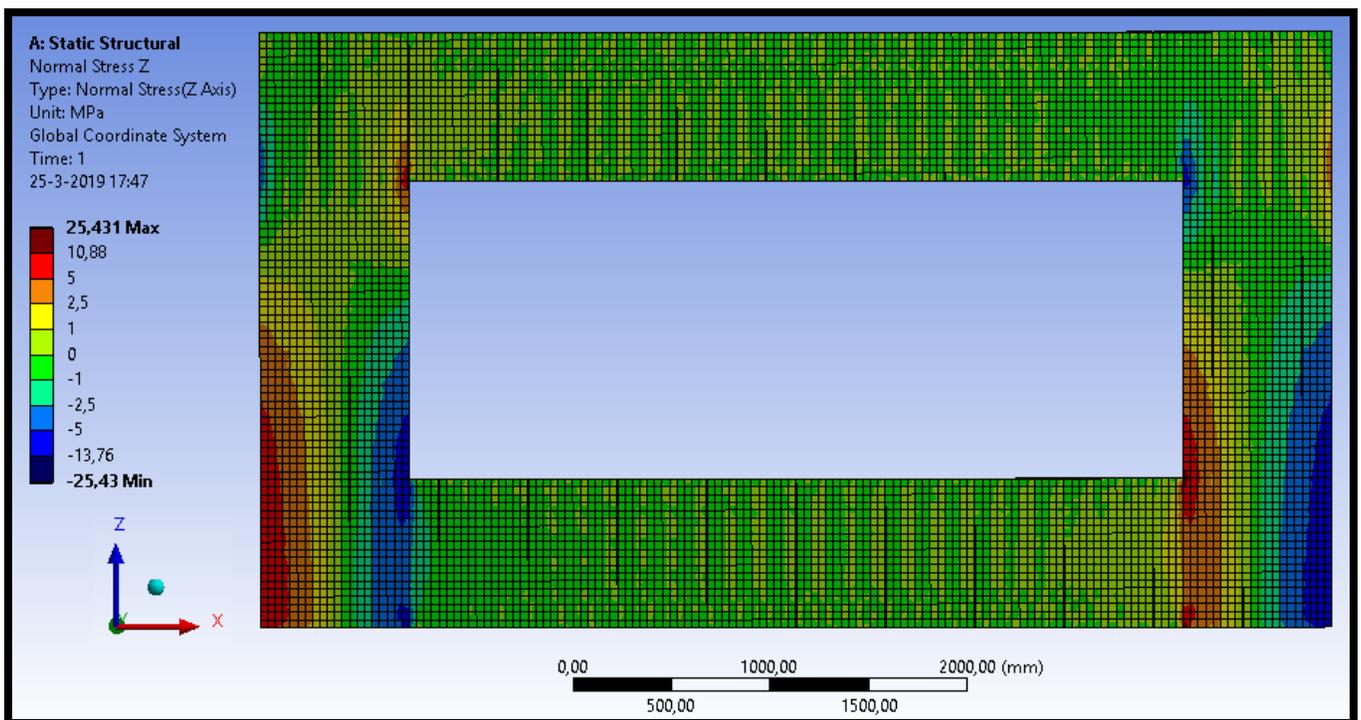


Figure 47 - Axial stress in z-direction in the vertical layers, 3900x1500mm hole

5. CLT-house of the Future: A structural overview of the variants

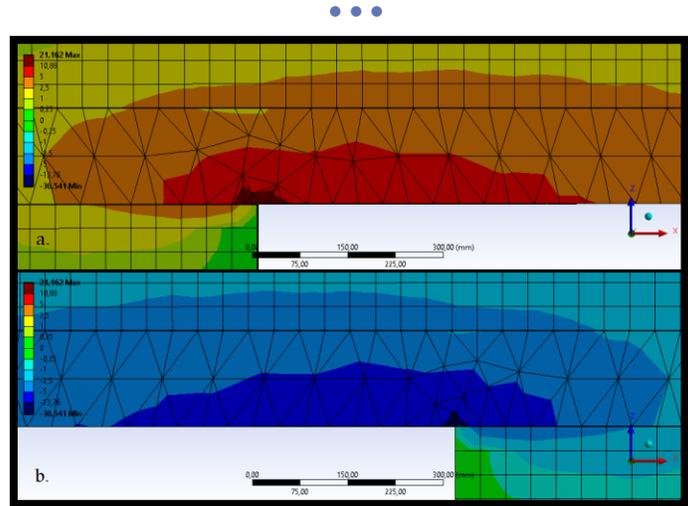


Figure 48 - Local stress peaks, a. tension, b. compression, 3900x1500mm hole

Figure 49 and Figure 50 show the regular shear stresses (x-direction on z-face, similar as in z-direction on x-face) in the timber boards in respectively the horizontal- and vertical layers. The stress peaks are most prominent in the vertical rods in both figures, explicable because these rods conduct the horizontal wind load down to the foundation. This causes shear stress in the vertical boards, which spreads to the horizontal layers. Again the maximum value of the shear stress exceeds the shear strength of C24 timber of 2,24 N/mm². Figure 51 shows the location of this stress peak, leading to local plastic deformation and redistribution of the stress. In the largest part of the CLT-slab, the shear stress is lower than the resistance.

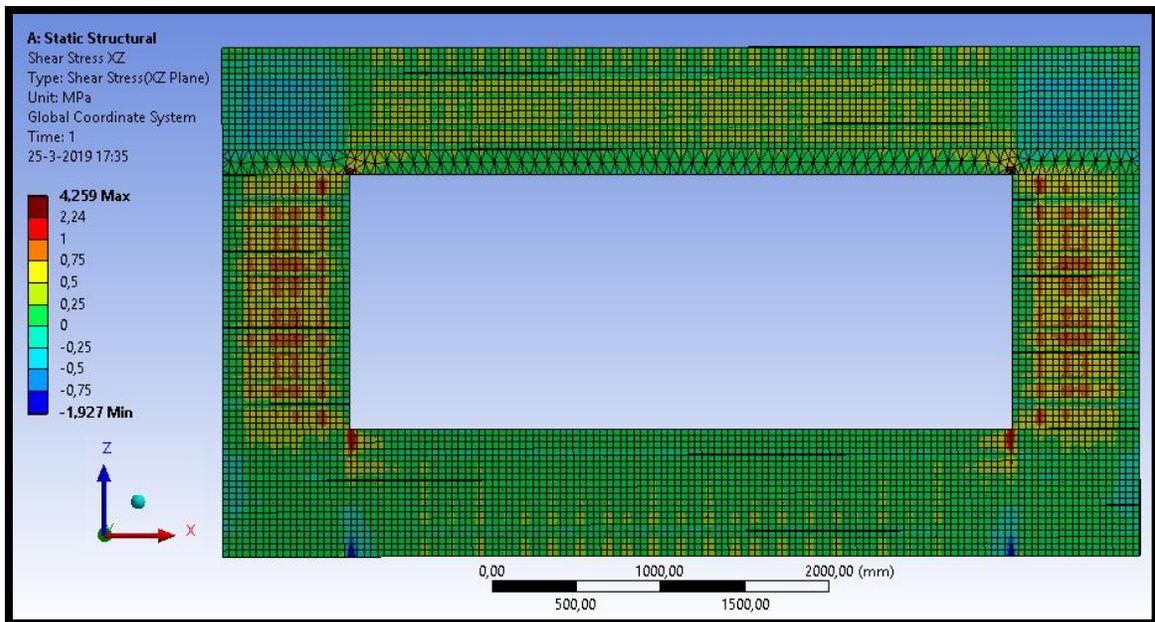


Figure 49 - Shear stress in x-direction on the z-face in the horizontal layer, 3900x1500mm hole

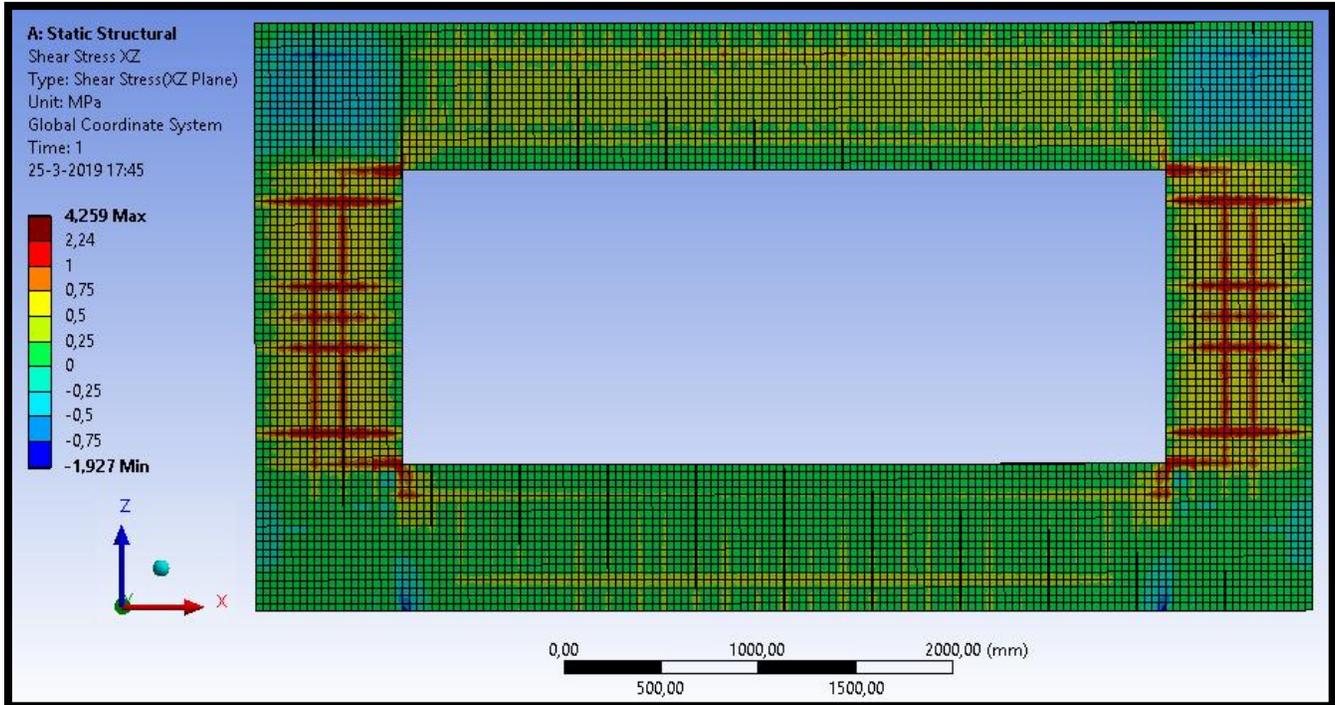


Figure 50 - Shear stress in x-direction on the z-face in the vertical layer, 3900x1500mm hole

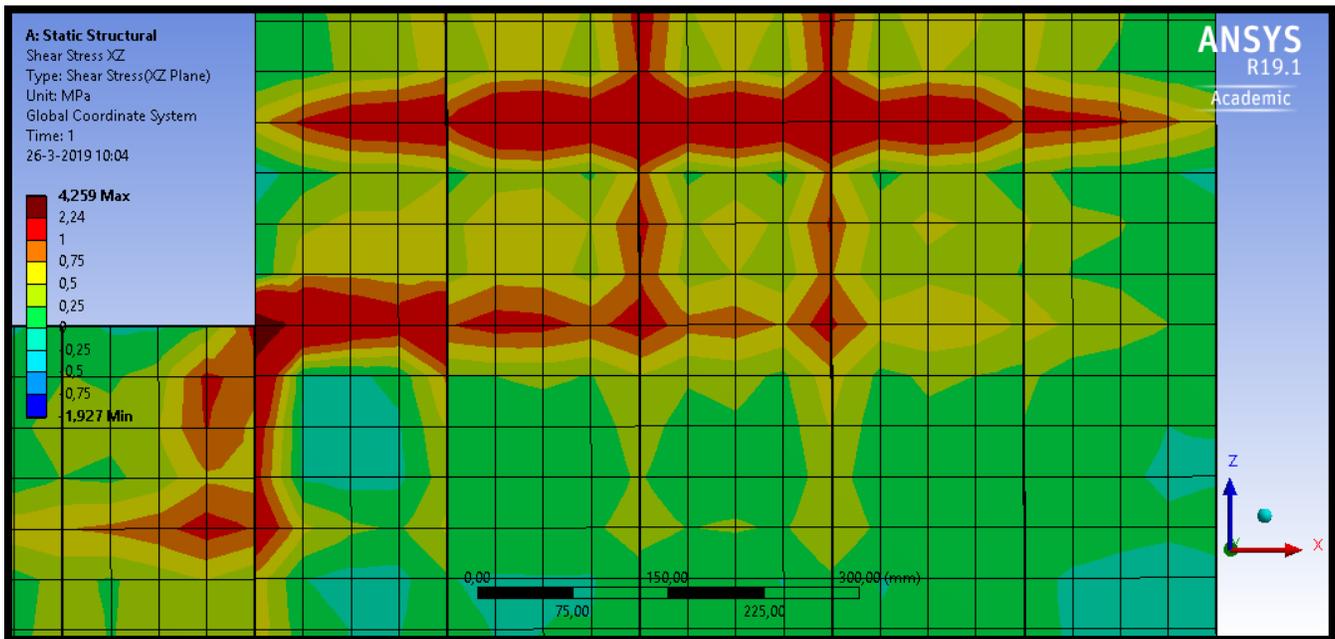


Figure 51 - Detail shear stress, 3900x1500mm hole

5. CLT-house of the Future: A structural overview of the variants



Figure 52 shows the frictional stress in the lamination layer. The maximum stress in the slab greatly exceeds the allowable frictional stress of $1,81 \text{ N/mm}^2$. However, again the location and spread of the stress peaks is of importance. Most of the peaks are visible in the vertical rod of the model. Assumed in the hand-calculation, these rods get their strength only from the vertical boards, and the connection between the layers is therefore irrelevant in terms of the functioning of the vertical rod. 4 stress peaks are visible at each corner of the hole (Figure 53). These stress peaks will cause local plastic deformation of the timber at the contact area, depending on the direction of the frictional stress. In case of vertical frictional stress, rolling shear will occur in the horizontal layer, whereas in case of horizontal frictional stress, rolling shear will occur in the vertical layer. Due to the local plastic deformation, the frictional stress is distributed over adjacent contact areas to decrease the maximum stress.

The rolling shear stresses in y-direction on the z-face for the horizontal layer (Figure 54) and in y-direction on the x-face for the vertical layer (Figure 55) again remain smaller than the rolling shear strength of $0,8 \text{ N/mm}^2$. The maximum values in the figures occur in the perpendicular layers.

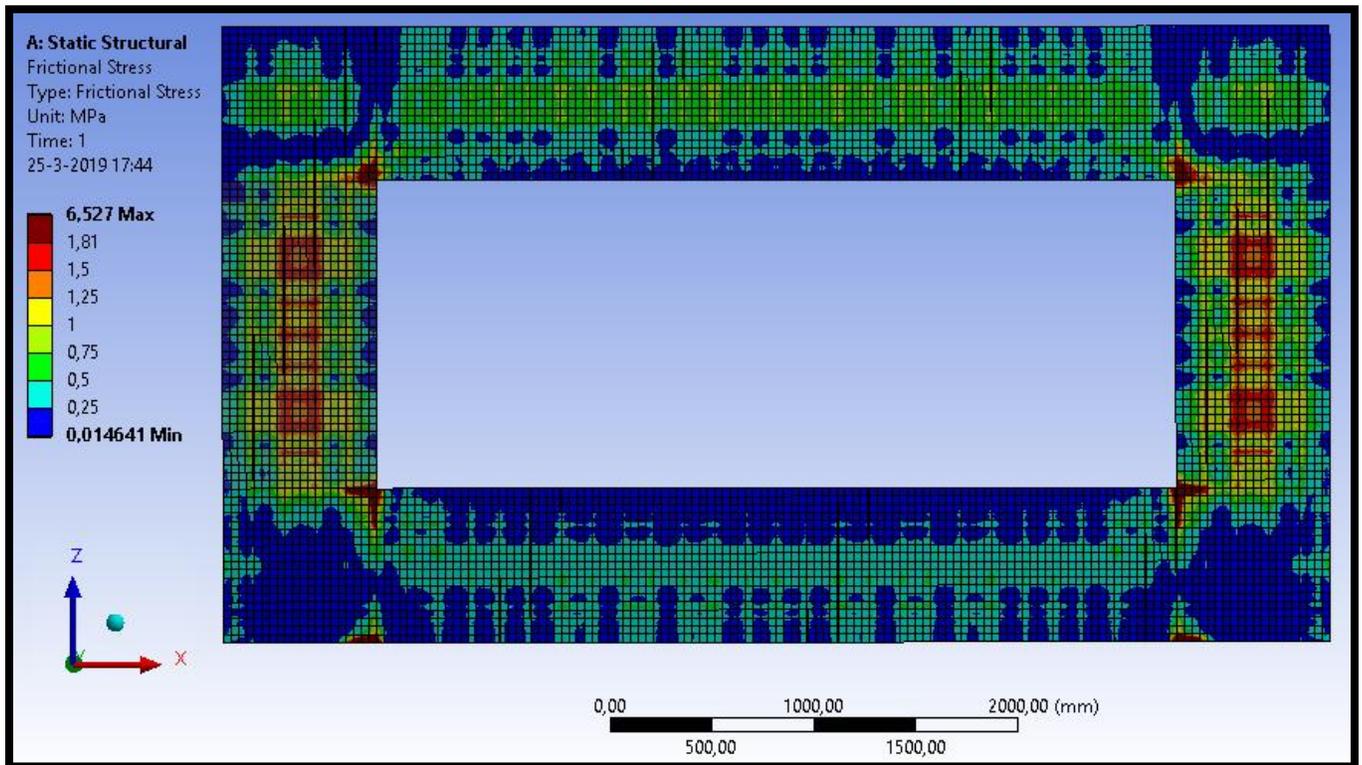


Figure 52 - Frictional stress in lamination layer, 3900x1500mm hole

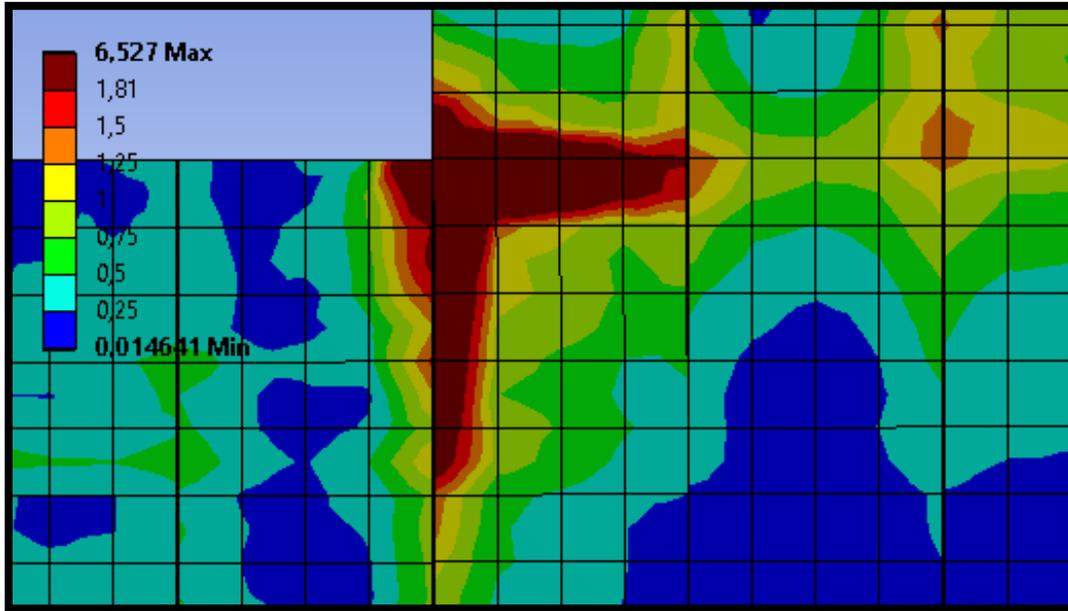


Figure 53 - Frictional stress detail, 3900x1500mm hole

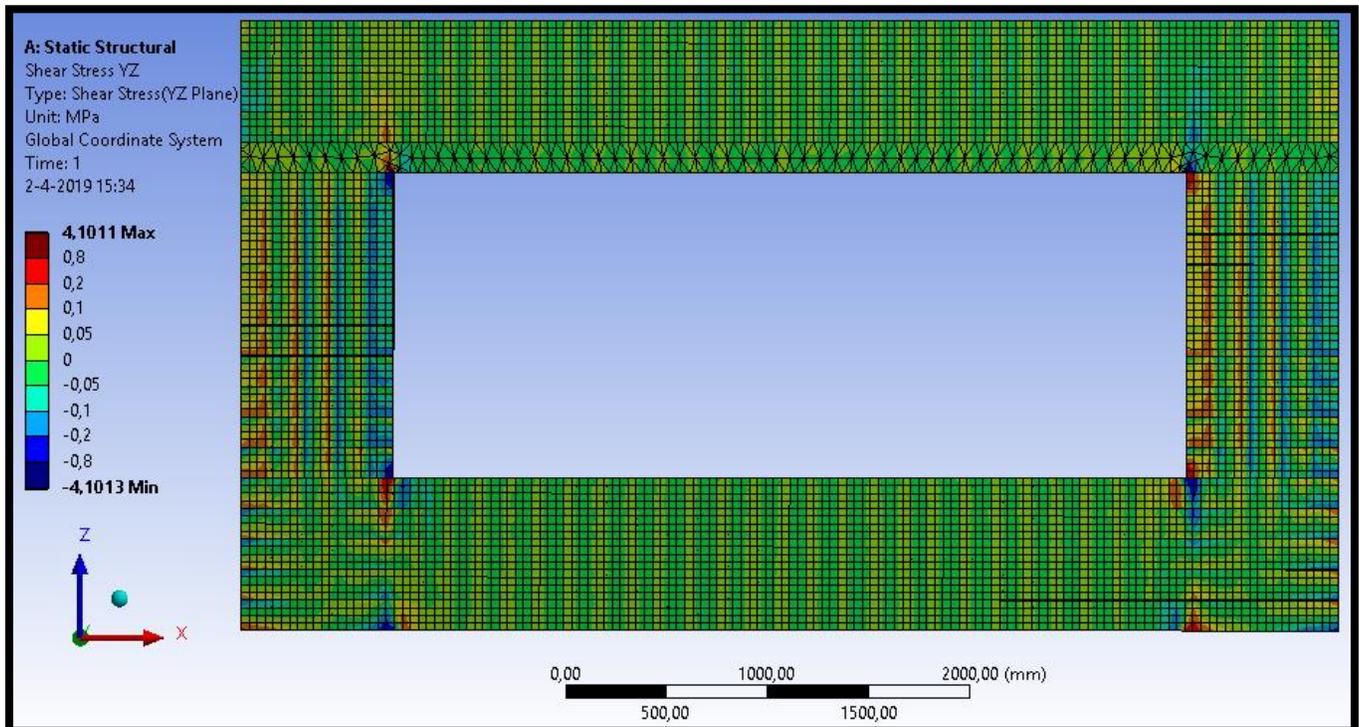


Figure 54 - Shear stress in y-direction on the z-face in the horizontal layer, 3900x1500mm hole

5. CLT-house of the Future: A structural overview of the variants

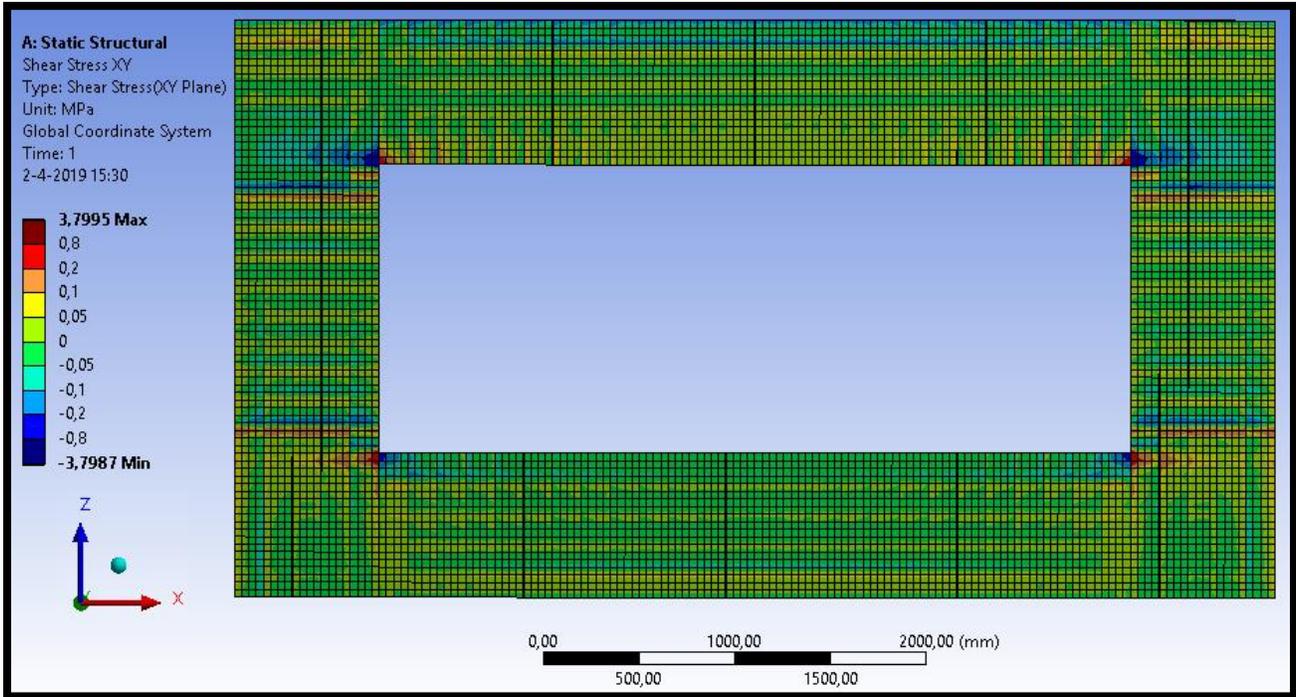


Figure 55 - Shear stress in y-direction on the x-face in the vertical layer, 3900x1500mm hole

The deformation, multiplied times 24, is shown in Figure 56. The maximum allowable deformation of 10mm is not exceeded for this composition.

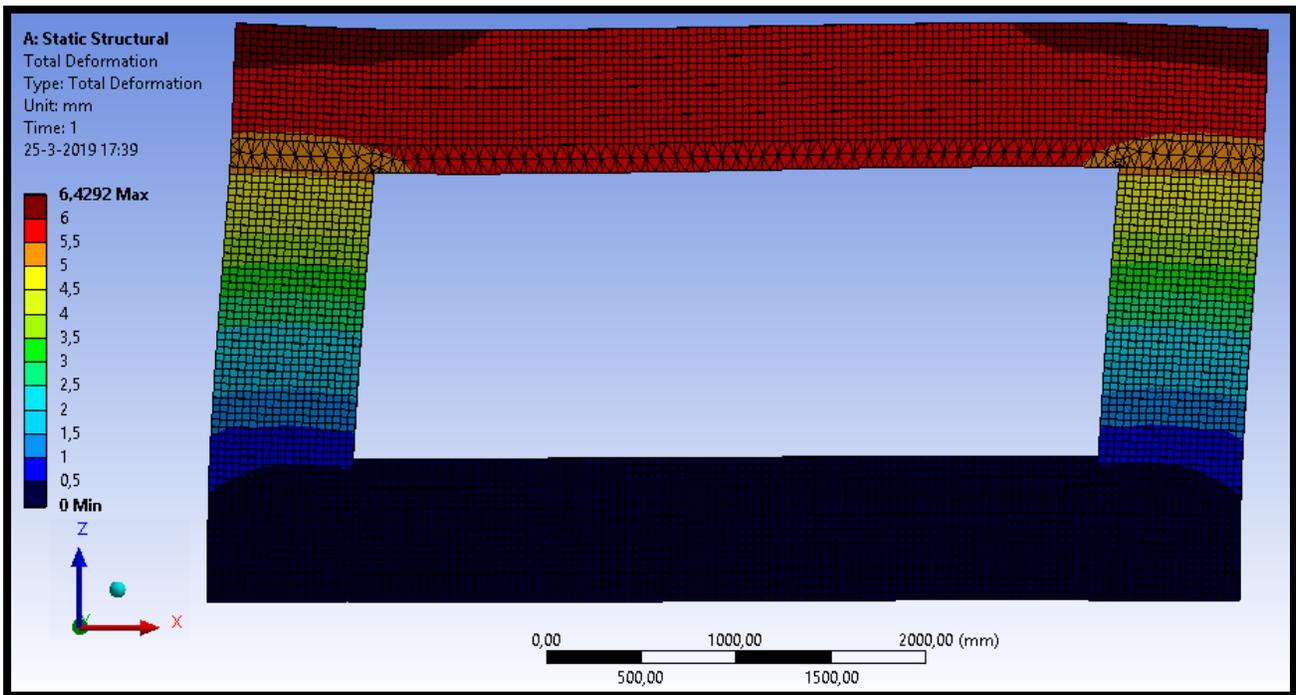


Figure 56 - Deformation, multiplied 24x, 3900x1500mm hole



The previous variant is a suitable application for the slabs above ground floor, but at ground floor an entrance into the building needs to be created. To investigate the possibilities of door holes, the third considered variant of the wall will contain a similar dimensioned hole as the previous composition, but moved to the bottom of the slab. This composition is not a realistic application but serves as a direct comparison with the previous variant to analyse the influence of the horizontal rod under the hole.

This comparison can be interpreted in two ways: as being equivalent to the comparison between an open and a closed tubular steel section subject to torsion, which leads to the prediction that the following results will be unfavourable compared to the second variant; or as being equivalent to the comparison of two cross-sections: a single rectangular cross-section of h times b mm, and two unconnected stacked rectangles of $\frac{1}{2}h$ times b , where the single rectangle has better structural performance than the two stacked rectangles even though the area of the total cross-section is equal meaning the following results should be better than the previous.

The frame is changed to the composition of Figure 57, shown in SpaceClaim in Figure 58. The moment in the corners of the frame is multiplied by 2, but because the horizontal rod now has a depth of 10 boards instead of 5, the number of contact areas and the theoretical resistance of the corner is multiplied by 2 as well.

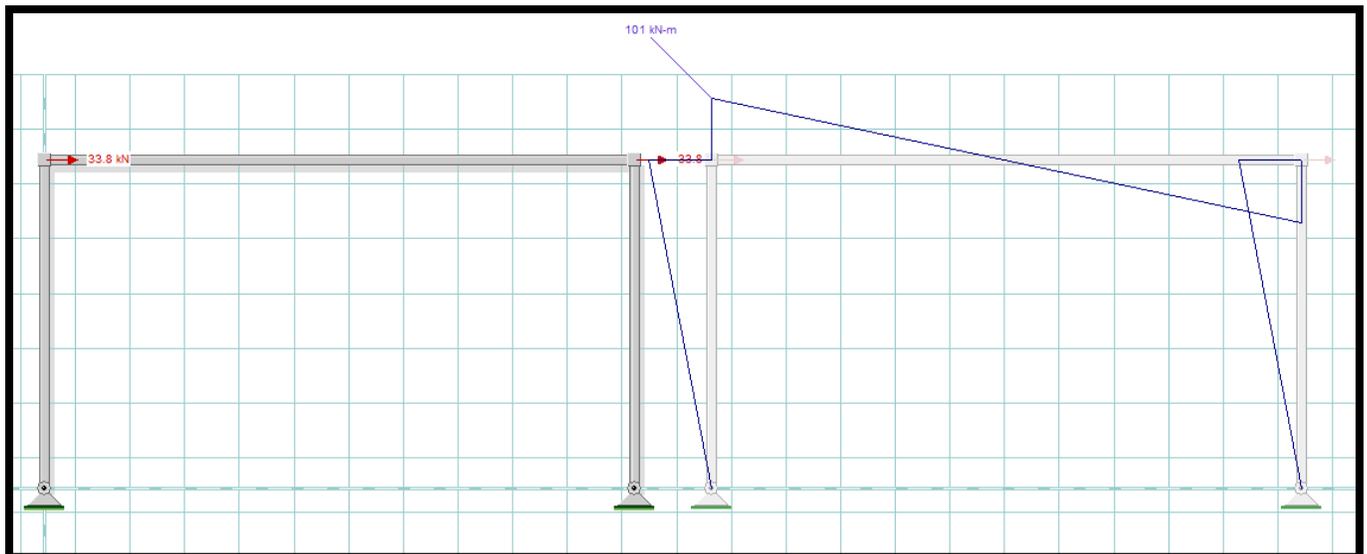


Figure 57 - Dr. Frame calculation for hole at bottom of the slab

5. CLT-house of the Future: A structural overview of the variants



Figure 59 till Figure 65 show the stress distributions and relevant details for this variant. When comparing the results with the results of the previous variant, the locations of stress peaks are corresponding; they occur at the corners of the hole, at the supports and the shear stress peaks show up in the vertical rods. However, the maximum- and minimum values are significantly lower, meaning the hypothesis was correct about the influence of the “height” of the horizontal rod being critical. This influence cancels out the effect of changing the composition from a hollow tubular section to an open tubular section.

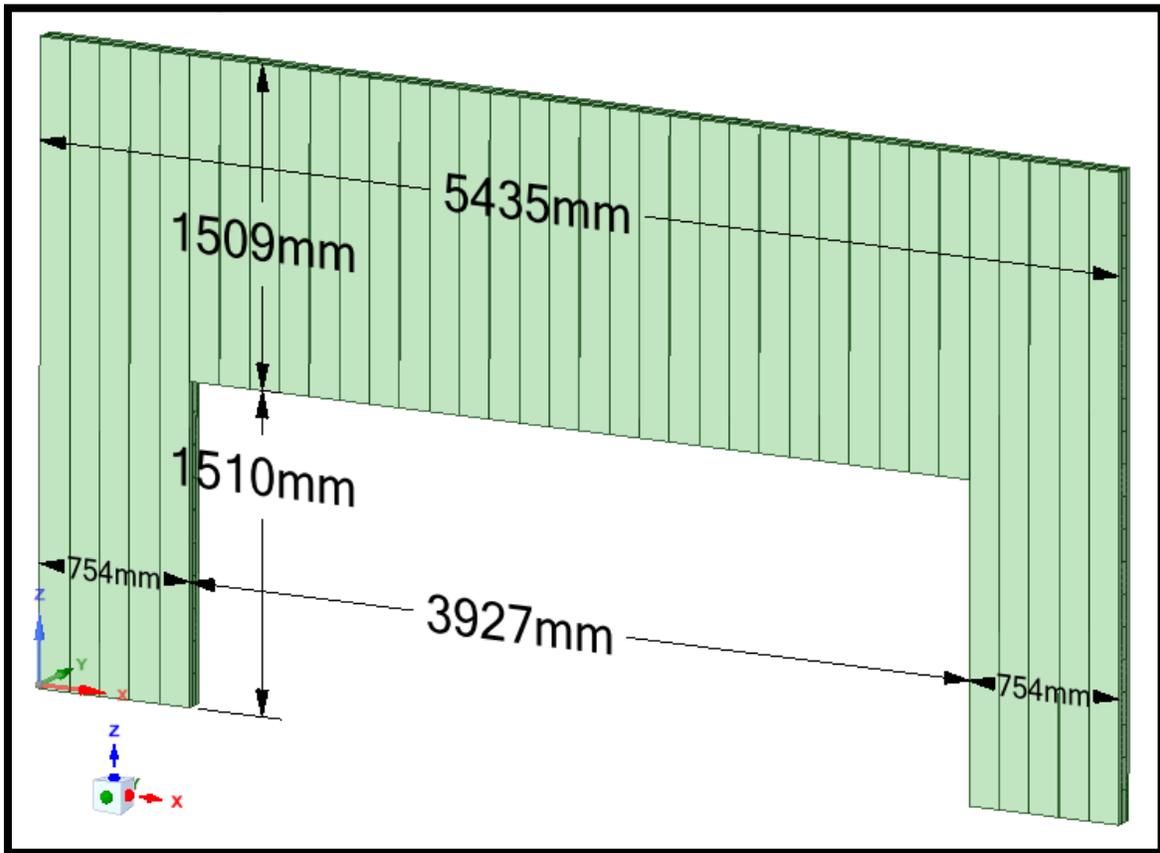


Figure 58 - SpaceClaim geometry, 3900x1500mm hole at bottom of slab

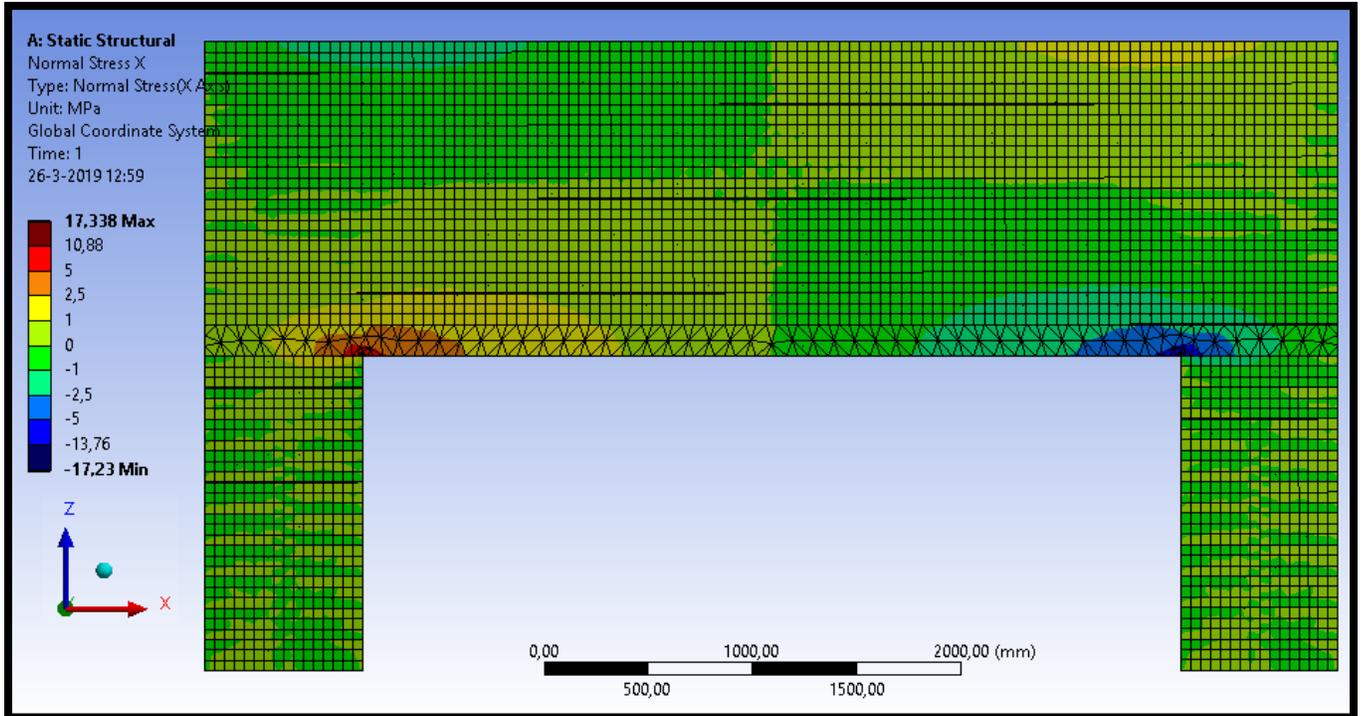


Figure 59 - Axial stress in x-direction in the horizontal layer, 3900x1500mm hole at bottom of slab

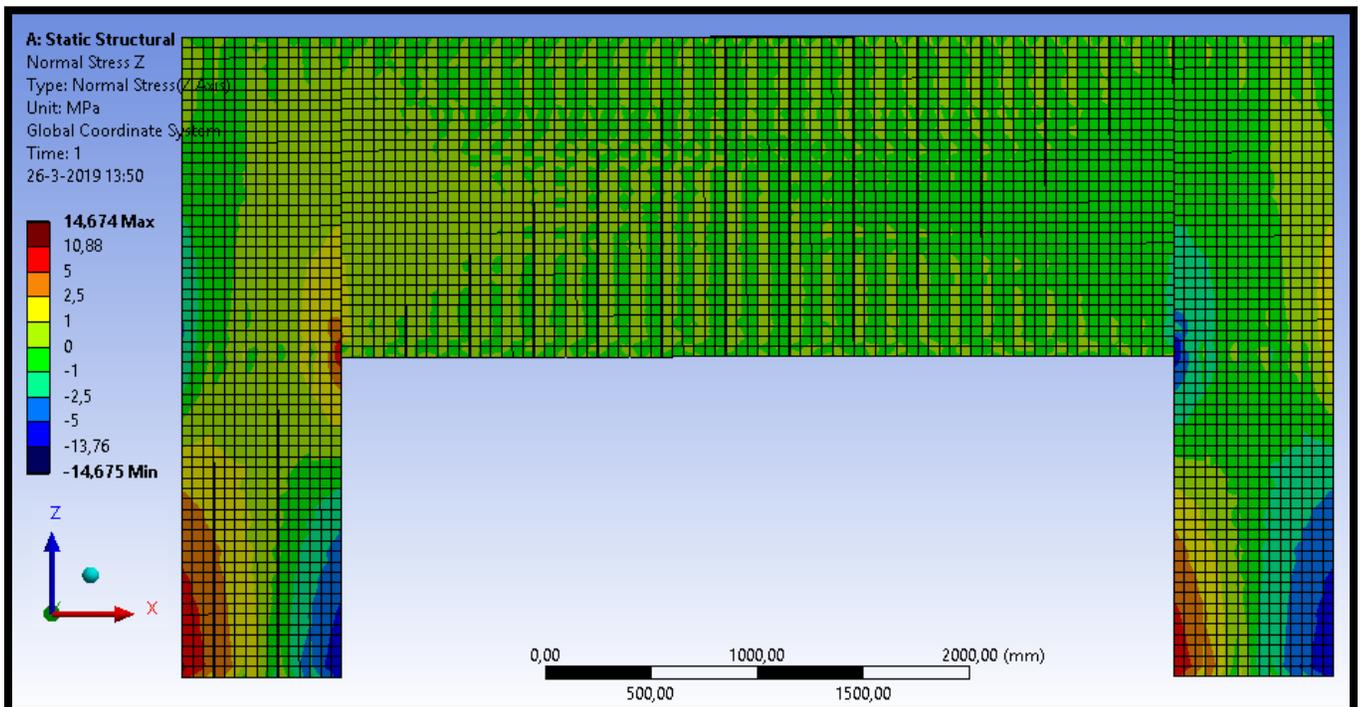


Figure 60 - Axial stress in z-direction in the vertical layers, 3900x1500mm hole at bottom of slab

5. CLT-house of the Future: A structural overview of the variants

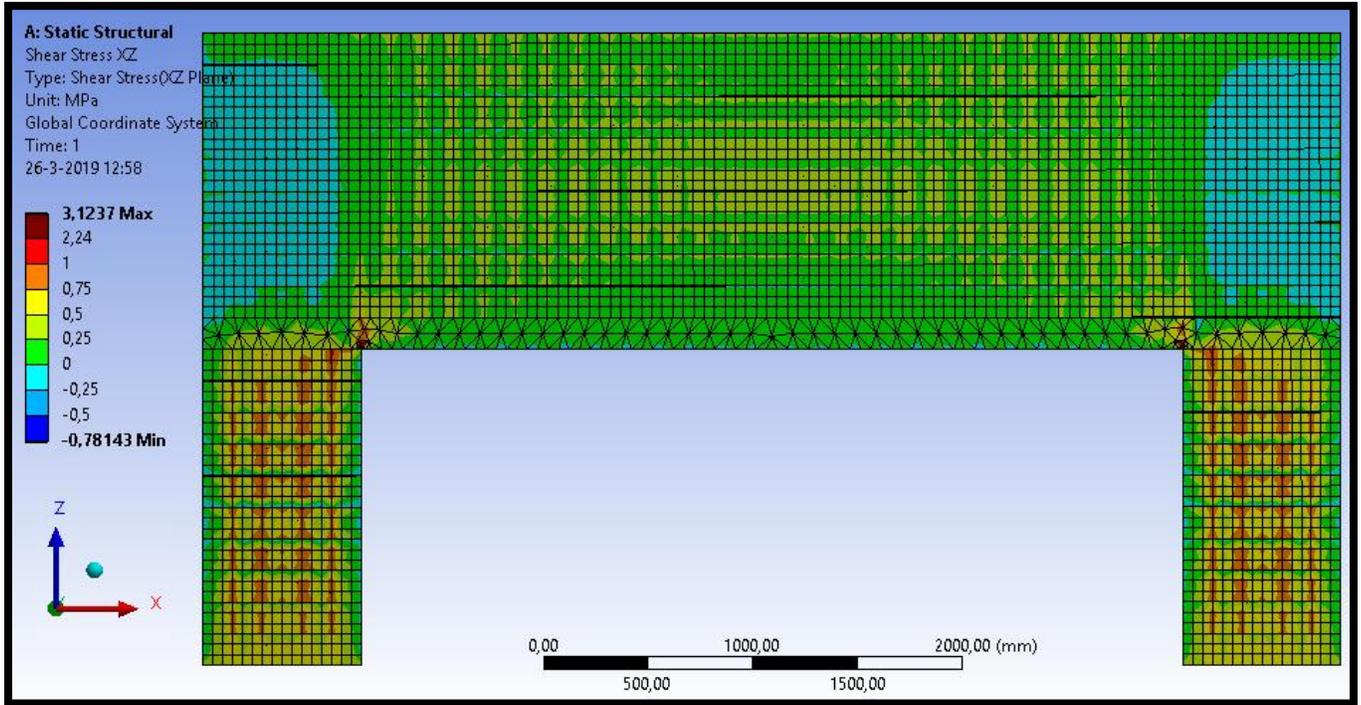


Figure 61 - Shear stress in x-direction on the z-face in the horizontal layer, 3900x1500mm hole at bottom of slab

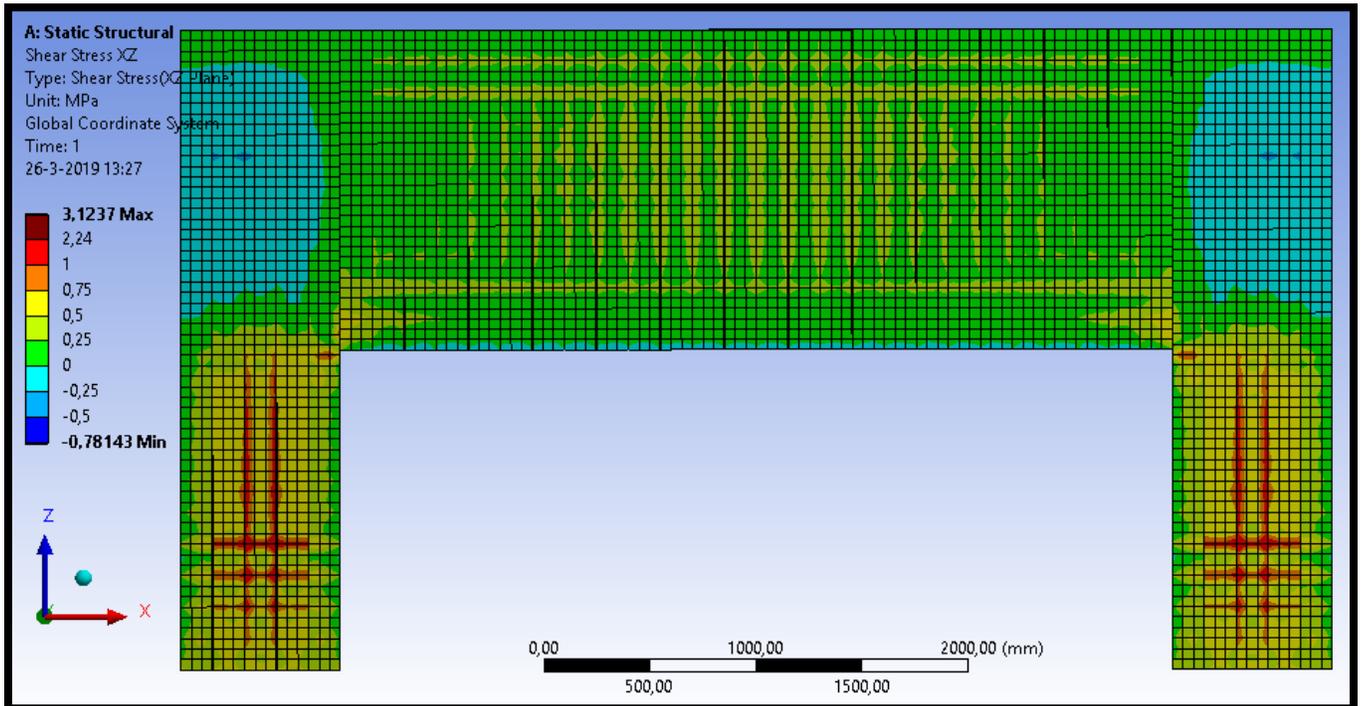


Figure 62 - Shear stress in x-direction on the z-face in the vertical layer, 3900x1500mm hole at bottom of slab

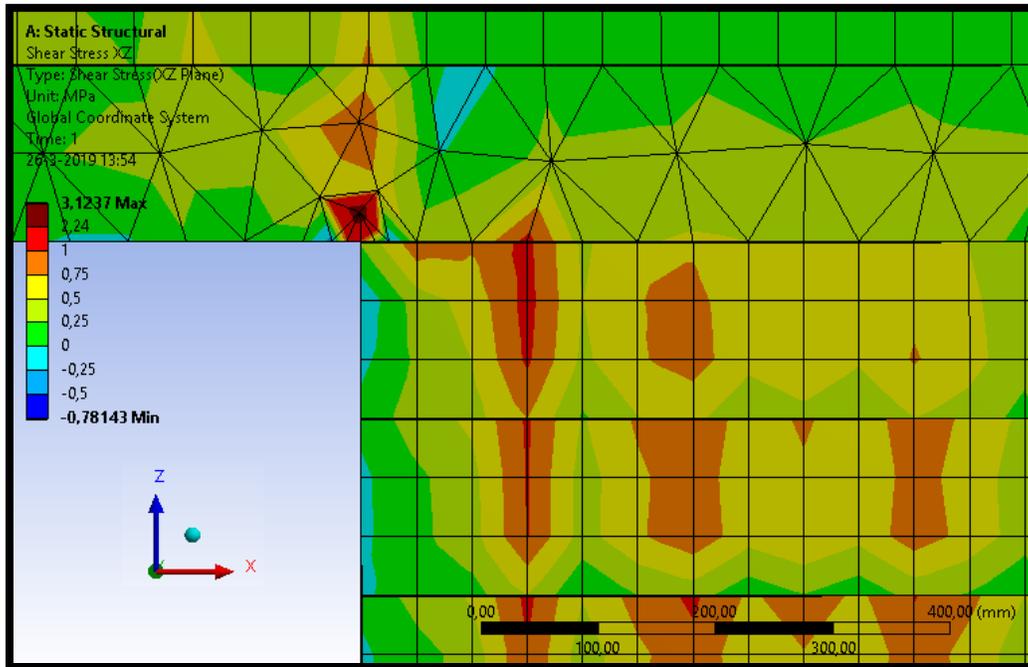


Figure 63 - Detail shear stress, 3900x1500mm hole at bottom of slab

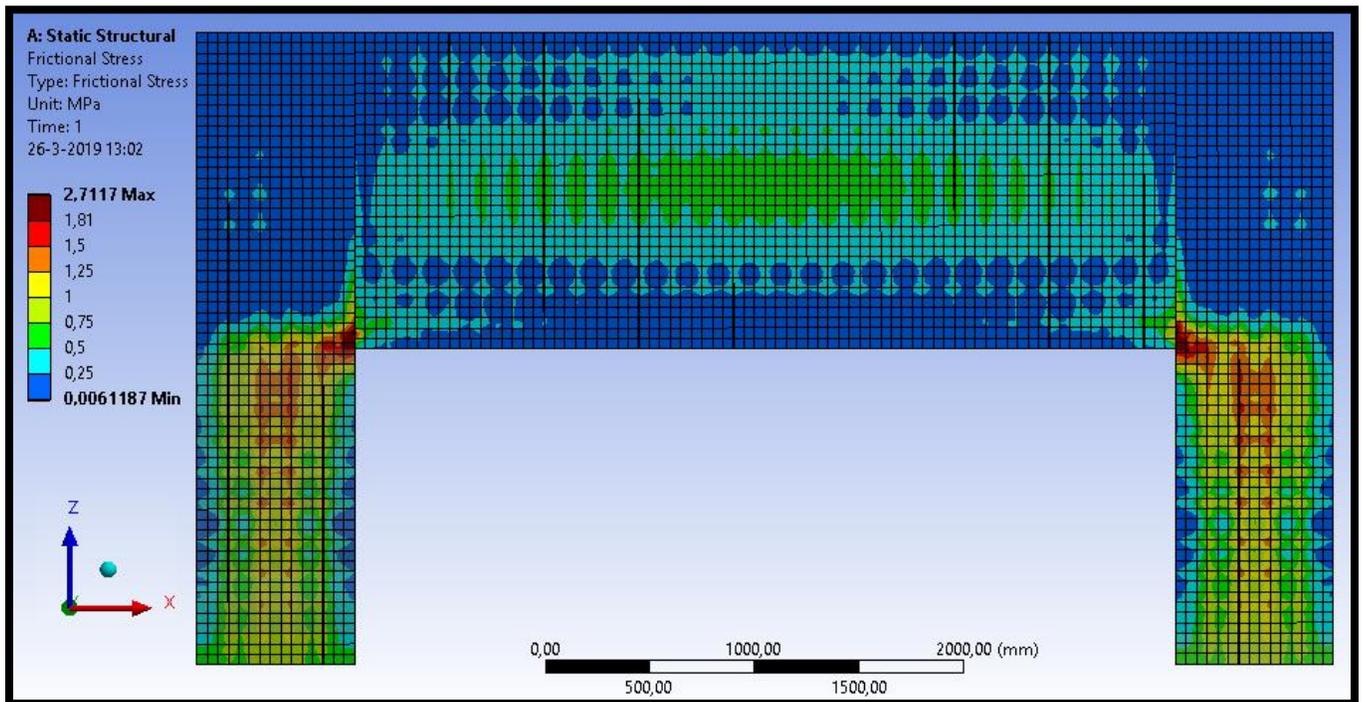


Figure 64 - Frictional stress in lamination layer, 3900x1500mm hole at bottom of slab

5. CLT-house of the Future: A structural overview of the variants

...

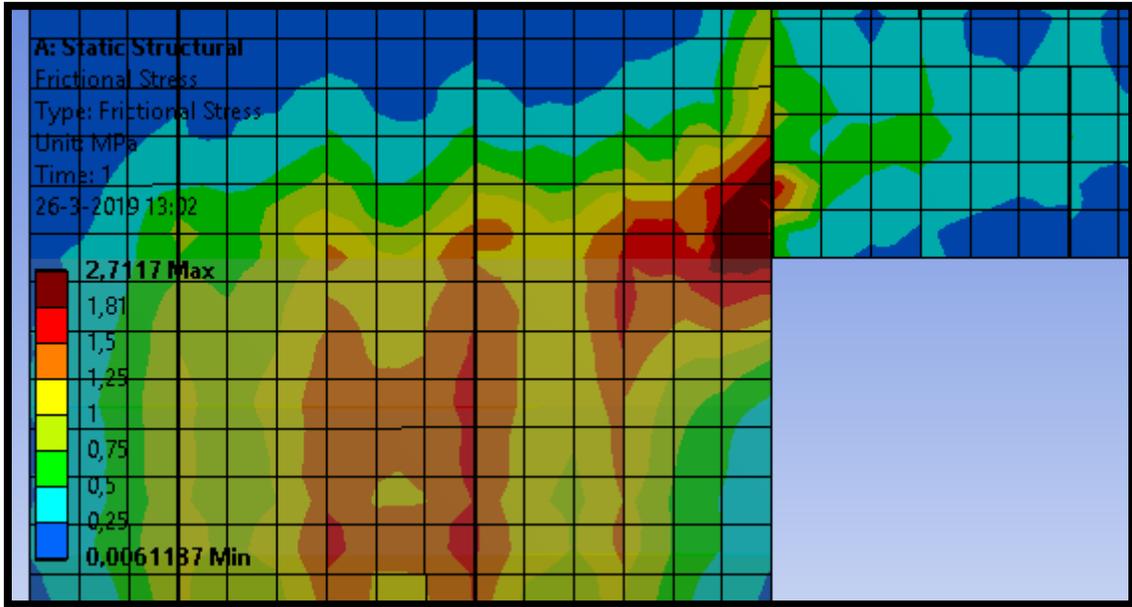


Figure 65 - Frictional stress detail, 3900x1500mm hole at bottom of slab

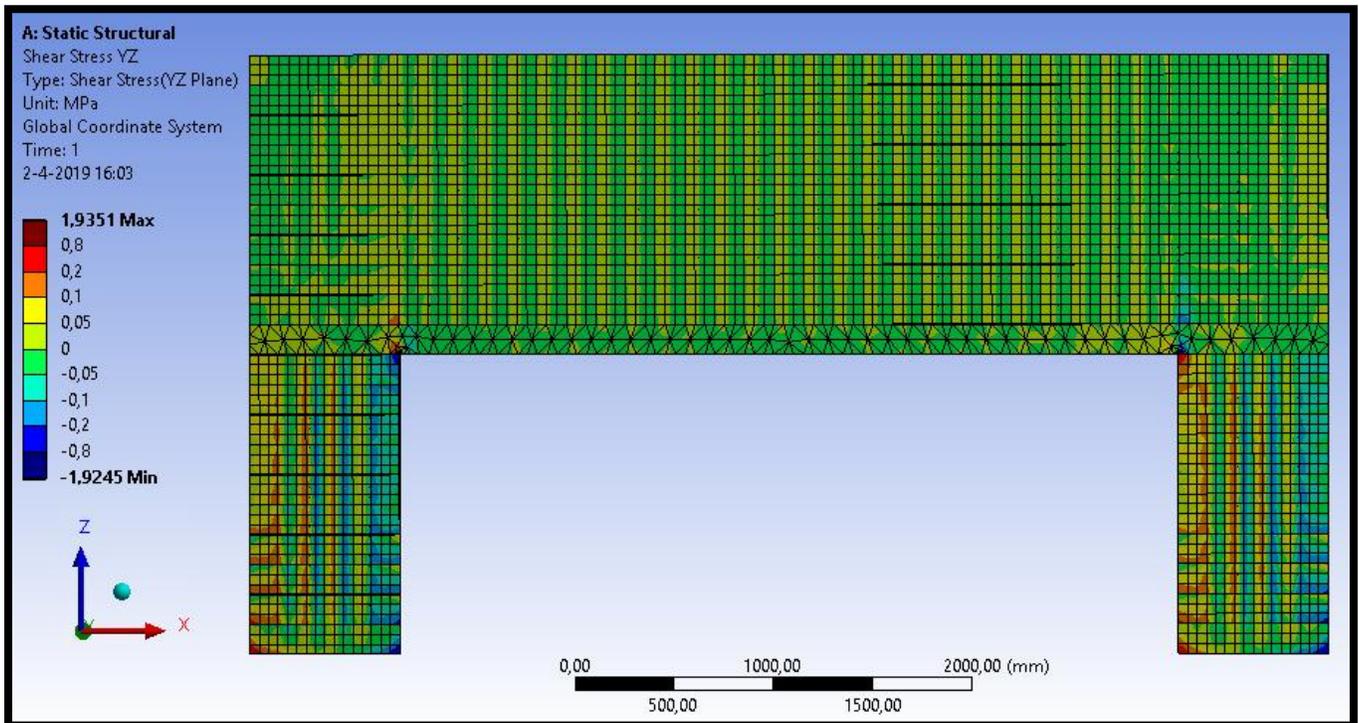


Figure 66 - Shear stress in y-direction on the z-face in the horizontal layer, 3900x1500mm hole at bottom of slab

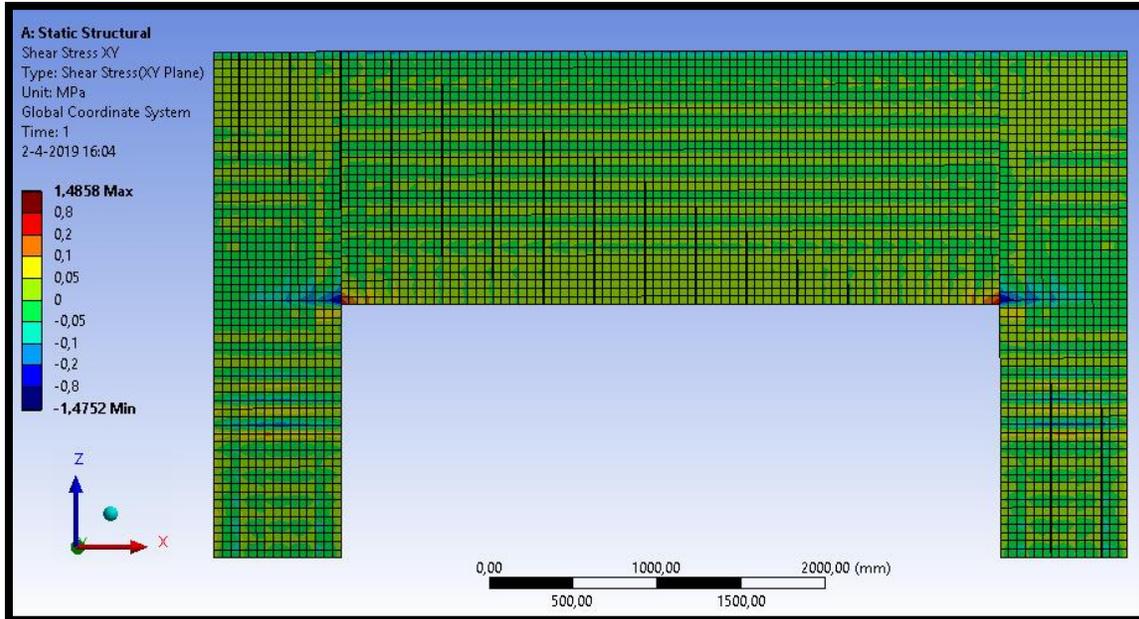


Figure 67 - Shear stress in y-direction on the x-face in the vertical layer, 3900x1500mm hole at bottom of slab

Also, in terms of the deflection, this variant performs better than the variant with the hole in the centre of the slab. This is mostly due to the stiffness of the top half of the slab. The maximum deflection is smaller than the required maximum deflection of 10 mm per storey.

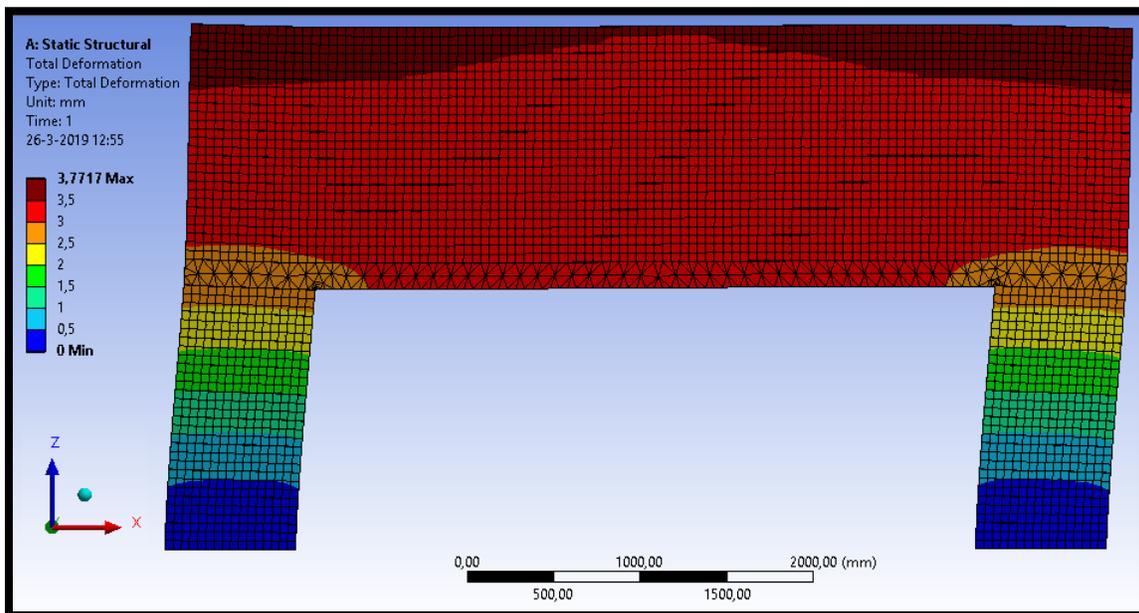


Figure 68 - Deformation, multiplied 41x, 3900x1500mm hole at bottom of slab

5. CLT-house of the Future: A structural overview of the variants



Besides the significance of the previous variant for research purposes, its application is non-existent. Article 4.21 and 4.22 of the Dutch Building Decree state that a door requires the minimum dimensions of 2300mm for the height and 1200mm for the width. A variant is investigated with a single door in the centre of the wall, followed by asymmetric compositions with additional window holes.

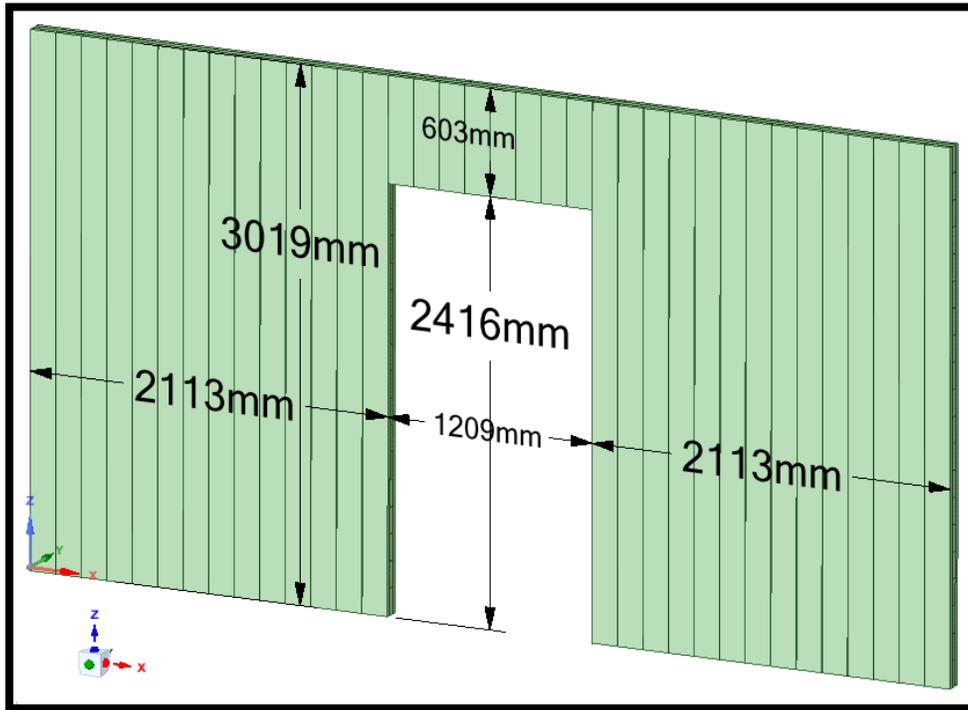


Figure 69 – Geometry, centered door hole

Compared to the previous variant, the height of the horizontal rod has decreased significantly, but this has been replaced by two vertical rods with increased height. The question is whether this structure can still be approximated with a hand calculation for a frame, or if the vertical rods provide enough in-plane resistance to neglect the horizontal connection above the door.

Figure 70 shows the deformations, multiplied by 44, for this composition using the old boundary conditions. One issue arises, due to the downward displacement of the right side of the left “vertical rod”. In reality, this displacement is prevented due to the foundation underneath the wall. ANSYS Mechanical provides an additional tool to implement a “compression only support”. This boundary condition is applied over the full bottom side of the slab. Figure 71 shows the deformation for this new situation, from which can be concluded that the additional boundary condition decreases the maximum deformation of the slab.

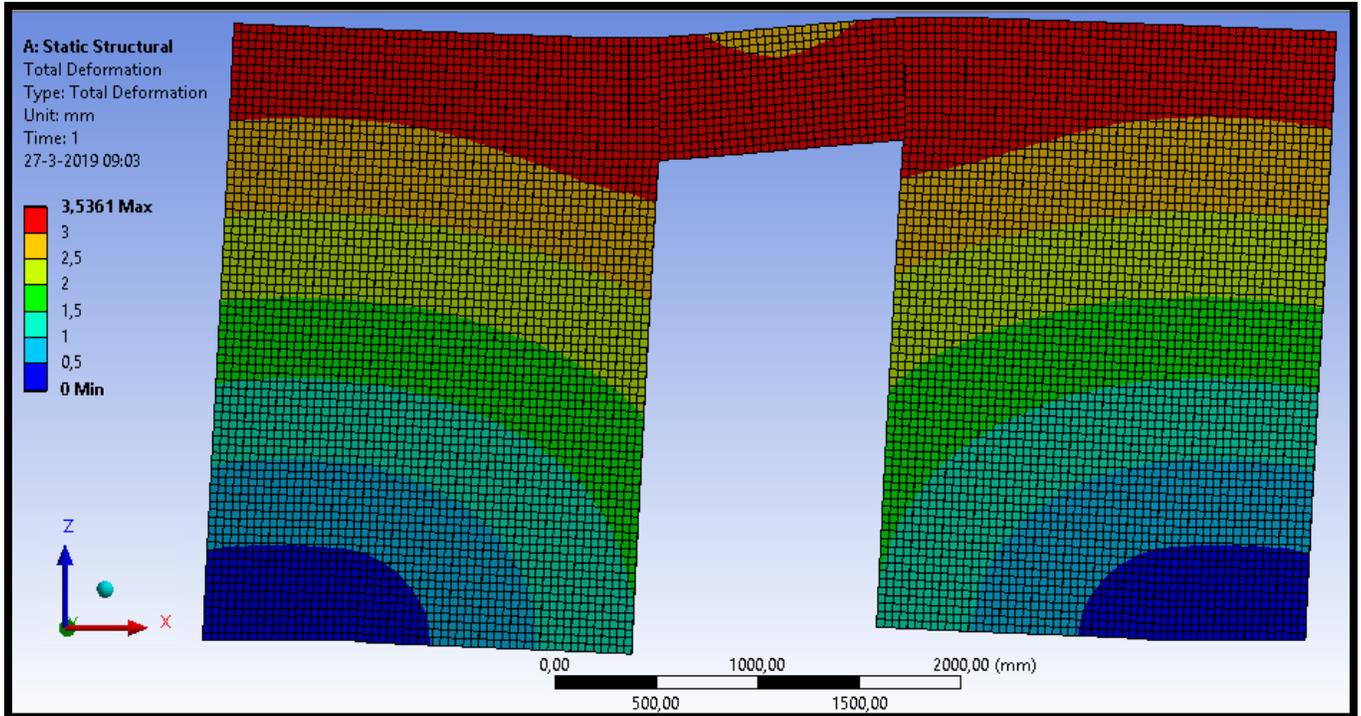


Figure 70 - Deformation, multiplied 44x, centered door hole

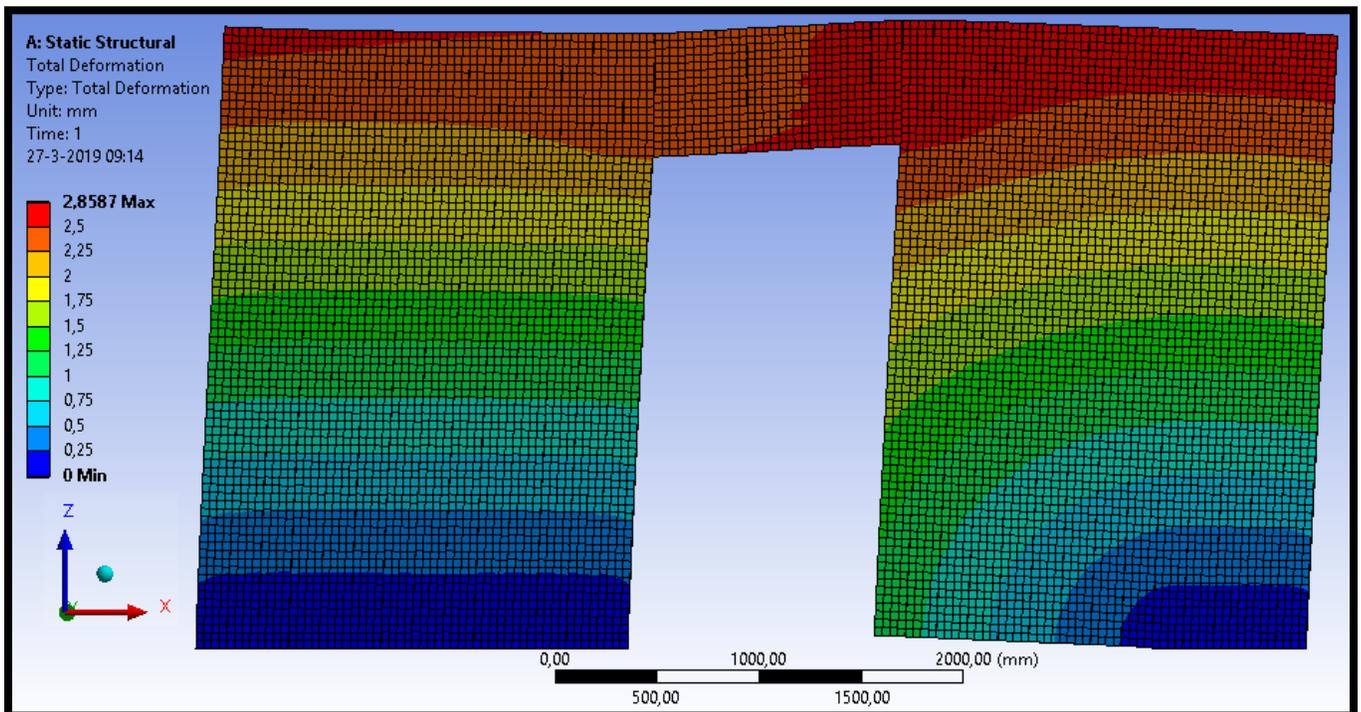


Figure 71 – Deformation, multiplied 55x, centered door hole, “compression only support” inserted

5. CLT-house of the Future: A structural overview of the variants



The stress distributions including details for this slab composition are shown in Figure 72 till Figure 78. The first noticeable change is the disappearing of symmetry in the slab due to the compression only support. This boundary condition creates more in-plane loading resistance from the left part of the slab, leading to local increases in frictional- and shear stress. Inserting tensile connectors to the bottom of the slab next to the door hole would reinstate the symmetry.

The axial stresses (Figure 72, Figure 73) exceed the resistance only in the tensile parts of the CLT (10,88 N/mm²), and only very locally at the corners of the holes or at the supports. The local exceedance causes a local plastic deformation which spreads the stress peak over a larger area, decreasing the maximum value.

The shear stress distributions (Figure 74, Figure 75) show small peaks at the corners of the door hole, but the peaks do not exceed the shear strength of 2,24 N/mm². This means no plastic deformation arises due to shear in the timber boards.

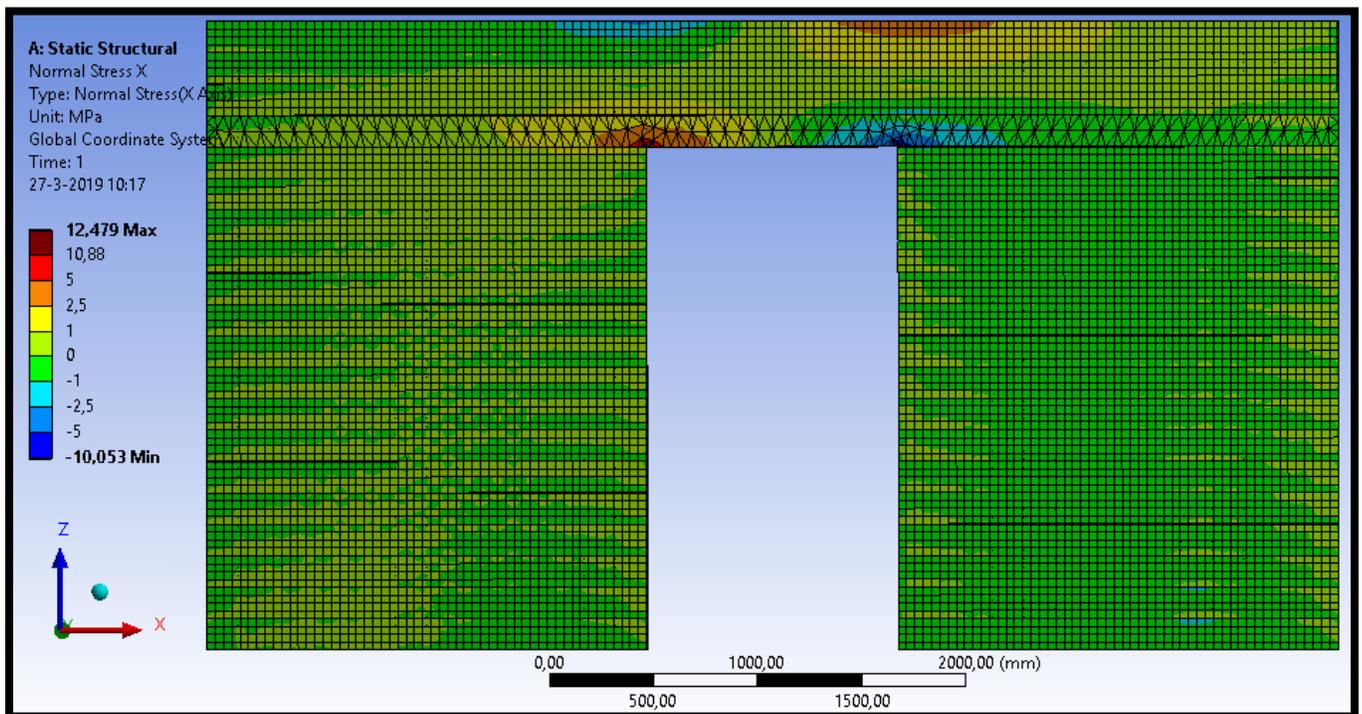


Figure 72 - Axial stress in x-direction in the horizontal layer, centered door hole, "compression only support" inserted

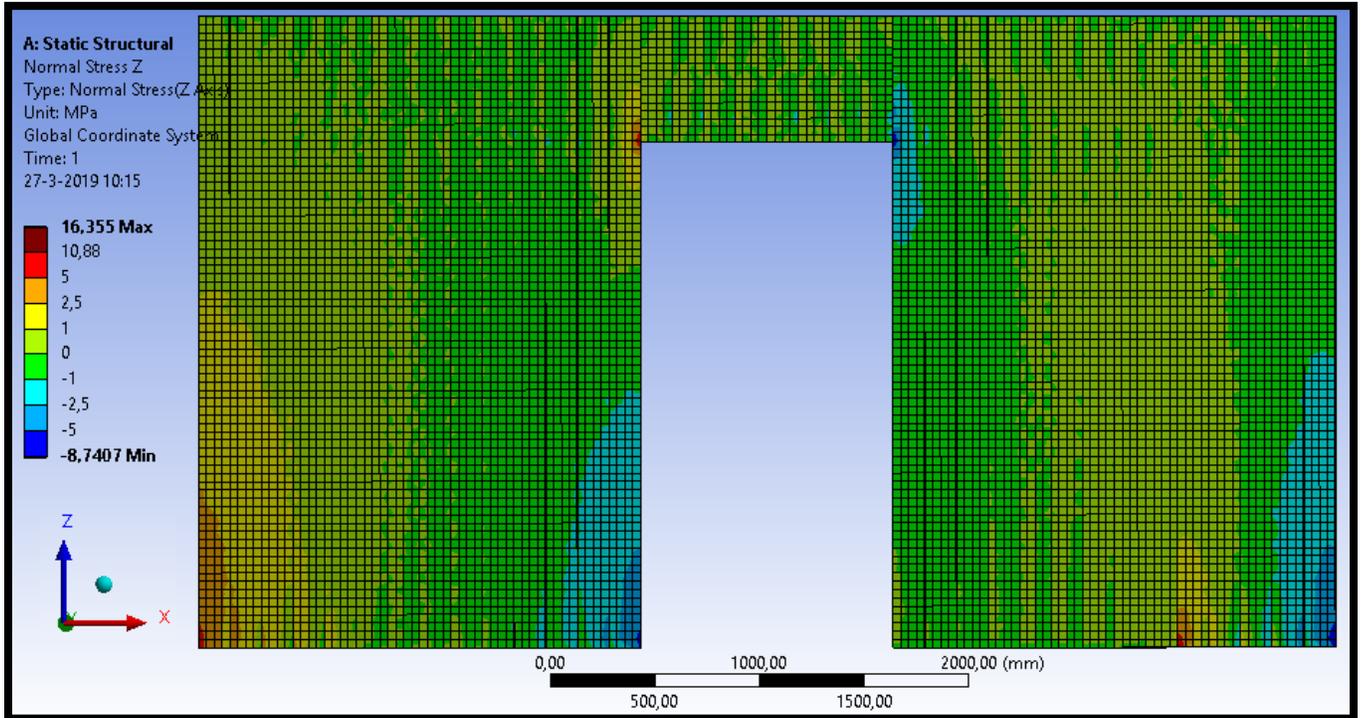


Figure 73 - Axial stress in z-direction in the vertical layers, centered door hole, "compression only support" inserted

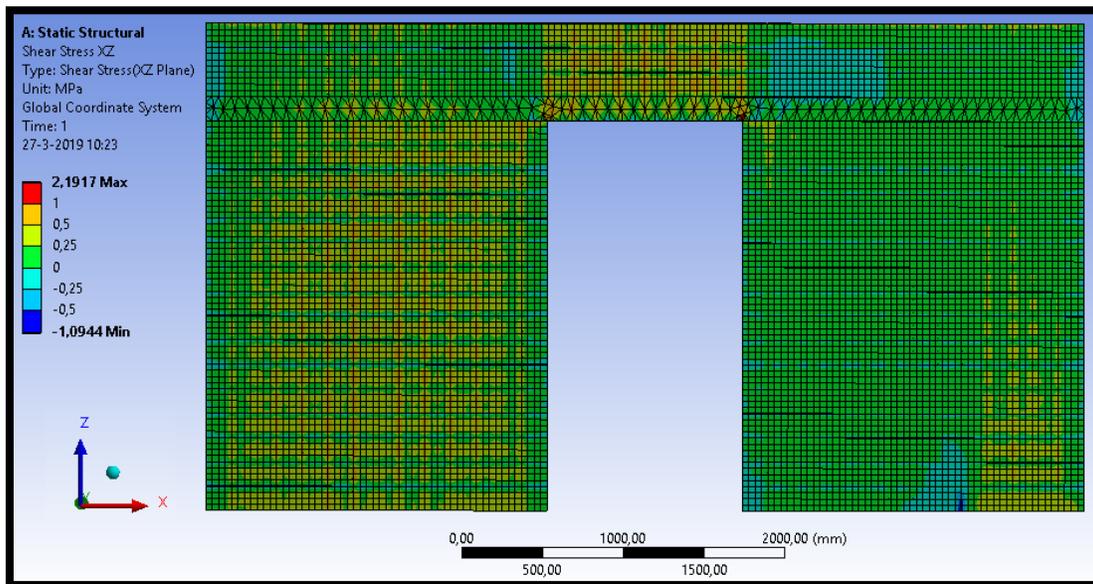


Figure 74 - Shear stress in x-direction on the z-face in the horizontal layer, centered door hole, "compression only support" inserted

5. CLT-house of the Future: A structural overview of the variants

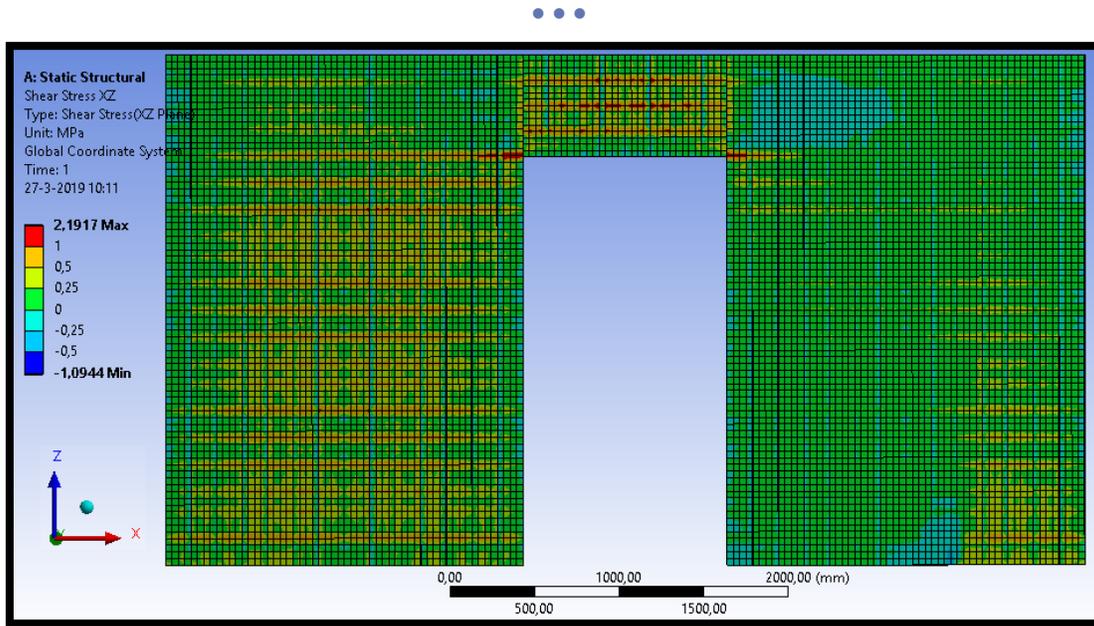


Figure 75 - Shear stress in x-direction on the z-face in the vertical layer, centered door hole, "compression only support" inserted

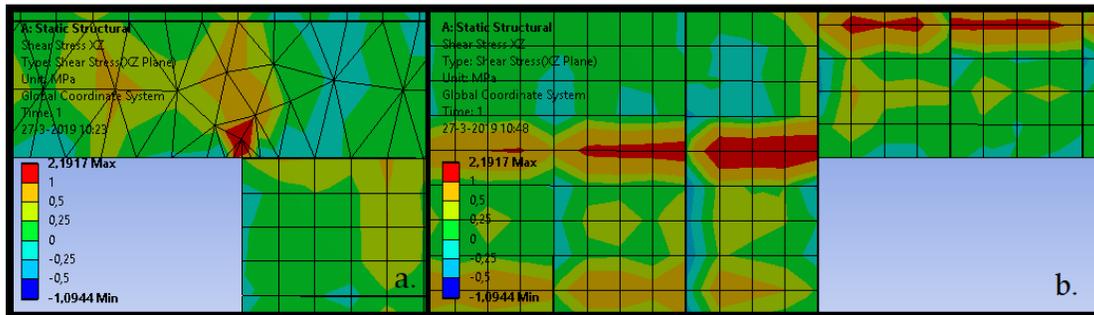


Figure 76 - Shear stress detail, a. horizontal layer, b. vertical layer; centered door hole, "compression only support" inserted

The frictional stress distribution shows values exceeding the resistance of $1,81 \text{ N/mm}^2$ (Figure 77). The stress peaks occur at the corners of the door hole and are again very local. The rolling shear stresses in these locations are lower than the relevant resistance of $0,80 \text{ N/mm}^2$ (Figure 79 and Figure 80). Plastic deformation will occur in the contact areas in these locations as a result of the combination of rolling and torsional shear, again spreading the stress peak over a larger area to decrease the maximum value to a desirable level.

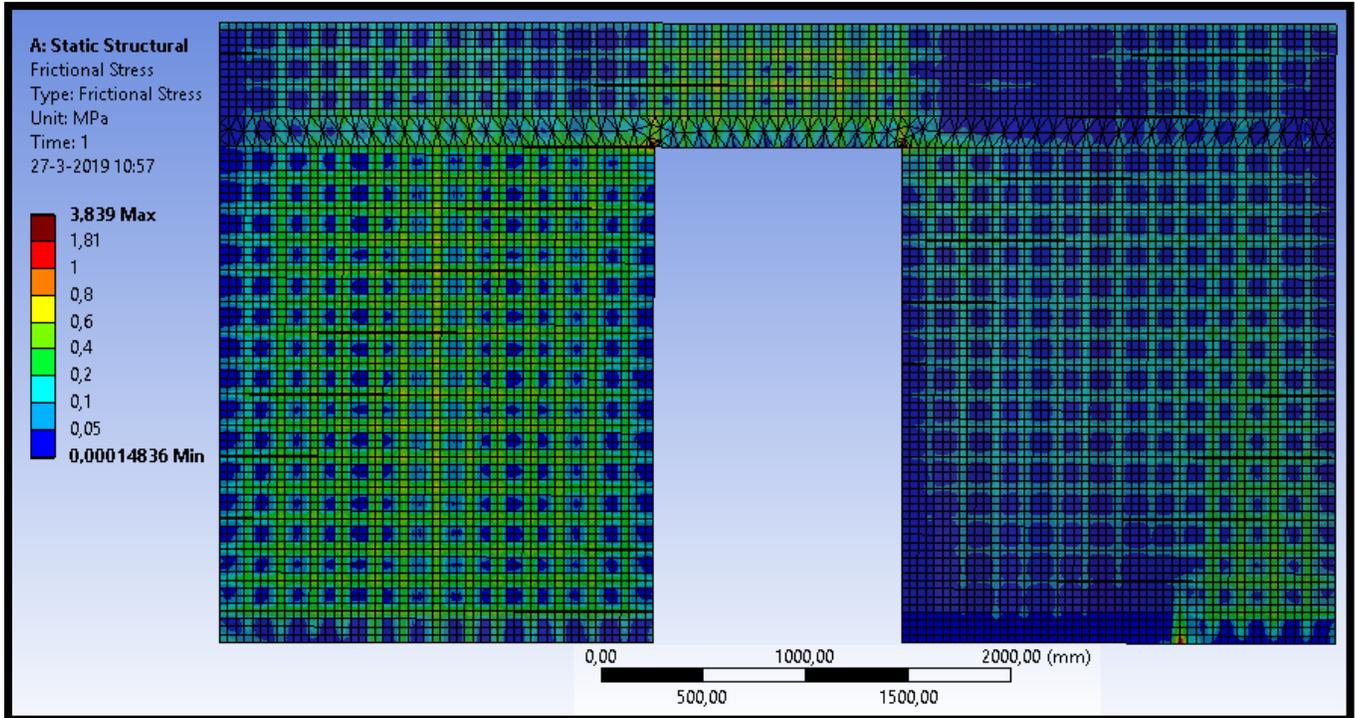


Figure 77 - Frictional stress in lamination layer, centered door hole, "compression only support" inserted

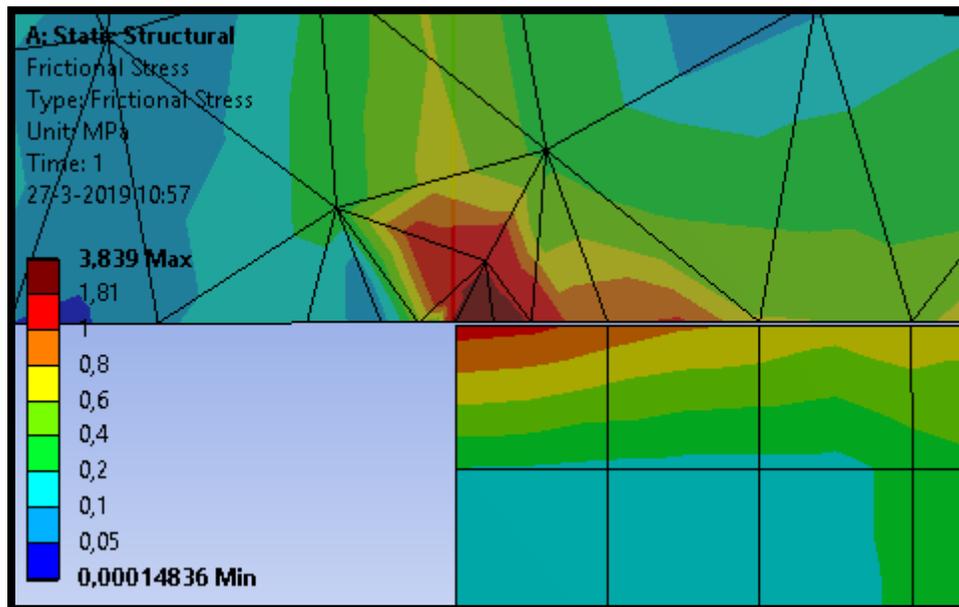


Figure 78 - Frictional stress detail, centered door hole, "compression only support" inserted

5. CLT-house of the Future: A structural overview of the variants

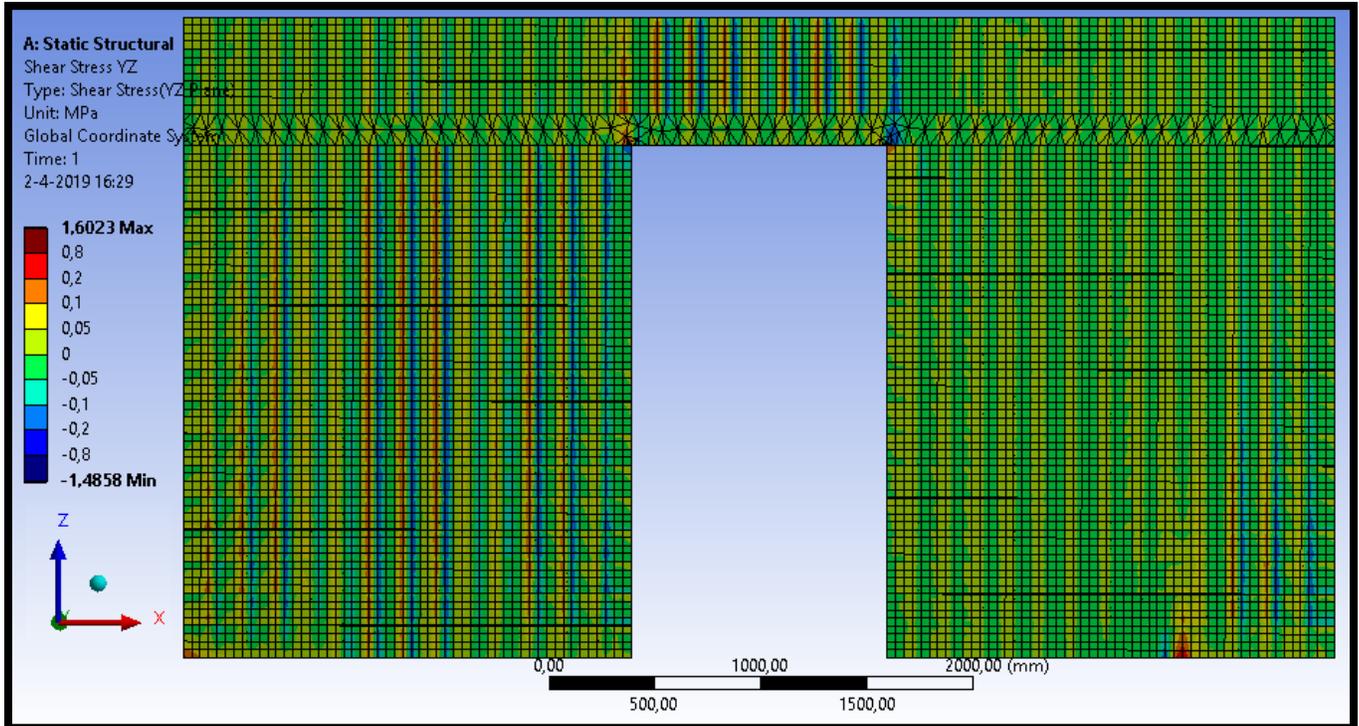


Figure 79 - Shear stress in y-direction on the z-face in the horizontal layer, centered door hole, "compression only support" inserted



Figure 80 - Shear stress in y-direction on the x-face in the vertical layer, centered door hole, "compression only support" inserted



Lastly, applications for the ground floor slab of the pilot project need to be investigated and analysed. According to article 3.75 of the Dutch Building Decree, the area of daylight entrance needs to be 10% of the floor area, with a minimum of 0,5 m². The floor area per storey of the house is equal to (9,6 m times 5,4 m is) 51,84 m². This means a total of 5,18 m² daylight entrance needs to be realized per storey. This area is divided over the front- and back facades, resulting in 2,6 m² of light entrance per façade. Including 50% of the door area in the calculation, this means a window is required next to the door with an area of 1,22 m², or with dimensions of 1 by 1,2 m.

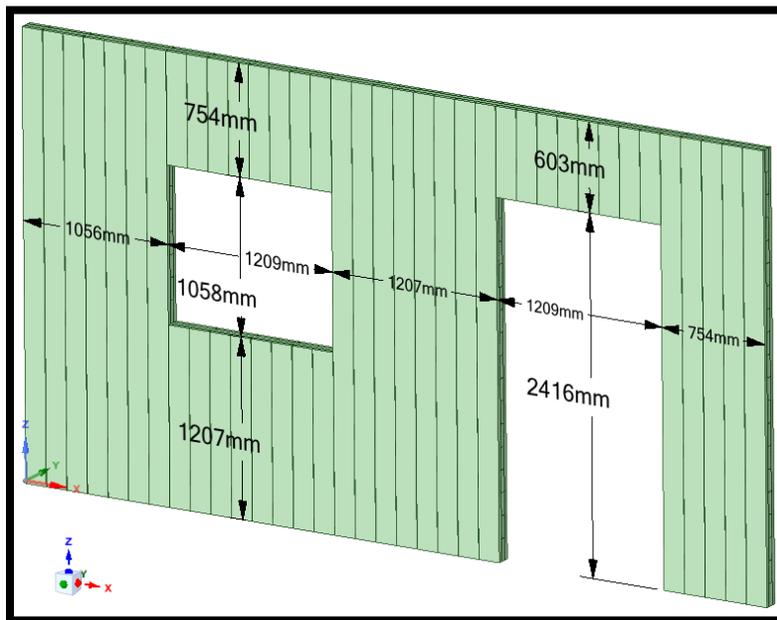


Figure 81 - Geometry, "door + window" variant

Due to asymmetry in the slab design, wind load cases from both sides of the slab need to be investigated for differences in the results. In case of a wind load from the left side of the slab, the maximum deformation (Figure 82) is less than 10mm. The stress limits (max and min) for the axial- (Figure 83 and Figure 84) and shear stresses (Figure 85 and Figure 86) are within the limits of the respective resistances. The stress peaks occur locally at the supports and corners of the holes. The frictional stress maximum exceeds the strength of the contact area of 1,81 N/mm², though the rolling shear stresses remain lower than the resistance of 0,80 N/mm² (Figure 89 and Figure 90). The detail in Figure 88 shows the stress peak occurs locally, causing local plastic deformation to spread the stress peak over a larger area of the connection.

5. CLT-house of the Future: A structural overview of the variants

...

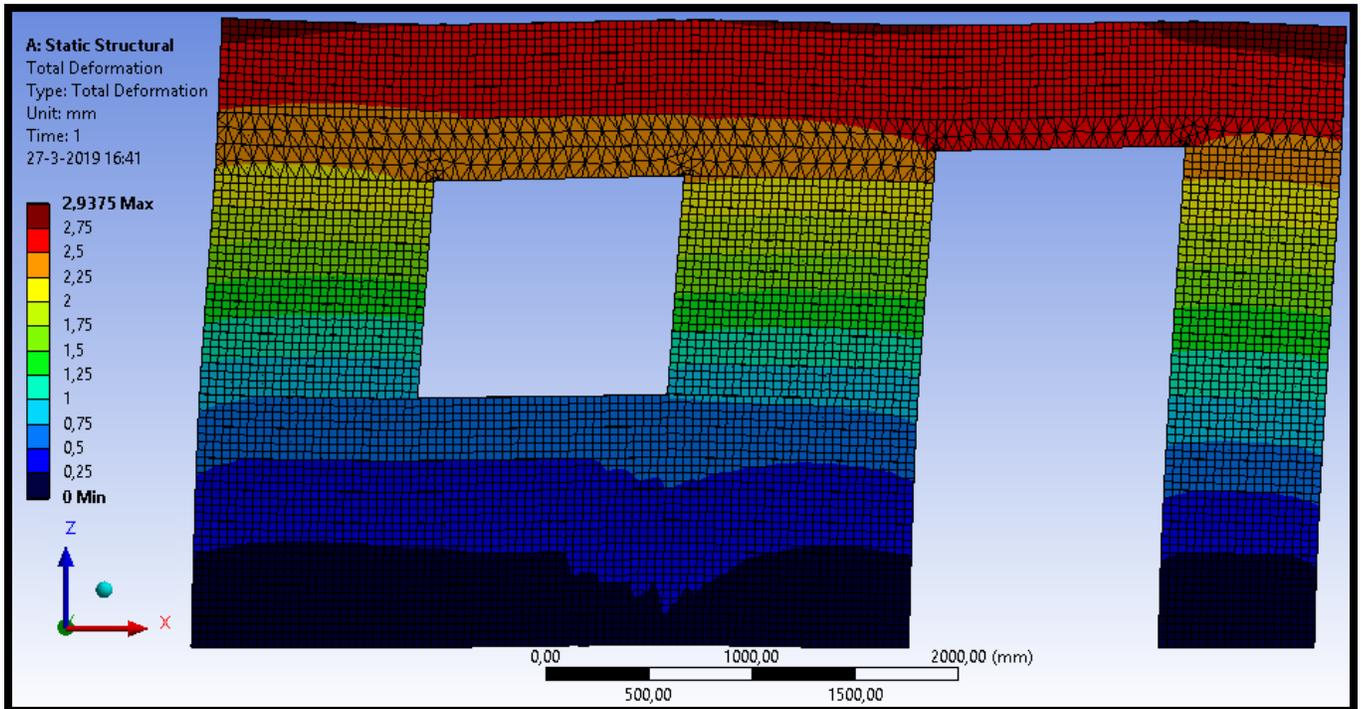


Figure 82 - Deformation, multiplied times 53, "door + window" variant, wind from left

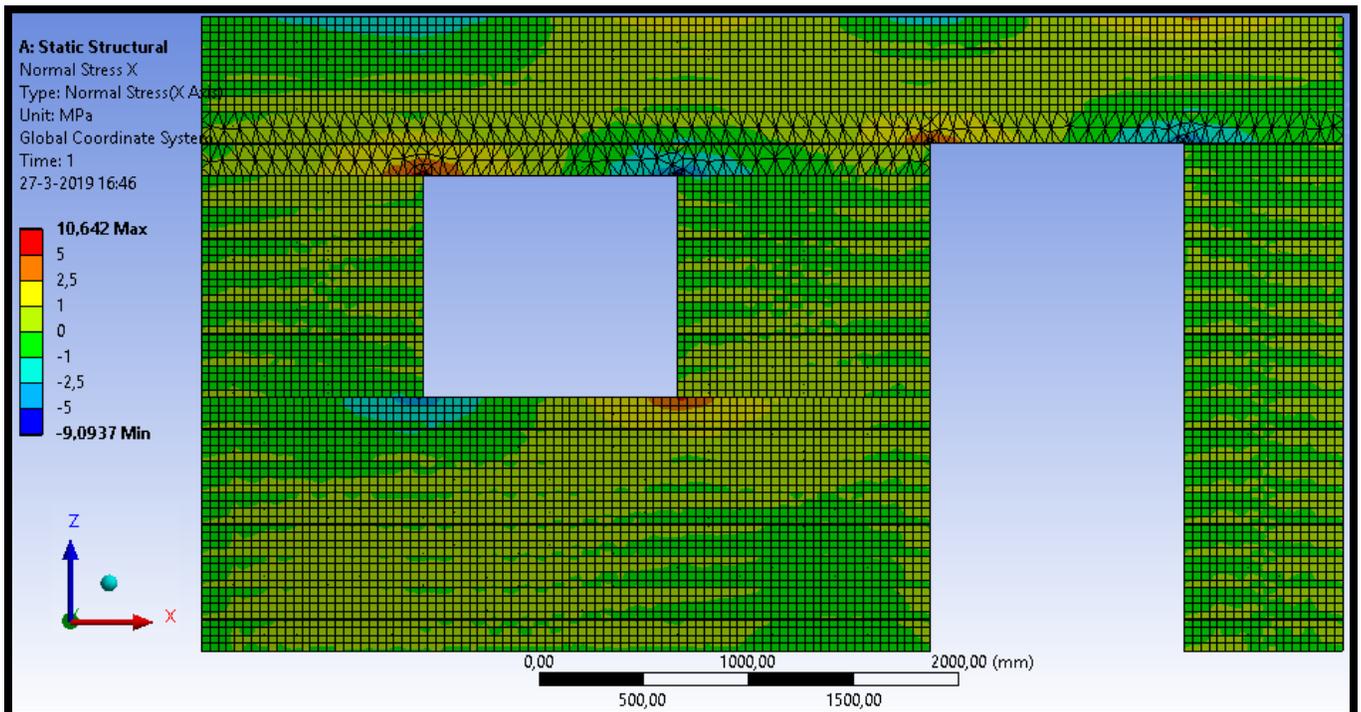


Figure 83 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from left

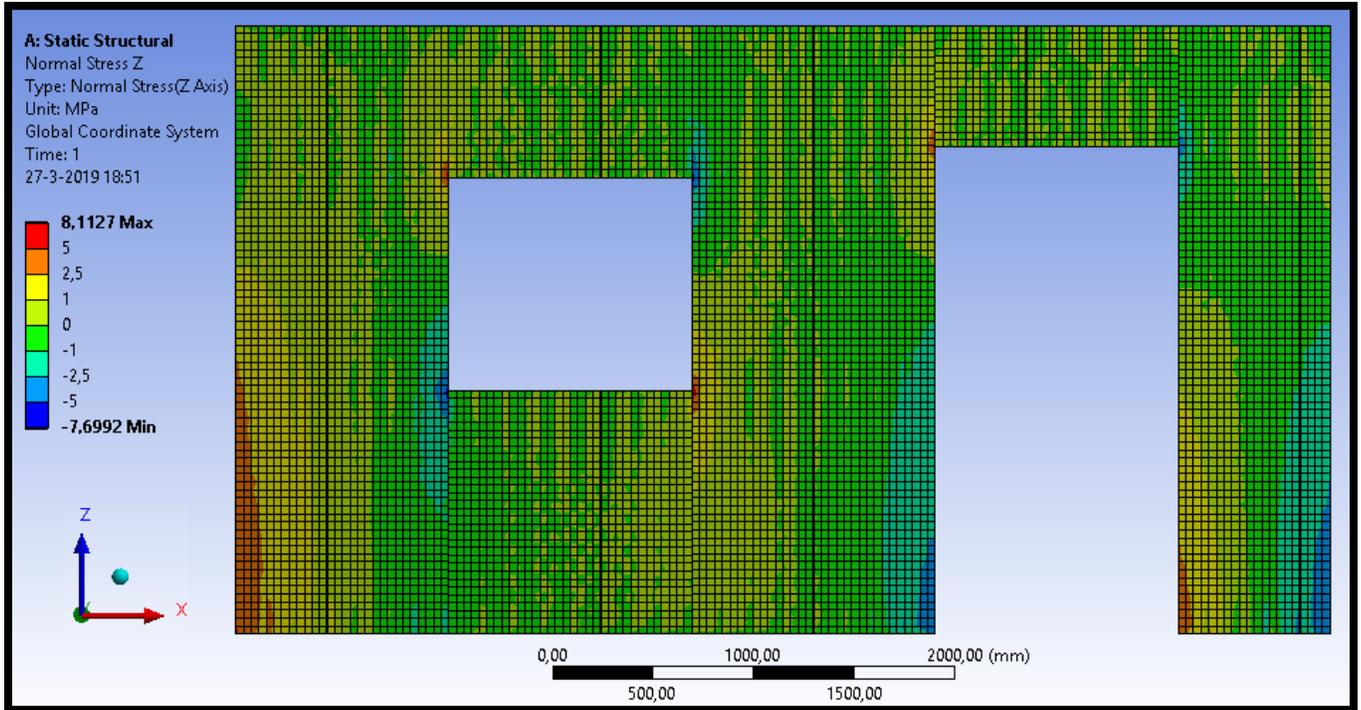


Figure 84 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from left

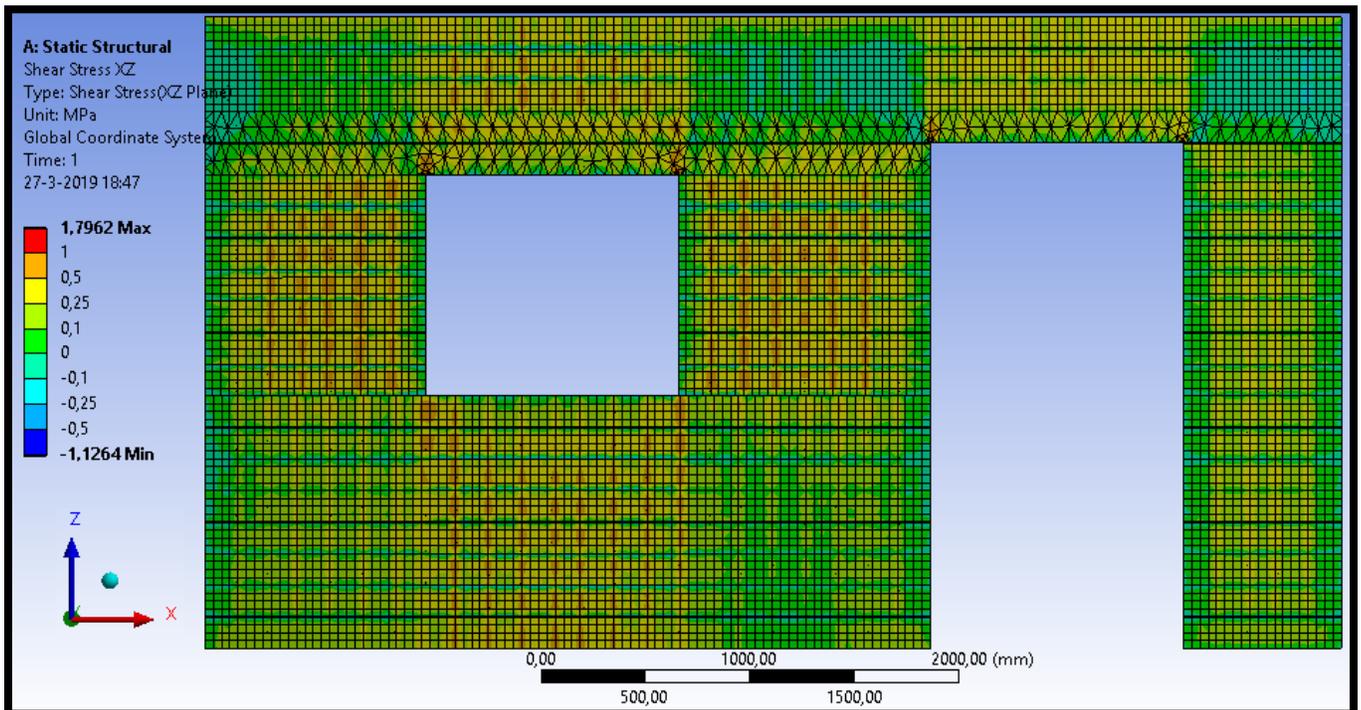


Figure 85 - Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from left

5. CLT-house of the Future: A structural overview of the variants

• • •

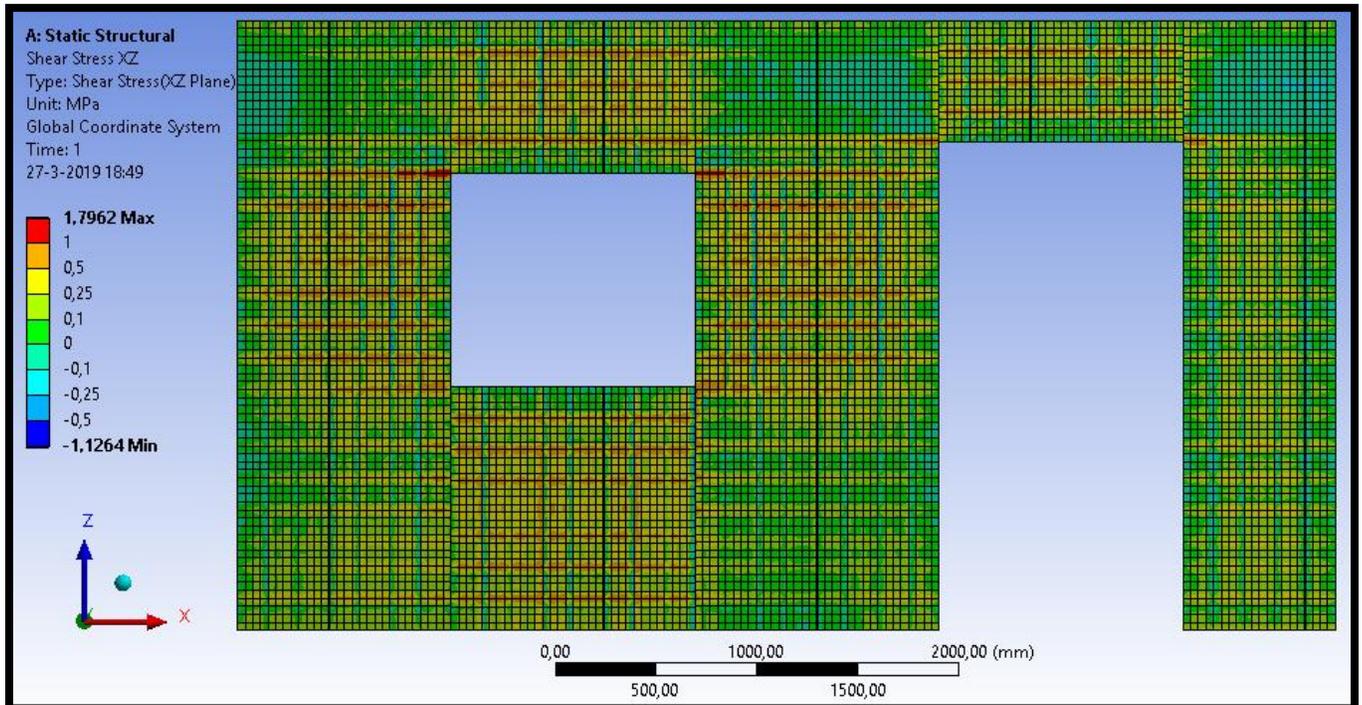


Figure 86 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from left

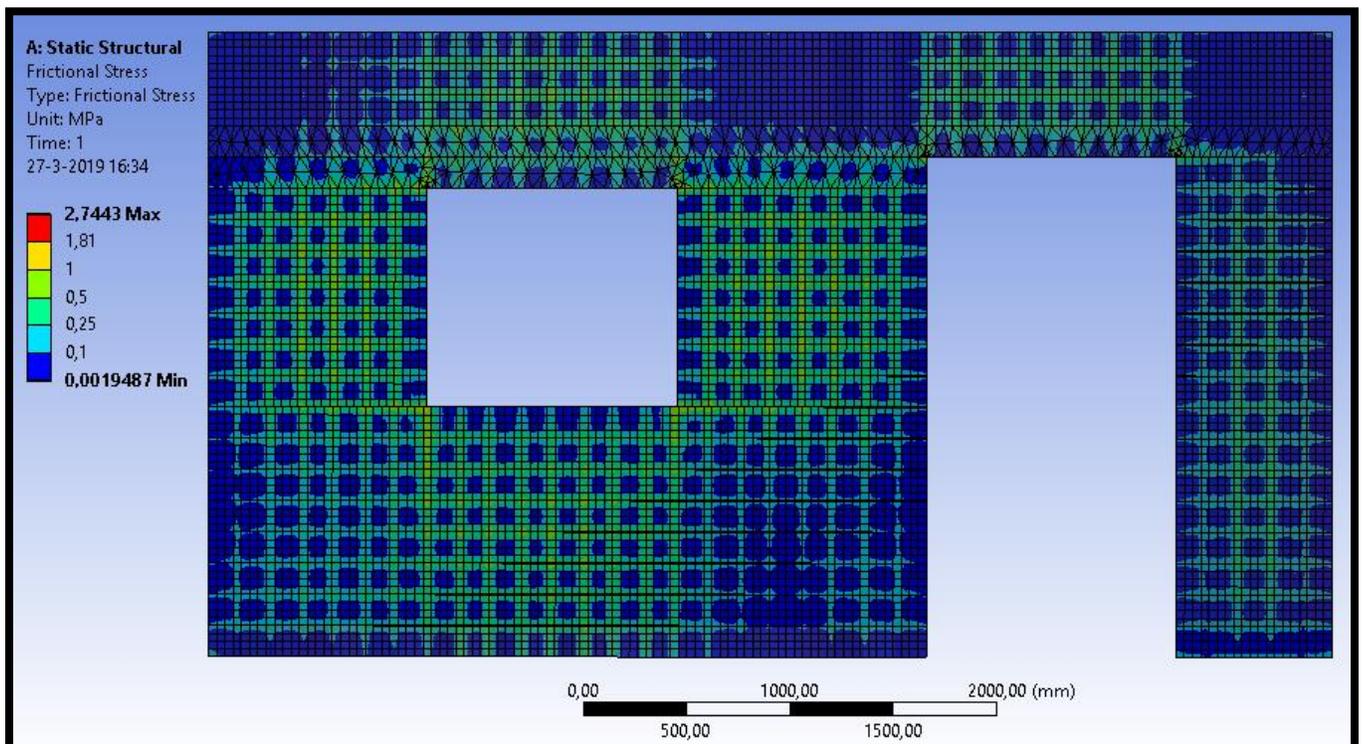


Figure 87 - Frictional stress in lamination layer, "door + window" variant, wind from left

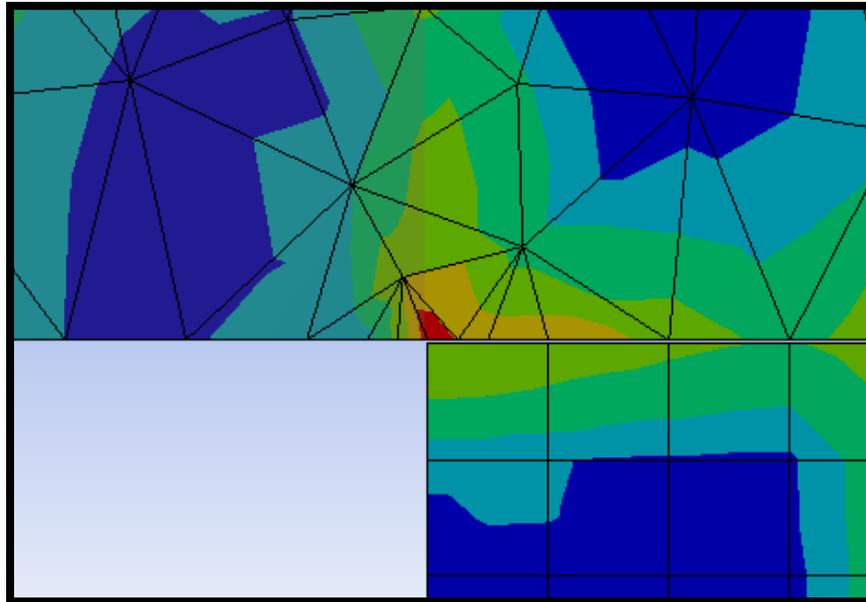


Figure 88 - Frictional stress detail, "door + window" variant, wind from left

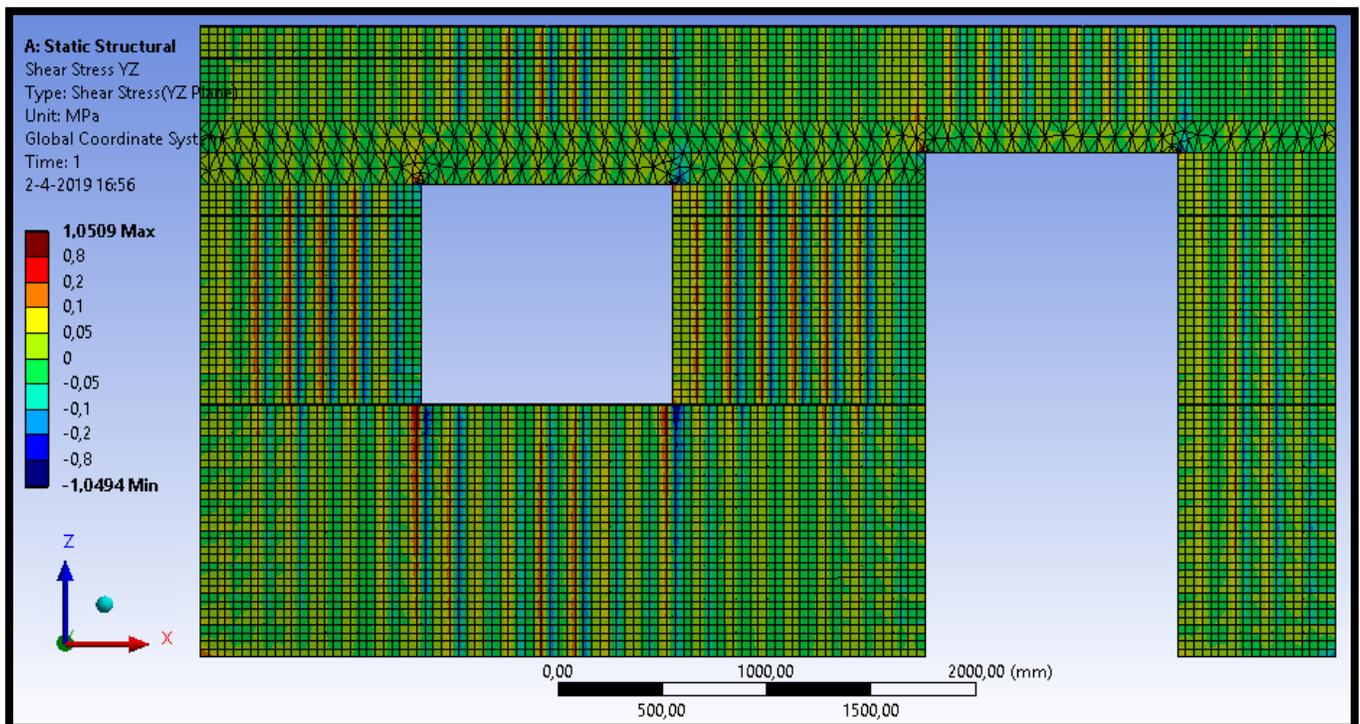


Figure 89 - Shear stress in y-direction on the z-face in the horizontal layer, "door + window" variant, wind from left

5. CLT-house of the Future: A structural overview of the variants

...

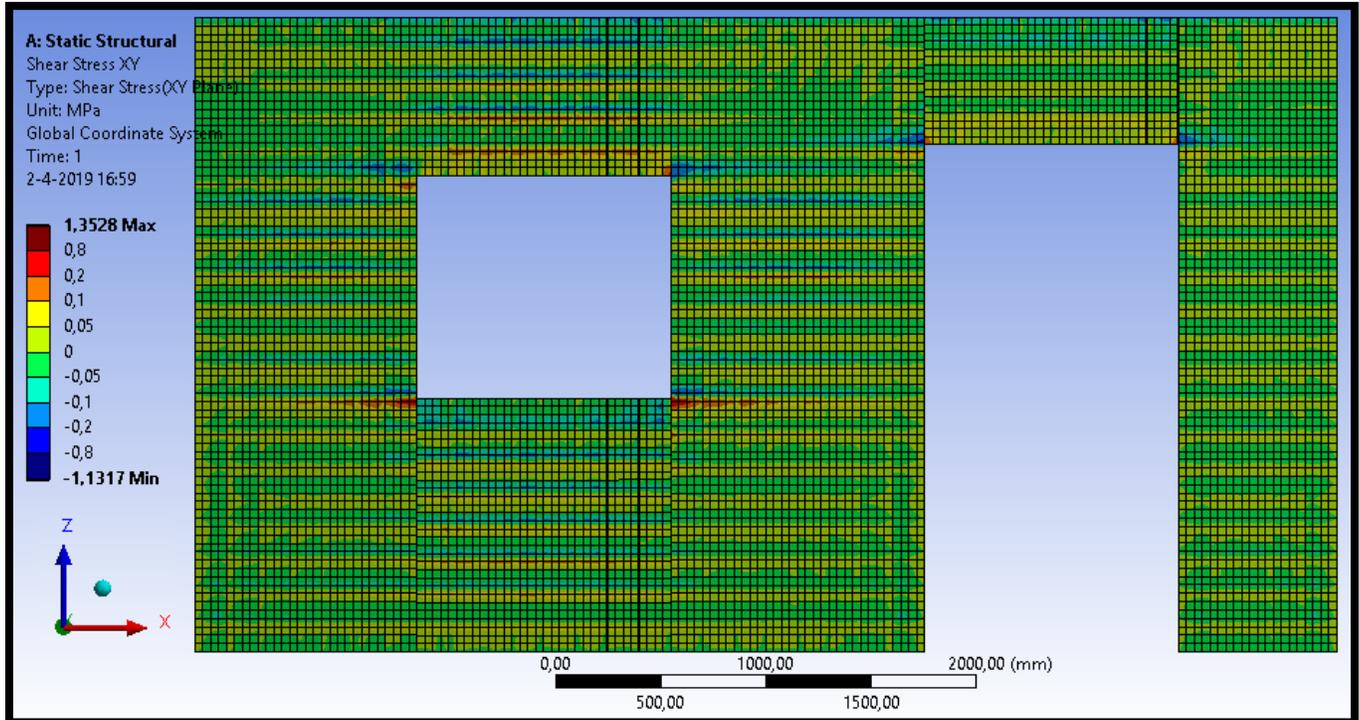


Figure 90 – Shear stress in y-direction on the x-face in the vertical layer, "door + window" variant, wind from left



Figure 91 shows the deformation of the slab for the wind load from the right side. Notable is the vertical displacement of approximately 3 mm of the timber to the left of the door hole. This displacement could become visible at the connection with the floor, and needs to be prohibited with the addition of tensile fasteners to the left of the door hole.

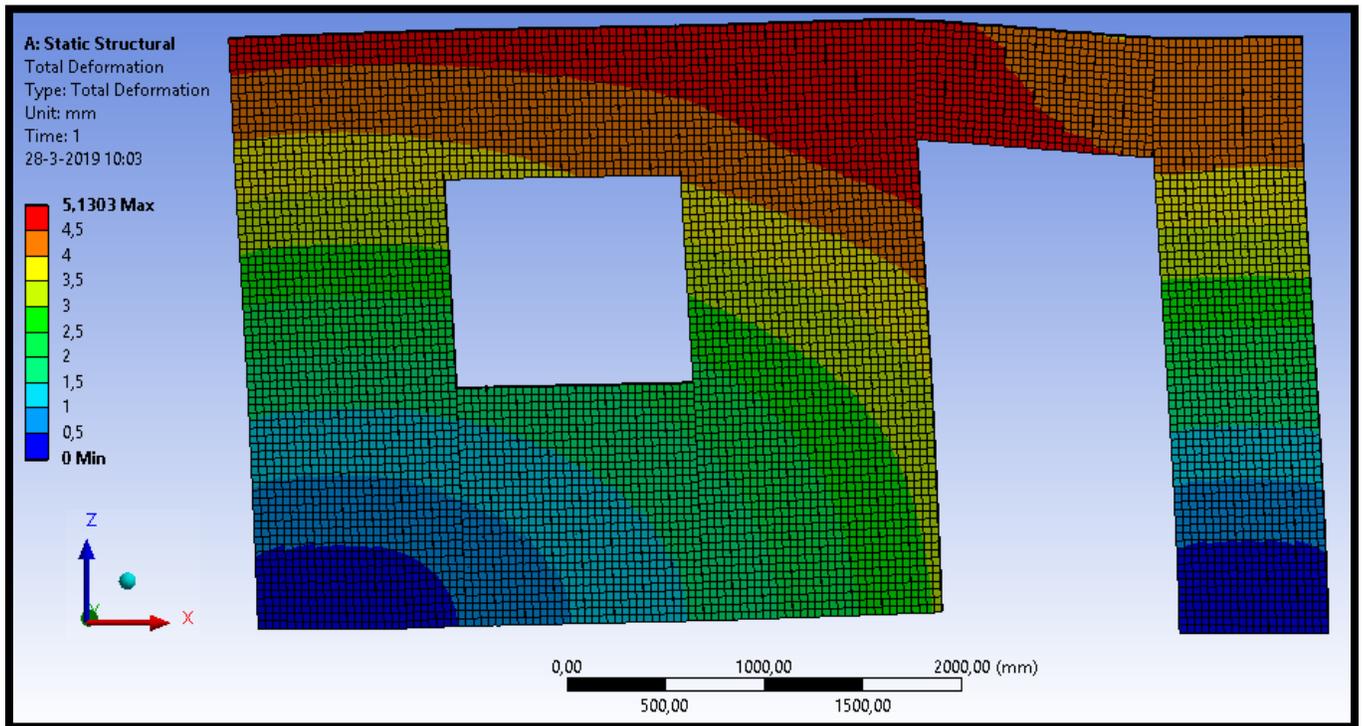


Figure 91 - Deformation multiplied times 33, "door + window" variant, wind from right

The stress distributions are shown in Figure 92 till Figure 98. Again, local stress peaks occur, mostly at the corners of the door hole where the horizontal "rod" has a minimum height. These stress peaks are caused for a large part by the large vertical deformation of the part of the slab to the left of the door hole compared to the right part. As previously discussed, local stress peaks exceeding the relevant strengths are conducted through local plastic deformation to distribute the peaks over a larger area, decreasing the maximum value. However, in this situation, this phenomenon is not applicable for the shear stress in the vertical layers (Figure 96). To the left of the door hole, shear stresses exceeding the strength of 2,24 N/mm² occur over the full width of the board, eventually leading to failure of the contact area between the timber part above the door and to the left.

5. CLT-house of the Future: A structural overview of the variants

...

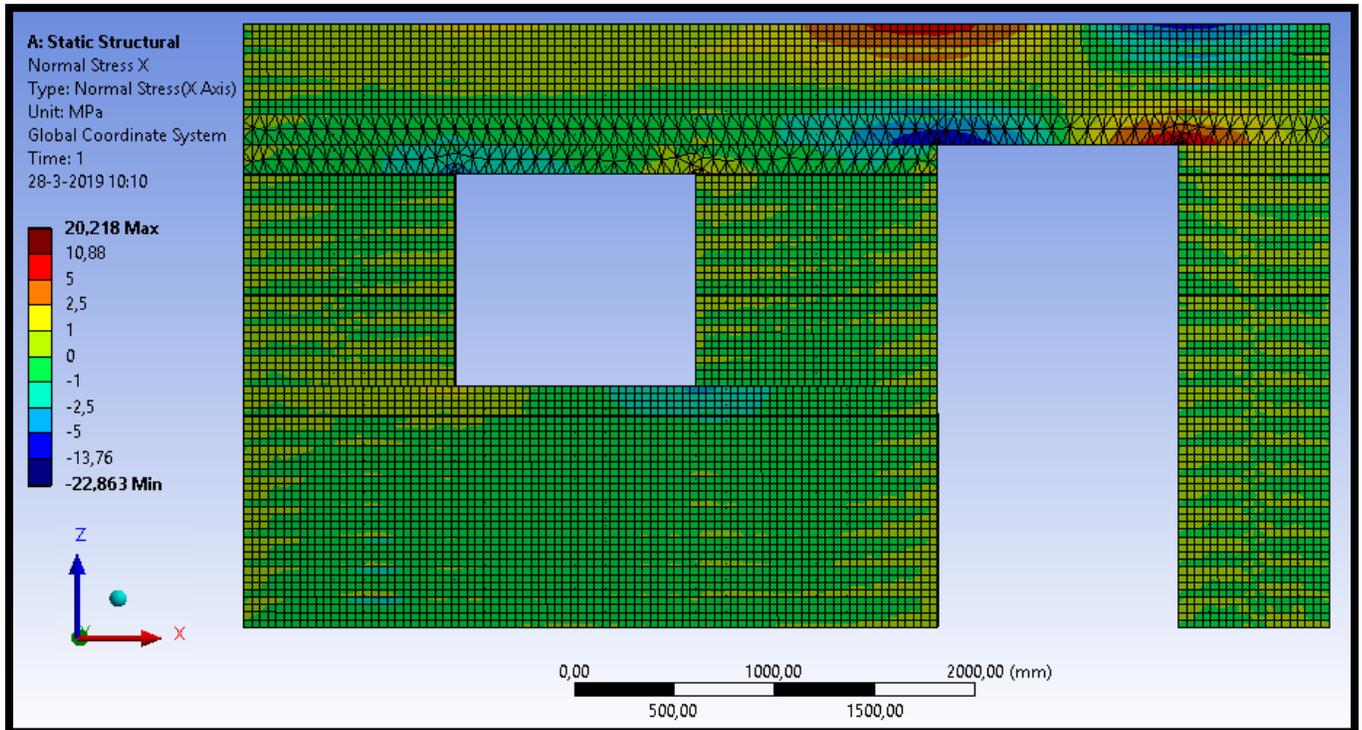


Figure 92 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from right

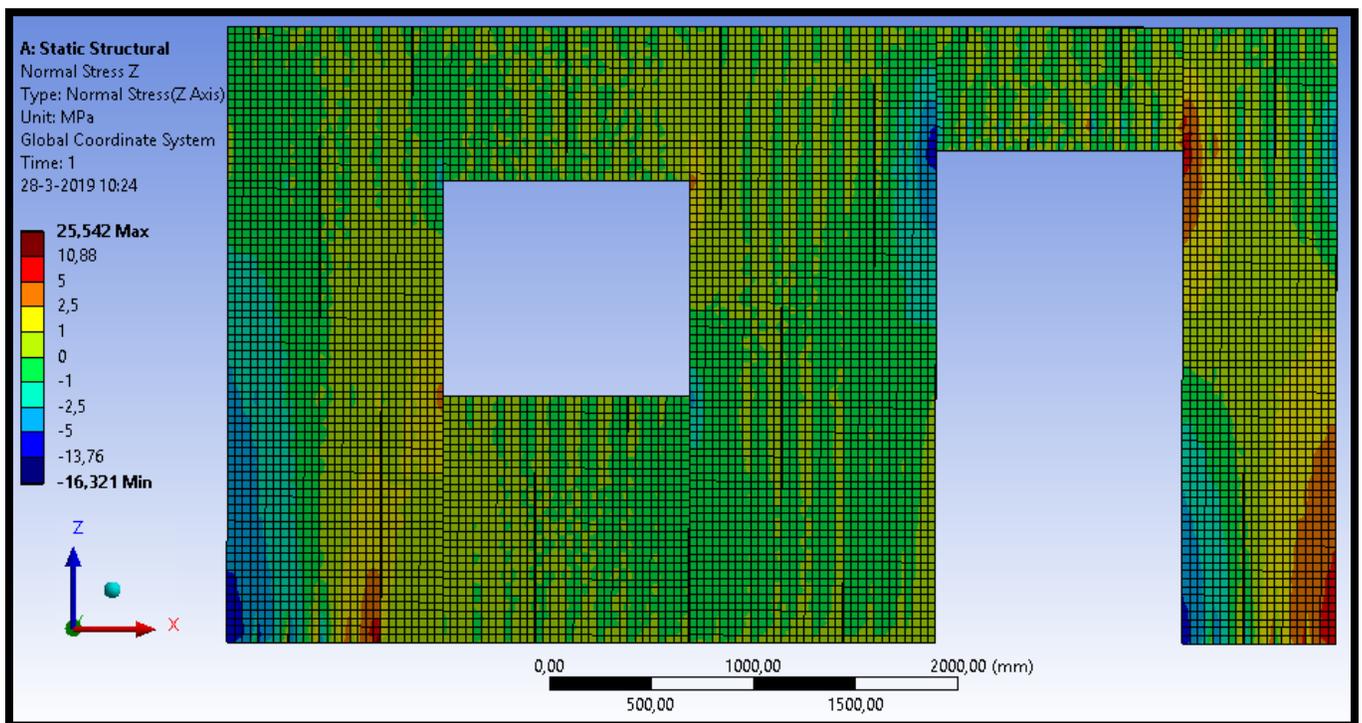


Figure 93 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from right

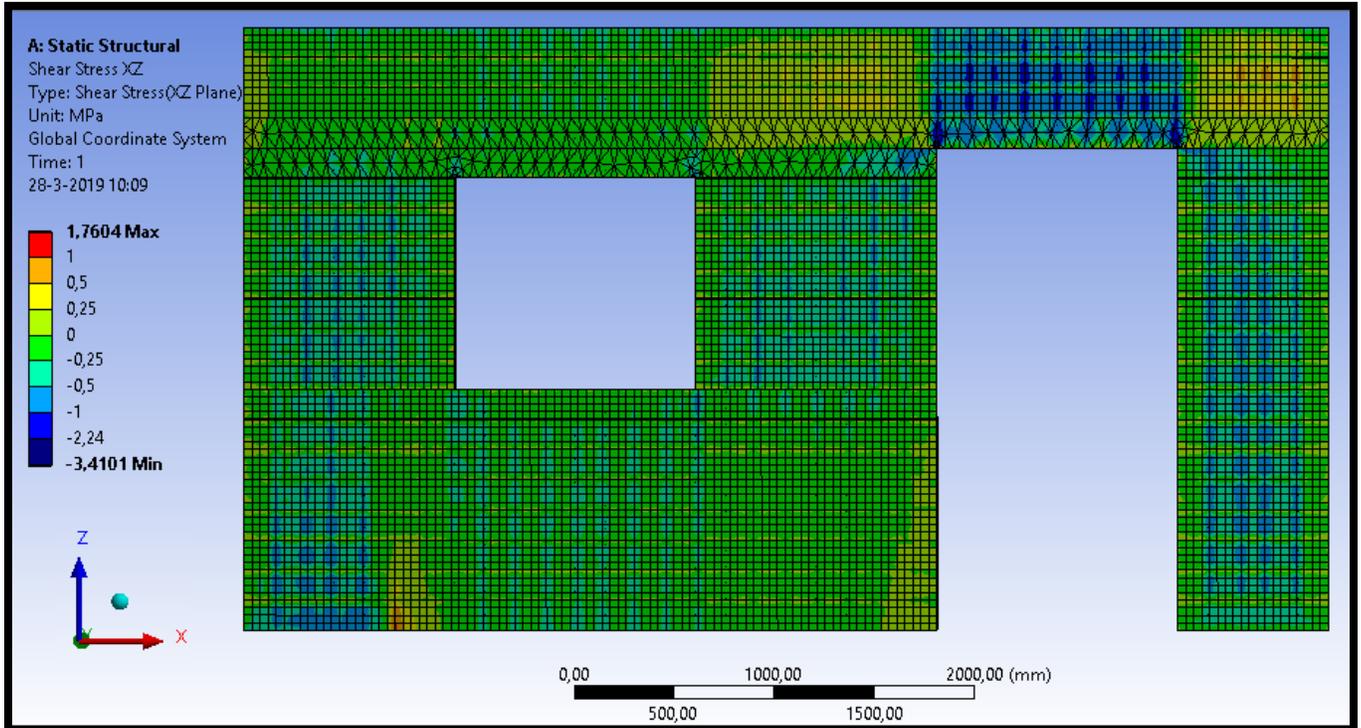


Figure 94 - Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from right

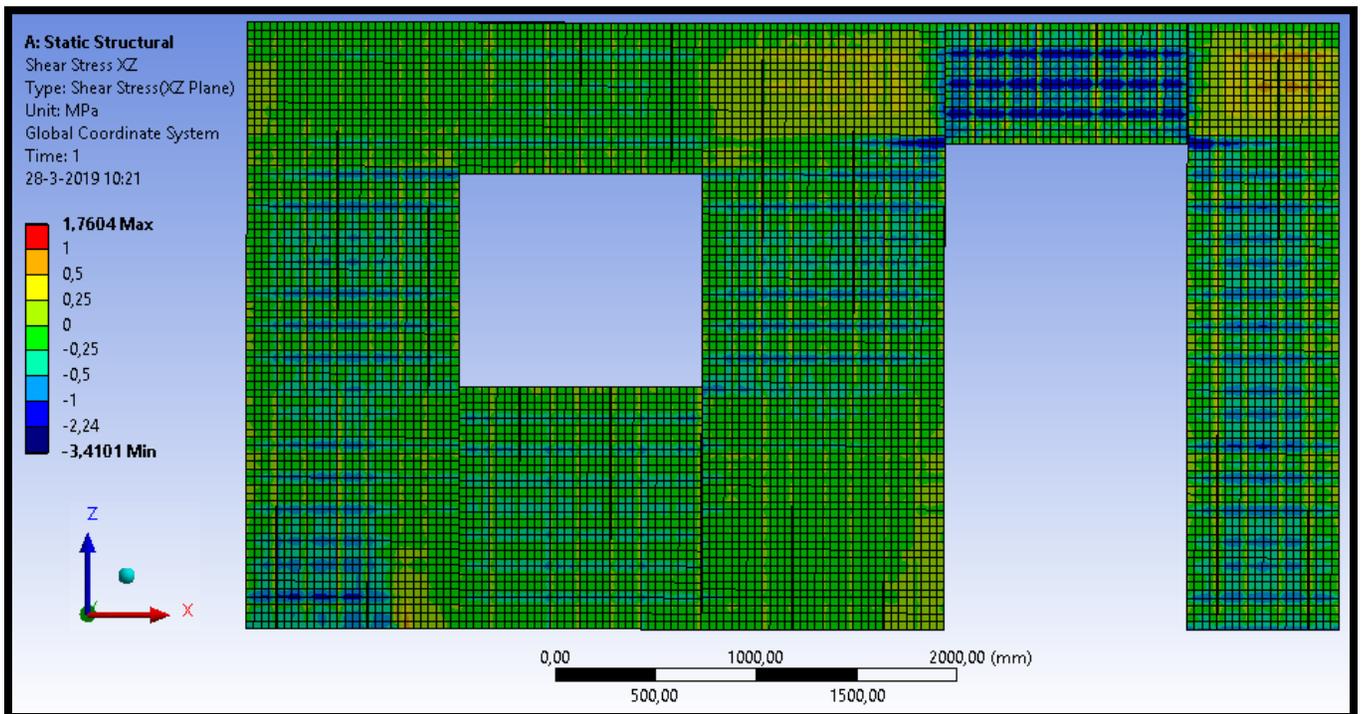


Figure 95 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from right

5. CLT-house of the Future: A structural overview of the variants

...

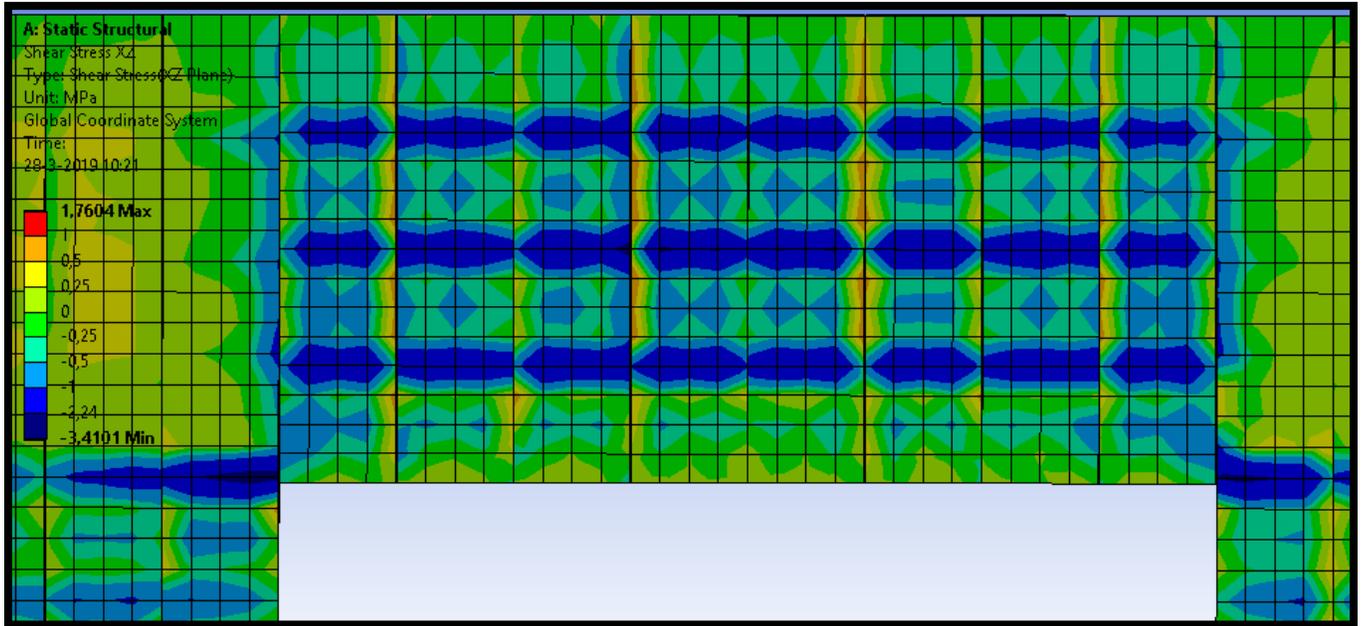


Figure 96 - Shear stress detail in vertical layer, "door + window" variant, wind from right

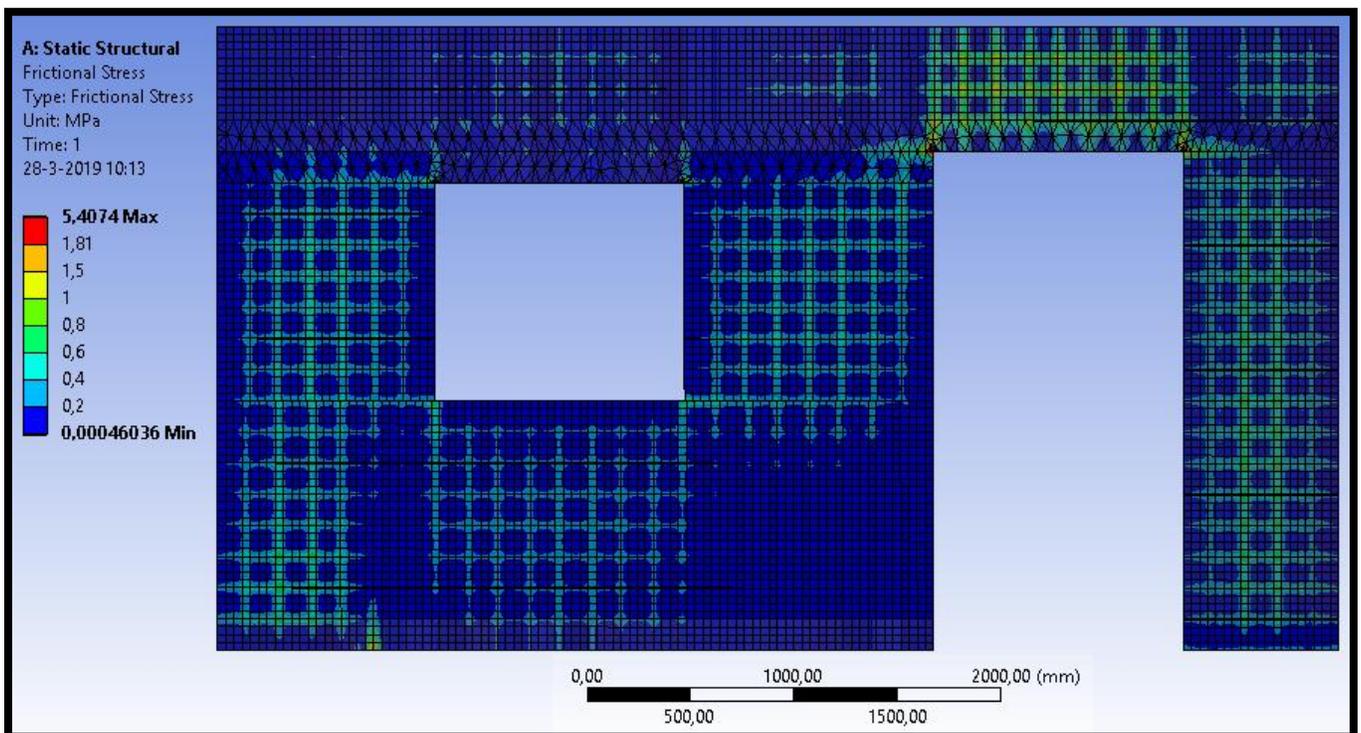


Figure 97 - Frictional stress in lamination layer, "door + window" variant, wind from right

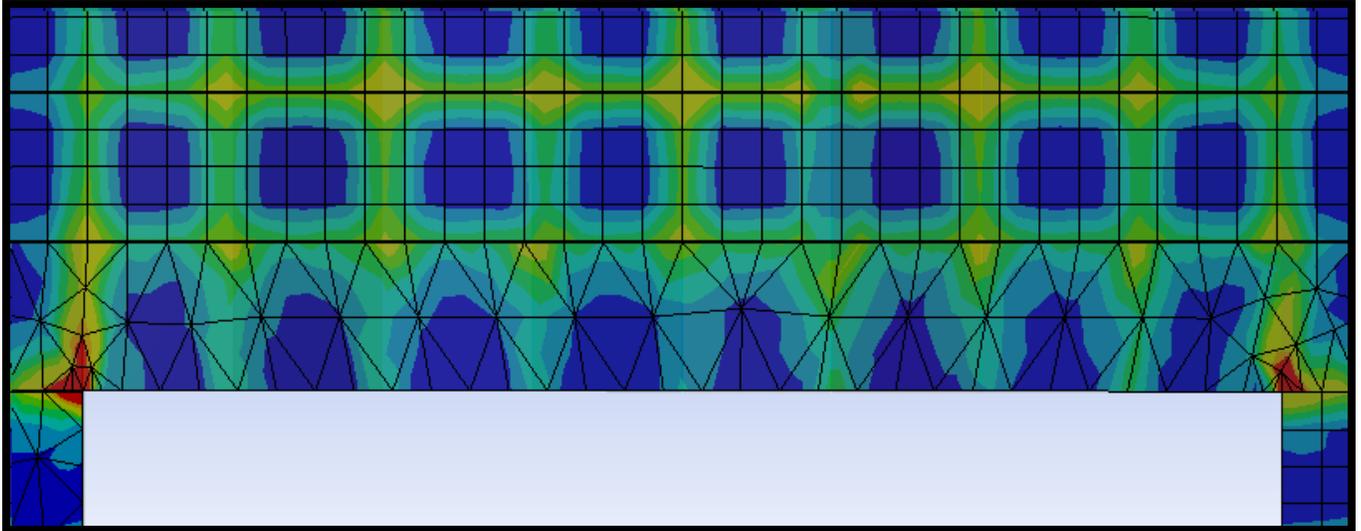


Figure 98 - Frictional stress detail at the top of the door, "door + window" variant, wind from right

5. CLT-house of the Future: A structural overview of the variants



Lastly, to counteract the vertical displacement of the timber part to the left of the door, application of a tensile connector is investigated. This connector is inserted as an extra boundary condition in ANSYS Mechanical. The limitation of the vertical movement leads to a decrease in stresses in the timber (horizontal) rod above the door. The deformed slab for the wind load from the right including the additional boundary condition is shown in Figure 99.

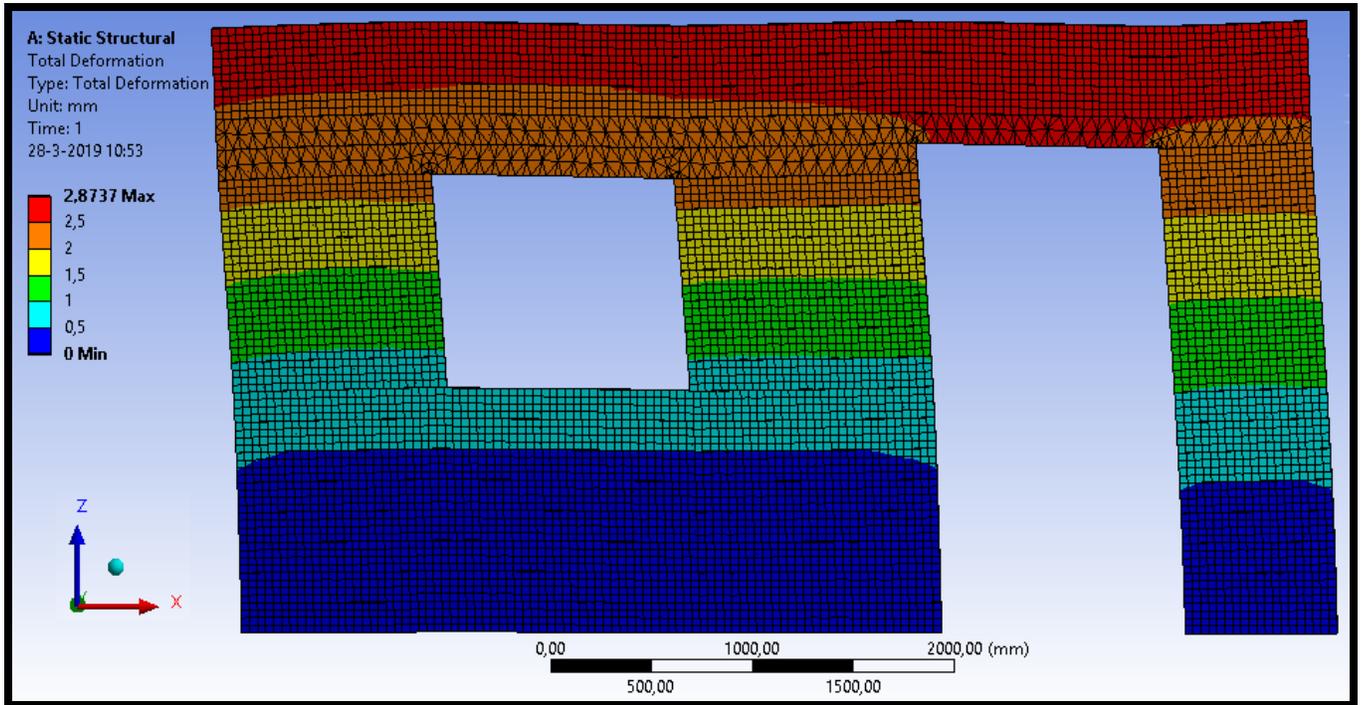


Figure 99 – Deformation multiplied times 55, "door + window" variant, wind from right, incl. extra tensile connector

Figure 100 till Figure 107 show the stress distributions for this variant with the mentioned boundary conditions. Notable is the predicted decrease in maximum- and minimum stress values. For all stress types except the frictional stress, the values are lower than the resistances, with the frictional stress showing a small exceedance of the strength. Figure 105 shows the peak appears locally, causing only local plastic deformation in reality, lowering the actual stress value in the lamination layer, preventing total failure of the contact area.

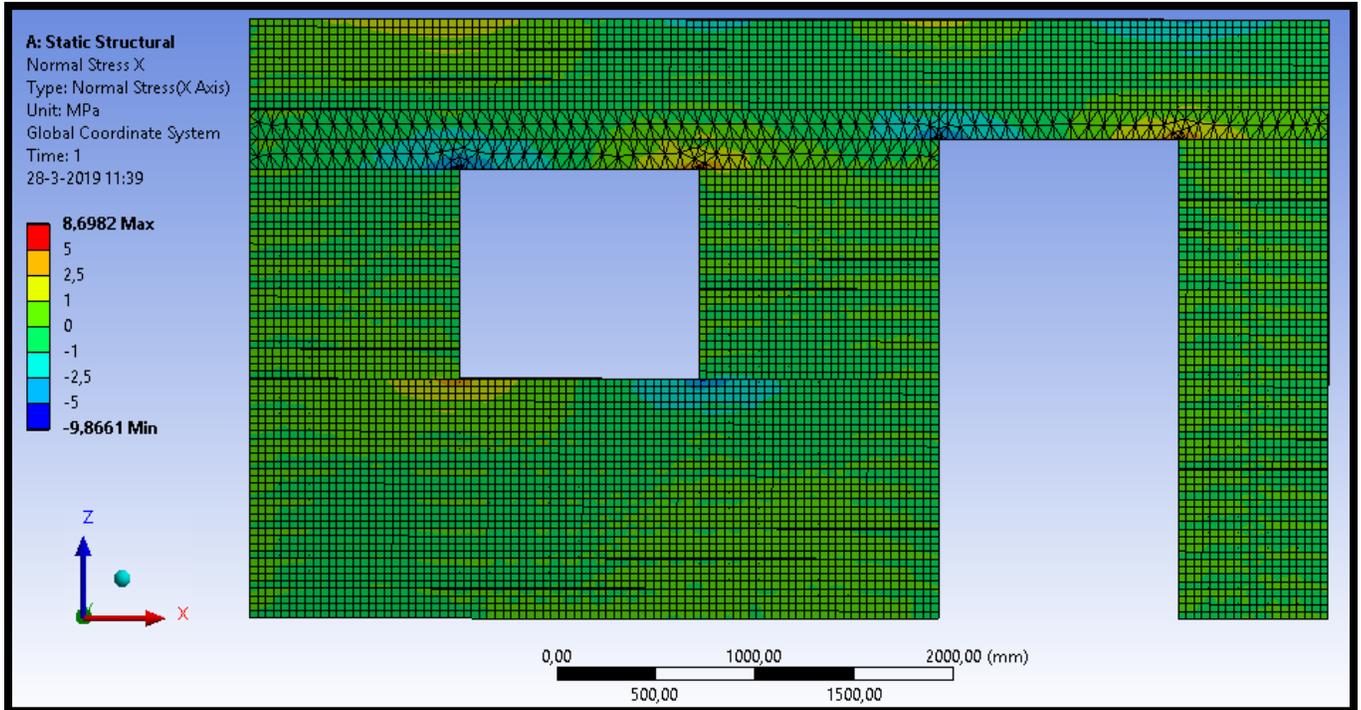


Figure 100 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from right, incl. extra tensile connector

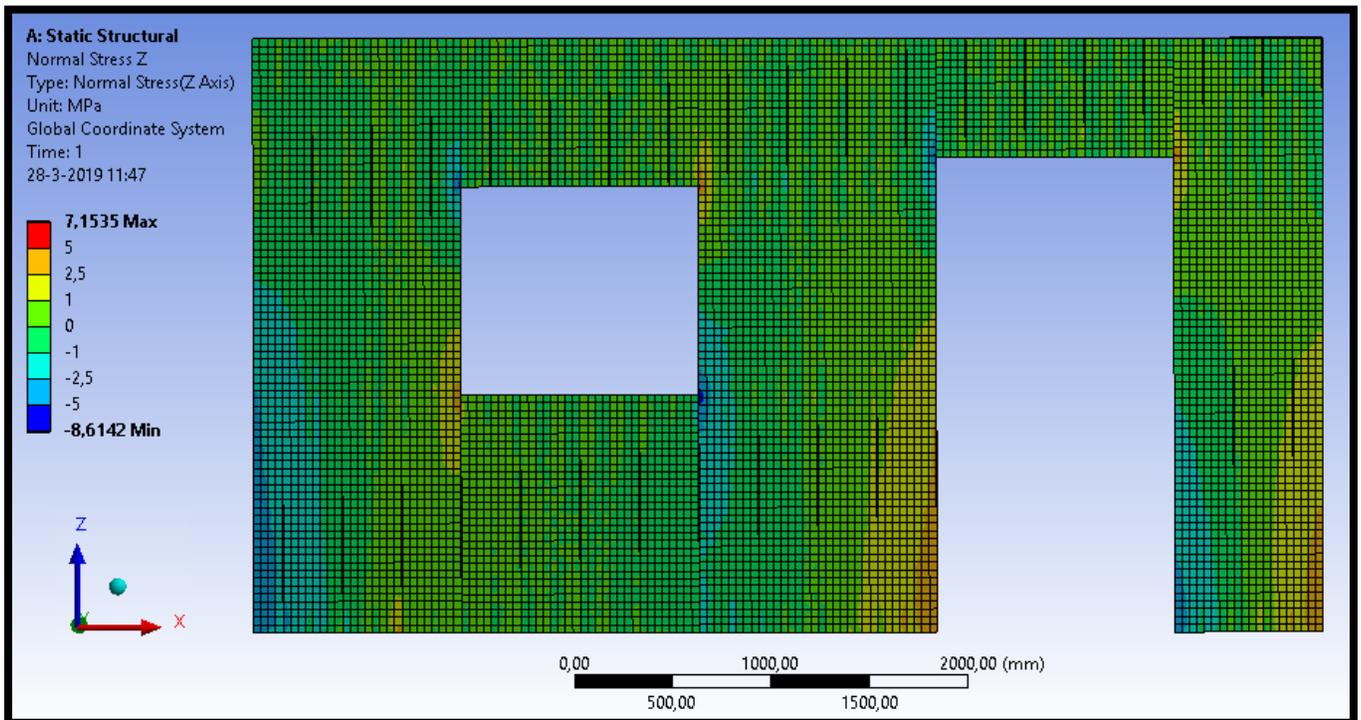


Figure 101 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from right, incl. extra tensile connector

5. CLT-house of the Future: A structural overview of the variants

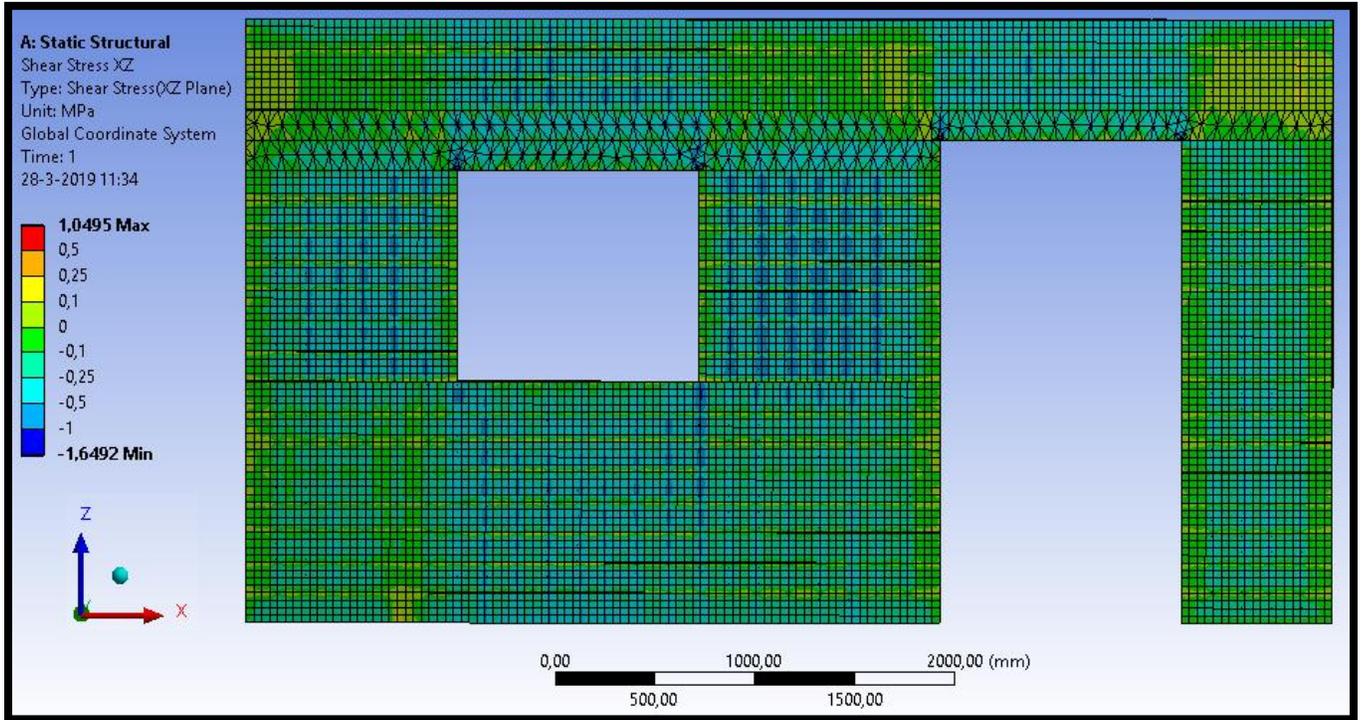


Figure 102 - Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from right, incl. extra tensile connector

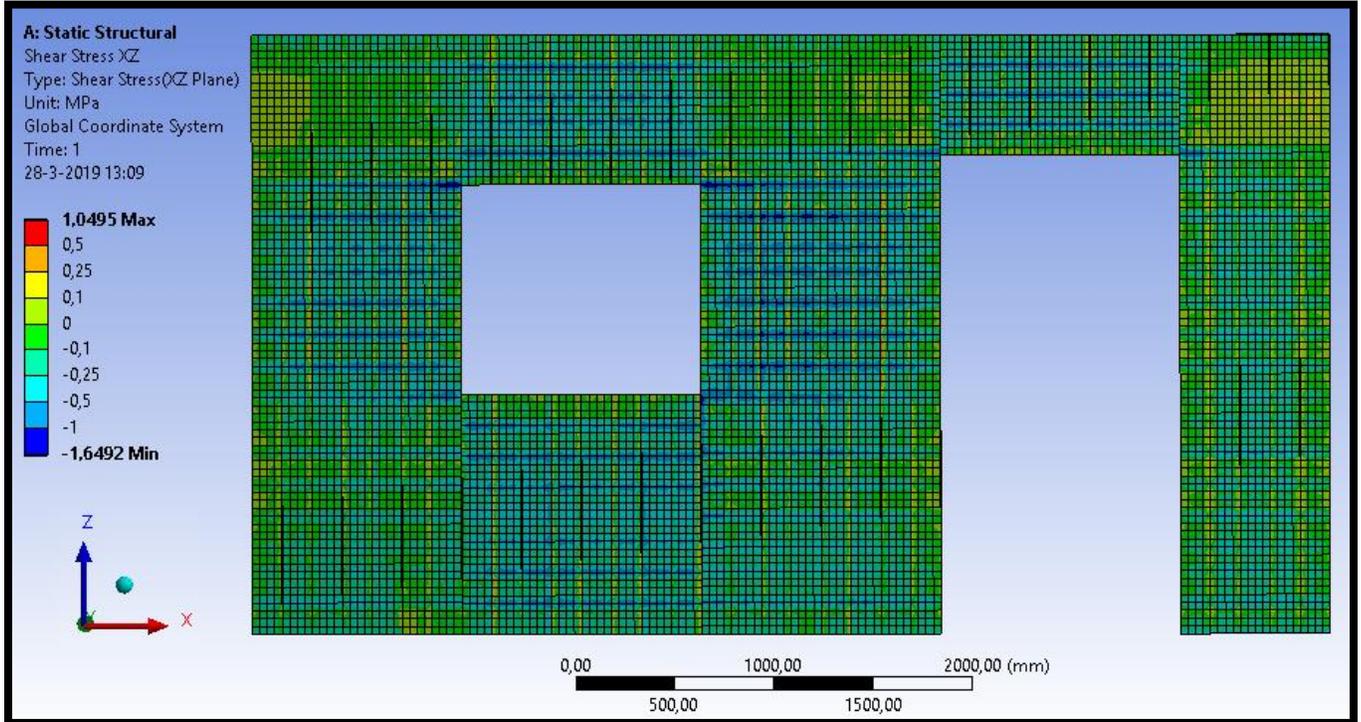


Figure 103 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from right, incl. extra tensile connector

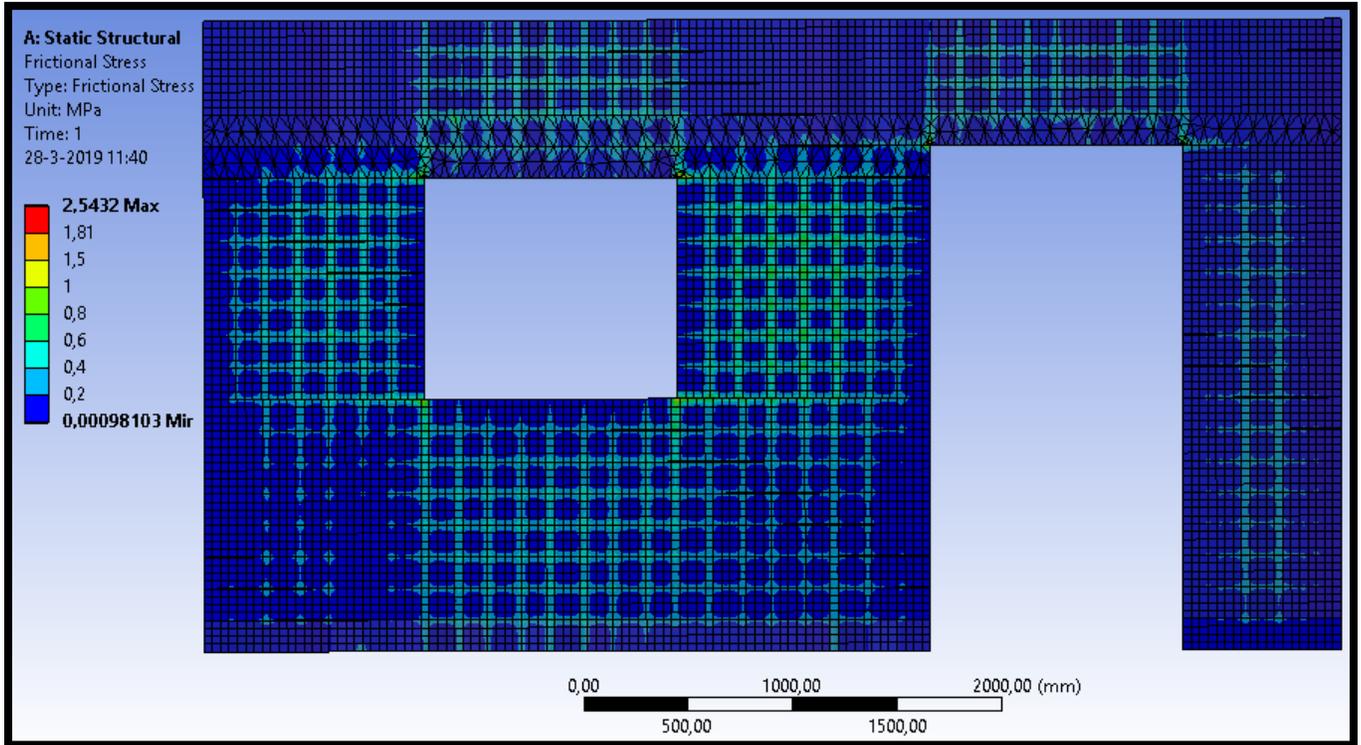


Figure 104 - Frictional stress in lamination layer, "door + window" variant, wind from right, incl. extra tensile connector

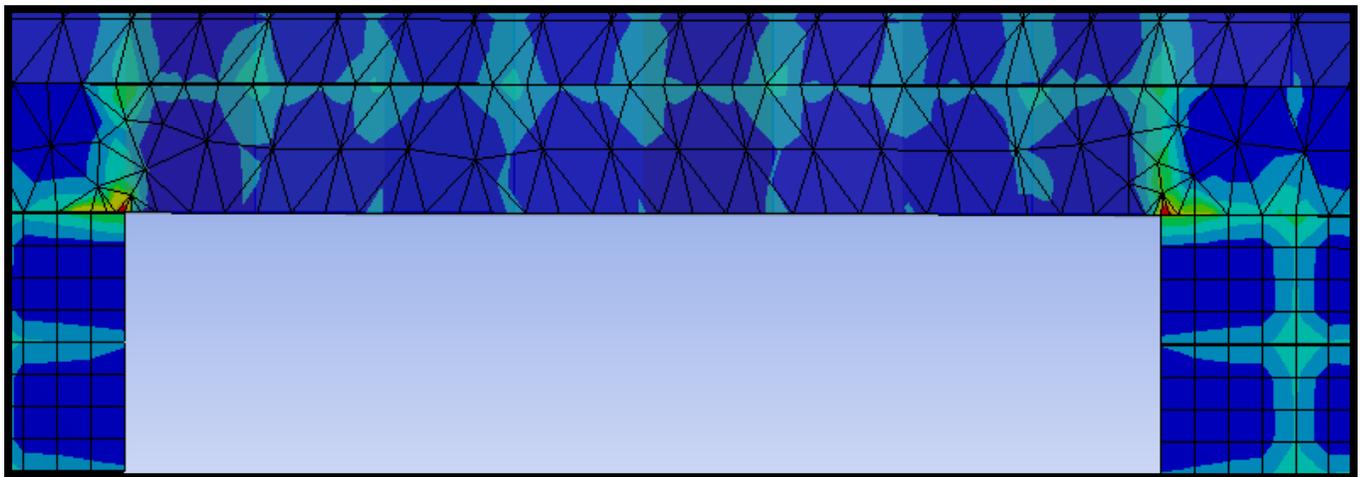


Figure 105 - Frictional stress detail at top of window, "door + window" variant, wind from right, incl. extra tensile connector

5. CLT-house of the Future: A structural overview of the variants

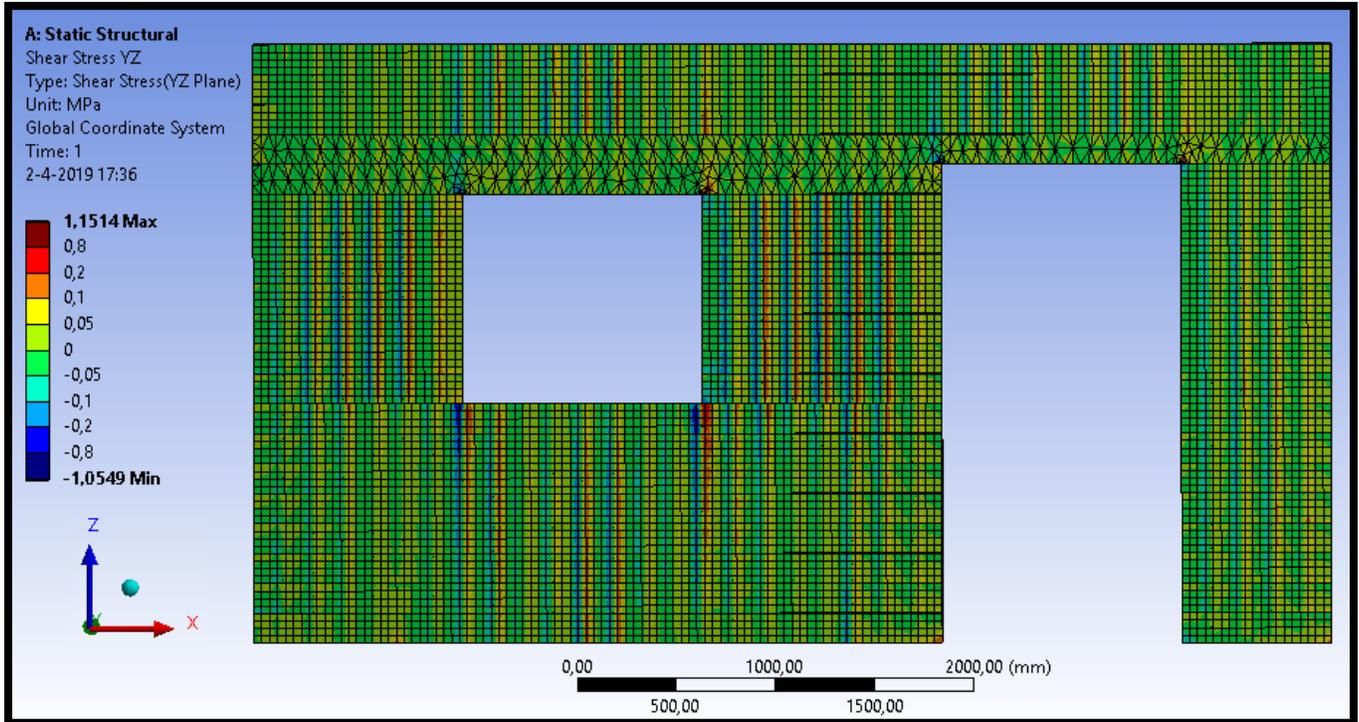


Figure 106 - Shear stress in y-direction on the z-face in the horizontal layer, "door + window" variant, wind from right, incl. extra tensile connector



Figure 107 - Shear stress in y-direction on the x-face in the vertical layer, "door + window" variant, wind from right, incl. extra tensile connector



In the investigated ANSYS models, the boundary conditions and contact area characteristics resemble a simplified version of the real situation. However, the fixed supports in reality are executed as tensile brackets and the contact areas consist of a glue layer which may not be fully bonded, meaning the stiffnesses are quantifiable and small deformations are permissible. ANSYS allows a factoring of the elastic slip of the contact areas, as well as input of an elastic support instead of the fixed supports. Considering tensile brackets (60mmx3mm, $E = 210.000 \text{ N/mm}^2$), the elastic support, divided over 2 boards per bracket has a stiffness of 2172 N/mm^2 . Inserting factors of 0,5 for the elastic slip and normal stiffness of the contact areas and the other supports results in bigger deformations (Figure 108), but the stress levels are comparable to the previous versions of the model, and therefore do not exceed the relevant strengths (Figure 109 till Figure 115). Due to the slip tolerance and applied stiffness factor for the contact areas, implying plastic deformation of the contact area, the frictional stress is much lower than in the previous models where restricted deformations led to an increased stress, sometimes greatly exceeding the determined strength of $1,81 \text{ N/mm}^2$.

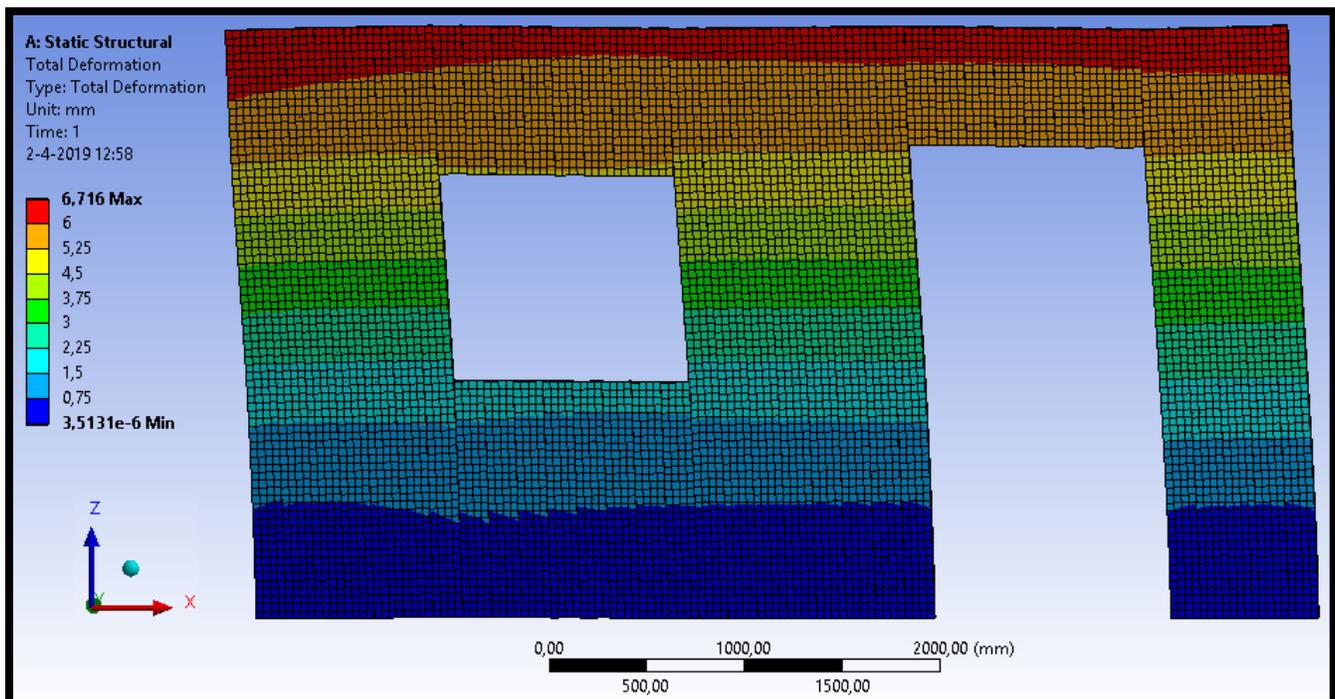


Figure 108 - Deformation, multiplied by 24, "door + window", wind from right, elastic supports and stiffness factors inserted

5. CLT-house of the Future: A structural overview of the variants

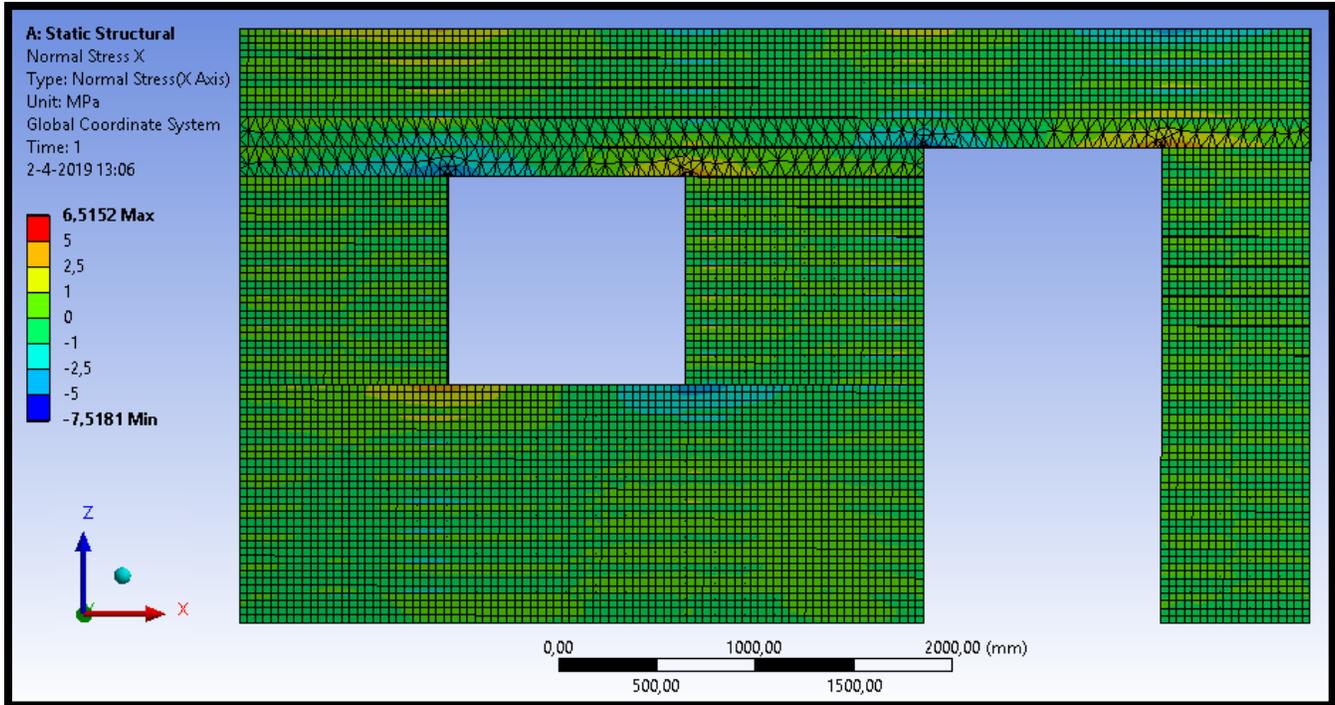


Figure 109 - Axial stress in x-direction in horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted

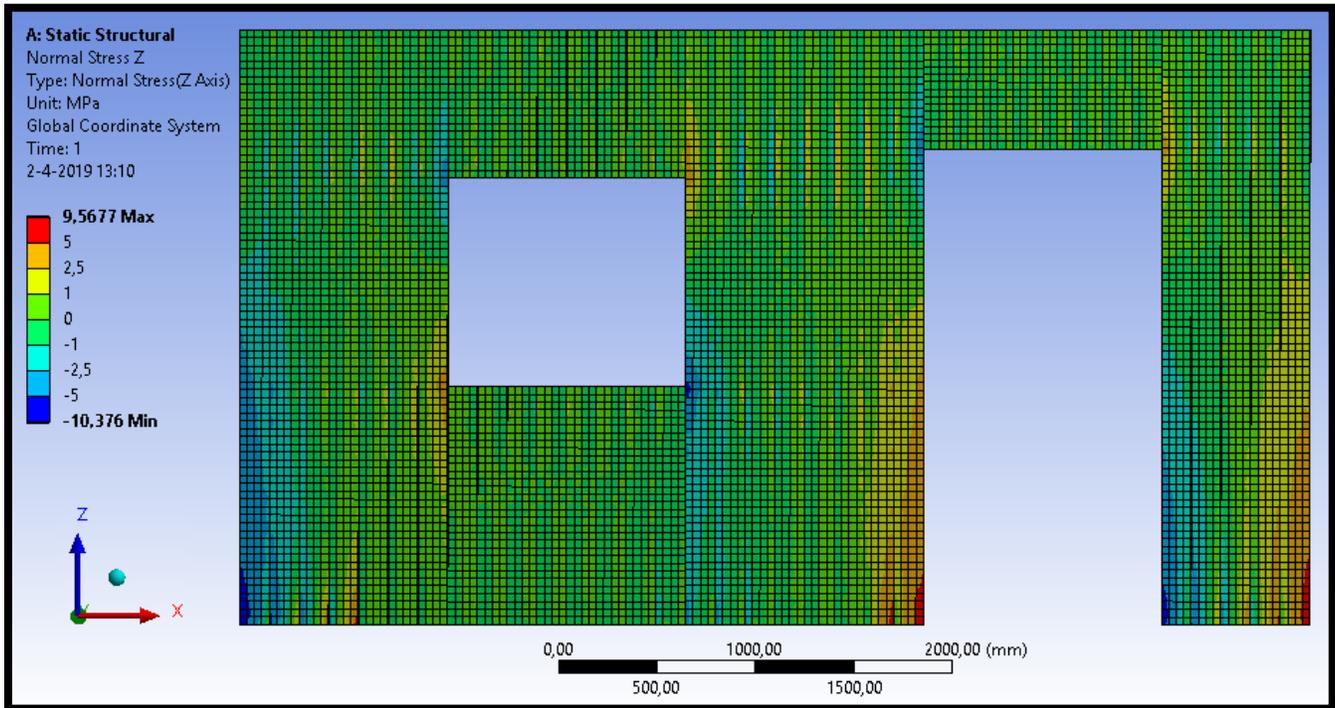


Figure 110 - Axial stress in z-direction in vertical layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted

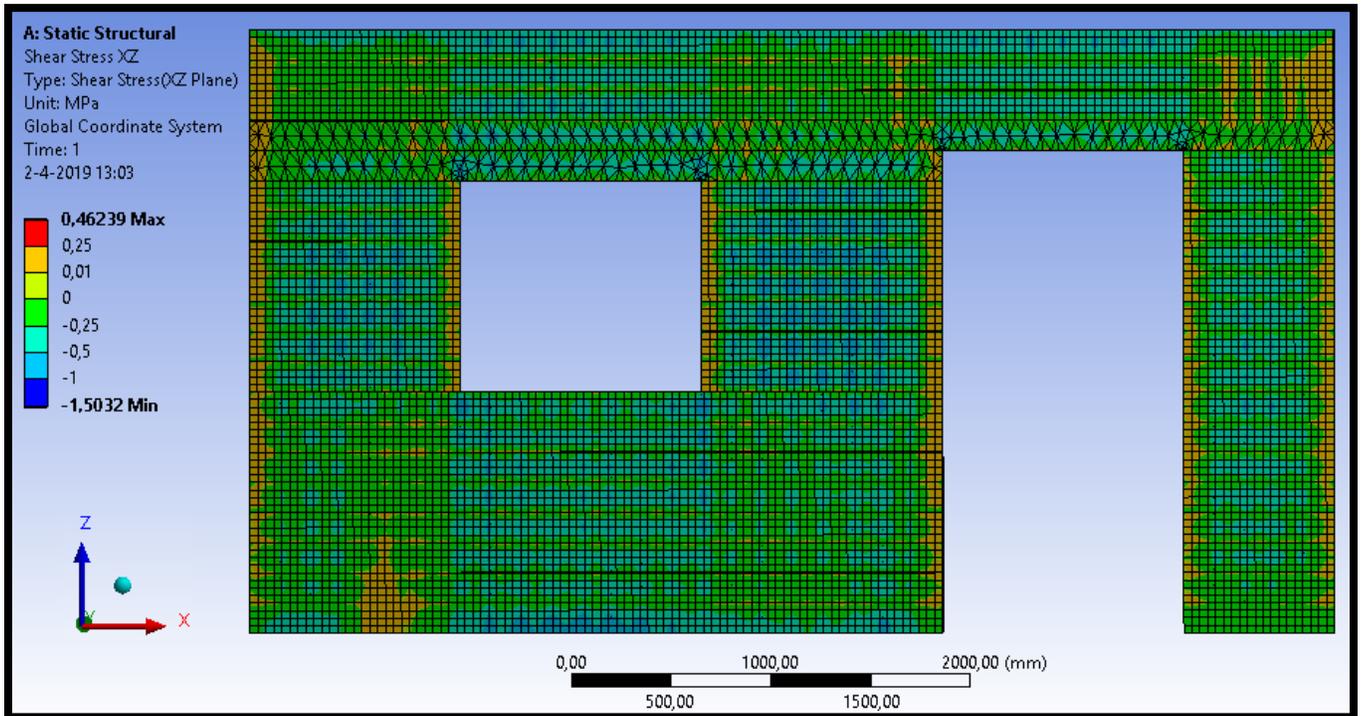


Figure 111 – Shear stress in x-direction on the z-face in horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted

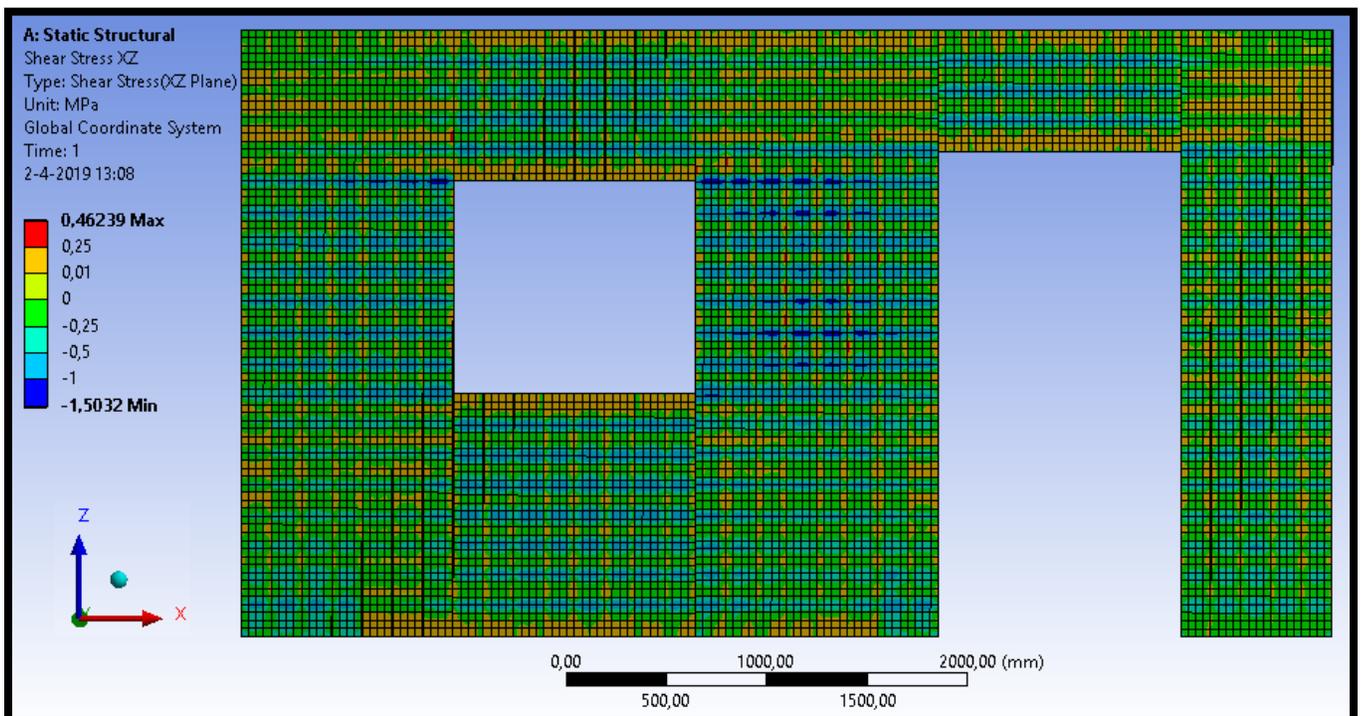


Figure 112 - Shear stress in x-direction on the z-face in vertical layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted

5. CLT-house of the Future: A structural overview of the variants

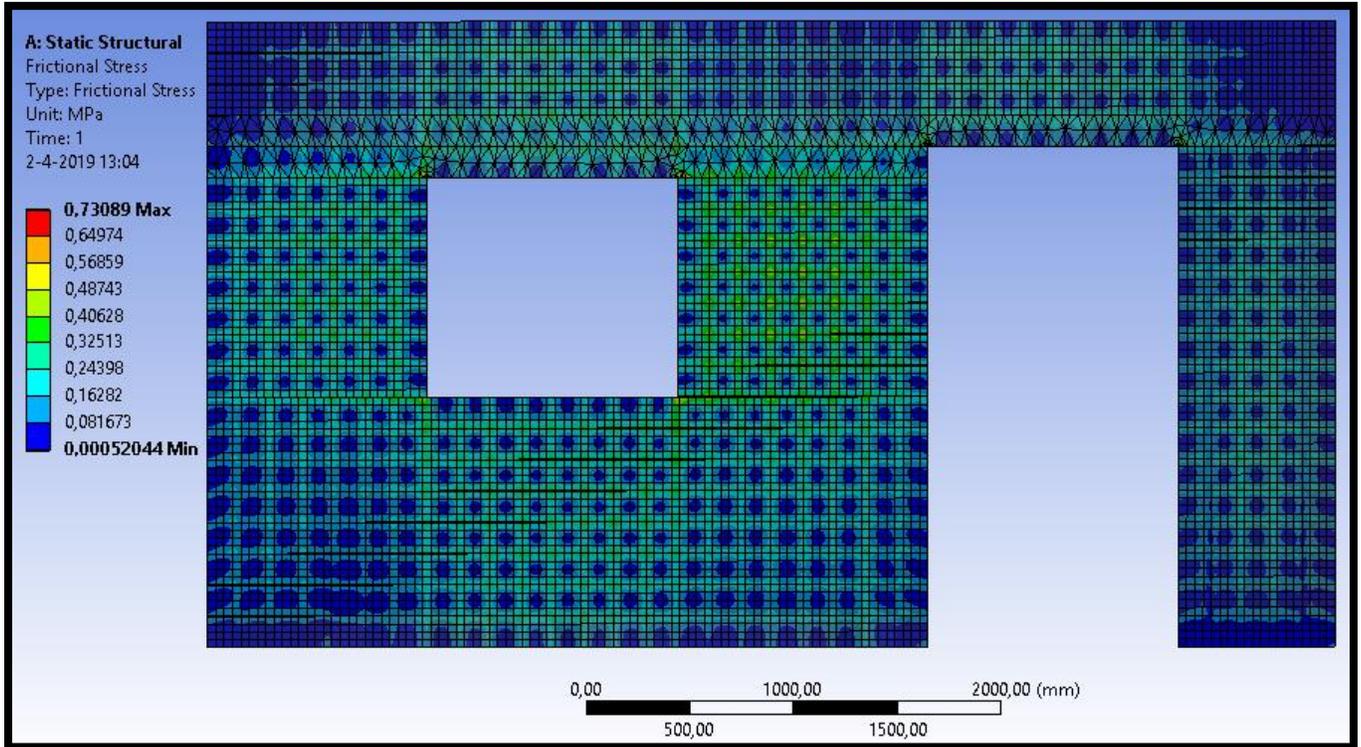


Figure 113 – frictional stress, "door + window" variant, wind from right, elastic supports and stiffness factors inserted

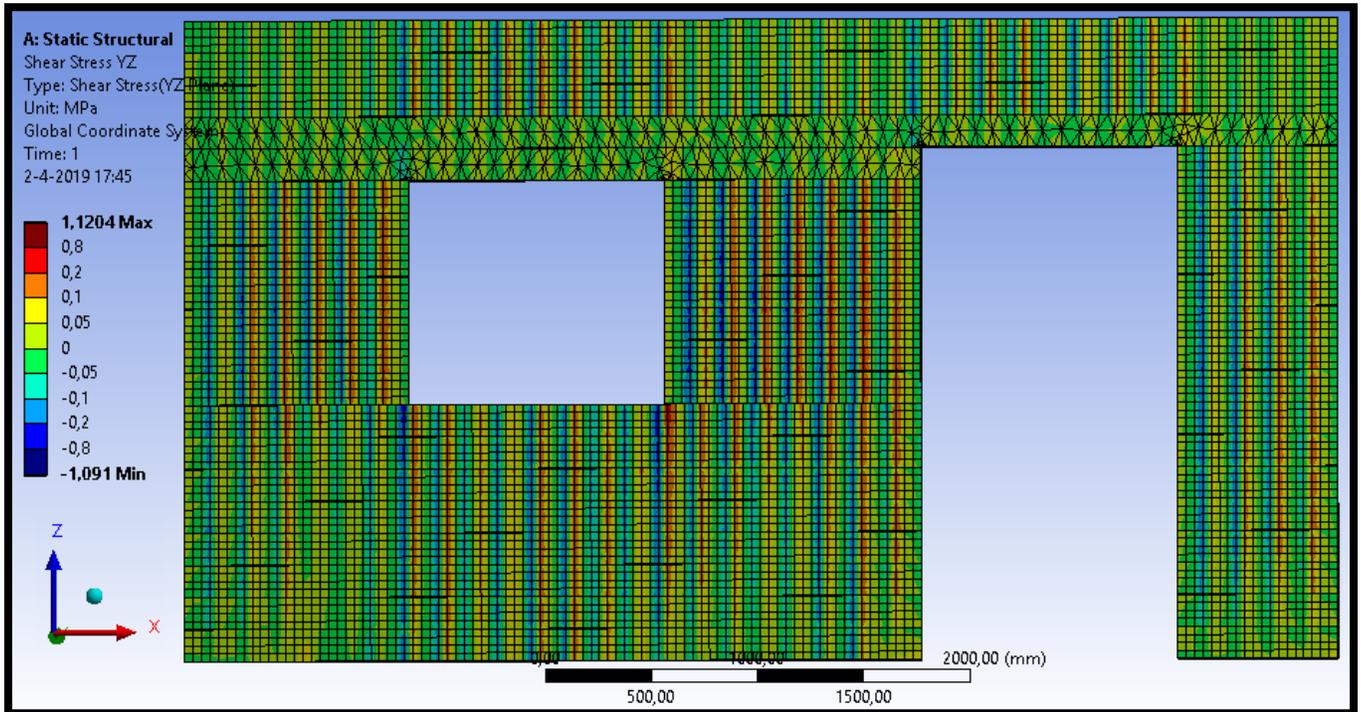


Figure 114 - Shear stress in y-direction on the z-face in the horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted

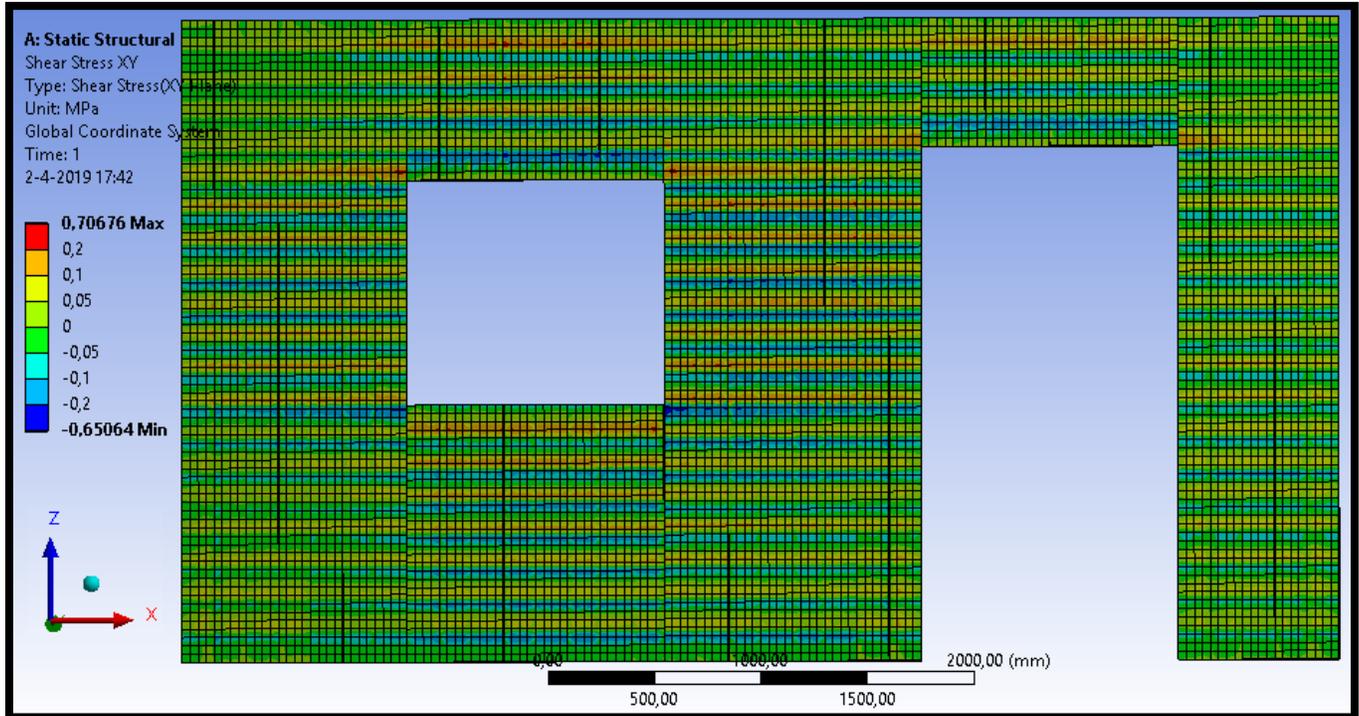


Figure 115 - Shear stress in y-direction on the x-face in the horizontal layer, "door + window" variant, wind from right, elastic supports and stiffness factors inserted

To conclude this analysis, the application of a "door + window" variant for the "ground floor"-facades, and a "single large window" variant for the 1st and 2nd floor facades have been investigated. Following the FEM results, these walls executed in a CLT thickness of 98 mm with a composition of 29-40-29 should be sufficient to resist the extreme wind load, with some local plastic deformation occurring in the extreme load cases.

	Max	Wall configurations				
		No holes	Hole in centre	Hole at bottom	Single door hole	Door + window
<i>Deformation [mm]</i>	10	1,03	6,429	3,772	2,859	6,716
<i>Tension [N/mm²]</i>	10,88	3,375	25,431	17,338	16,355	10,642
<i>Compression [N/mm²]</i>	13,76	3,375	30,541	17,23	10,053	10,376
<i>Shear [N/mm²]</i>	2,24	0,438	4,259	3,124	2,192	1,796
<i>Friction [N/mm²]</i>	1,81	0,588	6,527	2,712	3,839	2,744
<i>Rolling shear [N/mm²]</i>	0,8	0,549	4,101	1,935	1,602	1,353

Table 10 - FEM-results

5. CLT-house of the Future: A structural overview of the variants



The results of the FEM-analysis have been gathered in Table 10 - FEM-results. All of the exceedences of stresses have been caused by local stress peaks, which in reality are redistributed due to plastic deformations.

The last step in the stability calculation of the CLT terraced house is to link the hand-calculation with the results from the finite element analysis of the wall with the hole in the centre.

The axial stress distribution can easily be linked. In the FEM-results, at the connections between the “rods” and the “nodes” of the CLT element, a clear distinction in tensile and compressive stresses is visible, insinuating an in-plane moment in the connection. This corresponds exactly with the result from the hand-calculation where the nodes transfer in-plane moments from one rod to the next.

The shear stresses in the hand calculations were assumed constant over the height of the “rods”. This assumption was based on the fact that CLT consists of multiple boards with their own individual contribution to the overall resistance of the slab. The FEM results correspond with this assumption, since the stresses show small peaks within the boards, but each board has an equal contribution.

In the nodes, the comparison between hand- and FEM-calculation becomes more complex. Again in the hand calculation, the assumption was made that all of the lamination planes between the individual boards contribute equally to the in-plane resistance of the node. However, the FEM-results show that plastic deformation occurs in the CLT in the corner of the window hole due to local stress peaks (shear or frictional) and unequal division of the stresses. This means the hand-calculations should be done more conservatively and account for more required contact areas within the node than calculated with equal contribution. A safe assumption would be to at least disregard the contact areas between the boards in the node closest to the hole, shown with a red cross in Figure 116 - Disregarded contact areas in hand-calculation.

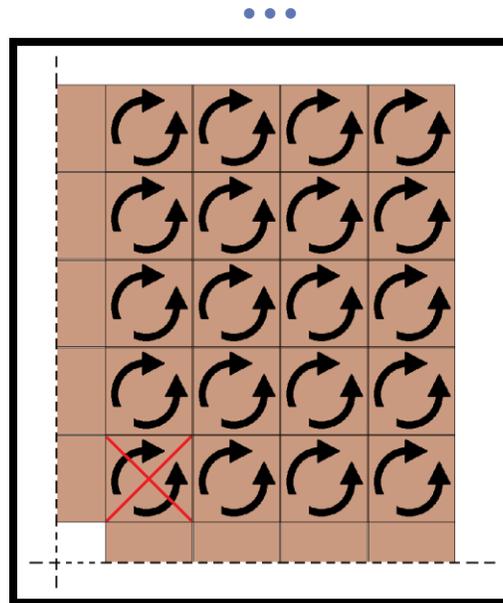


Figure 116 - Disregarded contact areas in hand-calculation

5.1.2. Design visualisation

Though application of CLT in building projects could still increased significantly across the globe, the robustness, good quality and production rates have led to CLT being an established building product in countries where timber structures take up a big share of the total building sector (e.g. Scandinavian countries, Austria). The following figures show the ideal CLT configuration for the application in a pilot project for Woodteq bv. This design is easily adapted to conform any similar framed structure with other dimensions.

Figure 117 shows a 3-dimensional overview of two attached low-rise, terraced CLT houses. To clarify the position of details, this overview is sectioned in two dimensions (Figure 118 and Figure 119). Suggested floor plans are supplied by the project initiator (Figure 118 and Figure 120), but due to assumptions in the calculations, inhabitants are free to choose any layout that is desired within the houses.

5. CLT-house of the Future: A structural overview of the variants

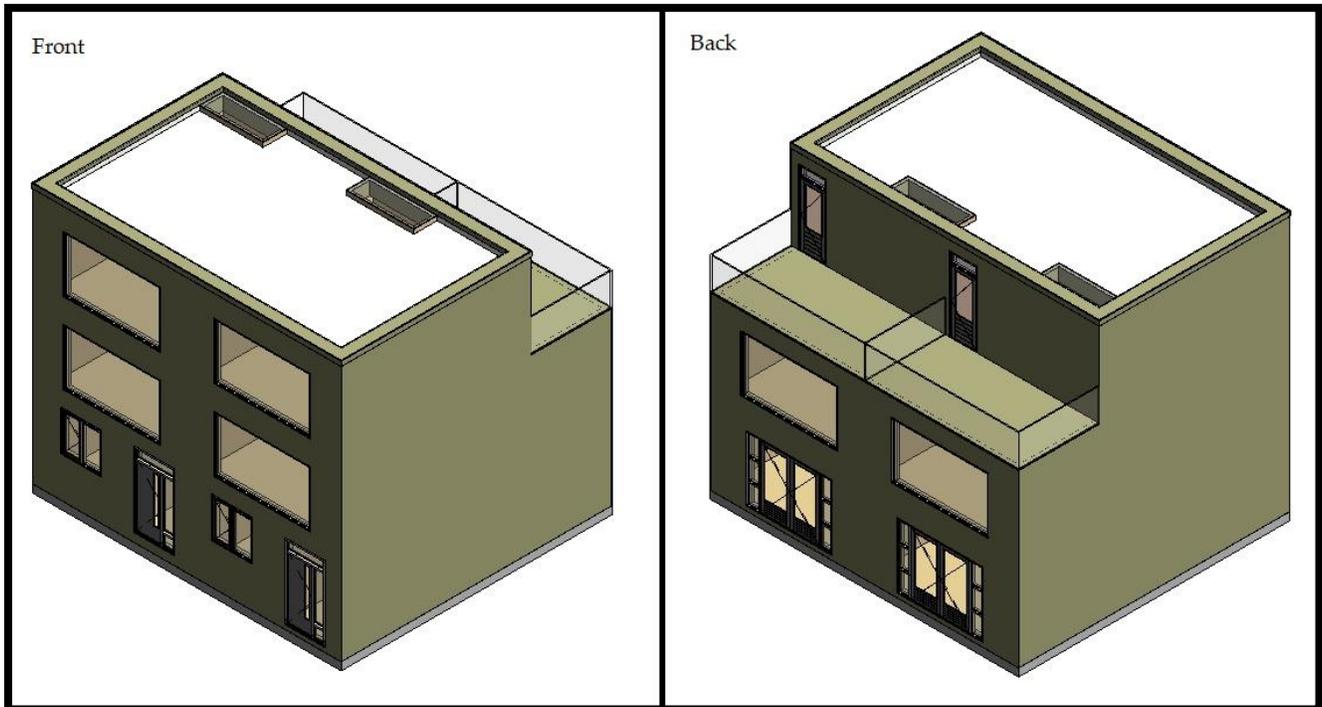


Figure 117 – 3d overview of two terraced houses

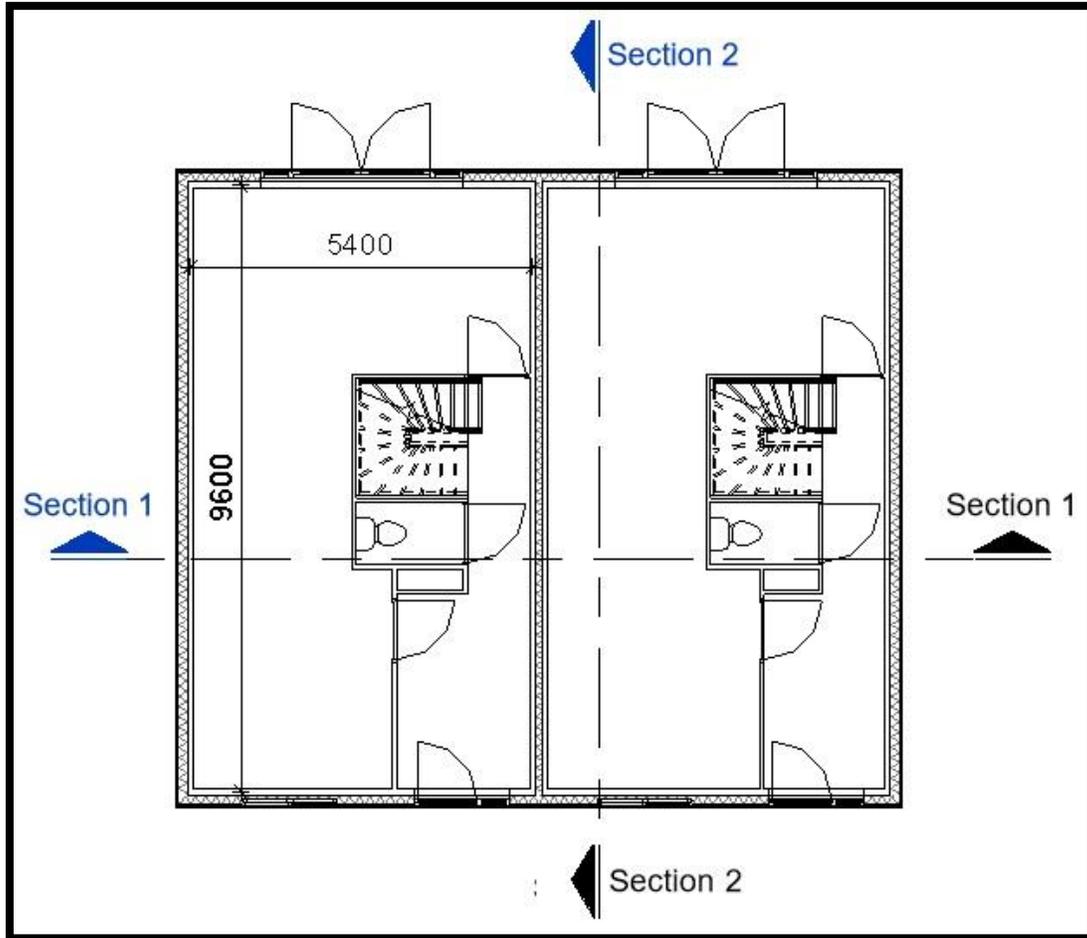


Figure 118 – Suggested area plan for ground floor, including section denotation

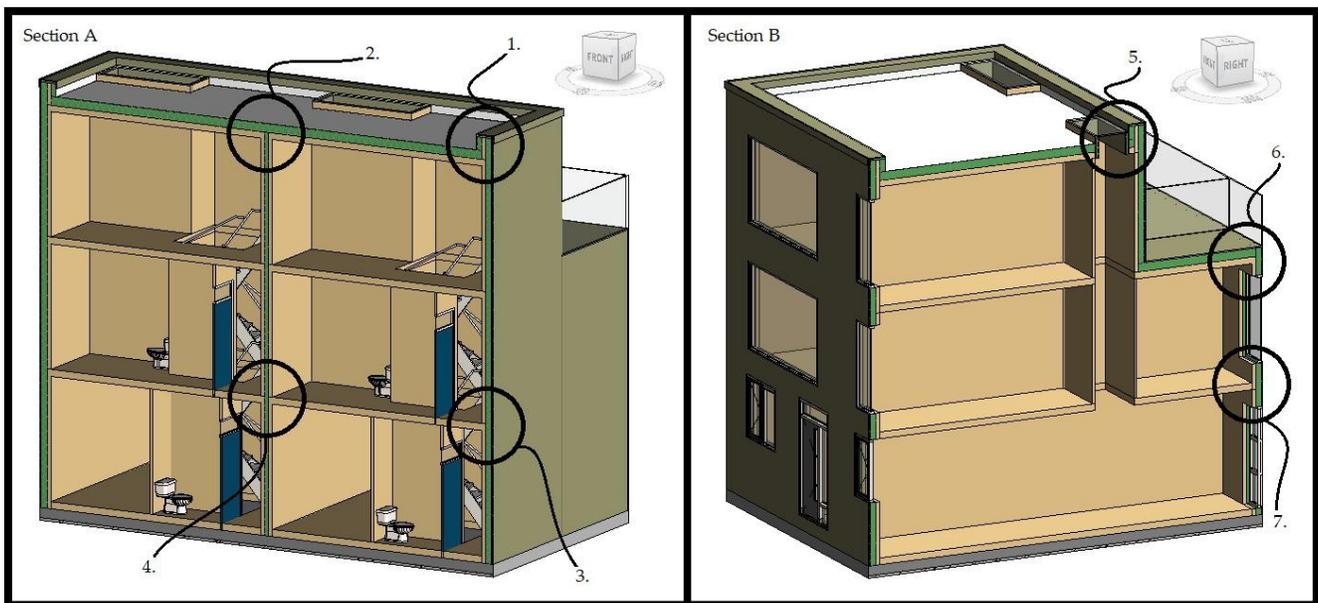


Figure 119 – 3d view of sectioned structures, including detailing denotation

5. CLT-house of the Future: A structural overview of the variants

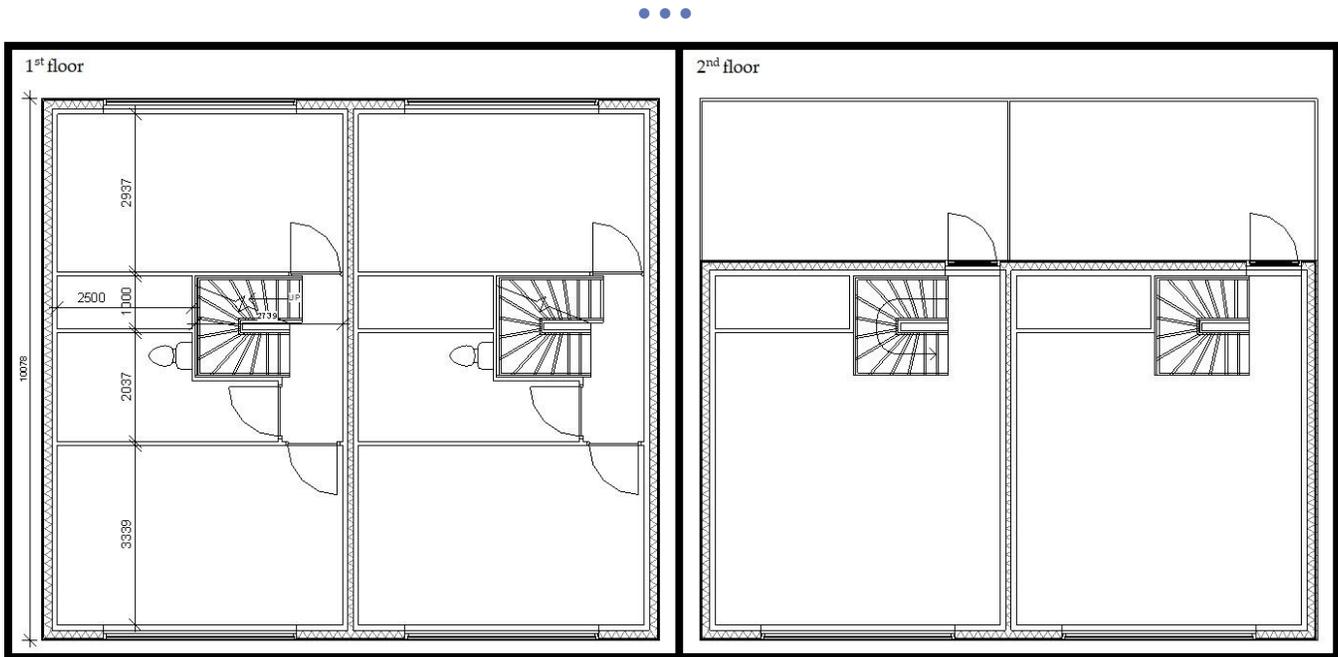


Figure 120 – Suggested area plan for 1st and 2nd floor.

Figure 121 till Figure 127 show the various details specified in the sectioned 3D view. The figures show the following dimensions for the structural elements. The slab compositions and governing requirements are determined in Appendix A:

- Roof slab: 145mm (29-29-29-29-29) CLT, long-term deflection governing
- Floor slab: 160mm (40-20-40-20-40) CLT, long-term deflection governing
- Load-bearing walls: 98mm (29-40-29) CLT, buckling

These walls are the house-separating walls in the structure. The floor- and roof slabs are rested on top of these walls to create the easiest vertical load transmission. This value is not the absolute minimum according to structural requirements, but for the purpose of other building criteria like acoustics, 98 mm thick CLT finds the best application.

- Stabilizing walls: 98mm (29-40-29) CLT, shear

These walls are the front- and back facades of the houses. Their only function is to provide stability for the system in case of a horizontal load perpendicular to the depth of the house, e.g. a wind load on the side façade.



The screwed connections are executed with fasteners, and require the following screw types and spacing dimensions per connection:

- Parapet to roof: d=8,l=180 at 300mm spacing Figure 121
- Roof to load bearing wall: d=10,l=240 at 300mm spacing Figure 121
- L.B. wall to intermediate floor: d=8,l=180 at 300mm spacing Figure 123
- Intermediate floor to L.B. wall: d=10,l=240 at 300mm spacing Figure 123
- Parapet to front-/back façade: d=8,l=180 at 125mm spacing Figure 125
- Front-/back façade to roof: d=10,l=180 at 250mm spacing Figure 126
- Between different front-/back façade slabs: d=8,l=180 at 125mm spacing Figure 127
- Front-/back façade to intermediate floors: d=10,l=180 at 250mm spacing Figure 127

This results in a maximum total number of screws required of:

- d=8,l=180 424 screws (5*32 over 9,6m depth, 6*44 over 5,4m width)
- d=10,l=180 192 screws (6*32 over 9,6m depth)
- d=10,l=240 132 screws (6*22 over 5,4m width)

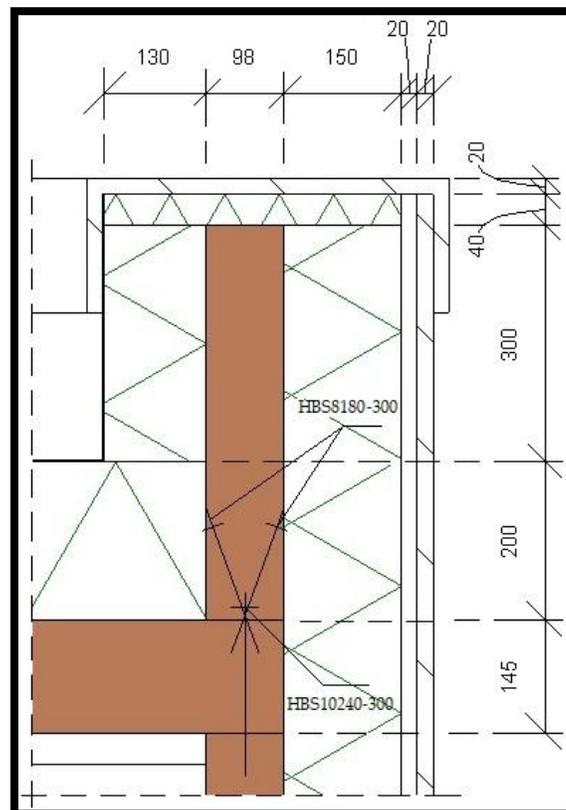


Figure 121 - Detail 1 in Figure 119

5. CLT-house of the Future: A structural overview of the variants

...

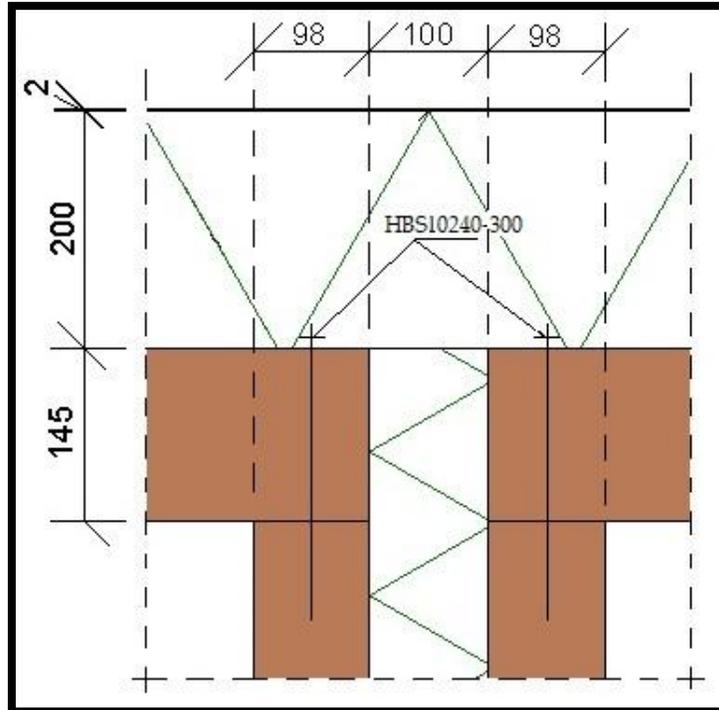


Figure 122 - Detail 2 in Figure 119

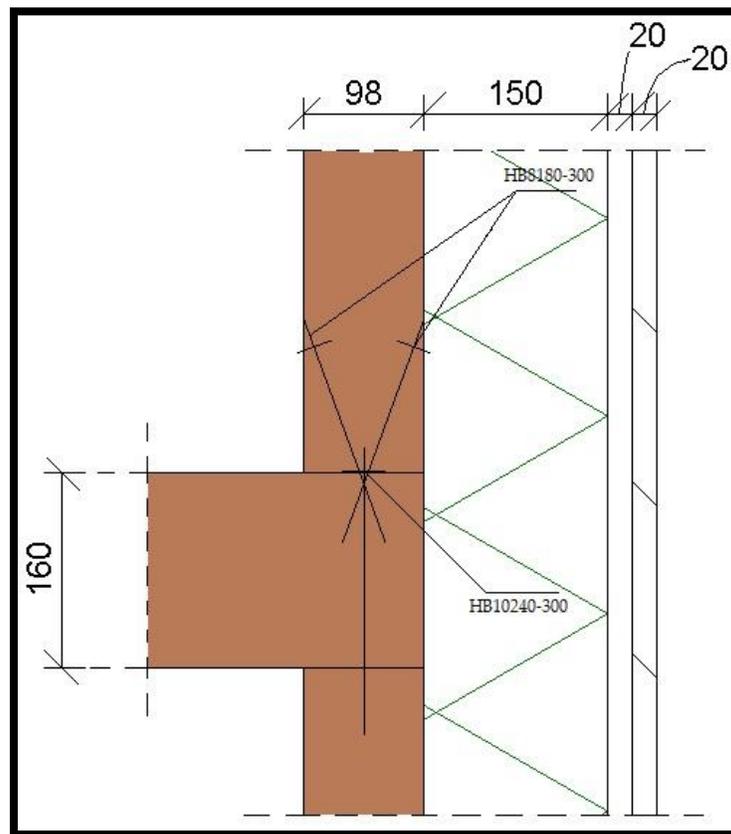


Figure 123 - Detail 3 in Figure 119

...

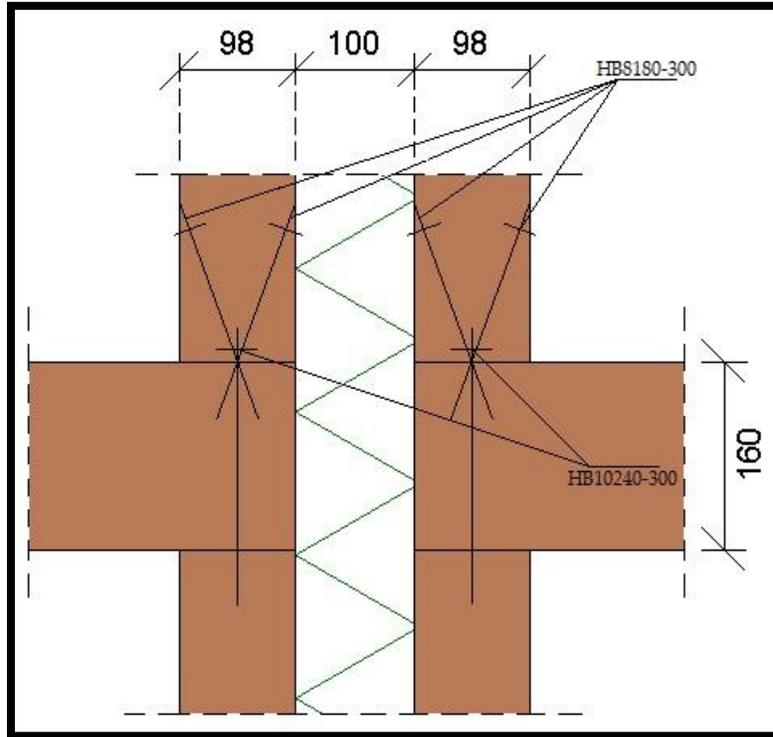


Figure 124 - Detail 4 in Figure 119

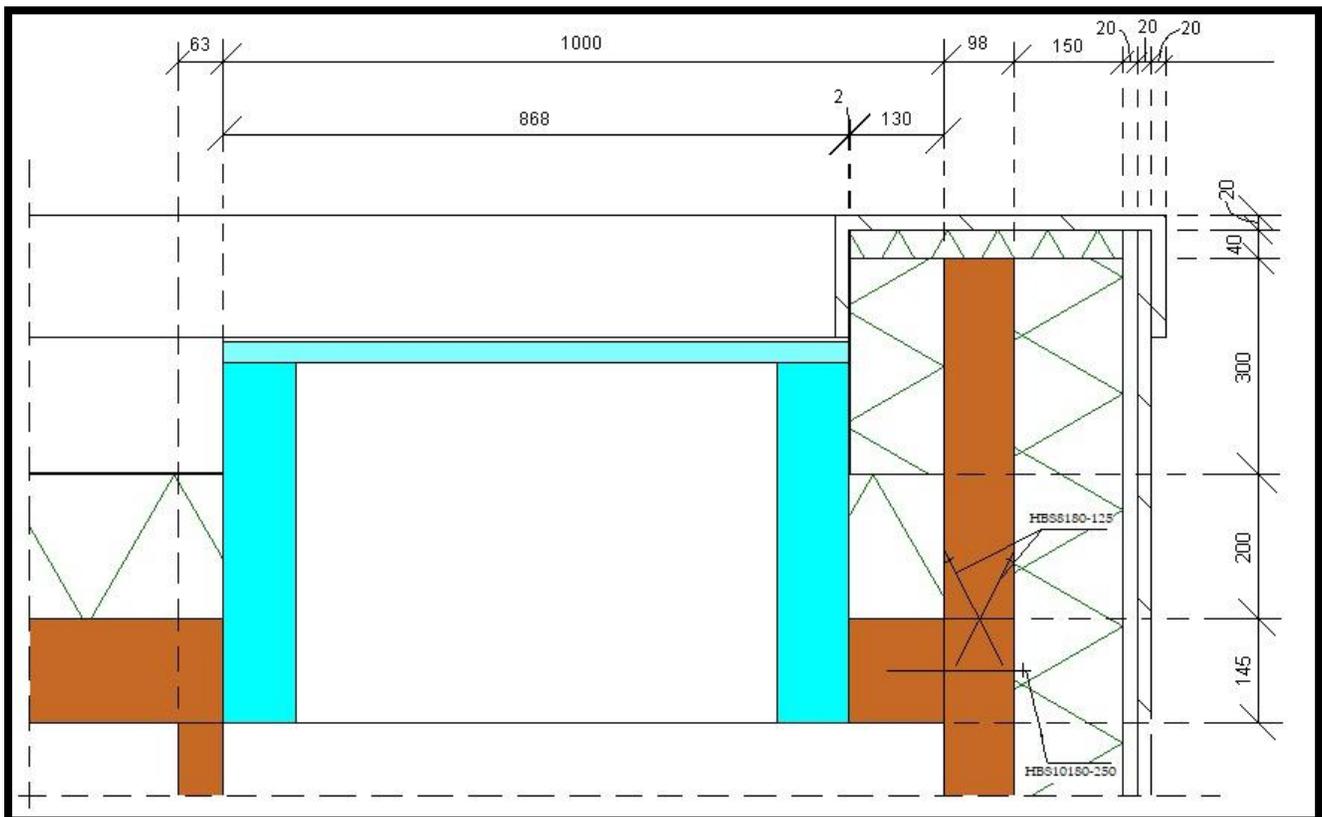


Figure 125 - Detail 5 in Figure 119

5. CLT-house of the Future: A structural overview of the variants

...

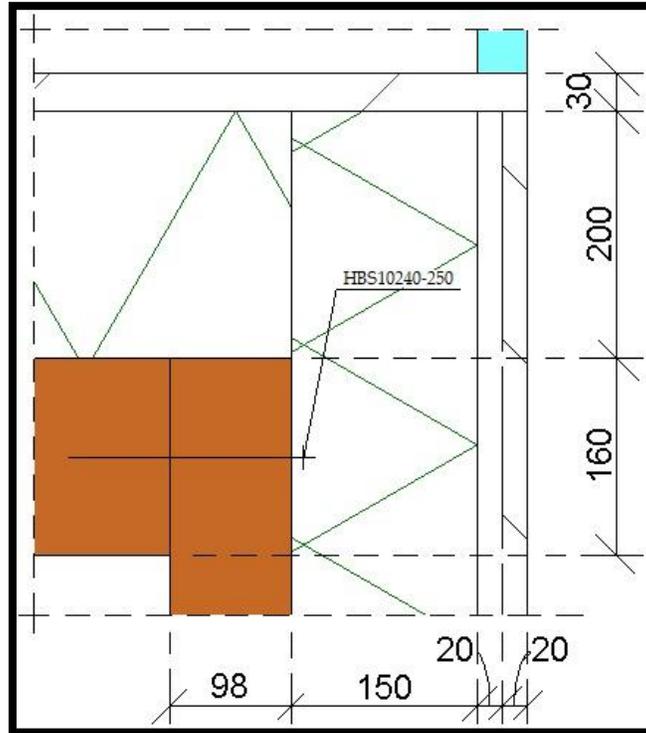


Figure 126 – Detail 6 in Figure 119

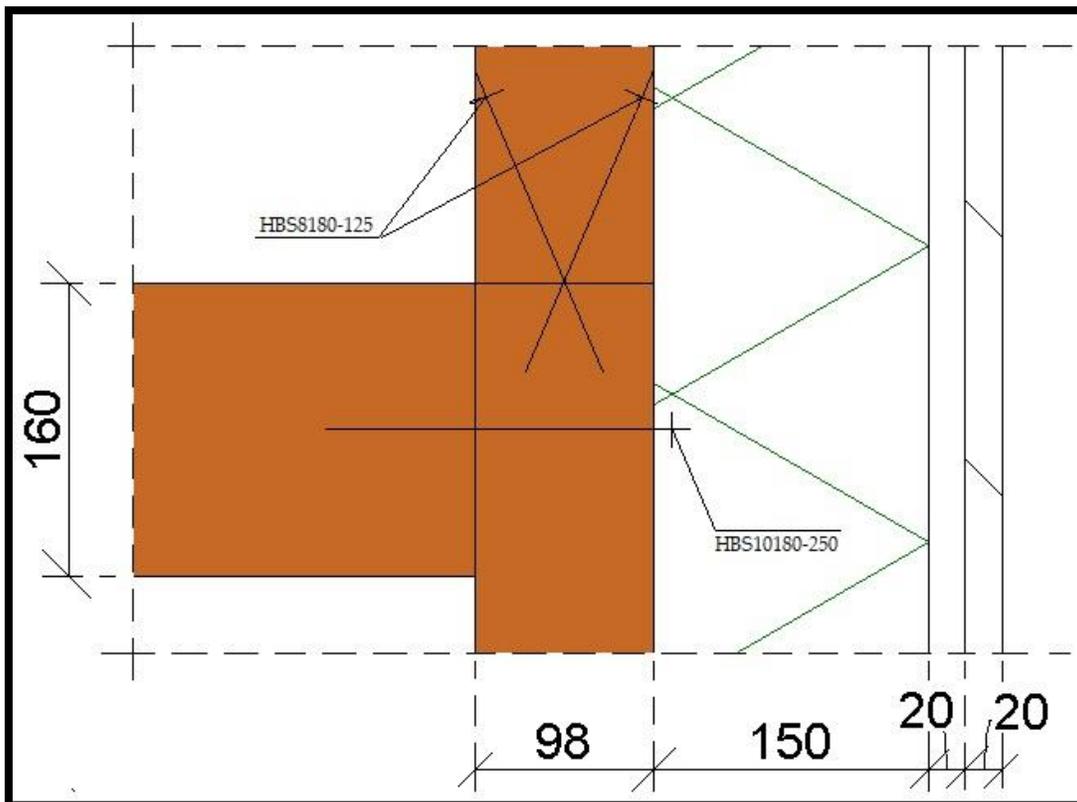


Figure 127 – Detail 7 in Figure 119



Appendix A mentions various additional measures required to fulfil the building physics requirements. A visual overview of these measures is shown in Figure 128.

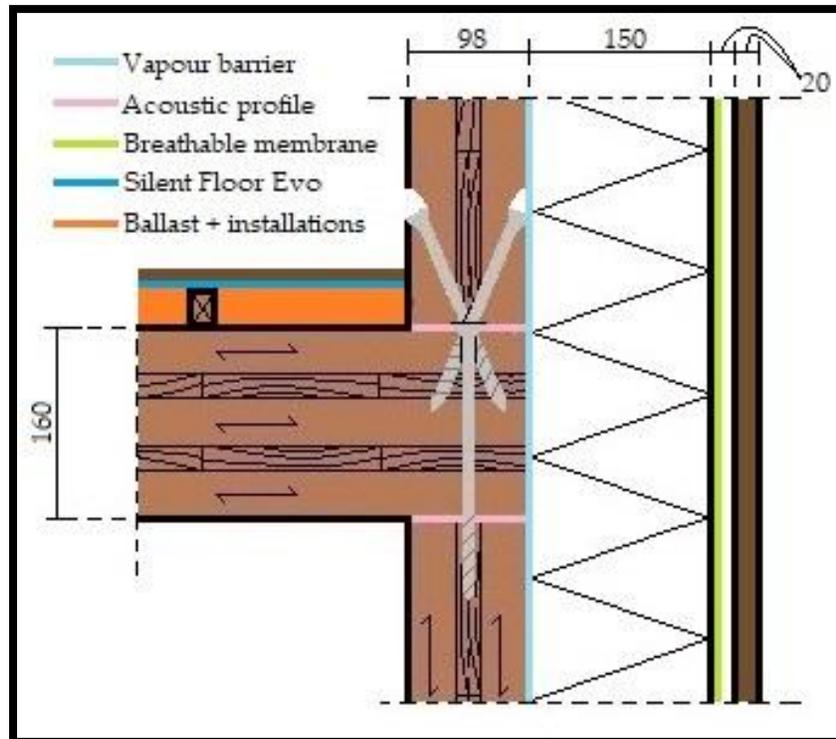


Figure 128 - Additional measures in intermediate floor/load-bearing wall connection

5.2. Variant: Hollow core CLT

As a variant to the standard CLT design, the principle is applied of placing as much material of a cross section, as far away from the neutral axis as possible. A sandwich panel is created with relatively stronger plating materials and weaker connectors in between (in terms of the contribution to the moment of inertia) keeping the plates (flanges) together; hollow core CLT (HCCLT). This means cavities are formed at the neutral axis in the cross section, resulting in a relatively larger moment of inertia than for a solid slab with equal material usage. The calculations for the HCCLT elements are performed in Appendix B.

The elements can be produced by either altering the regular CLT production line, or by creating the slabs out of already available engineered products like GLT webs and CLT flanges. Altering the CLT production line is a more efficient and time saving way of creating HCCLT (van Aken, 2017). This however leads to an issue when compressing the slabs in order to create sufficient strength in the lamination between the timber layers in the flanges. During the production, the flanges need to be

5. CLT-house of the Future: A structural overview of the variants

• • •

temporarily supported by a material with similar characteristics as the webs to achieve a correct structural element over the full dimensions of the slab.

The composition possibilities with regard to the dimensions result in hollow core CLT being only applicable for floor- and roof slabs (Figure 129). The wall slabs in the regular CLT design are so slender that applying hollow cores will not create an improved solution whilst still fulfilling all structural- and physical requirements. The hollow cores provide a good opportunity to incorporate any needed additions within the floor- and roof slabs. This includes installations, ballast and other measures for acoustic purposes, fire retardant materials (for the floor slabs) or insulation material (for roof slabs).

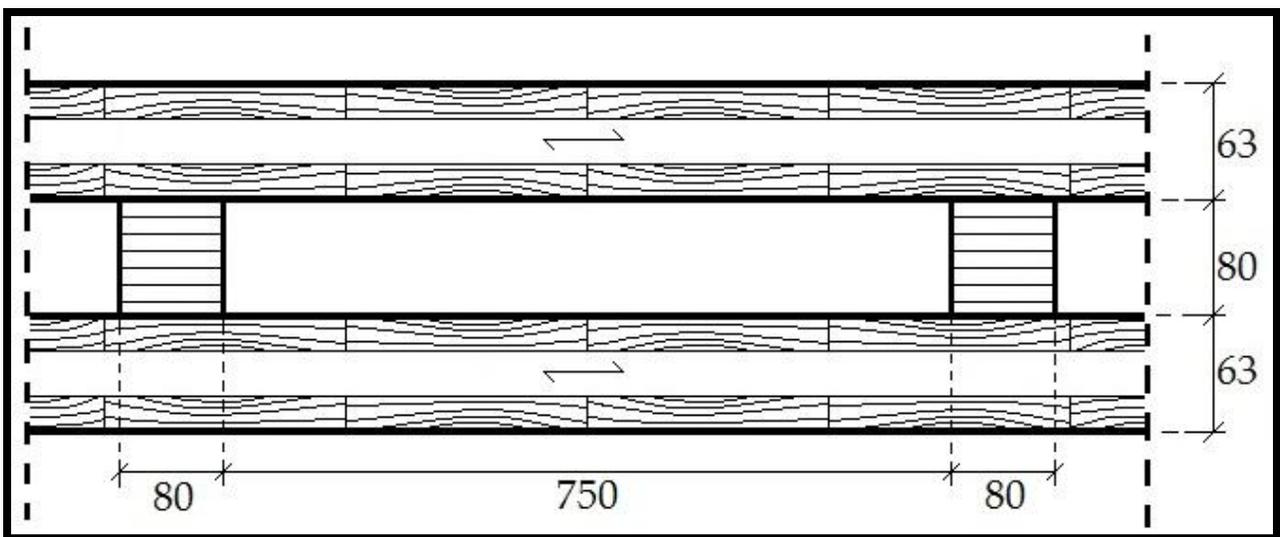


Figure 129 – Hollow core CLT design for floor- and roof slabs

Table C1 gives a net timber area of the cross section per meter width of $1,345 \cdot 10^5 \text{ mm}^2$. This means the application of HCCLT in the roofs and floors imply a building material saving of respectively 7,25% and 16%. Appendix C shows this accumulates to a total material saving for the structure of approximately 7,5%. Given the relatively low unity checks (maximum 0,41 for the deflection), the cross-section could be optimized further to find the economically most favourable option, by e.g. applying thinner layers of multiplex or laminated veneer lumber in the flanges. This optimization will not be a part of this research.

The unity check for fire safety of the hollow core CLT is favourable over this unity check for the bending strength. This performance is explicable due to the division of the strong parts of the cross-section. A fire in one compartment of the building only influences the cross-section on that side of the



engineered slab, while the other side of the slab remains fully functional. This structural benefit of HCCLT is accompanied by a functional drawback. If any additional measures and installations are placed inside the hollow cores, strength reduction of the bottom flange over time under fire conditions will eventually lead to the content of the cores falling down if they are not secured to the top flange of the slab.

Considering an application of HCCLT for the roof slabs, the hollow cores provide additional space for thermal insulation, lowering the required thickness of the insulation layer outside the structural slab and possibly lowering the total roof thickness. The thermal resistance of the HCCLT roof consists of 3 layers; CLT, GLT+insulation and insulation, and assuming equal thermal resistance for CLT and GLT is determined as:

$$\lambda_{combi} = \lambda_{ins} \frac{b_{ins}}{b_{ins} + b_{GLT}} + \lambda_{CLT} \frac{b_{GLT}}{b_{ins} + b_{GLT}} = 0,038 \cdot \frac{750}{750 + 160} + 0,13 \cdot \frac{160}{750 + 160} = 0,054$$

$$R_{\lambda,roof} = \frac{t_{CLT}}{\lambda_{CLT}} + \frac{t_{combi}}{\lambda_{combi}} + \frac{t_{ins}}{\lambda_{ins}} \geq 6,0$$

$$t_{ins} = \lambda_{ins} \left(R_{\lambda,roof} - \frac{t_{CLT}}{\lambda_{CLT}} - \frac{t_{combi}}{\lambda_{combi}} \right) \geq 0,038 \cdot \left(6,0 - \frac{2 \cdot 0,063}{0,13} - \frac{0,08}{0,054} \right) = 0,135 \text{ mm}$$

So the structural slab including the insulation has a thickness of 341 mm.

Terms like circularity, bio-economy and recyclability have shown up repeatedly throughout this research. They are trends in the building industry, and new products like HCCLT need to fit well into these trends for a bigger chance to eventually see enough production and application. The circularity of HCCLT is dependent on the configuration of the engineered slab. Reapplication of a used slab might require different holes for e.g. stairs, possibly exposing the hollow cores and influencing the efficacy of installations and other measures assimilated in the slab.

Considering HCCLT is made out of laminated timber, the biggest share of the materials is bio-based. Similar to conventional CLT, applicable adhesives up till now are not bio-based. Research shows much potential for bio-based replacements. Regardless of bio-based innovations, currently used glues are required to meet strict standards on sustainability for their production, which decreases the negative effect on the environment.

Uncertainty with regard to the recyclability and fit in the bio-economy is related to the assembly of the CLT and GLT to create HCCLT. The production process is time consuming due to the different production steps. When opting to instead use a regular CLT pressing process to directly laminate

5. CLT-house of the Future: A structural overview of the variants



boards into the desired composition leads to insufficient stress distributions in several parts of the HCCLT (van Aken, 2017). Mechanical connections are a possible solution for this problem, but steel fasteners negatively influence the fit in the bio-based economy, and timber dowelled connections are laborious and too complex to disassemble without destruction. In terms of the manufacturability, a better solution needs to be developed to produce the HCCLT in a more economically friendly and less laborious way. The connection between the webs and flanges is also the critical factor in the recyclability. Mechanical connections, if not executed with timber dowels, need to be disassembled before the timber parts can be processed for recycling purposes.

The durability of HCCLT is arranged in the same way as any timber structure; NEN-EN 1995 gives clear indications of factoring in the durability of structural members in certain service classes and for certain load-durations, through the application of k_{mod} .

The influence of HCCLT on the indoor climate, when the slab is considered to be a part of the building envelope, is more complex than for regular CLT. The volume of timber that is exposed to the indoor environment of the building will contribute to the indoor climate quality by absorption of moisture. The equivalent thickness of HCCLT was previously determined to be lower than the thickness of CLT, and therefore the volume is lower as well. Additionally, due to the application of insulation in the hollow core of the roof slab, a vapour barrier is required on the warm side of this insulation layer (Figure 130). This vapour barrier disallows the top half of the HCCLT slab to assist in the indoor climate control, and additionally provides a durability issue due to the exposure to moisture of the top half of the slab.

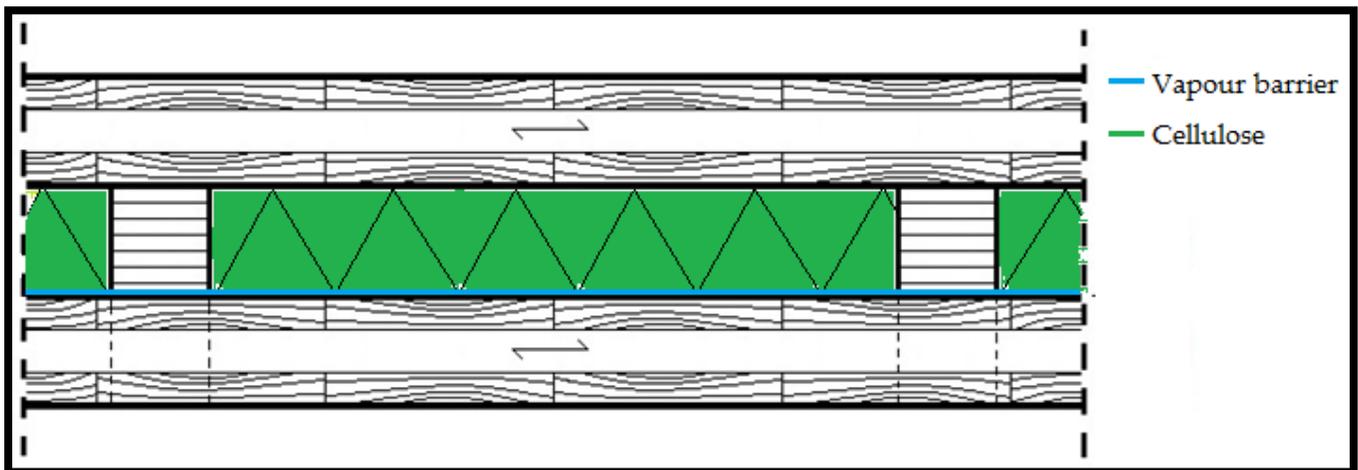


Figure 130 - HCCLT roof slab incl. insulation and vapour barrier



A full comparison on the basis of multiple judgement criteria is required between HCCLT and conventional CLT to make a substantiated decision on which variant currently provides a better application in low-rise housing.



6. Results

6.1. Multi criteria analysis

The designs described in Ch.5 will be compared to each other through a multi criteria analysis (MCA). The criteria are a result of the various building requirements described in Ch.4, in which the variants may show different levels of performance. As mentioned in Ch.2, the first step is to set the weight factors for the criteria.

6.1.1. Weight factors

The weight factors need to be determined as objectively as possible. Audit sampling would ideally be applied to find out which criteria are generally considered as most important in the decision-making amongst house-buyers. Such a sampling needs to be big enough to assure the average public opinion is well represented. In the scope of this research creating and executing a questionnaire with enough participants is too extensive. Alternatively, incorporating building owner's, project initiator's and contractor's knowledge would lead to weight factors which correctly represent the importance of criteria in the design, construction and exploitation phases of a building. In this thesis however, weight factors from 1 to 5 are attached to the criteria (1 being least important and 5 being most important) based on logic and enough substantiation. The criteria and their accompanying weight factors on which the variants are judged are:

1. Material efficiency, weight factor: 4

Amount of material necessary to design according to the structural requirements. The weight factor is 4 due to the effect it has on the total costs of the house, and therefore on the price for buyers. Additionally, higher material consumption to create a comparable structure automatically implies a higher environmental cost due to production, transport, etc. This criterion can be expressed in weight or in volume, but just the consistency in this expression is of importance to determine correct relative scores for the different variants.

2. Fire safety, weight factor: 3

The assuring of fire safety within the structural design is done through building requirements, but one variant may be more efficient than another. Weight factor 3 implies medium importance, since the actual safety needs to be guaranteed for any design, this criterion scores how well the variant secures this fire safety.



3. Energy efficiency in exploitation, weight factor: 5

This criterion describes the influence of the structural members on the energy efficiency of the building while it is in use. This includes energy consumption for heating. It has a big impact on the overall ecological footprint of the structure. Additionally, a score for energy efficiency is attached to structures, resulting in it having a big influence on the interest of buyers. Notable are the requirements stated by the Building Decree with regard to insulation; this criterion describes how efficient the variant assures the required isolative value, and is therefore in importance comparable to the fire safety criterion.

4. Circularity, weight factor: 3

The ability of the full reapplication of materials that fit into the “circular economy”. This includes members from disassembled structures or total structures, but excludes products made from parts of old, broken down members (this is considered as recycling (point 6)). This criterion can significantly influence the environmental cost of specific members, since it cuts down the extraction from nature and energy use for production of new elements. The generation of energy from structural materials is not incorporated in this criterion, since it requires breakdown of the members, and is taken into account in criterion 6: recyclability.

5. Bio-based, weight factor: 4

The extent to which the structure fits into the “bio economy”; a biological renewable resource and possible biodegradability of the building materials. This criterion influences the total environmental impact of the structure, and mitigates depletion of non-renewable natural resources. Additionally, bio-based is trendy and could be the deciding factor for buyers to opt for a house.

6. Recyclability, weight factor: 2

Recyclability is considered as a less energy-efficient version of the circularity of a structural element. The distinction in assumption is important, since the two terms overlap and are often used as synonyms. In this thesis, recycling is considered more often applicable, but requires energy consumption to convert old members into smaller particles which are applied in new products, and is therefore less efficient than totally circular use of materials.

6. Results



Recycling is also assumed to include the possibility to use old products to generate energy, in which the process needs to be analysed to determine by-products and emissions. Also considered in this criterion is the phenomenon of upcycling, where the structural members can be broken down into pure materials which may decrease certain scarcities.

7. Durability, weight factor: 4

This criterion covers both the structural safety over time (degradation of material characteristics) and the appearance of the structure over time. Structural safety over time needs to be guaranteed by meeting the requirements from the Eurocode, but the feeling of safety for inhabitants is equally as important. The appearance of a structure influences a buyers decision on his investment in a house. The buyer prefers an increase in house price, rather than a decrease due to a visual deterioration.

8. Indoor climate, weight factor: 2

This criterion relates to criterion 3: the energy efficiency in exploitation, as it is affected by the temperature regulation within the structure. However, the main focus of this criterion is the ventilation and dehumidification of the building. This is considered more as a favourable addition to the structure rather than a requirement, and it is less noticeable than other criteria when inspecting a house as a potential buyer. The breathability (and ability to absorb moisture) of the materials in sight contributes significantly to the design of a naturally neutralizing healthy environment in the building. Poor ventilation could always be adjusted through mechanical installations.

9. Acoustics, weight factor: 3

The acoustic performance of the structure is guaranteed to meet a minimum value due to requirements in the Building Decree. This criterion relates to the method in which acoustic measures are applied within the structure and how easily the acoustic performance can be upgraded through the application of additional materials. Acoustic comfort is considered as extremely important by building occupants, and therefore by building owners. Allowing the integration of measures within structural components is in this sense favourable over the application on top of the structure. During construction, this criterion is not so noticeable, but the effects of good acoustic design becomes clear during exploitation of the building.



10. Aesthetics, weight factor: 2

The aesthetics of a structure is possibly the most important criterion in the decision-making for buying a house. This does not relate to the psychological effects of timber in sight within a structure, since these are only noticeable after spending considerable time in the building. This criterion relates to a buyer's first opinion when looking at a house.

11. Manufacturability, weight factor: 5

Within the decision-making for structural members, the manufacturability is essential. A design team, including structural engineers, could create a structurally, economically and visually optimal structure, but if the structure or elements cannot be produced, the value of the design is limited to what cannot be built rather than what can be built. The weight factor of this criterion implies the importance of creating realizable structural members that can be assembled to create total structures in the design phase. This is only achieved by incorporating practical knowledge that is often only present at building companies from experiences in past projects.

12. Robustness, weight factor: 2

In the decision-making of house-buyers, robustness and the "feeling" of a solid structure is desired. Due to this reason, laity in the Netherlands often think concrete builds "better" structures than timber. This assumption has been dismissed in Ch.3, but the preference for a sturdy, robust structure stays.

6.1.2. Scoring

The result of this MCA is a full comparison between the two variants. In this paragraph, the variants will be analysed on the different criteria, judged and a score will be assigned to assist in the decision-making of the "ideal" timber structure. The judgement is done by first qualitatively comparing the variants, attaching numbers to these qualitative differences which eventually lead to a substantiated, quantified score for the variants.

6. Results



Material efficiency:

There is a quantifiable difference in material efficiency in the designed structures in the appendices. The hollow core CLT-variant is only applicable in the floor- and roof slabs, resulting in a total material saving for the total structure of 7.5% with respect to the regular CLT design. Additionally, a top floor is required for the regular CLT floors to create enough space for installations, whereas the HCCLT has this space built-in. Assume an additional saving of material of 2.5%.

$$\text{CLT: } 0,90 \qquad \text{HCCLT: } 1$$

Fire safety:

The efficiency of the fire safety of a variant can be expressed as the ratio between the fire safety bending unity check and the regular bending unity checks. The unity checks for regular CLT are 0,24 for fire safety and 0,31 for regular bending. The unity checks for HCCLT are 0,18 for fire safety and 0,16 for regular bending. The calculation of these unity checks is done in Appendices A and B. The ratios between the unity checks will be used as the scores for the variants:

$$\begin{aligned} \text{CLT:} & \quad \frac{0,24}{0,31} \approx 0,77 \\ \text{HCCLT:} & \quad \frac{0,18}{0,16} \approx 1,13 \end{aligned}$$

This means fire safety is more often governing for the design of regular CLT floors and is naturally incorporated in the HCCLT slabs. Additionally, the cavities in the HCCLT may be used for the application of a fire retardant to further increase the resistance in fire conditions.

Energy efficiency (during exploitation of the building):

The efficiency of the variants in terms of energy consumption mitigation is in this comparison dependent on the total thickness of the roof slabs. Appendix A shows the application of 145mm regular CLT requires an additional layer of cellulose of 186 to fulfil the thermal resistance requirement, resulting in a total thickness of 331mm. Ch.5 shows the minimal HCCLT variant requires a total thickness of 341mm for the structure plus insulation, caused by the combination of a thicker structural slab due to the hollow cores (even though the timber volume is lower) with only a slightly thinner insulation layer. The scores are:

$$\begin{aligned} \text{CLT:} & \quad \frac{0,331}{0,331} \approx 1 \\ \text{HCCLT:} & \quad \frac{0,331}{0,341} \approx 0,97 \end{aligned}$$



Circularity:

Ch.3 described a fit in the circular economy of CLT similar to that of prefabricated concrete. Considering the circularity of CLT as the reference, HCCLT scores worse. Ch.5 describes the additional challenges that arise when looking to reapply HCCLT slabs. The strength of HCCLT is the ability to customize the content of the hollow cores as desired, negatively affecting the possibilities of application in slightly different functions. Adaptations to the content of the cores is cumbersome and might mean recycling of the separate materials is a better option. Quantifying the negative effect on the circularity for HCCLT at 50% less than regular CLT due to the specialized form, this leads to scores:

CLT: 1 HCCLT: 0,5

Bio-economy:

Consider timber is 100% bio-based, the addition of adhesives to produce CLT slightly reduces the bio-basedness of the product. Less timber is applied in HCCLT than in conventional CLT, and therefore the quantity of adhesives per square meter is assumed less.. The effect on the biodegradability of adhesives is more significant, since the timber boards cannot be disassembled after gluing. HCCLT relies more on the strength of adhesives to produce the slabs due to the extra lamination layers between the CLT and the GLT, meaning the application of bio-based adhesives is less feasible, and is therefore less fit for the bio-economy than conventional CLT with a marginal difference. Scores:

CLT: 0,9 HCCLT: 0,85

Recyclability:

Similar to the fit in the bio-economy, the recyclability of timber relies on the quantity and strength of additives required to construct a product. Additionally, in this score the possibility of upcycling is considered; both HCCLT and CLT cannot be broken down to form new useable timber boards. In Ch.3 was discussed that the application of new timber from sustainable forests is very beneficial. Upcycling (the process of breaking down elements to retrieve valuable and scarce materials) is therefore not of importance. Only if HCCLT was constructed with GLT and CLT connected with metal fasteners instead of adhesives would upcycling have an impact. Scores, similar to bio-economy:

CLT: 0,9 HCCLT: 0,85

6. Results



Durability:

The durability of structures is assured through the minimal requirements in the Eurocode. These checks are based on an assumed life span, and the durability is guaranteed over this time with a set reliability which follows from the reliability class, determined for the structure as a whole and for separate members. The unity checks following the checks for HCCLT in Appendix B prove to be significantly lower than the same unity checks for conventional CLT in Appendix A. This implies both structures are considered safe, but the HCCLT variant is safer. The assumed design life of the structure is 50 years, meaning the lower UC for the HCCLT structure assures more structural safety after this design life since the deterioration of the material (spruce) is equal for the variants. For both variants, the long-term deflection requirement gives the critical unity check, leading to the following scores for the durability of the structures after their design lives:

$$\text{CLT:} \quad 1 - 0,75 = 0,25$$

$$\text{HCCLT:} \quad 1 - 0,41 = 0,59$$

Indoor climate:

Due to the HCCLT being produced out of CLT flanges and GLT webs, the air tightness is similar to that of conventional CLT. The difference in the effect on the indoor climate between the variants relates to the parts of the cross-section that can contribute to controlling the indoor moisture content and temperature. More timber in the indoor structure means more moisture control and less required mechanical ventilation. This means the material efficiency criterium counteracts this one. Additionally for the HCCLT-members with an external function, insulation material is placed inside the slab. A moisture barrier is applied on the warm side of the insulation material to avoid condensation of the diffused moisture between the wall layers, decreasing the total moisture absorption capacity even further. A full heat distribution calculation should provide more knowledge on whether this barrier is actually necessary, as it depends on the seasonal temperature differences between inside and outside and the thicknesses of the different layers, but to assure the durability of the timber in relatively colder climates it is always safe to assume application on the warm side of the insulation. Volume calculations for the absorbing layers are found in Appendices A and B. Assuming the CLT volume as the reference with a score of 1, this leads to:

• • •

$$V_{\text{CLT}} = 5,886 \cdot 10^{10} \rightarrow \text{CLT: } \frac{5,886 \cdot 10^{10}}{5,886 \cdot 10^{10}} = 1$$

$$V_{\text{HCCLT}} = 5,886 \cdot 10^{10} - 3,965 \cdot 10^9 - 0,544 \cdot 10^9 - 9600 \cdot 5400 \cdot 63 = 5,109 \cdot 10^{10} \rightarrow$$

$$\text{HCCLT: } \frac{5,109 \cdot 10^{10}}{5,886 \cdot 10^{10}} = 0,87$$

Acoustics:

The (indoor) acoustic performance of CLT has been determined in Appendix A. The critical requirements for the intermediate floors led to the application of a ballast layer of 50 kg/m² (airborne sound) and a layer of shock-absorbing foil (structure-borne sound). In terms of the structure-borne sound, the HCCLT performs better than the CLT due to the connections within the engineered slab which will increase the energy loss. Instead of applying a foil on top of the floor, adding a profile within the connections in the slab improves this performance. The airborne noise performance of the HCCLT is worse than for conventional CLT due to the lighter weight of the slab. More ballast is required to assure the same acoustic performance, which can be placed inside the hollow cores. The performance of the HCCLT varies compared to conventional CLT, and therefore the accumulated score for both variants are considered equal:

CLT: 1 HCCLT: 1

Aesthetics:

The HCCLT consists of CLT flanges, implying that aesthetically, the structural members in the variants do not vary. However, due to the application, a floor topping is required for the intermediate floors in the CLT variant, making it impossible to leave the CLT in sight and decreasing aesthetic design options. This topping is required for the application of acoustic foil and ballast to decrease the fundamental frequency of the floor. In HCCLT, these application can be processed within the slab. The difference in scoring however is not too big, since a floor could be chosen which looks similar to CLT.

CLT: 0,95 HCCLT: 1

Manufacturability:

The manufacturability of regular CLT is the reference for this criterion (=1). The process has been fully automated, leading to quick production and good quality control. HCCLT can be produced in an equal manner as CLT, by pausing the process to rearrange boards in specific layers. This part can also be automated if HCCLT proves to be a suitable alternative to regular CLT. Producing HCCLT out of CLT

6. Results



slabs and GLT beams is more time-consuming and not so easily automated, whilst requiring three production lines when setting up a new HCCLT factory (one for CLT, one for GLT, and one to combine them (even though the production lines for CLT and GLT already exist in current timber factories)).

The HCCLT from the automated gluing process needs to be pressed in order to create sufficient bonding strength in the laminations. However, due to the presence of the hollow cores, the flanges are able to deform during pressing, resulting in insufficient compression in the lamination layer and therefore insufficient bonding strength. Placing a supporting profile with equal characteristics as the timber web inside the hollow core could solve this problem, but generates difficulty in the automation of the process as well as the ease of manufacturing (van Aken, 2017). HCCLT is assumed to be 40% less effective in the manufacturability than regular CLT.

$$\text{CLT: } 1 \qquad \text{HCCLT: } 0,6$$

Robustness:

A comparison between the variants in terms of the robustness is quite complex. The comparison in terms of this criterion with concrete is mostly based on laymen's assumptions. The main issue is therefore to change this thought. The sturdy feeling of concrete arises from its relatively high weight and stiffness. The difference in weight between the variants is negligible, mainly due to the addition of ballast on the floors for acoustic purposes. The score for this criterion is therefore based on the deformation of the intermediate floors, representing the stiffness' of the variants:

$$\begin{aligned} \text{CLT:} \quad & \frac{0,41}{0,75} \approx 0,55 \\ \text{HCCLT:} \quad & \frac{0,41}{0,41} = 1 \end{aligned}$$

• • •

Criterion	Weight factor	CLT		Hollow core CLT	
		Score	Weighted	Score	Weighted
<i>Material efficiency</i>	4	0,9	3,6	1	4
<i>Fire safety</i>	3	0,77	2,31	1,13	3,39
<i>Energy efficiency</i>	5	1	5	0,97	4,85
<i>Circularity</i>	3	1	3	0,5	1,5
<i>Bio-based</i>	4	0,9	3,6	0,85	3,4
<i>Recyclability</i>	2	0,9	1,8	0,85	1,7
<i>Durability</i>	4	0,25	1	0,59	2,36
<i>Indoor climate</i>	2	1	2	0,87	1,74
<i>Acoustics</i>	3	1	3	1	3
<i>Aesthetics</i>	2	0,95	1,9	1	2
<i>Manufacturability</i>	5	1	5	0,6	3
<i>Robustness</i>	2	0,55	1,1	1	2
Total			33,31		32,94

Table 11 – MCA result

Table 11 shows the results of the multi-criteria analysis. Questions on the reliability of this analysis are valid. The intention of objective scoring and weight factors becomes clear in the previous sections, but there remains uncertainty due to the lack of available information into many topics. These topics must be researched individually and combined to create a reliable comparison, but for this thesis, such research would be too extensive.

The scores for both variants are very close to each other. This is mostly due to the (subjective) scoring of the manufacturing processes. Both slab-types have their advantages and disadvantages, but if the manufacturing process of HCCLT can become as automated as that of CLT, the scores will not only change, but a clear improvement in the costs can be made due to the saving of material.

The application would require much more optimization, due to the different functions that structural elements can fulfil (floors, walls, roofs, ground floors) and the endless options in dimensions (web- and flange thicknesses). Clearly, the development and spreading application of CLT is only a small portion of what may become possible in the future of timber structures.



6.2. Cost estimate ideal design

To complement the initial comparison between the “CLT-House of the Future” and one of the most conventional building methods for terraced housing in prefabricated concrete, an economic analysis of the structure is required. The costs of a two-storey prefabricated concrete housing structure have been determined at €25.220,- (83 €/m² for floors and roofs, 59 €/m² for external walls and 36 €/m² for inner walls), which includes transportation to the building site. Note that this cost number includes only the structure, excluding insulation and cladding. The following aspects contribute to the costs of the CLT-structure:

- Material
- Transportation
- Potential floor toppings including installations
- On site lifting

For comparison purposes, the top storey of the designed timber structure in Ch.5 will be neglected. This means the floors, the roof and the walls have thicknesses of 160mm, 145mm and 98mm, respectively.

Member	A [m ²]	t [mm]	V [m ³]	M [kg]
<i>Floors</i>	103,68	160	16,58	8290
<i>Roof</i>	51,84	145	7,51	3755
<i>Load-bearing walls</i>	115,2	98	11,28	5640
<i>Front-/back facades</i>	64,8	98	6,35	3175
Total	335,52		41,75	20860

Table 12 - Area, thickness, volume and mass per CLT member-type

The average price of CLT is approximately 500 €/m³ for larger suppliers (CBI - Ministry of Foreign Affairs, 2017), dependent on the thickness and number of layers in the CLT slab and the chosen visual quality. Woodteq however assumes a price of 700 €/m³ as an upper limit to assure a reliable cost estimation that does not underestimate the real costs. The total price of the timber structure based on this price is €29.225,-. This is 16% more expensive than the prefabricated concrete structure, but the comparison should include inner walls, fasteners, installations and foundation for both material-variants. The assumed inner wall area must be assumed equally for the CLT structure as for the



concrete structure as 47 m². With a wall thickness of 63mm, this results in a total price of €2.072,-. The acoustic performance could be increased by choosing thicker inner walls, resulting in higher costs.

Considering the relatively lower weight of the timber structure compared to concrete, the foundation will be cheaper. The foundation design will not be incorporated in this thesis, but the assumption is made that the foundation accounts for 10% of the costs for a concrete structure and 5% of the costs for a CLT structure.

The total number of screws required has been determined in Ch.5. Packages of 8mm diameter screws contain 100 pieces, of 10mm diameter screws contain 50 pieces per pack. Assuming an average box price of €50,- for all types, this accumulates to a total of €600,- for the fasteners. The CLT structure needs to be connected to the foundation with tensile- and shear brackets. Assume another €1000,- for the required brackets and screws.

Assuming a worst-case scenario, where the CLT mounting costs are equal to the mounting costs for concrete, which has been determined in the cost analysis of the concrete structure at €2877,-. This is the worst-case scenario, since the timber structure is much lighter than the concrete structure, and the connections are constructed in a dry manner, leading to easier working methods and better conditions on site. Realistically, this leads to higher speed of construction, saving money.

Additional costs need to be included for the transportation costs of the structural members. Considering the closest CLT supplier to the Netherlands, this lead to a transportation distance of approximately 200km from Amsterdam. The choice of CLT supplier is essential in finding the best economic solution, as transportation is an important contributor to the accumulated costs. The maximum dimensions of trucks for regular transportation in the Netherlands are 12m long, 2,55m wide and 4m high, resulting in a volume of 122,4m³. This means all prefabricated slabs can be transported regularly across the road without additional permits and costs, since the building is 9,6m long and 3m high per storey, but it also means the slabs need to be transported vertically to not exceed the maximum width of a truck. Considering the maximum mass of a vehicle of 40.000kg, the CLT members for one house can be transported with one truck. Concrete factories are abundant in the Netherlands, so it is safe to assume a factory is never further away from a building site than 50km. Assuming an estimated total volume of the structural members similar to the CLT structure, the total structure has a mass of 100.200kg. This means at least 3 transportations of 33.400kg are required to move the

6. Results



prefabricated concrete members from the factory to the building site. Though the total weight of the CLT structure is significantly lower, the relatively longer distance it needs to travel results in approximately equal transportations; transporting 20.000kg over 200km is similar in costs and environmental effects as transporting 33.400kg three times over 50km. Establishing CLT factories within the Netherlands would be the optimal method of decreasing the transportation costs of CLT for the Dutch market. The current costs of transportation of the CLT-structure, assuming transportation costs of 2 €/km due to the relatively heavy weight per shipment of (even though the CLT structure is considered to be lightweight), accumulate to €800,-, but as mentioned this same number can be assumed for the transportation of concrete.

Mentioned in the MCA, the regular CLT variant requires many different additions to create a well-functioning floor slab. It needs to be thick enough to carry the loads, it needs to

In Ch.3, the difference in thermal resistance between timber and concrete has been discussed, resulting in less insulation required for timber to create a sound structure according to the standards. Assuming no contribution from the concrete slabs to the insulating characteristics of the wall composition, this leads to an assumed total increase in required insulating material of at least 20%, with CLT requiring 150mm of cellulose on average, and concrete requiring a layer of 180mm. Assuming an average price of €100,-/m³ for cellulose, and an average external slab area of 200m², the total is shown in Table 13 - Cost comparison CLT and prefab concrete.

Costs	CLT	Concrete
<i><u>Structural members</u></i>		
<i>Floors</i>	€ 11.612,16	€ 8.605,44
<i>Roof</i>	€ 5.261,76	€ 4.302,72
<i>Load-bearing walls</i>	€ 7.902,72	€ 6.796,80
<i>Front-/back facades</i>	€ 4.445,28	€ 3.823,20
<i>Inner walls</i>	€ 2.072,70	€ 1.692,00
Total structure	€ 31.294,62	€ 25.220,16
<i>Foundation</i>	€ 1.564,73	€ 2.522,02
<i>Fasteners</i>	€ 1.600,00	-
<i>Mounting</i>	€ 2.877,00	€ 2.877,00
<i>Transportation</i>	€ 800,00	€ 800,00
<i>Floor topping</i>	€ 1.000,00	€ 550,00
<i>Insulation</i>	€ 3.000,00	€ 3.600,00
Total	€ 42.136,35	€ 35.569,18



Table 13 - Cost comparison CLT and prefab concrete

Currently, the quantifiable difference results in a CLT-structure that is approximately 20% more expensive than the concrete solution. An essential, unquantifiable entity in this cost comparison is the possible establishment of a different CO₂-tax in the near future. The CO₂-tax system currently acts as a fining system, where companies only need to pay for the emitted CO₂ when a certain threshold is exceeded. Plans have been suggested by the Dutch government to switch to a system with a constant cost per kg of emitted CO₂, which might significantly influence the economic viability of concrete negatively and of CLT positively.



7. Discussion

The findings in this thesis suggest that CLT as a building product is a capable replacement with regard to the structural performance of prefabricated concrete slabs in the application in low-rise housing. The safety and serviceability are guaranteed through minimum requirements in the Eurocode, and in terms of building physics, the amount of additional measures to assure good performance is comparable for both building materials. For many of the structural verifications, substantiated assumptions were necessary in the correct application of factors and coefficients.

The comparison of CLT and prefabricated concrete originated from the conventional building methods of Woodteq bv. and the project initiator, respectively. To increase the value of this thesis, more methods should be considered in the comparison, like methods with timber as the primary building material (e.g. prefabricated) timber framing or with other materials (e.g. limestone). Following this report, any contractor who builds with other methods or materials could state that their method is still more viable than building with CLT, as the contrary has not yet been proven.

The structural design in this thesis yields critical topics with regards to holes in CLT slabs for which clear design guides are either missing or could be enhanced to increase the theoretical optimizations of members. For the diaphragm action and the in-plane shear in slabs, standards suggest a constant distribution of the stresses over the strong parts of a slab e.g. the parts of a shear wall that span the total height between the point of load introduction and the point of support. For slabs without holes, this approach gives a good approximation of the real stress distribution, since all boards and every lamination layer between individual boards contribute equally to the resistance of the total slab. If holes are present in the slab however, this approach gives a lower limit of the resistance of the slab than reality, since weaker parts that connect stronger parts may increase the total resistance of the system but are neglected in the calculations. The current standards are still applicable, yet they are conservative and can be further optimized. This was proven in the FE analyses of different wall configurations in the results section of this report, since those walls would not pass in the current standard system, but considering stress distribution due to plastic deformation would in practice still function properly. Further research of this part of the structural design could provide much more insight in the optimization, and possibly lead to the establishment of new standards.



In terms of the product development, material efficiency and adhesive development are critical topics in the viability of engineered timber products in future application. The suggested variant of hollow core CLT provides a decrease in the timber demand to build a structure when compared to conventional CLT. The product was only applicable in some members of the structure due to the limited options of parts, manufacturability and change in structural characteristics. In that sense, more optimization can be done for individual structural members to determine the minimum material demand, since e.g. a wall may or may not need to act as a shear wall, influencing the requirement for strength in two directions. The suggested application in this thesis is therefore too specific, but in designing a structure, assumptions and choices need to be made. A designer must first determine the different functions of a member, then investigate how these functions are fulfilled through the structural characteristics and finally choose the appropriate critical dimensions to satisfy all requirements. The HCCLT design was not optimized for this application, since the functionality of the floors does not only depend on the ULS and SLS checks in the Eurocode, but also needs to provide in-plane resistance through diaphragm action, serve as an acoustic barrier between floors and mitigate noticeable vibrations due to human action. Additionally, assumed was a restriction to the application of available CLT-products, to elaborate a structure which is actually buildable at this moment. Altering the CLT production line to create HCCLT would be an alternative method of production, but this process is very complex to create qualitatively well made products. Additionally, when designing an engineered product with every type of timber part available (even small boards, plywood, etc.), more optimization can be accomplished.

Renewability and the biological resource (including bio-degradability) are among the strengths of timber as a building material and important reasons for the application in structures. The application of timber as a structural material means the structure acts as a carbon stock, and the carbon sequestration can continue subsequently by planting new trees. Therefore the demand for timber should get increased to equalize the supply, which is equal to the accumulated growth of forests and will maximize the absorbed CO₂ from the air. To successfully equalize supply and demand, an optimal design life of a structure should be found. Further research needs to provide more insight in the differences in environmental effects of circular reapplication of timber versus the application of new timber while using old timber as a source for e.g. bio-energy, as a base material for wood-based panels

7. Discussion



or dumping as waste (which maintains the carbon stock). The recycling rates of timber are relatively low, and more research should provide better solutions, in which the biodegradability and the influence on this biodegradability from additives (like adhesives) will possibly be crucial. The application of bio-based glues is currently not an option in structural members, but future developments may lead to an alteration in this application, affecting the biodegradability and possibly the options for recycling of timber members.

The final discussion topic, and possibly the most important part for the application in practice, is the cost comparison. Currently, there is a quantified difference in the costs between CLT and prefabricated concrete of about €7.000,- in favour of concrete. To bridge this gap, assumptions and expectations could be made towards the future, but no guarantees can be given. The plans of the Dutch government to change the law on CO₂-tax may be critical in this issue, but no specific plans have been established. Currently, the best option is to apply for a subsidy on the basis of the favourable sustainability of the timber structure. Sustainability is heavily promoted by the Dutch government, and individual subsidies are easier to push through than a CO₂-tax which affects the whole economy.



8. Conclusion

The first part of the conclusion relates to the decision-making in an early stage of the structural design; material selection. Woodteq bv. was given a pilot project to investigate innovative timber solutions. This project was initially considered to be built out of prefabricated concrete. This material has favourable structural characteristics, which have led to it being the second most consumed substance in the world, with water taking first place. Coincidentally, with the production of concrete much CO₂ is emitted as a by-product, accounting for at least 5% of the total global CO₂ emission. Investigating a replacement of concrete with timber, improvements are made in terms of e.g. carbon storage in a structure, carbon emission mitigation during production, renewability of building material sources and bio-degradability. The full material comparison is found in Table 1. Ultimately, timber is considered to be a better building material in terms of sustainability and quality of the indoor climate, especially considering future environmental issues and changes in the consumption mentality of humankind.

Taking from the first part of the conclusion that timber is a favourable building material over concrete, a suitable product is required for replacement of the prefabricated concrete members within a terraced housing structure. Timber, coming from trees, is only available with large dimensions in one direction, whereas prefabricated concrete in terraced houses functions as slabs, resisting both in-plane and out of plane loads. Cross-Laminated Timber (CLT) is the ideal timber engineered product to be applied as replacement, offering good strength characteristics in two directions of the slab, which creates resistance to out-of-plane and in-plane loads. It weighs approximately 21%(=500/2400) of the weight of concrete, and therefore provides a better solution in terms of transportation, lifting on site and manufacturability. The production process is fully automated and assures good quality.

According the Eurocode, structures need to meet requirements on (fire- and structural) safety and (dynamic- and static) serviceability. Additional requirements for the total building are compiled in the Dutch Building Decree, on topics like the thermal resistance (insulation), ventilation (breathability of structure) and acoustics. A total guideline analysis is done in Ch.3. Extra building criteria like the manufacturability of a structure and the aesthetics are not related to standards but have much influence in the design. The minimal CLT design for the Woodteq pilot project (three-storey with a height of 9m, 5,4m wide by 9,6m deep) consists of load-bearing, house-separating walls of 98mm (29-40-29) thick

8. Conclusion



with 100mm cellulose in between as a fire retardant, which has seen much application for this purpose already within the industry. These walls provide stability for horizontal loads in the direction of the depth of the house, and carry the floor- and roof slabs of 160mm (40-20-40-20-40) and 145mm (29-29-29-29-29) respectively. A floor topping needs to be applied to clear away installations and extra needed ballast for mitigation of vibration and acoustic issues. The front- and back facades are executed in 98mm (29-40-29) thick slabs. These walls provide the stability for horizontal loads in the direction of the width of the house. Structural analyses have indicated that holes of 1500mm by 3900mm at the first and second storey level, and door and window holes can be created while still maintaining the stability for the extreme wind loading situation. Visualizations of the total structure are found in Ch.5.1.2.

To find an economically friendlier solution to the CLT structure, a hollow core CLT slab design creates more material efficiency. The gross of the cross-section in this design is moved away from the neutral axis, improving the out-of-plane resistances of the members. Due to the thin walls in the CLT design, replacement of these members is not feasible. The roof- and floor slabs can be replaced by engineered hollow core slabs of 63mm (20-23-20) CLT flanges and 80x80mm GLT webs. This results in a total material saving for the structure of 7.5%. Decision-making on the basis of a multi-criteria analysis shows this material saving does not weigh up against the easy manufacturing of CLT: to create hollow core slabs, either a complex, altered CLT production line or three different production lines are required; CLT, GLT and an assembly line. Within this report, from the point of view of a project initiator, this is not a concern, however, it limits the possibilities within the market at this point in time. The hollow core variant scores similar to the conventional CLT variant in the weighted score of the MCA, but if an investment is made by a manufacturer to create a production line, in the long run this investment will pay out in the saving of material.

At last, a cost comparison between this best CLT design and a standard prefabricated concrete design for the pilot project shows the current economic unfeasibility of the timber structure, with a cost exceedance of approximately 20% of the costs of the concrete structure, but this calculation is still subject to uncertainties and ongoing changes within the industry with regard to unit prices. Future environmental policies with regard to the emission of CO₂ may be the deciding factor in the viability of CLT as a direct replacement for prefabricated concrete in the application in low-rise terraced housing structures.



References

- Berger, G., Kats, H., & Petutschnigg, A. (2006). What consumers feel and prefer: Haptic perception of various wood flooring surfaces. *Forest Products Journal*, 56(10), 42-47.
- Bradbury, M. (2017). A brief timeline of the history of recycling. Retrieved from <https://www.buschsystems.com/resource-center/page/a-brief-timeline-of-the-history-of-recycling>
- CBI - Ministry of Foreign Affairs. (2017, 1-11-2017). Exporting cross laminated timber (CLT) to Western Europe. Retrieved from <https://www.cbi.eu/market-information/timber-products/cross-laminated-timber/europe#what-are-the-end-market-prices-for-cross-laminated-timber->
- CEN. (2018). prEN16351 - Working draft of design of cross laminated timber in a revised Eurocode 5-1-1. In.
- Costes, L., Laoutid, F., Brohez, S., & Dubois, P. (2017). Bio-based flame retardants: When nature meets fire protection. *Materials Science and Engineering R: Reports*, 117, 1-25.
- de Graaf, P., & Banga, J. (2002). *Handboek Houtskeletbouw*. Rotterdam.
- Department for Communities and Local Government. (2009). *Multi-criteria analysis: a manual*. In. Retrieved from http://eprints.lse.ac.uk/12761/1/Multi-criteria_Analysis.pdf
- Egoïn. (2018). *Environmental Product Declaration - EGO-CLT Cross Laminated Timber wood panel*. Retrieved from <https://www.environdec.com/Detail/?Epd=14006>
- Flaig, M., & Blaß, H. J. (2013). *Shear strength and shear stiffness of CLT-beams loaded in plane*. Retrieved from <https://publikationen.bibliothek.kit.edu/1000037153/2850883>:
- Forest Europe. (2015). *State of Europe's Forests 2015*. Ministerial Conference on the Protection of Forests in Europe.
- Gifford, R., Steg, L., & Reser, J. P. (2011). Environmental Psychology. In P. R. Martin (Ed.), *The IAAP Handbook of Applied Psychology* (pp. 440-470): Blackwell Publishing Ltd.
- Government of the Netherlands. (2015). Basics on Dutch housing. Retrieved from <https://www.government.nl/topics/investing-in-dutch-housing/basics-dutch-housing>
- Gustavsson, L., Pingoud, K., & Sathre, R. (2006). Carbon dioxide balance of wood substitution: Comparing concrete- and wood-framed buildings. *Mitigation and Adaptation Strategies for Global Change*, 11(3), 667-691. doi:10.1007/s11027-006-7207-1
- Health Council of the Netherlands and Dutch Advisory Council for Research on Spatial Planning Nature and the Environment. (2004). *Nature and health. The influence of nature on social, psychological and physical well-being*.
- Hu, L., & Gagnon, S. (2012). *Controlling Cross-Laminated Timber (CLT) Floor Vibrations: Fundamentals and Method*. Paper presented at the World Conference on Timber Engineering, Auckland, New Zealand.
- Hu, L. J., & Chui, Y. H. (2004). *Development of a Design Method to Control Vibrations Induced by Normal Walking Action in Wood-Based Floors*. Retrieved from
- IPCC. (1996). *Revised 1996 IPCC Guidelines for National Greenhouse Gas Inventories*. Retrieved from <https://www.ipcc-nggip.iges.or.jp/public/gl/guidelin/ch2ref1.pdf>.
- Labannotte, N., & Malo, K. A. (2010). *Vibration properties of cross laminated timber floors*. Retrieved from
- Linsen, L., Karis, B. J., McPherson, E. G., & Hamann, B. (2005). Tree Growth Visualization. *The Journal of WSCG*(13), 81-88.

- Lo, T. Y., Tang, W. C., & Nadeem, A. (2008). Comparison of carbonation of lightweight concrete with normal weight concrete at similar strength levels. *Construction and Building Materials*, 22, 1648-1655.
- Malešev, M., Radonjanin, V., & Broćeta, G. (2014). Properties of recycled aggregate concrete. *Contemporary Materials*, 2(2014), 239-249. doi:10.7251/COMEN1402239M
- May, N. (2005). Breathability: The Key to Building Performance. Retrieved from <http://www.ecotimberframe.ie/pdf/BreathabilityinbuildingsNBT.pdf> website:
- Messmer, A. (2015). *Life cycle assessment (LCA) of adhesives used in wood constructions*. Retrieved from
- Nyrud, A. Q., & Bringslimark, T. (2009). *Is Interior Wood Use Psychologically Beneficial? A Review of Psychological Responses Toward Wood*. Retrieved from
- Pettersen, R. C. (1984). The Chemical Composition of Wood. In R. Rowell (Ed.), *The Chemistry of Solid Wood* (Vol. 207, pp. 57-126).
- Pizzi, A., & Mittal, K. L. (2011). *Wood Adhesives*: CRC Press.
- Rametsteiner, E., Oberwimmer, R., & Gschwandtl, I. (2007). *Europeans and Wood - What Do Europeans Think About Wood and its Uses?*
- Rice, J., Kozak, R. A., Meitner, M. J., & Cohen, D. H. (2006). Appearance Wood Products and Psychological Well-Being. *Wood and Fiber Science*, 38(4), 644-659.
- Rubenstein, M. (2012). *Mitigating Emissions from Cement*. Retrieved from <http://climate.columbia.edu/files/2012/04/GNCS-Cement-Factsheet.pdf>
- Schickhofer, G., Brandner, R., & Bauer, H. (2016). *Introduction to CLT, Product Properties, Strength Classes*.
- Shahnewaz, M., Tannert, T., Alam, M. S., & Popovski, M. (2015). *Experimental and Finite Element Analysis of Cross Laminated Timber (CLT) Panels*. Paper presented at the Conference on Intelligent Computer Mathematics 2015 (CICM), Dhaka, Bangladesh.
- Skogstad, H. B., Gullbrekken, L., & Nore, K. (2011). Air leakages through cross laminated timber (CLT) constructions. *Proceedings of the 9th Nordic symposium on Building Physics NSB*.
- Stichting Bouwkwaliiteit. (2019). *Bepalingsmethode Milieuprestatie Gebouwen en GWW-werken*. Retrieved from <https://www.milieudatabase.nl/imgcms//SBK%20Bepalingsmethode%20versie%203.0,%201%20januari%202019.pdf>
- Struble, L., & Godfrey, J. (2004). *How sustainable is concrete?* Paper presented at the International Workshop on Sustainable Development and Concrete Technology, Beijing.
- Tabsh, S. W., & Abdelfatah, A. S. (2009). Influence of recycled concrete aggregates on strength properties of concrete. *Construction and Building Materials*, 23(2), 1163-1167.
- Transparency Market Research. (2014). *Wood Adhesives and Binders Market: Global Industry Analysis, Size, Share, Growth, Trends and Forecast, 2014-2020*.
- van Aken, B. (2017). *Hollow Core Cross-Laminated Timber - Optimized for a more efficient use of material*. (Master of Science), TU Delft,
- van de Kuilen, J.-W., Kuijpers, H., Ravenshorst, G., Raadschelders, V., Smit, E., Brunner, M., . . . Frantzen, T. (2017). *Cursus Kruislaaghout: Constructies, brandveiligheid, aardbevingsbestendigheid en geluidswerendheid*. Rothoschool. Course presentations. Rothoblaas. Almere.
- van den Oever, M., Molenveld, K., van der Zee, M., & Bos, H. (2017). *Bio-based and biodegradable plastics - Facts and Figures*. Retrieved from



- Vnuček, D., Kutnar, A., & Goršek, A. (2016). Soy-based adhesives for wood-bonding - a review. *Journal of Adhesion Science and Technology*. doi:10.1080/01694243.2016.1237278
- World Business Council for Sustainable Development. (2002). *The cement sustainability initiative - our agenda for action*.
- Yeon, J.-O., Kim, K.-W., Yang, K.-S., Kim, J.-M., & Kim, M.-J. (2014). Physical properties of cellulose sound absorbers produced using recycled paper. *Construction and Building Materials*, 70, 494-500.
- Zhang, J., Liu, G., Chen, B., Song, D., Qi, J., & Liu, X. (2014). Analysis of CO2 Emission for the Cement Manufacturing with Alternative Raw Materials: A LCA-based Framework. *Energy Procedia*, 61, 2541-2545.



Appendix A: Calculations minimum CLT design

A.1 Assumptions

Before the requirements of the CLT-elements can be determined, some assumptions have to be made regarding the classification of the structure.

Service class: 2; e.g. houses; some moisture around structure

Consequence class: CC1

Design life class: 3 → Design life: 50 years

Strength class: C24^[1] (Spruce)

With these assumptions the necessary input can be taken from the Eurocode to calculate the timber structure as described in Ch.4.

The choice of supplier will influence the costs since the factory location and price per structural element will most likely differ. Most suppliers provide structural data for consumers to simplify the dimensioning of members. This data may be used in this appendix to supplement the calculations.

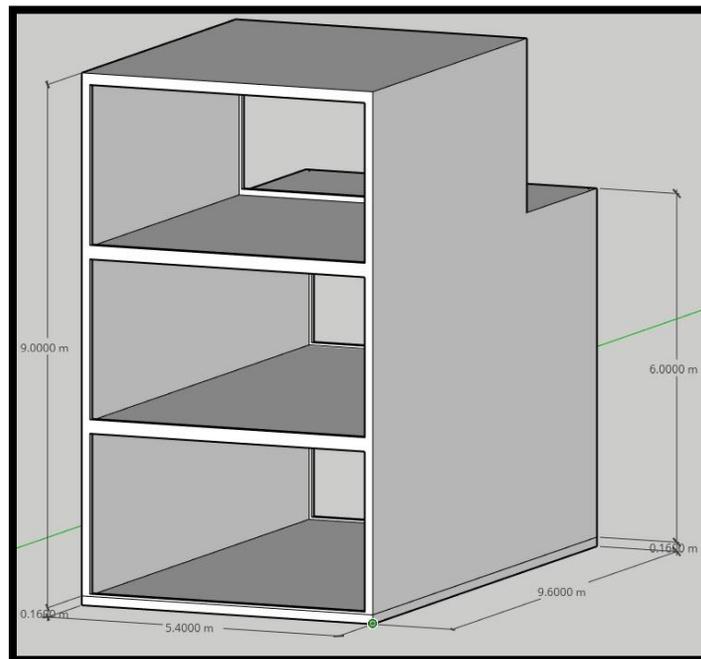


Figure A1 - 3d sketch of structure



A.2 Structural geometry

In this project the platform frame method is applied with a width of the house of $b = 5400$ mm. The total depth of the house is $d = 9600$ mm. Every storey has a height of $h = 3000$ mm, resulting in a height of $h_{total} = 9000$ mm for the whole house (Figure A1). At the top of the house there is a parapet of $h_p = 500$ mm. The top storey does not run over the full depth of the building, but instead has a depth of 6600 mm measured from the front of the house. The floor slabs are assumed to be separated into panels with a width of 1200mm, so 8 panels form one slab over the whole depth of the house.

Note: Figure A1 is a simplified visualization of the structure, in which the stabilizing front-/back facade are missing to show only the vertical load-bearing elements.

A.3 Structural checks

The critical load combination must be determined for each individual structural element and for the total structure, for both the ultimate limit state (ULS) and the serviceability limit state (SLS). See Figure A2 for a visualization of the different distributed loads, point loads and reaction forces per node or element.

Before calculating the individual elements, first the wind load on the building must be determined. With the height of the building set at 9000 mm, and assuming an unfavourable location without surrounding buildings within the Netherlands, the wind thrust is given in the national annex of NEN-EN1991-1-4 Table NB.5, with a value of $p_w = 0.98$ kN/m².

The next step is to perform the calculations, starting with elements at the top of the structure and working downwards, since the loads need to be transferred downwards towards the foundation (Figure A2).



Roof slab:

The roof slab is subject to 4 different loads, i.e. self-weight (permanent), imposed- (variable), snow- (variable) and wind load (variable). The self-weight depends on the required cross-sectional characteristics, which will be defined through an assumption in a later stage, but the variable loads can already be defined according to the Eurocode. The imposed load follows from the Dutch national annex of NEN-EN1991-1-1, Table NB.4 – 6.10, because roofs are in class H. For a roof angle of 0° , this load has a value of $1,0 \text{ kN/m}^2$. The snow load is determined through NEN-EN1991-1-3, with a resulting value of $s = 0,8 \cdot 0,7 = 0,56 \text{ kN/m}^2$.

The wind load on the roof is subdivided into three parts, all with a value of $p_w C_{pe,10}$:

1. Suction: vertical upward distributed load

Several zones per roof slab, depending on dimensions of the building (Figure A3).

Roof type:	Zone			
Parapet	F	G	H	I
$h_p/h = 0,05$	-1,4	-0,9	-0,7	$\pm 0,2$

Table A1 - $C_{pe,10}$ -values, source: NEN-EN 1991-1-4 Table NB.7 - 7.2

2. Internal pressure: vertical distributed load, can vary in direction

The internal pressure varies, and thus C_{pe} varies as well from $+0,3$ till $-0,3$

3. Wind loads on walls: horizontal line load, works in the connection between roof and wall

Several zones on wall, depending on dimensions of the building (Figure A4). In our case, $h/d \approx 1$ for both directions

Walls:	Zone				
	A	B	C	D	E
$h/d = 1$	-1,2	-0,8	-0,5	+0,8	-0,5

Table A2 - $C_{pe,10}$ -values, source: NEN-EN 1991-1-4 Table NB.6 - 7.1

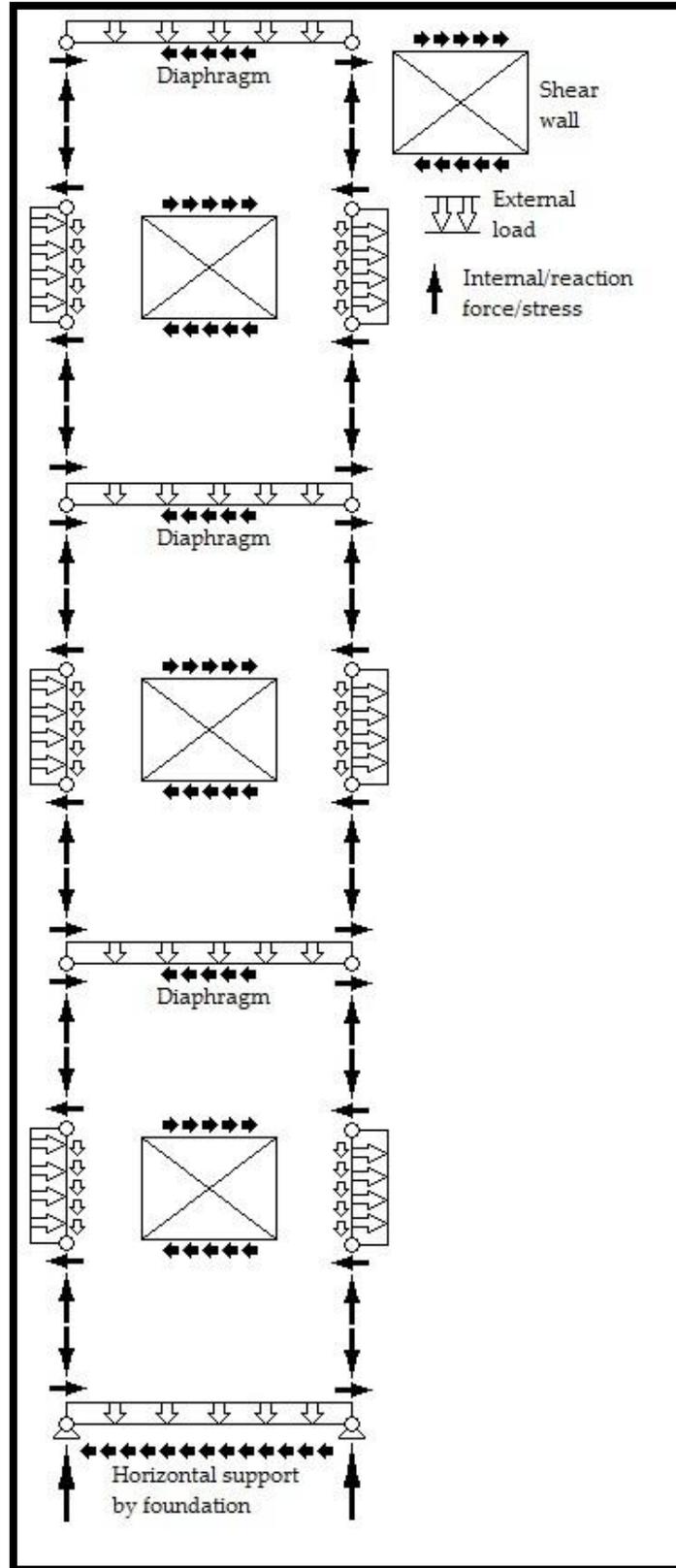


Figure A2 - Loads in structural members of terraced house

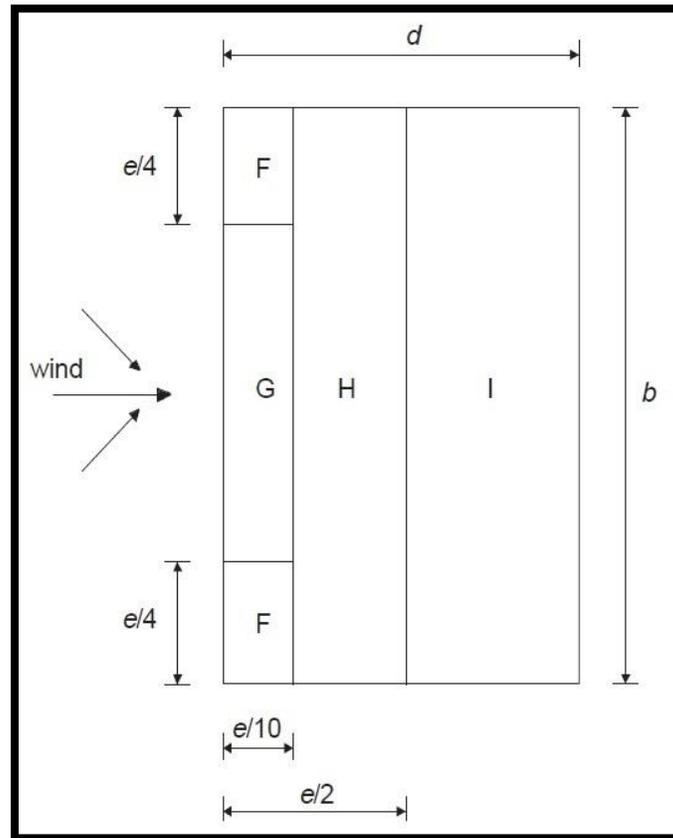


Figure A3 - Wind zones roof slab, source: NEN-EN 1991-1-4 Figure 7.6

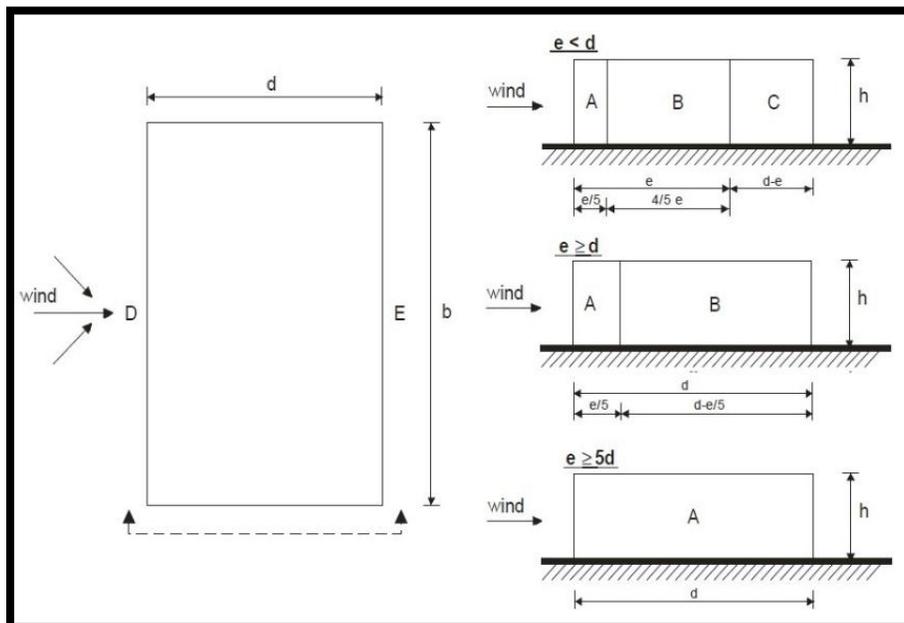


Figure A4 - Wind zones walls, source: NEN-EN 1991-1-4 Figure 7.5

Appendix A: Calculation CLT structure

• • •

The dimensions of zones F, G, H and I need to be calculated in order to determine the total loading situation on the roof.

$$e = \min(b; 2h) = \min(10; 2 \cdot 9) = 10 \text{ m}$$

The governing load for the wind is at either edge of the building, since this is where the maximum upward and downward pressure are found. A schematization of the combination for self-weight with imposed load or snow load can be seen in Figure A5.a, whereas Figure A5.b shows the load distribution for self-weight plus the upward wind load..

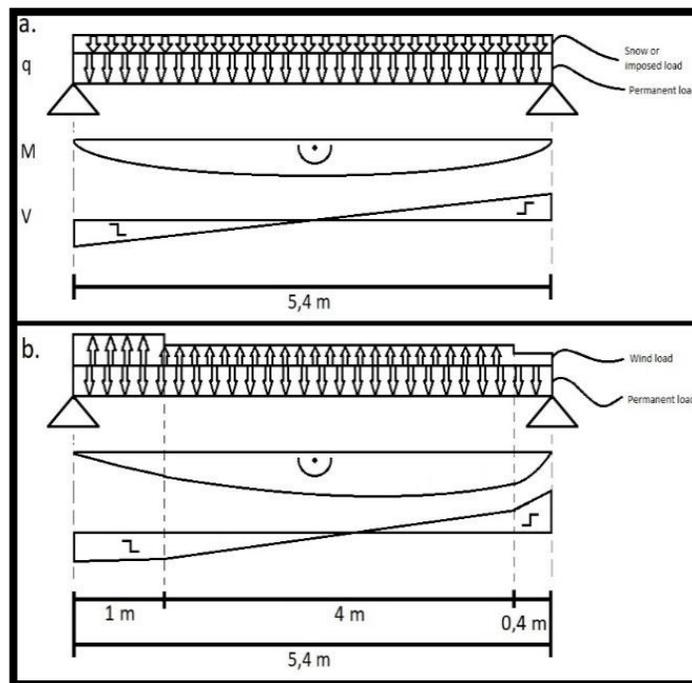


Figure A5 - a. self-weight + constant live load, b. self-weight + wind load

Assume a specific weight for CLT of 5 kN/m^3 . With a thickness of t [mm] and a calculation width of 1000 mm, the load on the floor due to self-weight is:

$$G_{k,j} = 5 \cdot \frac{t}{1000} \cdot 1 = \frac{t}{200} \text{ kN/m}$$

There are some more permanent loads working on the roof slab. First an isolation layer and an air- and watertight layer are placed over the structure. The isolation material is assumed to have a weight of 50 kg/m^3 and a thickness of 150 mm. Additionally, PV panels are placed on top of the roof to create an extra source of “green” energy. These panels weigh $15 \text{ [kg/m}^2]$ and require an extra ballast layer of 60 kg/m^2 to prevent the panels to fly away due to wind gusts.



The total additional permanent loads on the roof accumulate to a value of:

$$G_{k,add} = 0,15 \cdot 0,05 \cdot 9,81 + 9,81 \cdot (0,015 + 0,060) = 0,81 \text{ kN/m}$$

The ultimate limit state can be split up into several states, of which the following are of interest for our structure; EQU: loss of static equilibrium, FAT: failure of element due to fatigue, STR: failure of internal element due to strength.



Strength (ULS, STR):

In this situation, the load combination needs to consist of the permanent load, the design value of the main variable load and a combination design value of the other variable loads. All loads are multiplied by a safety factor to get the design value of the loads. These factors are 1,2 for permanent loads and 1,5 for variable loads (NEN-EN 1990 Table NB.4 – A1.2, eq. 6.10b). The unfavourable design load values are multiplied by a factor for loads (K_{FI}) depending on the consequence class (CC) of the structure. For regular houses of max. 3 storeys, the consequence class is CC1, meaning $K_{FI} = 0,9$ (NEN-EN 1990 Table B3). The combination factors (ψ) for the variable loads are found in NEN-EN 1990 Table NB.2-A1.1. However, all combination factors for roofs (for imposed, snow and wind loads) are equal to 0. The reduction factor for the permanent load (ξ) is assumed 0,89, taken from NEN-EN 1990 Table NB.4 – A1.2(B).

Characteristic combination:

The loads on the roof are accumulated into the following characteristic load combinations with equations 6.10a and b of NEN-EN 1990 NB:

Self-weight + imposed load:

$$q_d = 0,9 \left(0,89 \cdot 1,35 \cdot \left(\frac{t}{200} + 0,81 \right) + 1,5 \cdot 1 \right) = 2,23 + 1,08 \frac{t}{200} \text{ kN/m}$$

Self-weight + snow load:

$$q_d = 0,9 \left(0,89 \cdot 1,35 \cdot \left(\frac{t}{200} + 0,81 \right) + 1,5 \cdot 0,56 \right) = 1,63 + 1,08 \frac{t}{200} \text{ kN/m}$$

Self-weight + wind load (per zone):

$$\text{F: } q_d = 0,9 \left(\frac{t}{200} + 0,81 \right) - 0,9(1,5 \cdot 1,4 \cdot 0,98) = -1,12 + 0,9 \frac{t}{200} \text{ kN/m}$$

$$\text{G: } q_d = 0,9 \left(\frac{t}{200} + 0,81 \right) - 0,9(1,5 \cdot 0,9 \cdot 0,98) = -0,46 + 0,9 \frac{t}{200} \text{ kN/m}$$

$$\text{H: } q_d = 0,9 \left(\frac{t}{200} + 0,81 \right) - 0,9(1,5 \cdot 0,7 \cdot 0,98) = -0,20 + 0,9 \frac{t}{200} \text{ kN/m}$$

$$\text{I: } q_d = 0,9 \left(\frac{t}{200} + 0,81 \right) \pm 0,9(1,5 \cdot 0,2 \cdot 0,98) = 0,46 + 0,9 \frac{t}{200} \vee 0,99 + 0,9 \frac{t}{200} \text{ kN/m}$$

Given the above load combinations, the governing load combination for this structure is the self-weight plus the imposed load. This load causes a maximum bending moment halfway along the span of:

$$M_{Ed} = \frac{q_d l^2}{8} = \frac{5,4^2}{8} \cdot \left(2,23 + 1,08 \frac{t}{200} \right) = 8,13 + 0,02t \text{ kNm}$$



After choosing a certain thickness and composition of the CLT slab, the moment of inertia can be determined for the cross-section of the floor in the strong direction (assuming the strong direction is parallel to the span). This moment of inertia depends on the thicknesses of the individual layers in a certain direction and the distance from the centre of these layers to the neutral axis of the slab. The influence of the rolling shear of transverse layers is included in a reduction factor for the Steiner part of these layers. The Young's modulus parallel to the grain for the timber is assumed as $E_0 = 11.000 \text{ N/mm}^2$, and the shear modulus perpendicular to the grain (rolling shear) as $G_{90} = 50 \text{ N/mm}^2$. Additionally, the moments of inertia for different CLT types are often provided by manufacturers, but the rolling shear reduction factor in these datasheets is mostly neglected. With the moment of inertia and the maximum bending moment, the maximum bending stress in the cross-section can be calculated. The maximum bending moment occurs for a certain load combination. This leads to a k_{mod} corresponding with the shortest duration of the variable loads. The k_{mod} is found in NEN-EN 1995-1-1 Table 3.1. The imposed load has a medium-term duration, leading to $k_{mod} = 0,80$ for glued laminated timber. The upcoming CLT-part of Eurocode 5 will contain values for CLT, and in this specific case $k_{mod} = 0,80$ as well. γ_M is the partial factor for material characteristics, assumed at 1,25 for glued timber. It is found in NEN-EN 1995-1-1 Table 2.3. Assuming the usage of C24 timber to create the CLT slabs, $f_{c,k} = 24 \text{ N/mm}^2$. Then the design bending strength follows from the characteristic bending strength:

$$f_{m,d} = k_{mod} \cdot \frac{f_{m,k}}{\gamma_M} = 0,80 \cdot \frac{24}{1,25} = 15,36 \text{ N/mm}^2$$

The final result of the bending strength check is a unity check where the bending stress divided by the strength should be smaller than 1. The bending stress is equal to:

$$\sigma_{m,d} = \frac{M_{Ed}y}{I} = \frac{(8,13 + 0,02t) \cdot 10^6 \cdot 0,5t}{I} \leq 15,36$$

Type	t [mm]	Layers [mm]	I [mm ⁴]	M _{Ed} [kNm]	σ _{Ed} [N/mm ²]	UC
100L	100	40-20-40	8,27E+07	10,13	6,13	0,40
120L	120	40-40-40	1,39E+08	10,53	4,56	0,30
133L	133	29-23-29-23-29	1,63E+08	10,79	4,40	0,29
145L	145	29-29-29-29-29	2,01E+08	11,03	3,97	0,26
160L	160	40-20-40-20-40	3,04E+08	11,33	2,98	0,19

Table A3 – Strength check roof slab



Evidently, a CLT slab as thin as 100 mm thick with layers of 40-20-40 mm satisfies the strength requirements from the Eurocode, with the unity check below half of the requirement (Table A3). This might seem like an over-dimensioning, but the next checks will show why thinner CLT slabs will not be further investigated.

Fire safety (ULS, STR):

To assure the safety under fire conditions means for CLT that a strength check is carried out taking into account the burning in of the slab. These conditions are assumed as an accidental loading combination, meaning we apply the appropriate safety factors given in NEN-EN 1990.

Accidental combination:

In case of a fire in the structure, since the structural material is in sight from inside the building, part of the cross-section will burn and must be excluded from the part accounting for the leftover strength of the considered element. Assuming a timber structure will have a fire load > 500 MJ/m², the structure needs to satisfy the strength check in the accidental loading situation for 60 minutes. With a burning-in rate β_n given in NEN-EN 1995-1-2 of 0,7 mm/min for glued laminated softwood, over 60 minutes the slabs burn in over a distance of 42 mm. Comparing this distance to the layer compositions of the different CLT slabs previously analysed, the outer layer must be disregarded for every slab type in the calculation in the accidental load situation. This shows in the calculation of the reduced moment of inertia $I_{net,red}$. The load combination for the accidental situation is given in NEN-EN 1990 as:

$$q_d = \sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \vee \psi_{2,1}) Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$$

Clearly, partial factors for the dead- and live loads are ignored in this situation. The values of ψ_1 and ψ_2 are given in Table NB.2-A1.1 of the national annex of NEN-EN 1990. See Table A4 for the relevant values for the roof slab. The governing accidental load on the roof is then equal to:

Self-weight + snow load:

$$q_d = \left(\frac{t}{200} + 0,81 \right) + 0,2 \cdot 0,56 = 0,92 + \frac{t}{200} \text{ kN/m}$$

• • •

Load	ψ_1	ψ_2
<i>Imposed cat. H</i>	0	0
<i>Snow</i>	0,2	0
<i>Wind</i>	0,2	0

Table A4 – ψ -factors for buildings

The moment is determined with the standard formula for distributed loads. The net moment of inertia of the cross-section is decreased by the loss of the outer layer. With this value, the bending stress is calculated according to the same formula as in the regular bending strength check, but again the distance from the neutral axis to the outer fibre is adjusted to correspond with the decrease in cross-section.

$$M_{Ed} = \frac{q_d l^2}{8} = \left(0,92 + \frac{t}{200}\right) \frac{5,4^2}{8} \text{ kNm}$$

Type	t [mm]	Layers [mm]	$I_{\text{net,red}}$ [mm⁴]	M_{Ed} [kNm]	σ_{Ed} [N/mm²]	UC
100L	100	40-20-40	5,33E+06	5,18	19,41	1,26
120L	120	40-40-40	5,33E+06	5,54	20,78	1,35
133L	133	29-23-29-23-29	4,33E+07	5,78	5,41	0,35
145L	145	29-29-29-29-29	5,28E+07	6,00	4,94	0,32
160L	160	40-20-40-20-40	8,27E+07	6,27	3,79	0,25

Table A5 – Strength check under fire conditions

Table A5 shows a big difference between the fire resistance of three- and five layered slabs. This is explicable due to the loss of contribution of the Steiner parts to the net moment of inertia when the bottom layer is burnt up and the neutral axis of the cross-section coincides with the central axis of the top layer. The CLT slab of 120 mm thick does not satisfy the strength requirements under fire conditions.



Deformations (SLS):

Characteristic combination → instantaneous deflection:

After the strength checks, sufficient functionality of the structural components must be verified under the appropriate loading conditions. A combination of the same loads as in the ultimate limit state is applied, but all safety factors are set to 1, and K_{FI} and ξ are disregarded in the SLS. This results in:

Self-weight + imposed load:

$$q_{d,inst} = \left(\frac{t}{200} + 0,81\right) + 1 = 1,81 + \frac{t}{200} \text{ kN/m}$$

Self-weight + snow load:

$$q_{d,inst} = \left(\frac{t}{200} + 0,81\right) + 0,56 = 1,37 + \frac{t}{200} \text{ kN/m}$$

Self-weight + wind load (per zone):

$$\text{F: } q_{d,inst} = \left(\frac{t}{200} + 0,81\right) - (1,4 \cdot 0,98) = -0,56 + \frac{t}{200} \text{ kN/m}$$

$$\text{G: } q_{d,inst} = \left(\frac{t}{200} + 0,81\right) - (0,9 \cdot 0,98) = -0,07 + \frac{t}{200} \text{ kN/m}$$

$$\text{H: } q_{d,inst} = \left(\frac{t}{200} + 0,81\right) - (0,7 \cdot 0,98) = 0,12 + \frac{t}{200} \text{ kN/m}$$

$$\text{I: } q_{d,inst} = \left(\frac{t}{200} + 0,81\right) \pm (0,2 \cdot 0,98) = 0,61 + \frac{t}{200} \vee 1,01 + \frac{t}{200} \text{ kN/m}$$

Again the self-weight in combination with the imposed load gives the governing load combination. The instantaneous deflection of the roof slab is determined for the governing characteristic load combination as:

$$w_1 + w_3 = \left(\frac{5}{384}\right) \left(\frac{q_{d,inst} l^4}{EI}\right)$$

Table A6 shows the instantaneous deflections of different CLT slab configurations.

Type	t [mm]	Layers [mm]	I_{net} [mm ⁴]	$q_{d,inst}$ [N/mm]	w_1+w_3 [mm]
100L	100	40-20-40	8,27E+07	2,31	28,12
120L	120	40-40-40	1,39E+08	2,41	17,49
133L	133	29-23-29-23-29	1,63E+08	2,475	15,29
145L	145	29-29-29-29-29	2,01E+08	2,535	12,68
160L	160	40-20-40-20-40	3,04E+08	2,61	8,64

Table A6 – Instantaneous deflection of roof slab



Quasi-permanent combination → final deflection:

Service class 2 means the creep factor for CLT is $k_{def} = 1,00$. The quasi-permanent load is given in NEN-EN 1990 (6.16b). Values of ψ_2 for the live loads on the roof are given in Table A6. For this structure, this results in:

$$q_{d,fin} = \left(\frac{t}{200} + 0,81 \right)$$

The long term deflection of the roof is determined similarly as the instantaneous deflection, as:

$$w_2 = \left(\frac{5}{384} \right) \left(\frac{q_{d,fin} l^4}{EI} \right)$$

The total deflection is the sum of w_1 , w_2 and w_3 . The maximum allowable deflection according to the national annex of NEN-EN 1990 A1.4.3 is equal to $l_{rep}/250$. This results in a maximum deflection of the roof of $w_{max} \leq 5400/250 = 21,6$ mm. The results are shown in Table A7.

Type	t [mm]	Layers [mm]	I_{net} [mm ⁴]	$q_{d,fin}$ [N/mm]	w_2 [mm]	w_{max} [mm]	UC
100L	100	40-20-40	8,27E+07	1,31	15,95	44,07	2,04
120L	120	40-40-40	1,39E+08	1,41	10,23	27,73	1,28
133L	133	29-23-29-23-29	1,63E+08	1,48	9,11	24,40	1,13
145L	145	29-29-29-29-29	2,01E+08	1,54	7,68	20,36	0,94
160L	160	40-20-40-20-40	3,04E+08	1,61	5,33	13,97	0,65

Table A7 – Long-term and maximum deflection of roof slab

The minimum required composition following the deflection check for the roof slab is a 145 mm thick CLT-slab divided over 5 layers of 29 mm, but some additional checks need to be performed to assure the total safety and functionality of the roof.



Diaphragm action (ULS, EQU):

To distribute the wind force in a horizontal direction towards the shear walls, the roof needs to act as a diaphragm. Figure 17 in the main report shows the principle of this distribution, from which can be concluded that half the wind load of the top storey and the wind load on the parapet are horizontally distributed by the roof. This results in a height of 2 m. The roof has a depth of 6600 mm. Two situations are distinguished, being the wind on the side facade and wind on the front-/back facade (Figure A6). The horizontal wind load consists of two parts, the pressure on side D and the suction on side E (Figure A4). In this calculation, the friction is left out of consideration since the friction coefficients are negligible compared to the coefficients for pressure and suction.

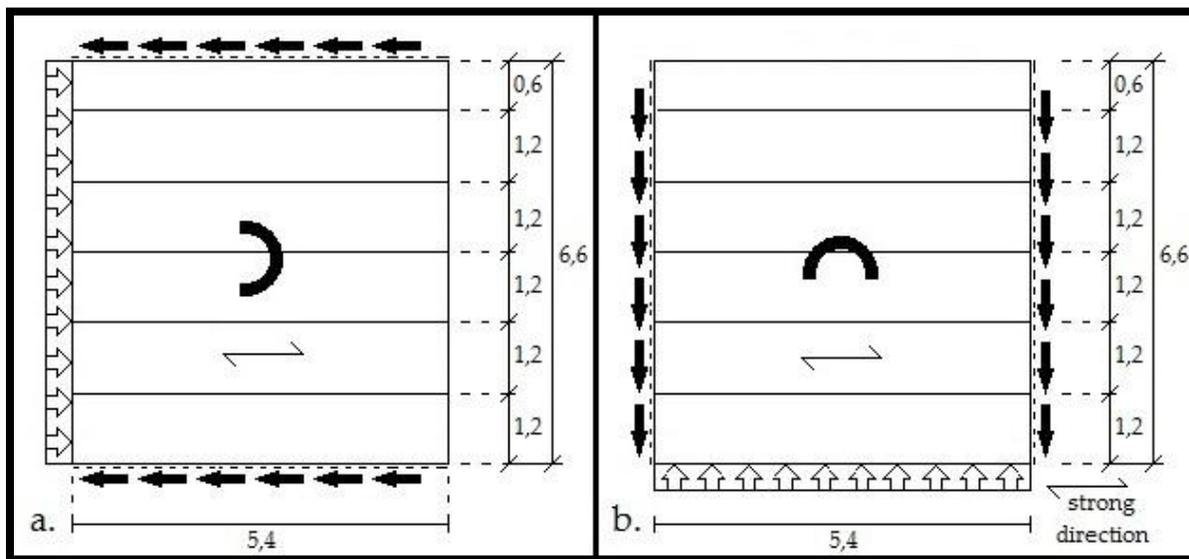


Figure A6 – Diaphragm action of roof; a. wind on side facade, b. wind on front-/back facade

According to the draft of the CLT-part for NEN-EN 1995, as long as the presence of openings in a diaphragm and a maximum spacing for fasteners has been considered, the shear forces in a diaphragm should be assumed to be uniformly distributed over the width of the diaphragm (Figure A7). This contradicts the general assumption of shear forces being distributed parabolic for uniform, rectangular cross-sections. This uniform distribution is explicable by the layered composition of CLT, meaning an introduced external force with a certain direction in a certain location is distributed in multiple directions through the different layers. The edges between boards in the same layer are not connected. The total shear resistance of the slab is therefore equal to the accumulated resistances of the independent crossing areas (Figure A8).

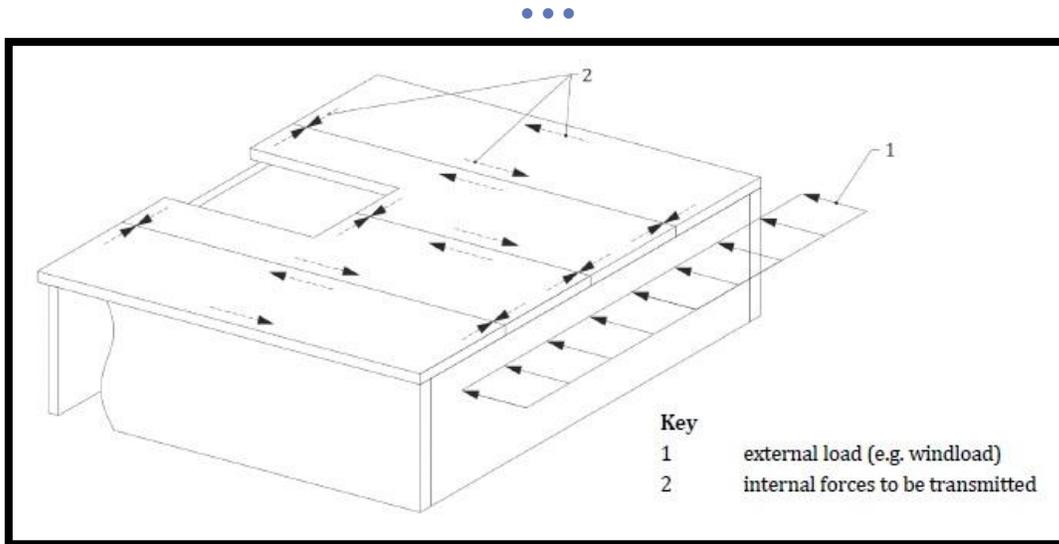


Figure A7 – Load distribution in a floor diaphragm, source: prEN16351

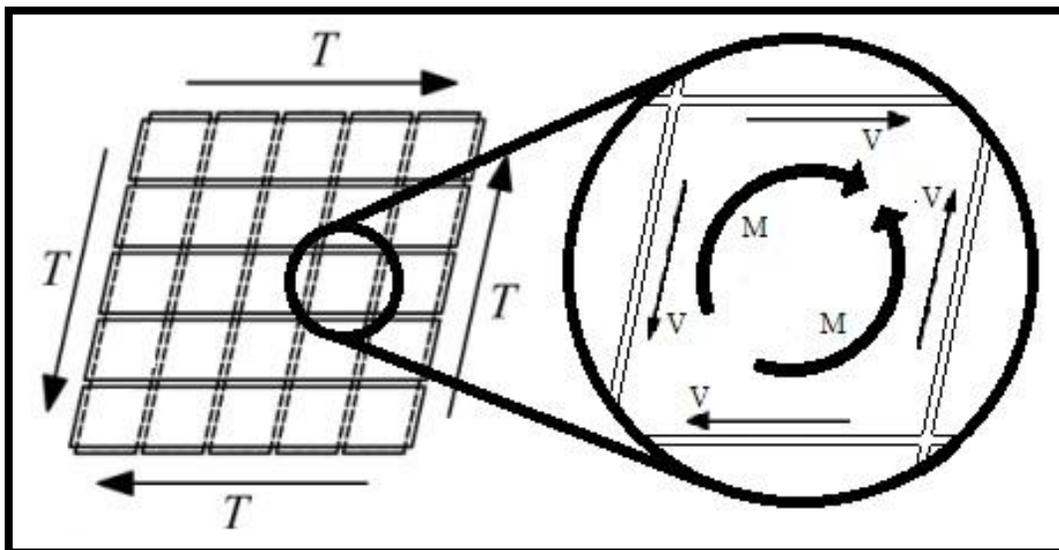


Figure A8 – Internal stresses per contact area in CLT

A minimum thickness of 145 mm was required according to the strength check under fire conditions, with composition 29-29-29-29-29. This leads to the following expressions for the shear strength (further elaborated in the Ch..5 of the main report):

$$m = \min \left\{ \begin{array}{l} \frac{5400}{150} = 36 \\ \frac{6600}{150} = 44 \end{array} \right.$$

Appendix A: Calculation CLT structure

$$f_{v,CLT} = \min \begin{cases} 3,5 \\ 5,5 \cdot \frac{58}{145} = 2,2 \vee 5,5 \cdot \frac{87}{145} = 3,3 \\ \frac{150 \cdot 4}{2 \cdot 145} \cdot \frac{1}{\frac{1}{2,5} \cdot \left(1 - \frac{1}{36^2}\right) + \frac{2}{0,8} \cdot \left(\frac{1}{36} - \frac{1}{36^2}\right)} = 4,43 \end{cases}$$

So the lowest shear strength for the total cross section is equal to 2,2 N/mm², with failure mechanism perpendicular to the grain in the weak cross-sectional direction. Failure perpendicular to the grain in the strong cross-sectional direction occurs at a stress of 3,3 N/mm² or higher. Consider the stress orientation of Figure A9. The difference in cross-sectional strength in the two directions of the CLT means the two layers of boards subject to v_{yx} fail at a lower resulting shear force than the three layers of boards subject to v_{xy} . Because v_{yx} and v_{xy} have the same value for an infinitesimal part in the slab, shear failure in the weak cross-section is inevitably the governing failure mechanism.

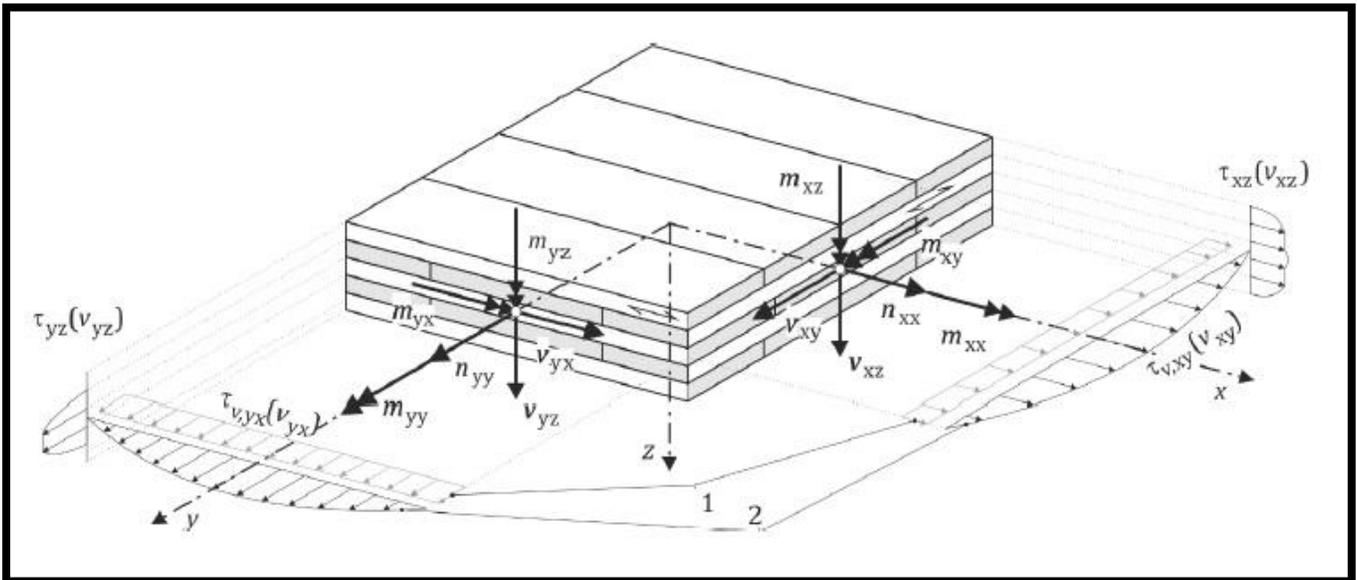


Figure A9 – Stress orientation in CLT, source: prEN16351

Examining the two different wind directions in Figure A6, the following reaction forces in the stabilizing walls are a result of the horizontal wind load on the roof:

$$H_{w,a} = Q_{d,w} \frac{6,6}{2 \cdot 5,4} = 2,33 \text{ kN/m}$$

$$H_{w,b} = Q_{d,w} \frac{5,4}{2 \cdot 6,6} = 1,56 \text{ kN/m}$$

The shear stresses following from these reaction forces, assuming a uniform distribution over the width of the slab, are equal to:

$$\tau_a = \frac{H_{w,a}}{b \cdot t_{roof}} = \frac{2,33 \cdot 10^3}{1000 \cdot 145} = 1,6 \cdot 10^{-2} \text{ N/mm}^2$$
$$\tau_b = \frac{H_{w,b}}{b \cdot t_{roof}} = \frac{1,56 \cdot 10^3}{1000 \cdot 145} = 1,1 \cdot 10^{-2} \text{ N/mm}^2$$

The uniform shear stresses are $\ll 2,2 \text{ N/mm}^2$. The diaphragm action of the roof is assured in both directions for the governing wind load.



Floor slabs:

The only considered variable load for these slabs is the imposed load, with a value of 1,75 kN/m². This value is found in NEN-EN1991-1-1, Table NB.1 – 6.2, in this case for category A: floors of residential areas.

Therefore the combination of permanent- and imposed load is also the governing combination.

Additional to the self-weight of the CLT slab a permanent load of 0,5 kN/m² is taken into account for internal walls. This means the total permanent load is equal to:

$$G_{k,j} = \frac{t}{200} + 0,5$$

Strength (ULS, STR):

Characteristic combination:

Permanent load + imposed load:

$$q_d = 0,9 \cdot \left(0,89 \cdot 1,35 \cdot \left(\frac{t}{200} + 0,5 \right) + 1,5 \cdot 1,75 \right) = 2,9 + 1,08 \frac{t}{200} \text{ kN/m}$$

$$M_{Ed} = \frac{q_d l^2}{8} = \frac{5,4^2}{8} \cdot \left(2,9 + 1,08 \frac{t}{200} \right) = 10,57 + 0,02t \text{ kNm}$$

$$\sigma_{m,d} = \frac{M_{Ed} y}{I} = \frac{(10,57 + 0,02t) \cdot 10^6 \cdot 0,5t}{I} \leq 15,36$$

Type	t [mm]	Layers [mm]	I _{net} [mm ⁴]	M _{ed} [kNm]	σ _{Ed} [N/mm ²]	UC
100L	100	40-20-40	8,27E+07	12,57	7,60	0,49
120L	120	40-40-40	1,39E+08	12,97	5,60	0,36
133L	133	29-23-29-23-29	1,63E+08	13,23	5,40	0,35
145L	145	29-29-29-29-29	2,01E+08	13,47	4,86	0,32
160L	160	40-20-40-20-40	3,04E+08	13,77	3,62	0,24

Table A8 – Strength check floor slab

In this calculation it is assumed there are no holes in the floors. However, for ventilation purposes through the building, the contractor desires a shaft running through the house from the ground floor till the roof. An additional hole for stairs is required for inhabitants to reach the first- and second floor.

The CLT slabs around the holes need to be able to carry the loads on the CLT parts that do not span the full length between the supporting walls due to the existence of the holes, depicted as the non-spanning areas in Figure A10.

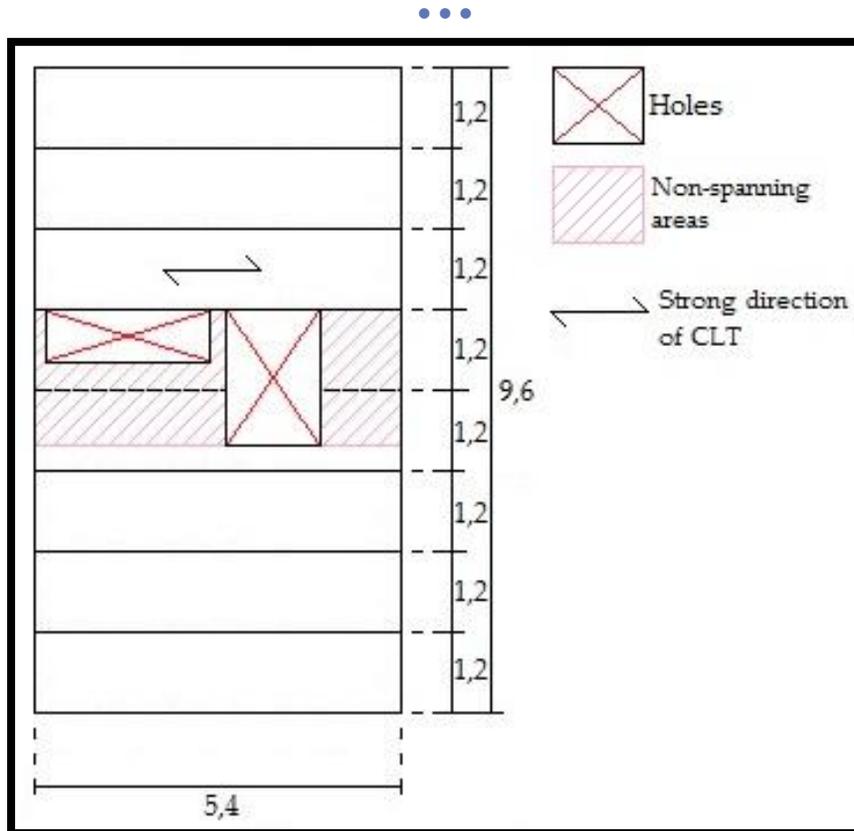


Figure A10 – Holes in floor slabs

Considering the load from the non-spanning areas to redistribute to attached spanning floor parts, assuming a redistribution width of approximately 1 meter, this leads to a load increase over the full span which must be smaller than 100% of the current load. Multiplying the unity check by a factor 2 shows all floor types would still suffice for the strength requirement.



Fire safety (ULS, STR):

Accidental combination:

Again, the fire safety is determined in the accidental loading situation. A similar burning rate is assumed as the burning rate for the roof, meaning after 60 minutes again a thickness of 42 mm is lost. For all considered slab types this results in the loss of one outer layer, which shows in the calculation of the reduced moment of inertia $I_{\text{net,red}}$.

Again, the load combination for the accidental situation is given in NEN-EN 1990 as:

$$q_d = \sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \vee \psi_{2,1}) Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$$

Table NB.2-A1.1 of the national annex of NEN-EN 1990 gives a value ψ_1 of 0,5 and ψ_2 of 0,3 for category A imposed loads (habitable spaces). ψ_1 is governing, leading to a load of:

Self-weight + imposed:

$$q_d = \left(\frac{t}{200} + 0,5 \right) + 0,5 \cdot 1,75 = 1,375 + \frac{t}{200} \text{ kN/m}$$

$$M_{Ed} = \frac{q_d l^2}{8} = \left(1,375 + \frac{t}{200} \right) \frac{5,4^2}{8} \text{ kNm}$$

The unity checks are found in Table A9.

Type	t [mm]	Layers [mm]	$I_{\text{net,red}}$ [mm ⁴]	M_{Ed} [kNm]	σ_{Ed} [N/mm ²]	UC
100L	100	40-20-40	5,33E+06	6,83	25,63	1,67
120L	120	40-40-40	5,33E+06	7,20	27,00	1,76
133L	133	29-23-29-23-29	4,33E+07	7,44	6,96	0,45
145L	145	29-29-29-29-29	5,28E+07	7,65	6,30	0,41
160L	160	40-20-40-20-40	8,27E+07	7,93	4,80	0,31

Table A9 – Strength check floor slabs under fire conditions



Deformations (SLS):

Characteristic combination → instantaneous deflection:

Permanent load + imposed load:

$$q_{d,inst} = \left(\frac{t}{200} + 0,5\right) + 1,75 = 2,25 + \frac{t}{200} \text{ kN/m}$$

$$w_1 + w_3 = \left(\frac{5}{384}\right) \left(\frac{q_{d,inst} l^4}{EI}\right)$$

The instantaneous deflections for the different CLT floor types are found in Table A10.

Type	t [mm]	Layers [mm]	I _{net} [mm ⁴]	q _{d,inst} [N/mm]	w ₁ +w ₃ [mm]
100L	100	40-20-40	8,27E+07	2,75	33,48
120L	120	40-40-40	1,39E+08	2,85	20,69
133L	133	29-23-29-23-29	1,63E+08	2,915	18,01
145L	145	29-29-29-29-29	2,01E+08	2,975	14,88
160L	160	40-20-40-20-40	3,04E+08	3,05	10,10

Table A10 – Instantaneous deflection of floor slabs

Quasi-permanent combination → long-term deflection:

Stated previously for CLT is k_{def} = 1,00, ψ₂ = 0,3. For this structure, this results in:

$$q_{d,fin} = \left(\frac{t}{200} + 0,5\right) + 0,3 \cdot 1,75 = 1,025 + \frac{t}{200}$$

The long-term deflection of the floor is determined similarly as the instantaneous deflection, as:

$$w_2 = \left(\frac{5}{384}\right) \left(\frac{q_{d,fin} l^4}{EI}\right)$$

The total deflection is the sum of w₁, w₂ and w₃. The maximum allowable deflection according to the national annex of NEN-EN 1990 A1.4.3 is equal to l_{rep}/250. This results in a maximum deflection of the roof of w_{max} ≤ 5400/250 = 21,6 mm. The results are shown in Table A11.

Type	t [mm]	Layers [mm]	I _{net} [mm ⁴]	q _{d,fin} [N/mm]	w ₂ [mm]	w _{max} [mm]	UC
100L	100	40-20-40	8,27E+07	1,53	18,57	52,05	2,41
120L	120	40-40-40	1,39E+08	1,63	11,79	32,48	1,50
133L	133	29-23-29-23-29	1,63E+08	1,69	10,44	28,45	1,32
145L	145	29-29-29-29-29	2,01E+08	1,75	8,75	23,64	1,09
160L	160	40-20-40-20-40	3,04E+08	1,83	6,04	16,14	0,75

Table A11 – Long-term and maximum deflection of floor slabs

Following the deflection check, a minimum CLT-thickness is required for the floor of 160 mm.



Vibrations from footfall (SLS):

The basic requirement on floor vibrations were stated in 4.3.2. Since the sum of the permanent and ψ_2 times the imposed load is smaller than 5 kN/m², the first natural frequency needs to be bigger than 3 Hz.

For CLT, the requirements regarding the dynamic behaviour of a floor depends on the desired functionality of the floor, are expressed as the floor performance level. High quality residential areas are aspired within this project, therefore demanding a floor performance level to be level III or better. Depending on the first fundamental frequency of the floor, which is either higher or lower than 4,5 or 8 Hz, an acceleration criterion or two criteria regarding velocity and stiffness need to be fulfilled to reach the floor performance level, respectively (See Table A12).

Criteria	Floor performance levels					
	I	II	III	IV	V	VI
Frequency f_1 [Hz] \geq	4,5			8		
Response factor R	4	8	12	16	20	24
Acceleration criteria when $f_1 < 8$ [Hz] a_{rms} [m/s ²] \leq	0,005·R			(NA)		
Velocity criteria when $f_1 \geq 8$ [Hz] v_{rms} [m/s] \leq	0.0001·R					
Stiffness criteria when $f_1 \geq 8$ [Hz] w_{1kN} [mm] \leq	0,25		0,5	0,8	1,2	1,6

Table A12 - Floor vibration criteria; source: subtask 7 for prEN16351 Table 9.2

The fundamental frequency is determined through the draft for the CLT part of EN1995:

$$f_1 = k_{e,1} k_{e,2} \frac{18}{\sqrt{\delta_{sys}}}$$

In this formula, $k_{e,1}$ is a multiplication factor in the case of a double span floor on supports, shown in Table A11. Since the floor in this project is spanning in only one direction, the biggest value of 1,41 for $k_{e,1}$ is assumed.

l_2/l ($l > l_2$)	1,0	0,9	0,8	0,7	0,6	0,5	0,4	0,3	0,2
$k_{e,1}$	1,00	1,09	1,16	1,21	1,25	1,28	1,32	1,36	1,41

Table A13 – factor $k_{e,1}$ for fundamental frequency of two-span floor; source: subtask 7 for prEN16351 Table 9.1



$k_{e,2}$ is a multiplication factor to take into account the stiffness characteristics in the two main directions of the floor, determined as:

$$k_{e,2} = \sqrt{1 + \left(\frac{l}{b}\right)^4 \frac{(EI)_T}{(EI)_L}}$$

Where l is the span of the floor, b is the width of the floor, $(EI)_L$ is the effective bending stiffness in the direction of the span and $(EI)_T$ is the effective bending stiffness in the transverse direction to the span. δ_{sys} is the deflection at midspan due to the permanent loads. As determined earlier, the permanent load is equal to:

$$G_{k,j} = \frac{t}{200} + 0,5$$

The fundamental frequencies per floor type are shown in Table A14.

Type	Layers [mm]	t [mm]	$k_{e,1}$	I_L [mm ⁴]	I_T [mm ⁴]	$k_{e,2}$	$G_{k,j}$ [kN/m]	δ_{sys} [mm]	f_1 [Hz]
120L	40-40-40	120	1,41	1,39E+08	5,33E+06	1,001919	1,1	7,97	9,01
133L	29-23-29-23-29	133	1,41	1,63E+08	3,31E+07	1,010121	1,165	7,19	9,56
145L	29-29-29-29-29	145	1,41	2,01E+08	5,28E+07	1,013074	1,225	6,13	10,38
160L	40-20-40-20-40	160	1,41	3,04E+08	3,73E+07	1,006129	1,3	4,30	12,31

Table A14 – Fundamental frequency floor slab

The frequencies seem relatively high, but this was expected for lightweight slabs. A minimum floor performance level of III is required. Therefore, the floor must meet two criteria; velocity and stiffness.

The draft of the CLT-part for NEN-EN 1995 gives the following formula for the deflection:

$$w_{1kN} = \frac{Fl^3}{48(EI)_L b_{ef}}$$

$$b_{ef} = \min \left\{ \begin{array}{l} l \\ 1,1 \sqrt{\frac{(EI)_T}{(EI)_L}} \\ b \end{array} \right.$$

Type	Layers [mm]	t [mm]	I_L [mm ⁴]	b_{ef} [m]	w_{1kN} [mm]	UC
120L	40-40-40	120	1,39E+08	2,172686	0,99	1,97
133L	29-23-29-23-29	133	1,63E+08	3,296016	0,56	1,11
145L	29-29-29-29-29	145	2,01E+08	3,515188	0,42	0,84
160L	40-20-40-20-40	160	3,04E+08	2,906075	0,34	0,68

Table A15 – Deflection due to 1kN point load halfway the span

Appendix A: Calculation CLT structure

• • •

Table A15 shows the unity checks of the stiffness for floor performance level III. A slab of at least 145 mm thick satisfies the required conditions. The specific velocity criterion is not available for CLT in the latest draft of the CLT-part for NEN-EN 1995. The regular NEN-EN 1995 gives a similar approach for the serviceability with regard to the vibrations in residences, with the requirement for the velocity as:

$$v \leq b^{(f_1 \xi - 1)} (= 120^{(f_1 \xi - 1)}) \text{ m/(Ns}^2\text{)}$$

A damping ratio (ξ) of 0,01 (1%) is given as an assumption in NEN-EN 1995. No formula is available to determine the exact velocity of the specific floor dimensions and composition in this project, but NEN-EN 1995 gives an approximation formula for rectangular floors, simply supported on all sides:

$$v = \frac{4(0,4 + 0,6n_{40})}{mbl + 200}$$

$$n_{40} = \left\{ \left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{b}{l} \right)^4 \frac{(EI)_l}{(EI)_b} \right\}^{0,25}$$

Where n_{40} describes the number of first order vibrations with a natural frequency smaller than 40 Hz and m is the mass of the CLT slab per square meter. The results are given in Table A16.

Type	Layers	t [mm]	I _L [mm ⁴]	I _T [mm ⁴]	f ₁ [Hz]	n ₄₀	v [m/Ns ²]	v _{adm} [m/Ns ²]	UC
120L	40-40-40	120	1,39E+08	5,33E+06	9,01	8,35	6,54E-03	1,28E-02	0,51
133L	29-23-29-23-29	133	1,63E+08	3,31E+07	9,56	5,34	3,95E-03	1,32E-02	0,30
145L	29-29-29-29-29	145	2,01E+08	5,28E+07	10,38	4,79	3,31E-03	1,37E-02	0,24
160L	40-20-40-20-40	160	3,04E+08	3,73E+07	12,31	5,28	3,28E-03	1,50E-02	0,22

Table A16 – Floor vibration velocity

The final analysed design method to control vibrations in CLT is again related to the fundamental frequency and the static deflection due to 1kN point load (L. Hu & Gagnon, 2012; L. J. Hu & Chui, 2004). Two requirements were set in 2004 and 2012 as respectively:

$$\frac{f}{d^{0,44}} \geq 18,7 \text{ and } \frac{f}{d^{0,7}} \geq 13$$

The deflection due to 1kN point load was previously determined as w_{1kN} . The unity checks for this design method are shown in Table A17. A floor of 160 mm thick fulfils both requirement.

Type	Layers	t [mm]	f ₁ [Hz]	w _{1kN} [mm]	f/d ^{0,44}	f/d ^{0,7}
120L	40-40-40	120	9,01	0,99	9,06	9,09
133L	29-23-29-23-29	133	9,56	0,56	12,38	14,43
145L	29-29-29-29-29	145	10,38	0,42	15,17	18,99
160L	40-20-40-20-40	160	12,31	0,34	19,85	26,32

Table A17 – Additional vibration control design method



Besides the requirements in the different parts of the Eurocode, CLT-producers often supply information regarding the allowed span with certain slab thicknesses. According to Dutch company Solid Timber, related to Swedish CLT-manufacturer Martinsons, creating a span of 5,4 metres for class-A buildings (houses) is achievable with a thickness of a CLT slab of at least 180mm. However, the information indicates this thickness is derived from the deflection calculations, which have already been checked previously. No additional information on the vibration of CLT-floors is given by producers.

Research has shown some simple modifications to a CLT floor to improve the vibration properties (Labannotte & Malo, 2010). The improvements in performance are expressed in a difference in the vibration parameter described by Hu and Chui (L. J. Hu & Chui, 2004). Self-explanatory modifications are e.g. the increase of cross-sectional properties in the direction of the span or an improvement in material characteristics like choosing a higher strength grade timber for the outer layers. Less obvious modifications are the application of inner layers with higher shear stiffness', or changing the direction of the cross layers by 45 degrees. Additionally, the configuration of floor-slabs may have an influence on the vibration properties of the system. The panels are assumed to have a certain width, which automatically leads to a certain configuration. Depending on possibilities with regard to transportation purposes, different panel widths or composition will lead to different vibration scenarios.

Further research on this topic would be advised to get a total understanding and clearer requirements for the design of CLT-floors with respect to vibrations. Testing of different floor configurations will give insight in those inspected cases, but a uniform expression is desired to cover the total spectrum of possibilities within CLT design.

The design of the foundation is left out of consideration in this project. However, in this design the vibrations of the surrounding soil and the foundation itself should be included to cover all dynamic loads leading to vibrations within the structure. The foundation, assuming it is made out of concrete for durability reasons, adds much weight to the total structure, and therefore an analysis of vibrations from surrounding soil on the CLT structure is not relevant without knowing what the foundation looks like.



Diaphragm action (ULS, EQU):

Just like with the roof, for the distribution of the wind force in a horizontal direction towards the stabilizing shear walls, the floors need to act as a diaphragm. The principle is equal to the diaphragm action of the roof, except for the dimensions (see Figure A11).

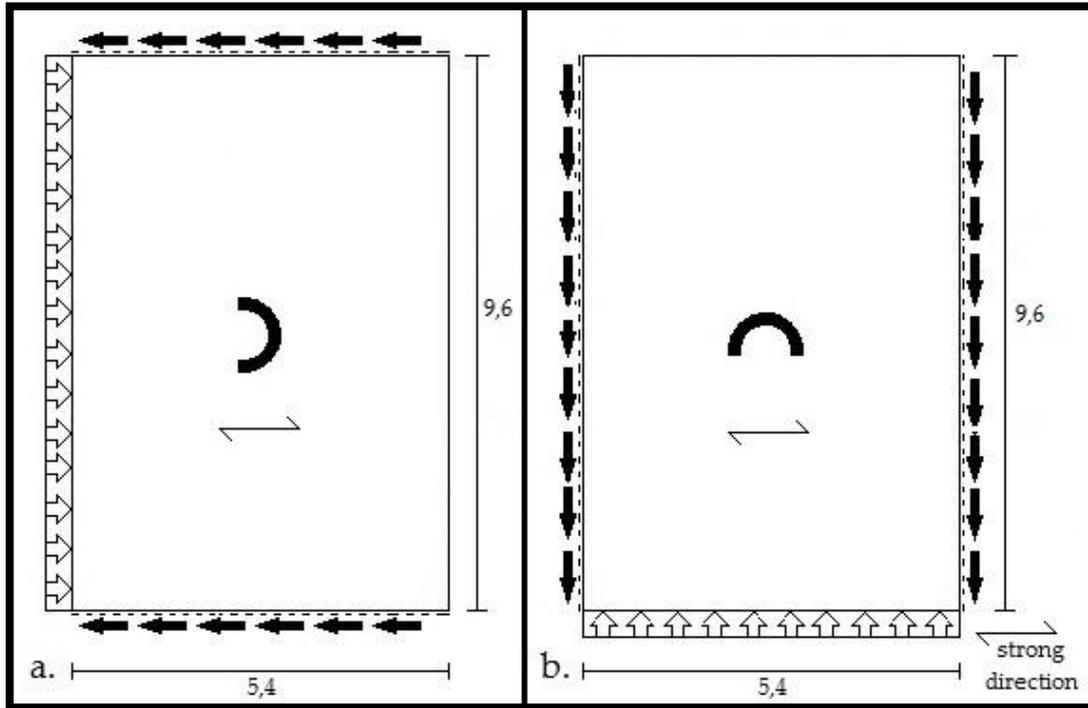


Figure A11 – Diaphragm action floor; a. wind on side facade, b. wind on front-/back facade

The floor slabs have a different thickness than the roof slab, so again the shear strength for the panel must be determined. A minimum thickness of 160 mm was required from the deformation check in the SLS, with composition 40-20-40-20-40. This leads to the following expressions for the shear strength:

$$m = \min \begin{cases} \frac{5400}{150} = 36 \\ \frac{9600}{150} = 64 \end{cases}$$

$$f_{v,CLT} = \min \begin{cases} 3,5 \\ 5,5 \cdot \frac{40}{160} = 1,375 \vee 5,5 \cdot \frac{120}{160} = 4,125 \\ \frac{150 \cdot 4}{2 \cdot 160} \cdot \frac{1}{\frac{1}{2,5} \cdot \left(1 - \frac{1}{36^2}\right) + \frac{2}{0,8} \cdot \left(\frac{1}{36} - \frac{1}{36^2}\right)} = 4,01 \end{cases}$$

Again, the governing failure mechanism is shear perpendicular to the grain in the weak cross-section, with shear strength for the total cross section being equal to 1,375 N/mm².



The value of the wind load per running meter on the floor slabs are equal, since they both conduct the wind load from an equal area of the facade. This area has a height of 3 metres, 1,5 metres above the floor and 1,5 metres below the floor. This results in a horizontal load on the floor of:

$$Q_{d,w} = \gamma_{Q,w} (p_w (C_{pe,10,D} + C_{pe,10,E}) h) = 1,5 \cdot (0,98 \cdot (0,8 + 0,5) \cdot 3) = 5,73 \text{ kN/m}$$

Examining the two different wind directions in Figure A11, the following reaction forces in the stabilizing walls are a result of the horizontal wind load on the floor:

$$H_{w,a} = Q_{d,w} \frac{9,6}{2 \cdot 5,4} = 5,1 \text{ kN/m}$$

$$H_{w,b} = Q_{d,w} \frac{5,4}{2 \cdot 9,6} = 1,61 \text{ kN/m}$$

The maximum shear stresses in the floor slab following from these reaction forces, assuming a uniform distribution over the width of the slab, are equal to:

$$\tau_a = \frac{H_{w,a}}{b \cdot t_{roof}} = \frac{5,1 \cdot 10^3}{1000 \cdot 160} = 3,19 \cdot 10^{-2} \text{ N/mm}^2$$

$$\tau_b = \frac{H_{w,b}}{b \cdot t_{roof}} = \frac{1,61 \cdot 10^3}{1000 \cdot 160} = 1 \cdot 10^{-2} \text{ N/mm}^2$$

The uniform shear stresses are $\ll 1,375 \text{ N/mm}^2$. The diaphragm action of the roof is assured in both directions for the governing wind load.

The holes in the floor slabs may lead to significant local stress peaks, where reinforcement may be required. The stress distributions are best analysed through the application of the Finite Element Method (FEM). A possible good approach within the scope of this method is to model the panel as separate boards with elastic material properties in each orthogonal direction of the lamella. The glue layer is then modelled in between through the application of contact elements with a high frictional coefficient (Figure A12). This way of modelling is a sufficient approach of reality for CLT-panels without holes (Shahnewaz, Tannert, Alam, & Popovski, 2015). New research is required to verify whether the FE models with holes are acceptable by comparing results from modelling with results from experiments.

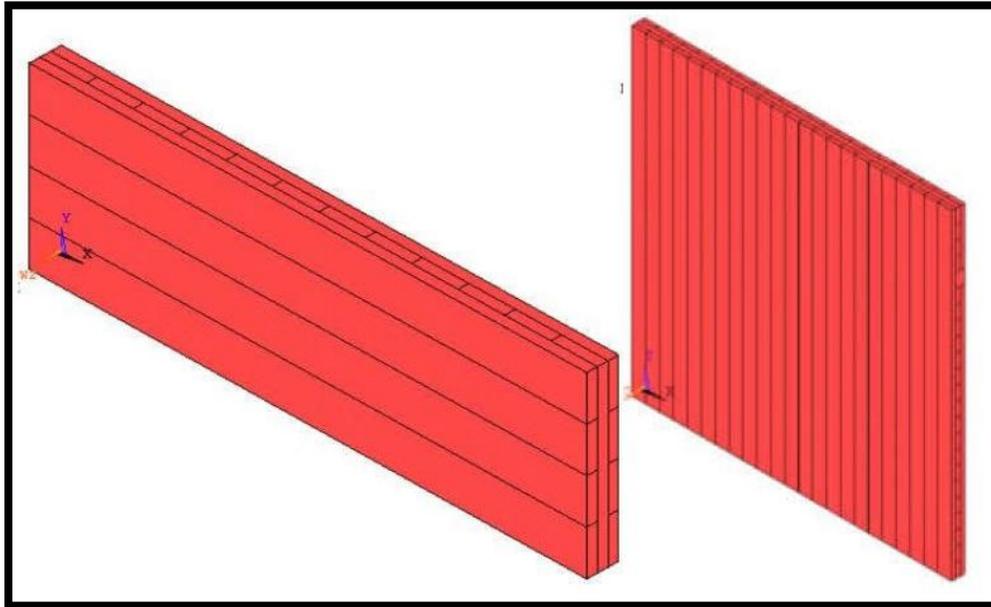


Figure A12 – FE-models of beams and walls, source: (Shahnewaz et al., 2015)

To theoretically approach the presence of holes in the floor slabs, some basic assumptions can simplify the problem to investigate a minimum value of the resistance. Neglecting the non-spanning areas in Figure A13 leads to the assumption that only the spanning parts of the floor contribute to the diaphragm action. The reaction forces in the walls supporting the horizontal loads can be seen in Figure A13.a for wind on the side façade, and in Figure A13.b for wind on the front-/back façade.

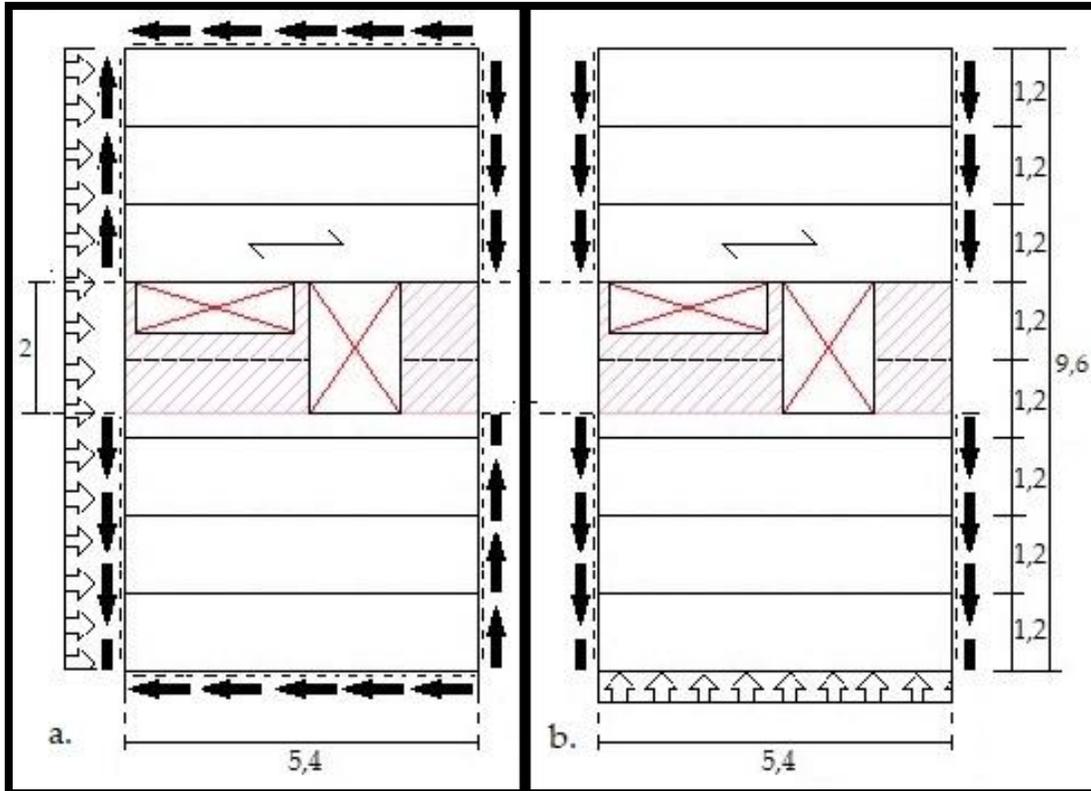


Figure A13 – Diaphragm with holes in slab; a. wind on side façade, b. wind on front-/back facade

The maximum shear stress for situation b. in Figure A13 is calculated by considering the reaction force only works over a distance of 7,6 meter instead of the original 9,6 meter.

$$H_{w,b} = 5,73 \cdot \frac{5,4}{2 \cdot 7,6} = 2,04 \text{ kN/m}$$

$$\tau_b = \frac{H_{w,b}}{b \cdot t_{roof}} = \frac{2,04 \cdot 10^3}{1000 \cdot 160} = 1,3 \cdot 10^{-2} \text{ N/mm}^2$$

This stress is still well below the shear strength of the CLT. For situation a. the shear stress is calculated differently, since the division of the floor slab in two parts introduces a bending moment due to the eccentricity of the load compared to the stabilizing wall. In Figure A13.a, the bottom part is governing for this calculation since it has larger dimensions than the top part, featured in Figure A14. The wind load on the left side of the building leads to a resulting force at 2,5 meters away from the stabilizing wall. This force leads to a counterpart in the front façade and therefore to an in-plane moment equal to the force times 2,5, all shown in red in Figure A14. To counter the in-plane moment due to the red forces, two reaction forces in the side facades are present, shown in blue. The reaction forces in the walls and shear stress in the diaphragm are calculated as:

Appendix A: Calculation CLT structure

• • •

$$H_{w,a,front} = Q_{d,w} \frac{5}{5,4} = 5,31 \text{ kN/m}$$

$$H_{w,a,side} = H_{w,a,front} \frac{2,5}{5,4} = 2,46 \text{ kN/m}$$

$$\tau_a = \frac{H_{w,a,front} + H_{w,a,side}}{b \cdot t_{roof}} = \frac{(5,31 + 2,46) \cdot 10^3}{1000 \cdot 160} = 4,85 \cdot 10^{-2} \text{ N/mm}^2$$

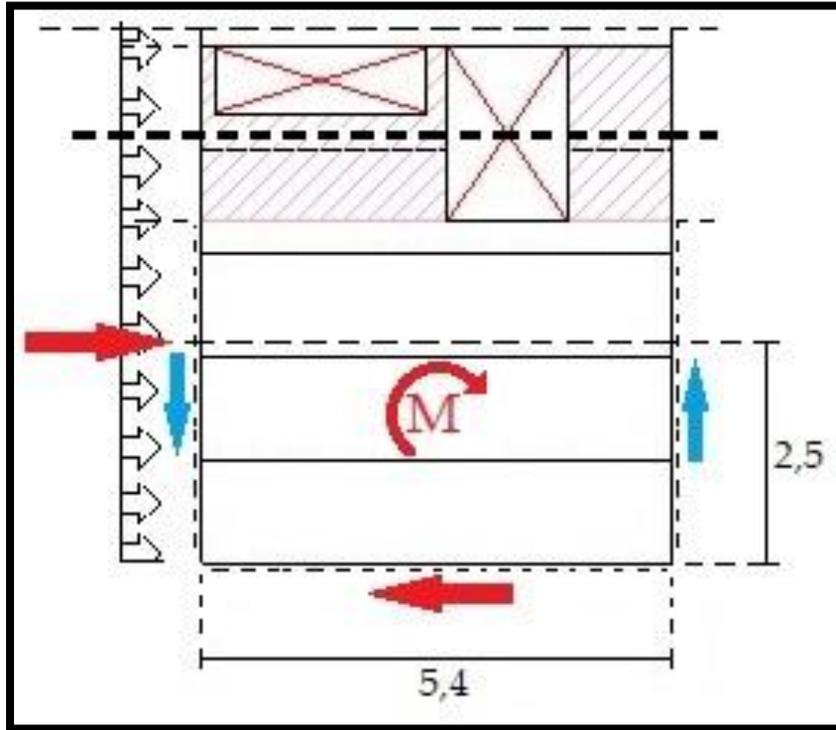


Figure A14 – Floor diaphragm detail



House-separating, load-bearing wall slabs:

The walls that separate the houses are also the walls that carry all the vertical loads from roof and floors and convey them towards the foundation. For the structural verification of the vertical load bearing walls, first the reaction forces in the supports of the floors and roof are determined. To increase the repetition factor for the project, all load-bearing wall slabs over the full height and depth of the houses are assumed to be similar in composition (layering), thickness and width.

Strength (ULS, STR):

Two strength checks are required for the load bearing walls; the maximum downward loading situation and the maximum upward lifting loading situation. The maximum reaction forces for different loads need to be determined from the roof- and floor slabs on the load-bearing walls. The reaction forces will be determined for the different loads separately instead of for the total load situations, because this allows for separate application of the safety factors, and may result in different critical load combination.

Self-weight:

$$G_{roof} = \left(\frac{t_{roof}}{200} + 0,81 \right) \cdot \frac{5,4}{2} = 2,19 + \frac{2,7t_{roof}}{200} \text{ kN/m}$$

$$G_{floor} = \left(\frac{t_{floor}}{200} + 0,5 \right) \cdot \frac{5,4}{2} = 1,35 + \frac{2,7t_{floor}}{200} \text{ kN/m}$$

Imposed load:

$$Q_{roof} = 1 \cdot \frac{5,4}{2} = 2,7 \text{ kN/m}$$

$$Q_{floor} = 1,75 \cdot \frac{5,4}{2} = 4,73 \text{ kN/m}$$

Snow load:

$$Q_s = 0,56 \cdot \frac{5,4}{2} = 1,51 \text{ kN/m}$$

Maximum uplifting and downward vertical wind loads:

$$Q_{w,uplift} = 1,4 \cdot 0,98 \cdot 2,5 + 0,9 \cdot 0,98 \cdot \left(\frac{5,4}{2} - 2,5 \right) = 3,61 \text{ kN/m}$$

$$Q_{w,downward} = 0,2 \cdot 0,98 \cdot \left(\frac{5,4}{2} \right) = 0,53 \text{ kN/m}$$



Depending on the wind-direction, the maximum upward and downward wind loads are located at the front and the back of the house respectively, or the other way around (Figure A3). The strength checks with these loads are thus only local. The self-weight of the walls are required to determine some of the local loads:

$$G_{wall} = \left(\frac{t_{wall}}{200}\right) \cdot 3 = \frac{3t_{wall}}{200} \text{ kN/m}$$

The maximum tensile stress in case of the maximum uplifting wind load occurs in the top of the wall at the 2nd floor (see Figure A15), because the downward, accumulated self-weight of the supported elements in this point is at a minimum, limited to only the weight of the roof. In this situation, the tensile force in the load bearing walls is equal to the combination of the self-weight of the roof and the maximum uplifting wind load on the roof:

$$q_{d,up} = 0,9 \cdot 1,5Q_{w,uplift} - 0,9G_{roof} = 0,9 \cdot 1,5 \cdot 3,61 - 0,9 \cdot \left(2,19 + \frac{2,7t_{roof}}{200}\right) \text{ kN/m}$$

Note: the value for the uplifting load is calculated per running meter wall. The cross-sectional properties, like A_{net} and I_{net} are also determined per running meter. This must be taken into account when performing further calculations.

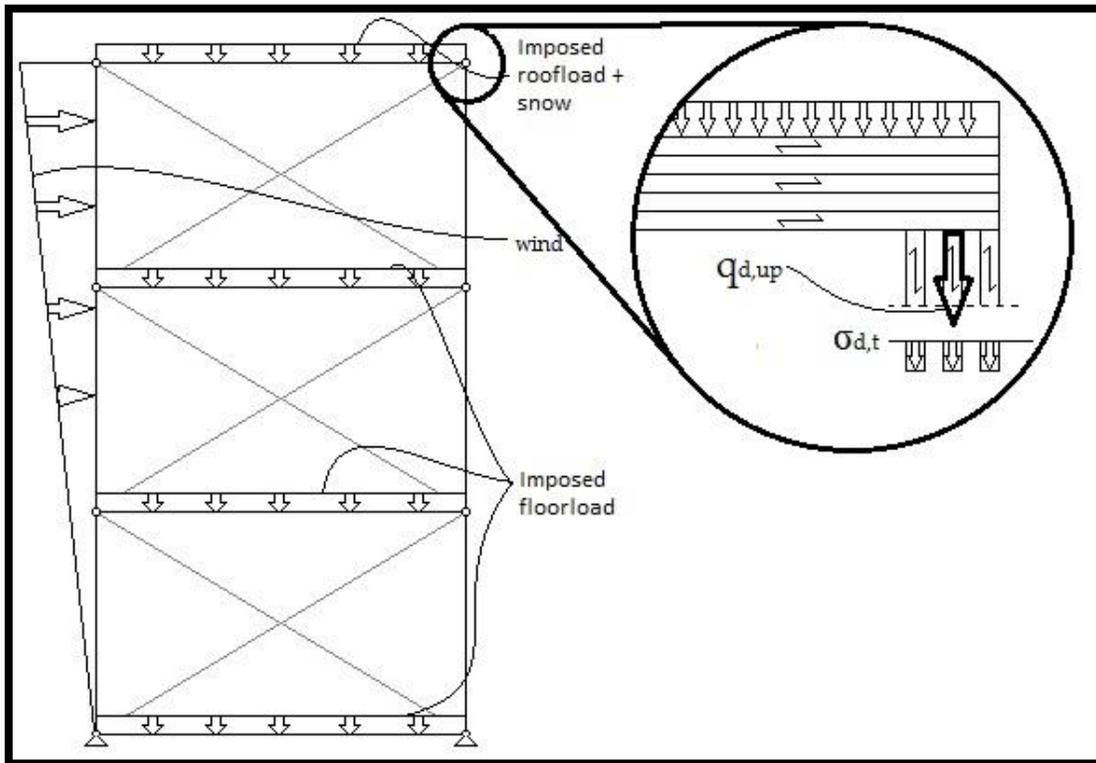


Figure A15 – Detail roof/wall connection



The design tensile strength of the applied timber is equal to:

$$f_{t,0,d} = k_{mod} \frac{f_{t,0,k}}{\gamma_M} = 0,8 \cdot \frac{17}{1,25} = 10,88 \text{ N/mm}^2$$

The roof thickness was previously set at 145mm. The tensile stress is calculated as:

$$\sigma_{t,0,d} = \frac{q_d}{A_{net}}$$

The maximum compressive stress in case of the maximum downward loading situation occurs in the bottom part of the wall at ground level (see Figure A16). This is the location in the structure where most of the self-weight of individual elements is accumulated. The compressive load in this location is equal to the accumulated self-weights and a combination of the variable live loads:

$$q_{d,down} = 0,9 \cdot 1,5Q_k + 0,9 \cdot 0,89 \cdot 1,35G_k$$

$$G_k = 3 \cdot G_{wall} + 2 \cdot G_{floor} + G_{roof}$$

The combination of the variable loads needs to be determined by the means of the combination factors ψ_0 from NEN-EN 1990. These factors for the live loads on the roof are all equal to 0, whereas ψ_0 for the imposed load on the floor is 0,4. The governing variable roof load is the imposed load.

$$Q_k = \max \begin{cases} Q_{roof} + 2 \cdot 0,4Q_{floor} = 2,7 + 2 \cdot 0,4 \cdot 4,73 = 6,48 \\ Q_{floor} + 0,4Q_{floor} = 4,73 + 0,4 \cdot 4,73 = 6,62 \end{cases} \text{ kN/m}$$

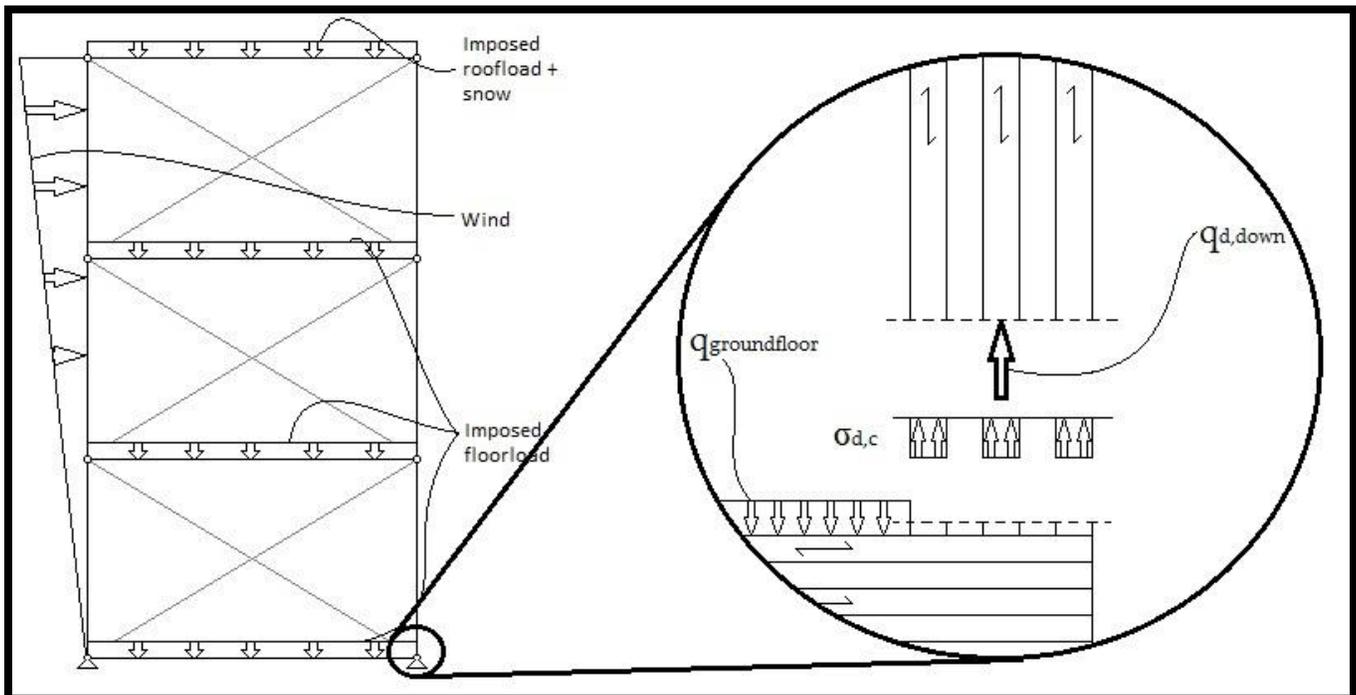


Figure A16 – Detail wall/foundation connection

Appendix A: Calculation CLT structure

• • •

The wall slabs are all executed in the same thickness to avoid confusion during construction. Previous calculations set the roof thickness at 145 mm and the floor thickness at 160 mm. The design compressive strength in these checks are equal to:

$$f_{c,0,d} = k_{mod} \frac{f_{c,0,k}}{\gamma_M} = 0,8 \cdot \frac{21,5}{1,25} = 13,76 \text{ N/mm}^2$$

Type	t [mm]	Layers [mm]	A _{net} [mm ²]	I _{net} [mm ⁴]	q _{d,up} [kN/m]	σ _{t,0,d} [N/mm ²]	UC
98W	98	29-40-29	5,80E+04	7,31E+07	1,71	0,03	0,00
103W	103	40-23-40	8,00E+04	9,01E+07	1,65	0,02	0,00
120W	120	40-40-40	8,00E+04	1,39E+08	1,44	0,02	0,00
133W	133	29-23-29-23-29	8,70E+04	1,63E+08	1,29	0,01	0,00
145W	145	29-29-29-29-29	8,70E+04	2,01E+08	1,14	0,01	0,00

Table A18 – Load-bearing wall tension check

Type	t [mm]	Layers [mm]	A _{net} [mm ²]	I _{net} [mm ⁴]	F _G [kN/m]	q _{d,down} [kN/m]	σ _{c,0,d} [N/mm ²]	UC
98W	98	29-40-29	5,80E+04	7,31E+07	15,58	25,78	0,44	0,03
103W	103	40-23-40	8,00E+04	9,01E+07	15,80	26,03	0,33	0,02
120W	120	40-40-40	8,00E+04	1,39E+08	16,57	26,85	0,34	0,02
133W	133	29-23-29-23-29	8,70E+04	1,63E+08	17,15	27,48	0,32	0,02
145W	145	29-29-29-29-29	8,70E+04	2,01E+08	17,69	28,07	0,32	0,02

Table A19 – Load-bearing wall compression check

Wall compositions are checked for tension in Table A18 and for compression in Table A19. The unity checks are <<1, so the requirements are easily met for all wall types.



Stability → Buckling (ULS, EQU):

With regard to the stability of the load-bearing walls, the elements need to be checked for flexural buckling. Lateral torsional buckling is not an issue since the slabs are continuous over the full depth of the building, and therefore the bending stiffness along the buckling length of the wall is much smaller than the bending stiffness in transverse direction.

The classical theory for buckling is the Euler buckling theory. This theory is applicable to ideal columns, so columns made out of a homogeneous material that are perfectly straight with no initial stresses. These criteria cannot be assured, since timber is inhomogeneous, anisotropic and prone to imperfections with regard to the dimensions due to sawing. However, Euler’s critical buckling load gives a first insight in the buckling theory. This load is determined as:

$$F_c = \frac{\pi^2 EI_{net}}{l_{buck}^2}$$

The connections at the top and bottom of the walls are hinged connections, and therefore the buckling length is equal to the storey height. NEN-EN 1995-1-1 6.3.2 together with the draft of the new CLT-part gives a different approach for the instability due to buckling for beams and columns, where the compressive strength is being reduced to consider the slenderness and straightness of an element:

$$\lambda_y = \frac{l_{buck}}{\sqrt{I_{net}/A_{net}}}, \lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}, k_y = 0,5(1 + \beta_c(\lambda_{rel,y} - 0,3) + \lambda_{rel,y}^2), k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}}$$

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

β_c is given in the CLT-draft for NEN-EN 1995-1-1 (6.29) as 0,1. Since the walls are connected with hinges, and the stability in the perpendicular direction is assured by the application of shear walls in the front- and back facades, this results in the absence of out-of-plane bending stresses (in y-direction). With $E_{0,05}=9100 \text{ N/mm}^2$, the previous formulas lead to values per wall type shown in Table A20.

Type	t [mm]	Layers [mm]	A _{net} [mm ²]	I _{net} [mm ⁴]	λ _y	λ _{rel,y}	k _y	k _{c,y}
98W	98	29-40-29	5,80E+04	7,31E+07	84,50	1,31	1,41	0,52
103W	103	40-23-40	8,00E+04	9,01E+07	89,42	1,38	1,51	0,47
120W	120	40-40-40	8,00E+04	1,39E+08	72,06	1,11	1,16	0,67
133W	133	29-23-29-23-29	8,70E+04	1,63E+08	69,33	1,07	1,11	0,71
145W	145	29-29-29-29-29	8,70E+04	2,01E+08	62,38	0,97	1,00	0,80

Table A20 – Slenderness and k-factors for load-bearing wall

Appendix A: Calculation CLT structure

• • •

The load combination for stability checks in the ULS is given in the national annex of NEN-EN 1990 as:

$$q_d = 1,1 \cdot \sum_{j \geq 1} G_{k,j} + 1,5 Q_{k,1} + 1,5 \cdot \sum_{i > 1} \psi_{0,i} Q_{k,i} = 1,1 \cdot \sum_{j \geq 1} G_{k,j} + \max \left\{ \begin{array}{l} 1,5 \cdot Q_{roof} + 2 \cdot 1,5 \cdot 0,4 \cdot Q_{floor} \\ 1,5 \cdot Q_{floor} + 1,5 \cdot 0,4 \cdot Q_{floor} \end{array} \right.$$

$$= 1,1 \cdot \sum_{j \geq 1} G_{k,j} + \max \{ 9,73 \\ 9,93 \}$$

Due to the horizontal wind load, an in plane moment may be present in the load-bearing walls. However, this moment causes the biggest stress at the edges of the wall. At these edges, the load-bearing walls are connected to the front- and back facades of the building, which provide horizontal support over the height of the wall, and therefore decrease the slenderness λ_y and increases the instability factor $k_{c,y}$. Additionally, assuming the imposed floor load is dominant in the calculation of the load combination, this imposes the wind load should be multiplied by factor ψ_0 , which is equal to zero for wind.

The stabilizing effects of the front- and back facades on the structural behaviour is not easily predicted without further research, and will therefore be neglected in the scope of this thesis, with the assumption that the following check will suffice:

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} \leq 1$$

The load per running meter q_d is to be divided by the net area of the wall to calculate the compressive stress. The design compressive strength is determined to be 13,76 N/mm², resulting in the unity checks shown in Table A21.

Type	t [mm]	Layers [mm]	A _{net} [mm ²]	I _{net} [mm ⁴]	G _k [kN/m]	q _d [kN/m]	σ _{c,0,d} [N/mm ²]	UC
98W	98	29-40-29	5,80E+04	7,31E+07	15,5775	27,06525	0,47	0,07
103W	103	40-23-40	8,00E+04	9,01E+07	15,8025	27,31275	0,34	0,05
120W	120	40-40-40	8,00E+04	1,39E+08	16,5675	28,15425	0,35	0,04
133W	133	29-23-29-23-29	8,70E+04	1,63E+08	17,1525	28,79775	0,33	0,03
145W	145	29-29-29-29-29	8,70E+04	2,01E+08	17,6925	29,39175	0,34	0,03

Table A21 – Buckling strength check for load-bearing wall

Again, the unity checks are <<1, so the requirements are met for all wall types. This corresponds with the buckling loads given by the supplier for these wall types, which are significantly bigger than the design load q_d , shown in Table A21.



Fire safety (ULS, STR):

Consider a fire at the ground floor of the building, since the loads at this level reach the maximum values of the structure (Figure A17). Again a fire resistance of 60 minutes is required, meaning the cross section is again decreased by 42 mm. The correct accidental load combination must be determined for all structural elements which together form the fire compartment of this space.

Again, the load combination for the accidental situation is given in NEN-EN 1990 as:

$$q_d = \sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \vee \psi_{2,1}) Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$$

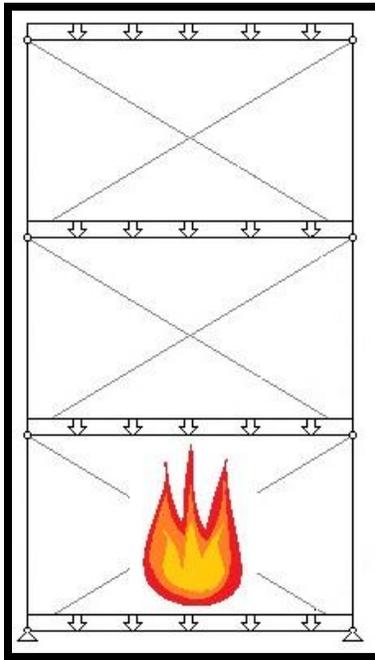


Figure A17 – Considered fire situation

The load combination for the compressive force in the wall at ground floor level consists of the self-weight of the two wall slabs located above the considered wall, the floors and the roof, and combination values of the live loads; snow load, wind load and imposed loads. The combination factors ψ_1 and ψ_2 have been given previously. Filling in the formula results in:

Appendix A: Calculation CLT structure

$$\begin{aligned}
 q_d &= G_k + \max \left\{ \begin{array}{l} \psi_{1,s} Q_s + 2\psi_{2, floor} Q_{floor} \\ \psi_{1, floor} Q_{floor} + \psi_{2, floor} Q_{floor} \end{array} \right. \\
 &= 3G_{wall} + 2 \cdot \left(1,35 + \frac{2,7 \cdot 160}{200} \right) + \left(2,19 + \frac{2,7 \cdot 145}{200} \right) \\
 &\quad + \max \left\{ \begin{array}{l} 0,2 \cdot 1,51 + 2 \cdot 0,3 \cdot 4,73 = 3,14 \\ 0,5 \cdot 4,73 + 0,3 \cdot 4,73 = 3,78 \end{array} \right.
 \end{aligned}$$

The decrease in cross-section is shown in $A_{net,red}$ and $I_{net,red}$. With these reduced properties, a new critical compressive resistance can be determined for the wall, which can be compared to stresses from the governing loading situation. The results are shown in Table A22.

Type	t [mm]	Layers [mm]	$A_{net,red}$ [mm ²]	$I_{net,red}$ [mm ⁴]	G_k [kN/m]	q_d [kN/m]	$\sigma_{c,0,d}$ [N/mm ²]	UC
98W	98	29-40-29	2,90E+04	2,03E+06	15,5775	19,3575	0,67	0,05
103W	103	40-23-40	4,00E+04	5,33E+06	15,8025	19,5825	0,49	0,04
120W	120	40-40-40	4,00E+04	5,33E+06	16,5675	20,3475	0,51	0,04
133W	133	29-23-29-23-29	5,80E+04	4,33E+07	17,1525	20,9325	0,36	0,03
145W	145	29-29-29-29-29	5,80E+04	5,28E+07	17,6925	21,4725	0,37	0,03

Table A22 – Strength check for load-bearing wall under fire conditions

Once again, the unity checks are $\ll 1$, so the requirements are met for all wall types.



Stability → Shear wall (ULS, EQU):

The load bearing walls also have a stabilizing function for the horizontal wind loads on front- or back facades. This function is fulfilled through in-plane shear resistance, but because CLT is a composed material with different characteristics in each direction, this theory is not as simple as it would be for a homogeneous slab. The loads on the wall are shown Figure A18. The wall at ground level is governing for the shear resistance, since the horizontal load is an accumulation of the horizontal load on walls located above the considered wall and the horizontal reaction force from the diaphragm action of the connected floor slab.

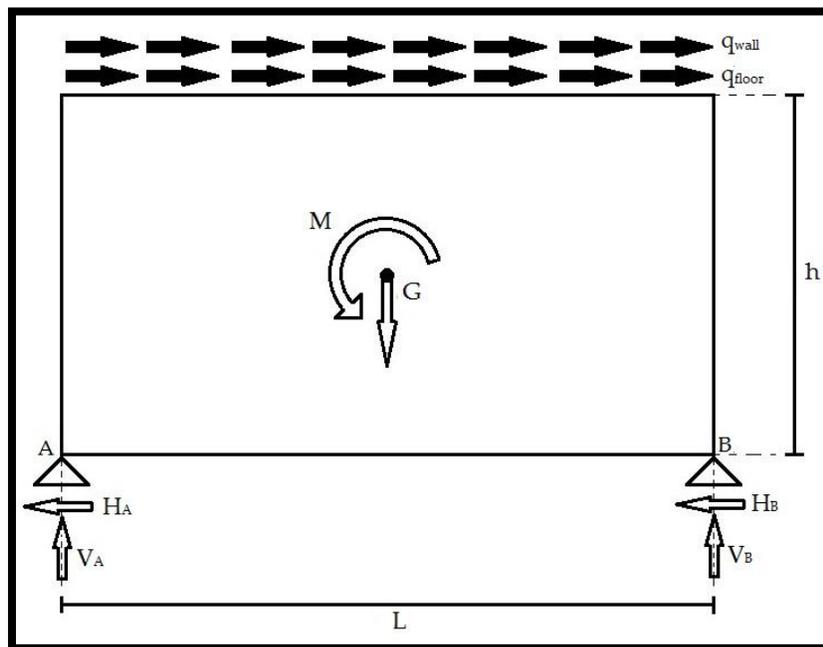


Figure A18 – Wind load on shear wall

Previously determined wind load is $p_w = 0,98 \text{ kN/m}^2$. The examined minimum wall thickness of 98 mm with composition 29-40-29 is chosen. The wind load on the front- or back façade is conducted sideways by the floor- and roof slabs, meaning due to symmetry that the load-bearing walls conduct equal parts of the resulting wind force down towards the foundation.

The reaction force in the load-bearing wall due to diaphragm action of the roof was determined as:

$$H_{w,b,roof} = Q_{d,w} \frac{5,4}{2 \cdot 6,6} = 1,56 \text{ kN/m}$$

The reaction force in the wall due to diaphragm action of the floors was determined as:

$$H_{w,b,floor} = Q_{d,w} \frac{5,4}{2 \cdot 9,6} = 1,61 \text{ kN/m}$$

Appendix A: Calculation CLT structure

• • •

Assuming a board width (b) of 15 cm, then the expression of the shear strength is as follows:

$$m = \min \begin{cases} \frac{3000}{150} = 20 \\ \frac{9600}{150} = 64 \end{cases}$$

$$f_{v,CLT} = \min \begin{cases} 3,5 \\ 5,5 \cdot \frac{40}{98} = 2,24 \vee 5,5 \cdot \frac{58}{98} = 3,26 \\ \frac{150 \cdot 2}{2 \cdot 98} \cdot \frac{1}{\frac{1}{2,5} \cdot \left(1 - \frac{1}{20^2}\right) + \frac{2}{0,8} \cdot \left(\frac{1}{20} - \frac{1}{20^2}\right)} = 2,96 \end{cases}$$

So the shear strength for the total cross-section is 2,24 N/mm². The total horizontal force in the wall at ground floor level is equal to:

$$H_{w,b} = H_{w,b,roof} + 2H_{w,b,floor} = 1,56 + 2 \cdot 1,61 = 4,78 \text{ kN/m}$$

The shear stress is then determined as:

$$\tau_a = \frac{H_{w,b}}{b \cdot t_{roof}} = \frac{4,78 \cdot 10^3}{1000 \cdot 98} = 4,88 \cdot 10^{-2} \text{ N/mm}^2$$

This stress is $\ll 2,24 \text{ N/mm}^2$, so the shear resistance of the load-bearing walls for the wind load in situation b is assured.



Roof-/floor slabs:

Strength → Reaction forces at support (ULS, STR):

With the determined forces in the wall, the stress perpendicular to the grain of the horizontal slabs can be determined to assure the slabs do not fail locally. Additionally, the out-of-plane shear stress at the support is checked with the known reaction forces, since this is the location where the shear force is at a maximum. Due to the varying thickness of the roof- and floor slabs, both need to be checked. The floor slab at the first floor is governing due to the larger reaction force within the load-bearing wall.

As shown in Figure A19, the shear force in the roof slab is equal to the reaction force in the load bearing wall. The governing load combination for the roof was previously determined as the sum of the self-weight and the imposed load, as:

$$q_d = 2,23 + 1,08 \frac{t}{200} = 2,23 + 1,08 \cdot \frac{145}{200} = 3,01 \text{ kN/m}$$

The reaction force per running meter wall is equal to the shear force in the cross-section at the support, with a value of:

$$F_V = V = \frac{1}{2} q_d l = \frac{1}{2} \cdot 3,01 \cdot 5400 \cdot 10^{-3} = 8,14 \text{ kN}$$

The shear distribution is a combined function due to the presence of “strong” layers with shear perpendicular to the grain and “weak” layers subject to rolling shear. The correct shear stress needs to be compared to either the shear strength or the rolling shear strength:

$$\tau_d = \frac{VES}{bEI} \leq f_{v,d} = k_{mod} \frac{f_{v,k}}{\gamma_M} = 0,8 \cdot \frac{3,5}{1,25} = 2,24 \text{ N/mm}^2 \vee f_{r,d} = k_{mod} \frac{f_{r,k}}{\gamma_M} = 0,8 \cdot \frac{0,8}{1,25} = 0,512 \text{ N/mm}^2$$

The shear stress distribution is shown in Figure A18.1. In the 2nd and 4th layer, this shear stress must be lower than the design rolling shear strength. The stress in these layers has a value of:

$$\tau_d = \frac{VES}{bEI} = \frac{8140 \cdot 1000 \cdot 29 \cdot \frac{29}{2}}{1000 \cdot 201000000} = 0,017$$

So the rolling shear stress is lower than the rolling shear strength of the roof where it is supported by the wall.

...

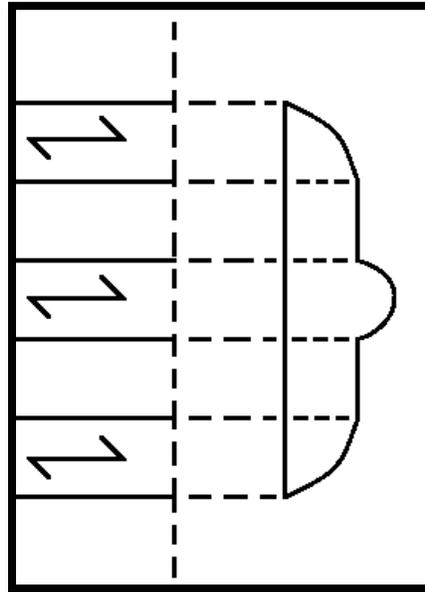


Figure A18.1 – Shear stress distribution

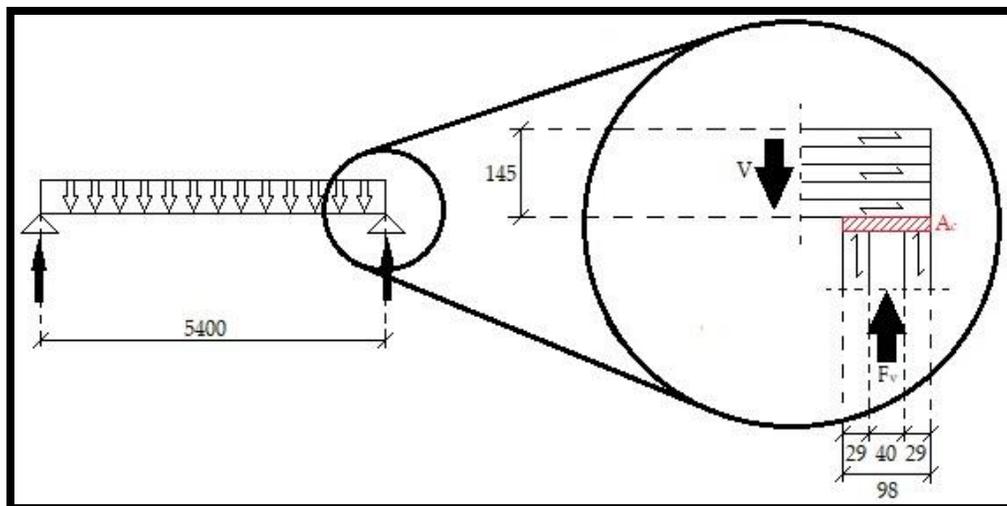


Figure A19 – Detail roof-wall connection 2

The same force of 8,14 kN per running meter causes a compressive stress perpendicular to the grain at the location of the support of the wall, shown in Figure A19 as the red area A_c . The working draft of CLT-part for NEN-EN 1995 describes a design method with regard to these stresses. The distribution of the stresses play an important role in the local resistance of a CLT slab, assumed at an angle of 35° (see Figure A20). This roof slab is comparable to the situation in Figure A20.c, except for the load distributing to one side only, since the wall is connected to the end of the roof slab.

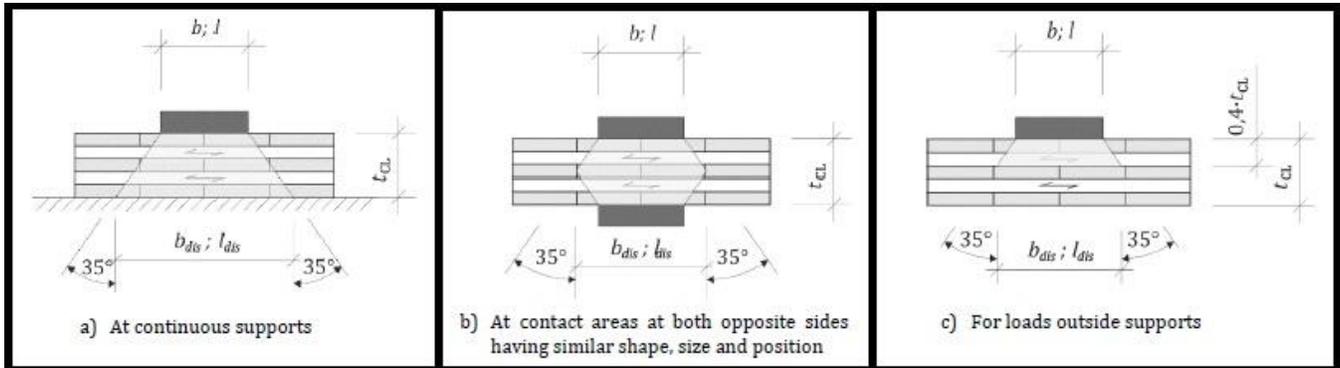


Figure A20 - Load distribution width and dispersion length, source: prEN16351

The verification of the compression perpendicular to the plane is done by assuring:

$$\sigma_{c,z,d} = \frac{F_V}{A_c} = \frac{8,14 \cdot 10^3}{98 \cdot 1000} = 8,3 \cdot 10^{-2} \leq k_{c,90,CL} f_{c,z,d} = \sqrt{\frac{b_{dis} l_{dis}}{bl}} f_{c,z,d} = \sqrt{\frac{138,6}{98}} \cdot 3 = 3,57 \text{ N/mm}^2$$

$$b_{dis} = (0,4 \cdot t_{CL} \cdot \tan(35^\circ)) + b = 0,28 \cdot 145 + 98 = 138,6 \text{ mm}$$

The governing load combination for the floor slabs was previously determined as the sum of the self-weight and the imposed load, as:

$$q_d = 0,9 \cdot \left(0,89 \cdot 1,35 \cdot \left(\frac{160}{200} + 0,5 \right) + 1,5 \cdot 1,75 \right) = 3,76 \text{ kN/m}$$

$$V = \frac{1}{2} q_d l = \frac{1}{2} \cdot 3,76 \cdot 5400 \cdot 10^{-3} = 10,16 \text{ kN}$$

$$\tau_d = \frac{VEs}{bEI} = \frac{10,16 \cdot 10^3 \cdot 1000 \cdot 40 \cdot 20}{1000 \cdot 304000000} = 2,7 \cdot 10^{-2} \leq f_{v,d} = 0,512 \text{ N/mm}^2$$

So the shear stress is lower than the rolling shear strength of the floor where it is supported by the wall.

Appendix A: Calculation CLT structure

• • •

The compressive force in the load-bearing wall per running meter at the location where it supports the first floor ($F_{v,2}$ in Figure A21) is equal to a combination value of the shear force in the floor and the compressive force in the wall which is located on top of the floor. This force is also equal to the previously calculated value of the compressive force in the ground level wall minus the weight of that wall itself:

$$q_{d,down} = 0,9 \cdot 1,5Q_k + 0,9 \cdot 0,89 \cdot 1,35G_k = 25,78 \text{ kN}$$

$$F_{v,2} = q_{d,down} - 0,9 \cdot 0,89 \cdot 1,35 \cdot G_{wall} = 25,78 - 1,08 \cdot \frac{3 \cdot 98}{200} = 24,19 \text{ kN}$$

This floor slab is comparable to the situation in Figure A20.b, except for the load distributing to one side only, since the wall is connected to the end of the roof slab.

The verification of the compression perpendicular to the plane is done by assuring:

$$\sigma_{c,z,d} = \frac{F_V}{A_c} = \frac{24,19 \cdot 10^3}{98 \cdot 1000} = 2,47 \cdot 10^{-1} \leq k_{c,90,CL} f_{c,z,d} = \sqrt{\frac{b_{dis} l_{dis}}{b l}} f_{c,z,d} = \sqrt{\frac{154}{98}} \cdot 3 = 3,76 \text{ N/mm}^2$$

$$b_{dis} = (0,5 \cdot t_{CL} \cdot \tan(35)) + b = 0,35 \cdot 160 + 98 = 154 \text{ mm}$$

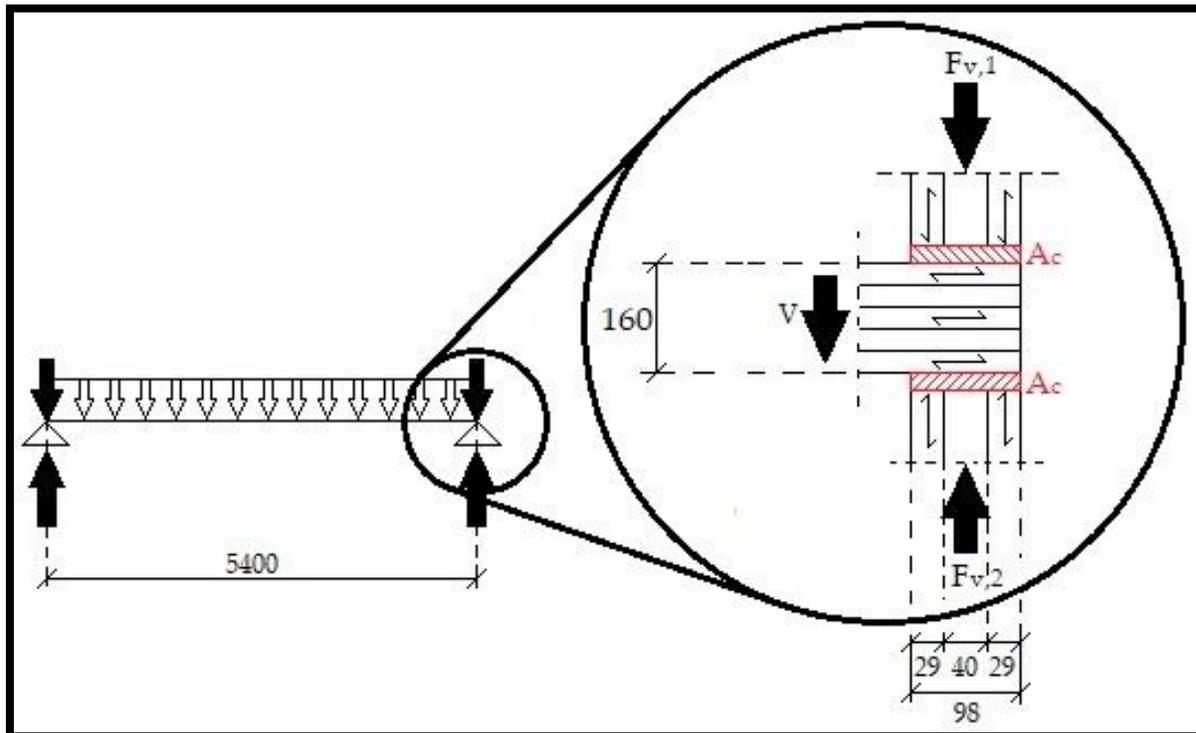


Figure A21 – Detail intermediate floor/wall connection



Front- and back facades:

Stability → Shear wall (ULS, EQU):

Again the governing wall part is the wall at the ground floor. The wind pressure is assumed constant over the height of the building, with previously determined value of 0,98 kN/m². The reaction forces in the front-/back facades were determined for the diaphragm action of roof- and floor slabs as:

$$H_{w,a,roof} = Q_{d,w} \frac{6,6}{2 \cdot 5,4} = 2,33 \text{ kN/m}$$

$$H_{w,a,floor} = Q_{d,w} \frac{9,6}{2 \cdot 5,4} = 5,1 \text{ kN/m}$$

For these facades, the shear resistance is the only structural requirement. Previous checks have shown the governing shear strength is often linked to the ratio between the thickness of the “weak” layers and the thickness of the total CLT slab. To maximize the shear resistance of these facades, it is therefore beneficial to select a CLT option where the “weak” layers have approximately the same thickness as the “strong” layers. The optimal choice provided by the supplier’s data again leads to a thickness of 98 mm with a composition of 29-40-29 mm. The governing shear strength is equal to the shear strength in the load-bearing walls:

$$m = \min \left\{ \begin{array}{l} \frac{3000}{150} = 20 \\ \frac{5400}{150} = 36 \end{array} \right.$$

$$f_{v,CLT} = \min \left\{ \begin{array}{l} 3,5 \\ 5,5 \cdot \frac{40}{98} = 2,24 \vee 5,5 \cdot \frac{58}{98} = 3,26 \\ \frac{150 \cdot 2}{2 \cdot 98} \cdot \frac{1}{\frac{1}{2,5} \cdot \left(1 - \frac{1}{20^2}\right) + \frac{2}{0,8} \cdot \left(\frac{1}{20} - \frac{1}{20^2}\right)} = 2,96 \end{array} \right.$$

So the shear strength for the total cross-section is 2,24 N/mm². The total horizontal load in the wall at ground floor level is equal to:

$$H_{w,a} = H_{w,a,roof} + 2H_{w,a,floor} = 2,33 + 2 \cdot 5,1 = 12,53 \text{ kN/m}$$

The shear stress is then calculated for a constant distribution as:

$$\tau_a = \frac{H_{w,b}}{b \cdot t_{roof}} = \frac{12,53 \cdot 10^3}{1000 \cdot 98} = 1,28 \cdot 10^{-1} \text{ N/mm}^2$$

Again, this stress is << 2,24 N/mm², so the shear resistance of the front- and back facades for the wind load in situation a is assured.



Contrary to the load-bearing walls, where no holes are desired as they have a separating function, holes are a necessity for the front- and back facades. These walls must contain doors and windows for the entrance of inhabitants and daylight. Depending on the preference of inhabitants, these holes may have big dimensions and therefore significant impact on the performance with respect to shear resistance. As described earlier, this problem is best solved through FE analysis. To supply inhabitants with the most freedom in their furnishing as possible, investigating a minimal design of the front-/back facades is desired. The FE analysis is further elaborated in Appendix B.



CLT connections:

Two connection types can be distinguished in this CLT design, related to the relative orientation of the connected slabs; longitudinal (in-plane) joints and transverse (out-of-plane) joints.

Three types of longitudinal joints are specified; the leaf joint (Figure A22.a), the tenon joint (Figure A22.b) and the half-lapped joint (Figure A22.c). These types of joints are used to elongate a slab if necessary due to transportation limits or for practicality reasons.

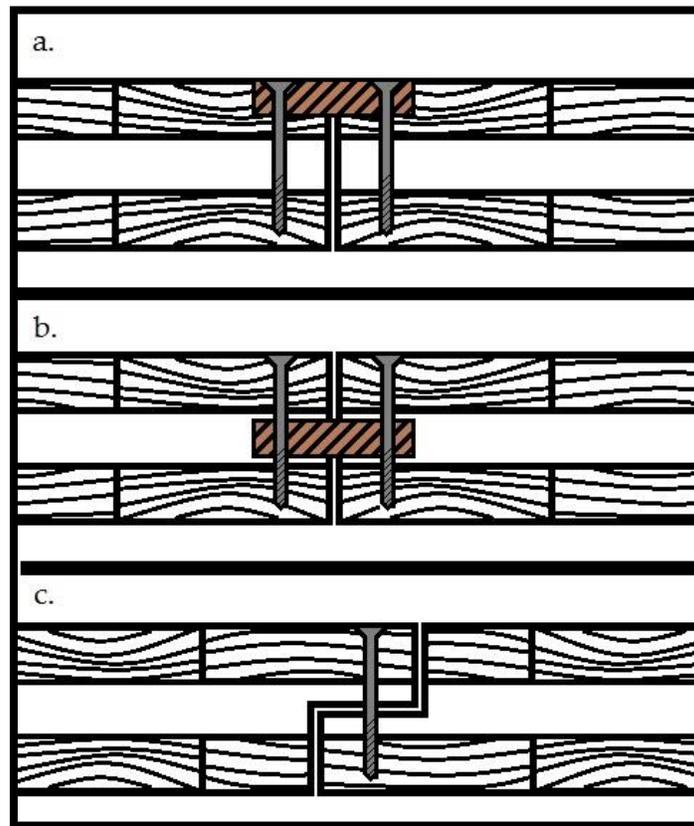


Figure A22 – Longitudinal joint options in CLT; a. leaf joint, b. tenon joint, c. half-lapped joint

The design of the connections in a CLT structure is very complex due to the various criteria mentioned in the main report. These criteria cause for conflicting motives in the decision-making of the connection types. When aiming for a connection type which corresponds with the circular economy aspirations, the disassembly of the connection becomes a very important characteristic. If however the bio economy is regarded as being more important than the circular economy, the natural source of the connection material and the biodegradability become the leading criteria in the decision-making.



However, the most important criterion is the safety, so the design of the connections needs to assure this. Different connections are present in the considered CLT-structure:

1. Longitudinal joints in roof-/floor slabs
2. Transverse joint between floor and load-bearing wall
3. Transverse joint between floor and front-/back facade
4. Transverse joint between load-bearing wall and front-/back facade

Before designing the connections in a structure, one needs to realize what type of requirements are attached to them. Low-rise structures in the Netherlands are generally subject to static loads with the exception of small dynamic wind loads, which are converted to static loads with appropriate weight factors through the Eurocode. When however the structure is subject to more dynamic load cases like in earthquake areas, this could lead to different conditions in the design of the connections, e.g. ductility could play a more important role in the dissipation of kinematic energy of the structure. These factors are crucial in the selection of appropriate fasteners.

Longitudinal joints:

The longitudinal joints need to be designed on shear forces in the connective elements in two directions; due to sliding between plates and due to tensile forces between plates as a result of in-plane moments. The shear stress is assumed to be constant over the length of the floor (Figure A22.a), thus the connectors in this situation need to be separated equally. In case of an in-plane moment, the shear connectors need to be present in the tensile zone of the slabs. Application of deep-beam theory results in a stress distribution as in Figure A23.b. The stresses in the compression zone are controlled through direct contact. Consider a situation like Figure A23.b, but with a reversed moment. This result in connectors being required on both sides of the slabs. The zones where shear connectors are required in Figure A23 are shown in grey.

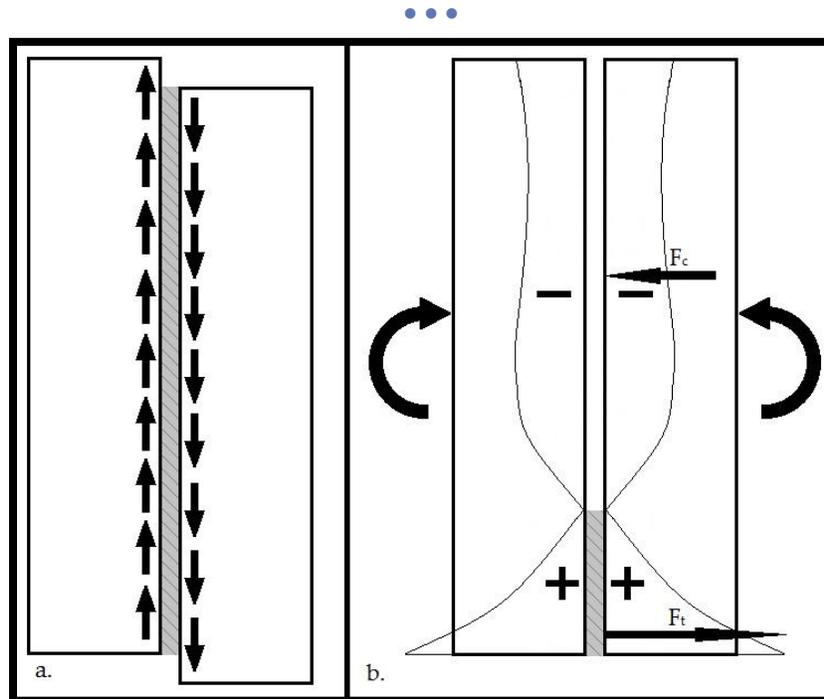


Figure A23 – Longitudinal joint stresses; a. due to sliding (shear), b. due to in-plane moment

The possible solutions for longitudinal joints shown in Figure A22 show the orientation of the connective elements is perpendicular to the grain of parts of the CLT, due to the crosswise orientation of the boards. The connections however get their resistance from the parallel layers, since these layers split at a higher stress. For this reason, different applications may result in a different solution for the longitudinal joint, depending on the location and direction of the required resistance. Given the desire of quick realization of the structures, the half-lapped joint is assumed for the floors in these buildings, since these joints allow for simple lifting and putting in place of the successive slabs.

Commonly applied and easily applicable fasteners in CLT are screws and nails. These steel elements have a high shear resistance combined with small dimensions, so they are easily concealed. For the roof connections, the maximum reaction force of 2,33 kN/m is assumed to be the maximum shear force between the panels. Shear and tension are the governing loads on the screws. The half-lapped joint implies there is one sliding surface where the screw needs to provide shear resistance.

Assume a screw diameter of 10 mm with a length of 140 mm and the screws inserted with pre-bored holes. The shear resistance of this screw type is 170 kg (provided by the screw supplier), equal to 1,67 kN. This implies a total of 2 screws per meter is sufficient to resist the maximum shear of 2,33 kN/m.

The maximum moment in the diaphragm is equal to:

$$M_d = \frac{1}{8}ql^2 = \frac{1}{8} \cdot 3,82 \cdot 9,6^2 = 44 \text{ kNm}$$

Assuming an internal lever arm for the bending stresses of approximately half the “depth” of the deep beam and a tensile zone of approximately a quarter of this “depth”. This results in a tensile force of:

$$F_t = \frac{M_d}{\frac{1}{2}h} = \frac{44}{0,5 \cdot 5,4} = 16,3 \text{ kN}$$

This is the resultant force in a zone of 1,35 m. Thus the following number of fasteners is required:

$$n_t = \frac{F_t}{\frac{1}{4}h \cdot v_{adm}} = \frac{16,3}{0,25 \cdot 5,4 \cdot 1,67} = 7,23$$

So 8 fasteners per meter are required, angled perpendicular to the joint interface. Spacing requirements are often given by screw suppliers, in this case leading to the total assumed connection as shown in Figure A24. Spacing requirements are more tolerant when the fasteners are inserted in pre-bored holes.

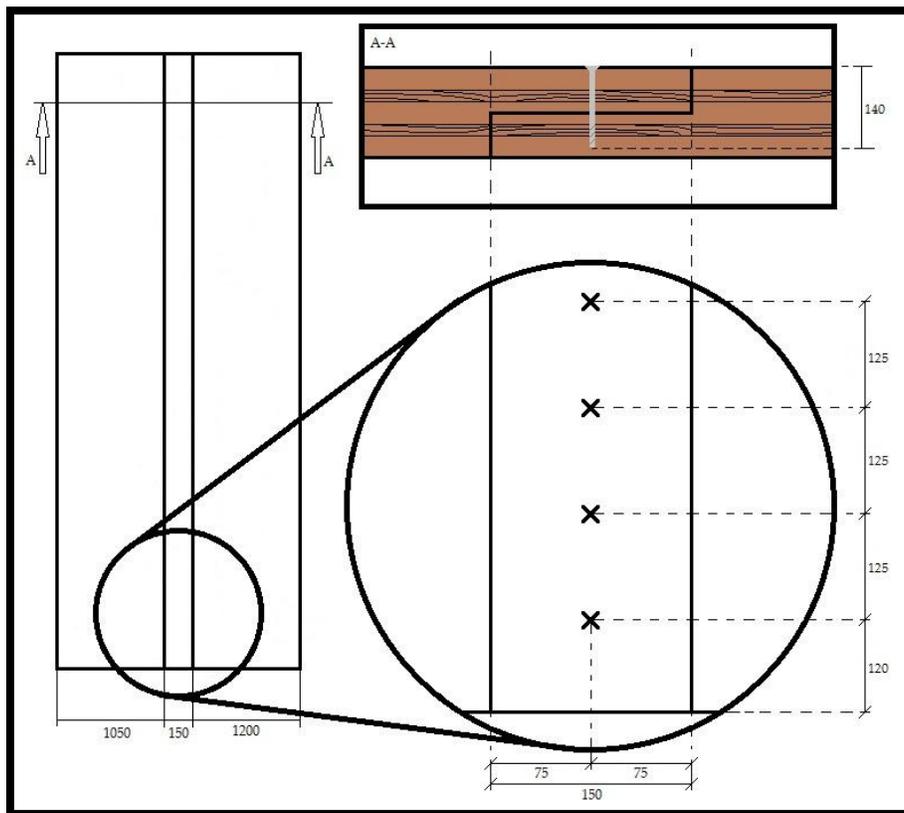


Figure A24 – Longitudinal leaf joint between two horizontal (floor- or roof) slabs, dimensions in mm

An example of spacing requirements are shown in Table A21. In case of CLT, with layers in two directions, determining the relevant spacing requirement is difficult, since only the assumption of the biggest value in Table A23 guarantees total safety without splitting of the timber.

• • •

	0°	90°
$a_1 \rightarrow$ between fasteners parallel to grain [mm]	50	40
$a_2 \rightarrow$ between fasteners perpendicular to grain [mm]	30	40
$a_{3,t} \rightarrow$ distance to stressed end [mm]	120	70
$a_{3,c} \rightarrow$ distance to discharged end [mm]	70	70
$a_{4,t} \rightarrow$ distance to stressed edge [mm]	30	70
$a_{4,c} \rightarrow$ distance to unload edge [mm]	30	30

Table A23 – Spacing requirements VGS fasteners, source: Rothoblaas

As shown in Figure A24, the spacing between the screws is equal to 125 mm, being bigger than requirement $a_1 = 45$ mm. The end spacing in the strong direction of the slabs is set at 120 mm. In this direction, 120 mm of the total 160 mm (75%) of timber has an angle of 0° with the required shear strength direction of the fastener, leading to requirement $a_{3,t} = 120$ mm being assumed to be governing. This requirement is relatively high because the timber in this direction is prone to splitting. The end spacing in the weak direction is 75 mm, since the connection has a total width of 150 mm and the fasteners are placed in the centre to create equal spacing in both sides. 75% of the timber grain is directed at an angle of 90° with the direction of the required shear strength. For this part of the cross-section, a spacing of 70 mm is sufficient. For the residual 25% of the cross-section with an angle of 0° between the grain and the required strength, again requiring a spacing of 120 mm. The spacing of 75 mm should be further tested for this layout to assure the safety.

Appendix A: Calculation CLT structure

• • •

Applying steel fasteners in this structure implies the addition of a non-bio-based material to a very sustainable solution for low-rise buildings, negatively affecting the fit in the bio-economy. Furthermore, screws and nails are not easily dismantlable and leave their mark when the connections are taken apart. This issue needs resolving in the reapplication of structural members.

Investigating bio-based solutions leads to the consideration of timber dowels as fasteners, although one might instantly question the dismantlability of dowelled connections. In practice, if it proves impossible to remove the dowels in one piece, boring out provides a solution to disassemble the structure without causing damage to the structural elements. For reapplication, only new timber dowels would be required. This type of connector already finds its application in relatively new timber products.

Timber dowels are available in many different strength grades and sizes. Considering an equal layout as the layout for steel screws (Figure A25, showing cross section A-A from Figure A24 but for a timber doweled connection), there are $1000/125 = 8$ dowels present in the connective zone, leading to a minimum required strength per dowel of:

$$V_{adm} = \frac{F_t}{\frac{1}{4}h \cdot n_t} = \frac{16,3}{0,25 \cdot 5,4 \cdot 8} = 1,51 \text{ kN}$$

Assuming a standard diameter of 30 mm for all timber dowels in the structure, the area of the dowel is equal to 707 mm^2 , requiring a strength of 2.14 N/mm^2 . This implies any timber species will satisfy the requirements, and the choice can be fully based on the sustainability and price of the dowel.

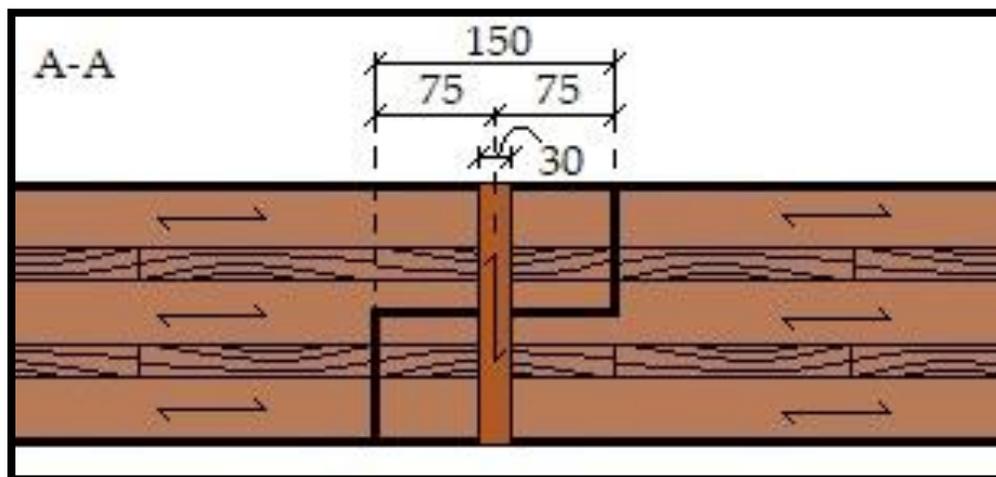


Figure A25 – Dowelled leaf joint



Transverse joints:

The first considered transverse joint is the connection between the horizontal (roof and floor) slabs and the load-bearing walls. These connections conduct the horizontal- and vertical tensile loads due to wind from the horizontal diaphragms towards the wall. The horizontal wind load causes a shear stress in the fastener in the in-plane direction of the slabs, shown in Figure A26.b. The vertical load leads to tensile stress in the fastener, shown in Figure A26.a.

In the consideration of screws as fasteners, the length needs to be sufficient for the screw to reach far enough into the vertical slab, with the screw diameter again set at 10mm. A screw length of 240mm is required to firmly connect the wall through the floor slab.

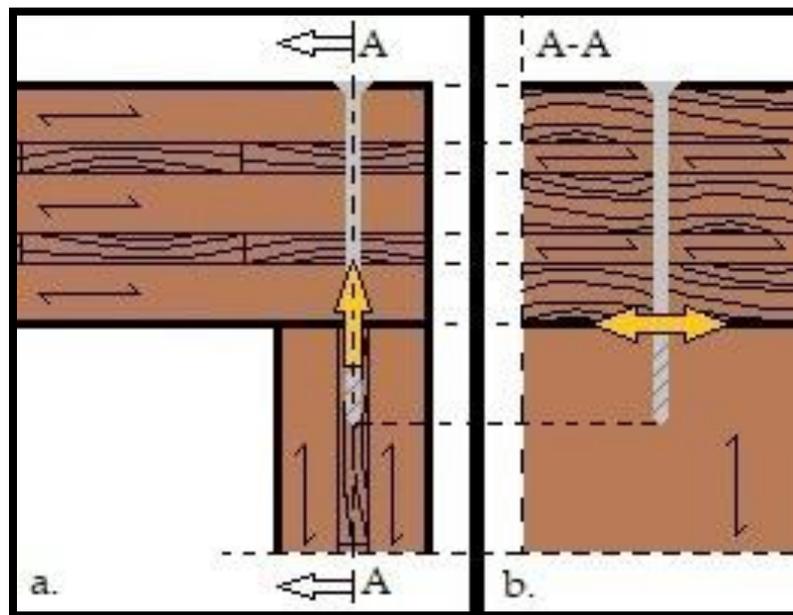


Figure A26 – a. Screwed roof/wall connection, b. View A-A

The reaction force in the wall due to diaphragm action in the roof was previously determined at 1,56 kN per running meter. The shear resistance is 170 kg, equal to 1,67 kN. The thread withdrawal resistance for this screw is 400kg, equal to 3,92 kN. The resistance to head penetration is 150 kg, equal to 1,47 kN. The maximum uplifting load had already been determined for the tensile stress check in the load-bearing walls. It was given by the formula per running meter:

$$q_{d,up} = 0,9 \cdot 1,5Q_{w,uplift} - 0,9G_{roof} = 0,9 \cdot 1,5 \cdot 3,61 - 0,9 \cdot \left(2,19 + \frac{2,7t_{roof}}{200} \right) \text{ kN/m}$$

• • •

t_{roof} has been set at 145 mm, resulting in a total uplifting load of 1,14 kN per running meter. So for both shear and axial loads, 1 screw per running meter is sufficient.

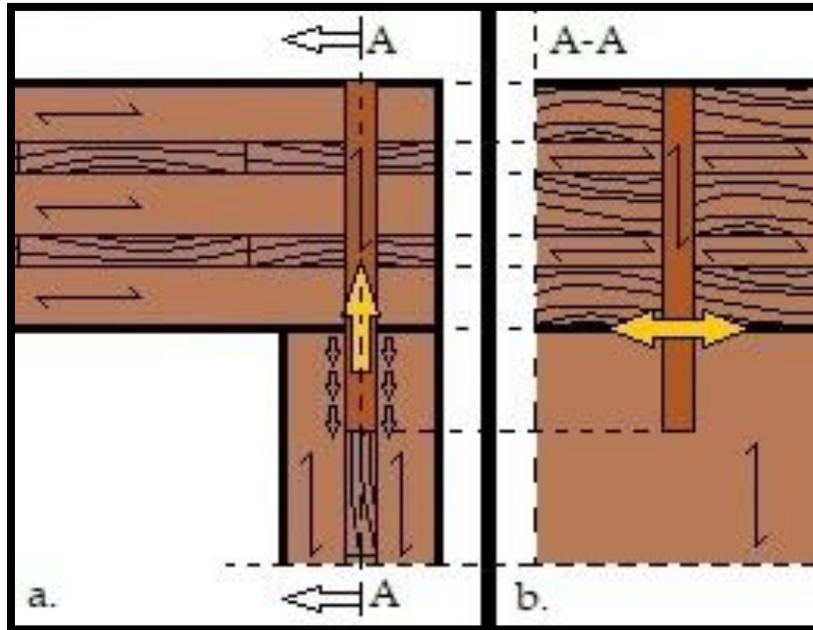


Figure A27 – a. Dowelled roof/wall connection, b. View A-A

Again a dowelled connection is considered in an attempt to increase sustainability. The required length of the dowel entering the wall slab depends on the required frictional force to withstand the uplifting wind load (Figure A27.a). Considering C24 dowels of 30 mm diameter leads to a shear resistance per dowel of:

$$R_V = f_{v,d} A_d = 0,8 \cdot \frac{3,5}{1,25} \cdot \pi \cdot 30^2 = 6,33 \cdot 10^3 \text{ N}$$

Thus, one dowel per meter would already be sufficient to conduct the horizontal wind load.

The tensile resistance per individual dowel is equal to:

$$R_t = f_{t,d} A_d = 0,8 \cdot \frac{16}{1,25} \cdot \pi \cdot 30^2 = 28,95 \cdot 10^3 \text{ N}$$

This tensile resistance is easily sufficient when applying e.g. 1 dowel per running meter of wall.

A frictional stress in the contact area between the dowel and the slabs needs to eventually keep the materials together. This friction depends on the ratio between the hole diameter and the diameter of the dowel. Generally, adhesive is inserted in the dowel connection to increase the bonding. In this structure, bio-based adhesive is the only option for sustainability reasons, and the dowel connections with this type of glue require more research.



Since the local tensile stress due to wind uplift in the load-bearing walls was governing in the top of the building since the influence of self-weight is smallest in that location, the tensile force in the fastener of the roof-wall connection is also governing. The screws in the wall-floor connections do therefore not need to be checked for tension.

The shear force between the first floor and the wall at ground floor level is governing in this building, with the maximum horizontal (wind) load previously determined at 4,78 kN/m. The shear force between the first floor and the wall at first floor level is equal to the shear in the ground floor wall minus the shear in the floor slab due to diaphragm action:

$$H_{w,1st} = H_{w,b} - H_{w,b,floor} = 4,78 - 1,61 = 3,17 \text{ kN/m}$$

The connection between the first floor and the ground floor level wall is easily constructed. In the platform framing method, the floor is lifted on top of the supporting walls. This is the moment when the screws for this connection are applied, similar to the application of the screws in the roof-wall connection. The next step is to fasten the first floor wall on top of the floor. This wall must be connected with angled screws, assume an angle of 60° with the floor slab. The screw requires a sufficient length to fully pierce through the floor slab, equal to 180mm. Consider screw thickness of 8mm, because more screws are desired for this direction to keep the connection as concentric as possible (Figure A28).

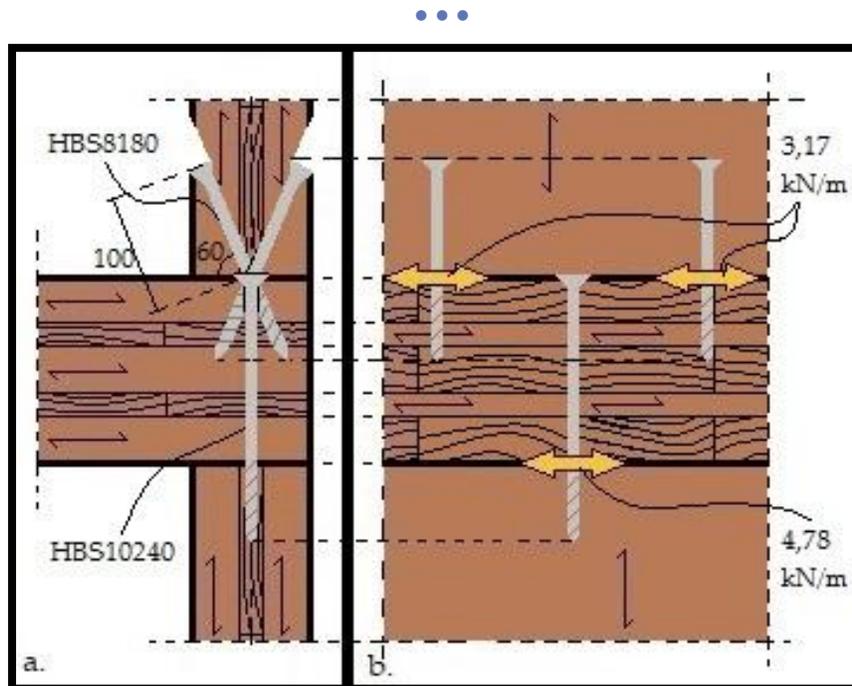


Figure A28 – a. Side view screwed wall/floor connection, b. Front view

The shear resistance of the screw with length of 240mm and diameter 10mm was previously determined as 1,67 kN. So for the connection between the ground floor level wall and the first floor the following number of screws per meter is required:

$$n_{1,g} = \frac{4,78}{1,67} = 2,86$$

So 3 screws per meter are sufficient, and assume a spacing of 300 mm. The shear resistance is provided as 109 kg, or 1,07 kN. This leads to a required number of screws per meter for the first floor wall connection with the floor of:

$$n_{1,1} = \frac{3,17}{1,07} = 2,96$$

So 3 screws per meter would already be sufficient, and assume a spacing of 300 mm.

The dowelled version for this connection is more complex than the screwed version. The first thought might lead to a connection with one dowel going from one wall slab, through the floor, into the other wall slab (Figure A29). However, considering the manufacturability, this connection is not easily constructed since the dowels require a tight fitting inside the timber slabs. Additionally, the desired dowel dimensions need to be available with producers. In the Netherlands, the dowels from two producers have received a KOMO-certificate according to the BRL 2908 guideline. These producers



supply dowels with a diameter of up to 16mm and a length of up to 150mm. These dimensions are not sufficient for the connections in this structure. Other producers do supply the desired dimensions.

Theoretically, with a shear resistance of the dowel of 6,33 kN, this means one dowel per meter of wall is sufficient to conduct the shear force from the first floor to the supporting wall underneath.

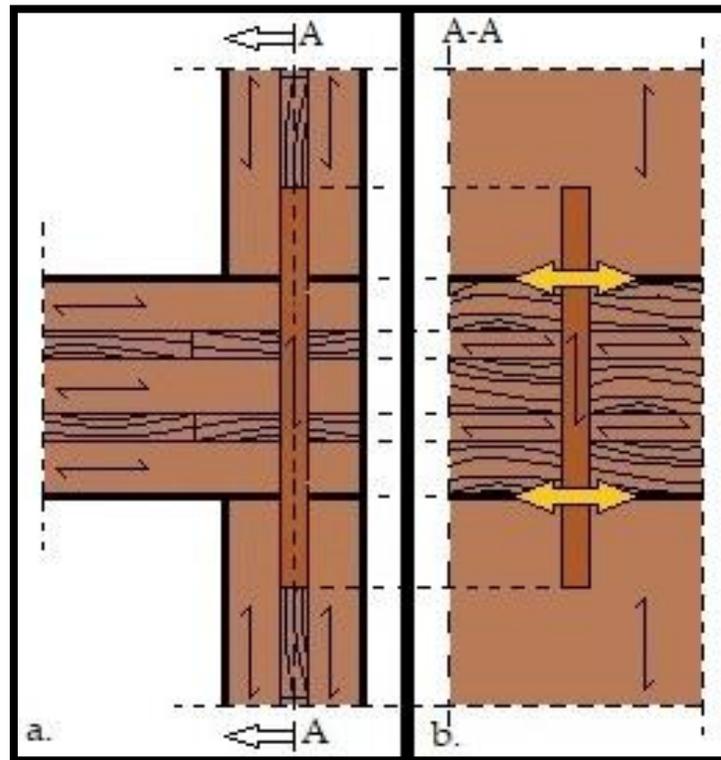


Figure A29 – a. Side view dowelled wall/floor connection, b. view A-A in a.

The connection between the horizontal slabs and the stabilizing, front-/back facades only needs to fulfil a shear resistance requirement. This connection is more complex than the connection between the floor and the load-bearing wall, since the floor is not supported on top of the stabilizing facades, but is connected against it with fasteners in horizontal direction (Figure A30). The maximum shear force needed to be conducted from the horizontal slabs to the front-/back facades is 5,1 kN/m. The assumed screw diameter for this connection is 10mm, with a shear resistance of 1,67 kN. So a total of 4 screws with diameter of 10mm and length 180mm are required per meter. Assume a spacing of 250 mm.

The maximum shear force in the connection between the different wall slabs of the front-/back facades is equal to 7,43 kN/m. The assumed screw diameter for this connection is 8mm, with a shear resistance



of 1,07 kN. So a total of 8 screws with diameter 8mm and length 180mm are required per meter. Assume a spacing of 125 mm.

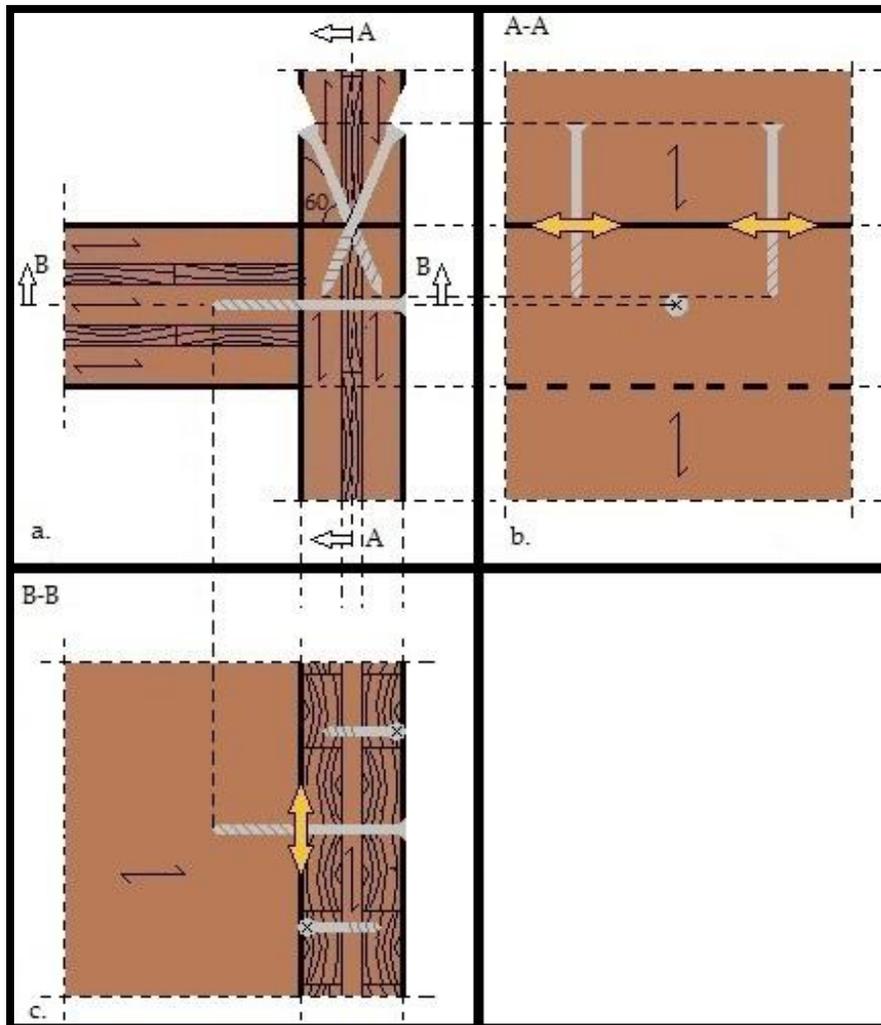


Figure A30 – a. Side view of screwed floor/front facade connection, b. view A-A in a., c. view B-B in a.

Again, the dowelled connection is not a practical solution due to difficulty in the application. Theoretically, the connection would be comprised of both horizontal and vertical dowels (Figure A31). Given the shear resistance of a single dowel of 6,33 kN, one horizontal dowel and two vertical dowels are required in the connection.

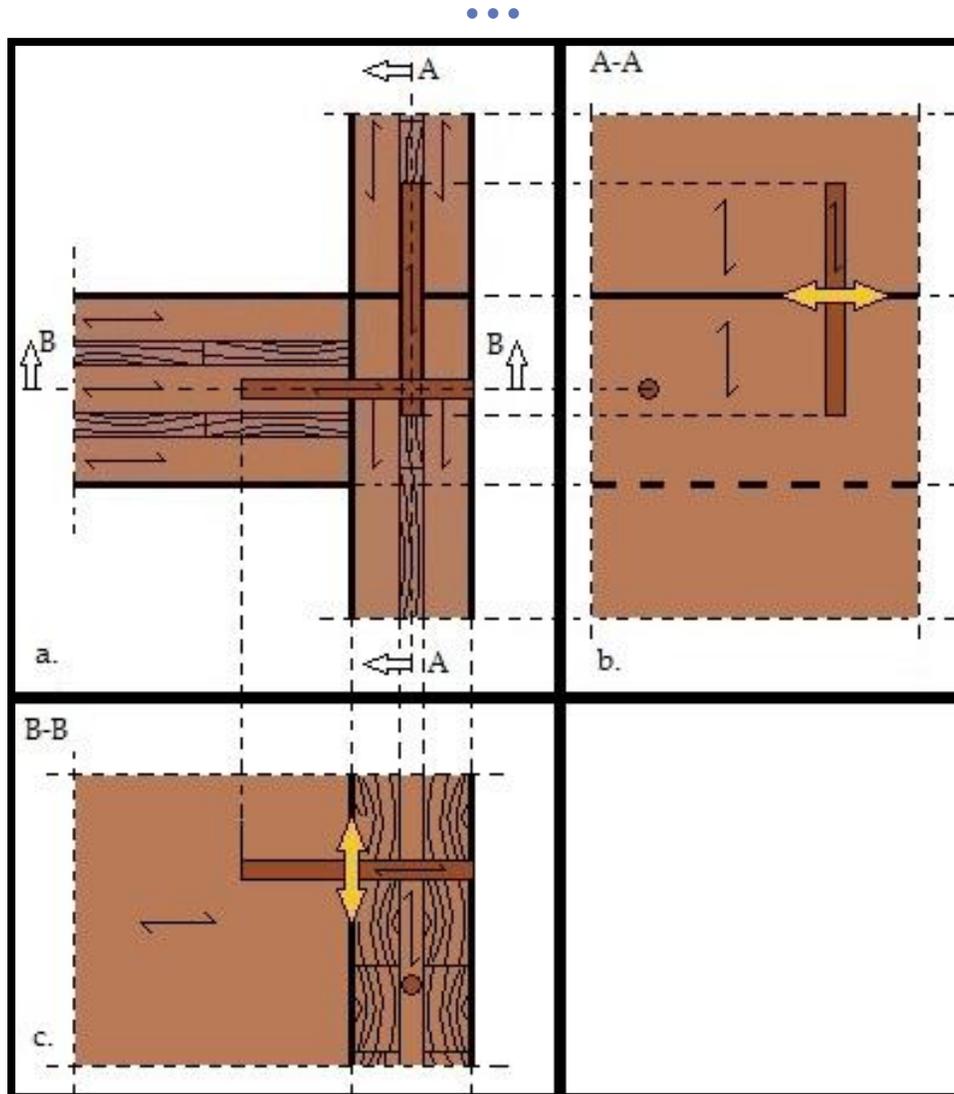


Figure A31 – a. Side view of dowelled floor/front facade connection, b. view A-A in a, c. view B-B in a.

A.3.1 Total timber volume

The total material usage of the CLT structure, assuming a 160mm thick CLT ground floor slab, is equal to:

$$\begin{aligned}
 V_{CLT} &= 3 \cdot 160 \cdot 9600 \cdot 5400 + 145 \cdot 9600 \cdot 5400 + 6 \cdot 98 \cdot 9600 \cdot 3000 + 6 \cdot 98 \cdot 5400 \cdot 3000 \\
 &= 5,886 \cdot 10^{10} \text{ mm}^3
 \end{aligned}$$



A.4 Additional fire safety measures

The basic requirement for the structure to resist failure for the first 60 minutes after a fire starts within the building has been assured in the previous part for the accidental loading combination. To check whether this requirement of 60 minutes is actually necessary, the fire load of the building needs to be calculated. If it is smaller than 500 MJ/m², the requirement may be lowered to 30 minutes. The formula for the fire load was given in Ch. 4.4. The thicknesses of structural elements have been determined as 145 mm, 160 mm and 98 mm for respectively the roof-, floor-, and wall slabs.

The net calorific value for timber has a value of approximately 15 MJ/kg. The total mass and the fire load of the CLT is determined as:

$$m_{CLT} = \rho_{CLT} \cdot V_{CLT} = 500 \cdot (145 \cdot 5400 \cdot 6600 + 3 \cdot 160 \cdot 5400 \cdot 9600 + 4 \cdot 98 \cdot 3000 \cdot 9600 + 2 \cdot 98 \cdot 3000 \cdot 6600 + 6 \cdot 98 \cdot 3000 \cdot 5400 \cdot 10 - 9) = 27373,5 \text{ kg}$$

$$A = 2 \cdot 9,6 \cdot 5,4 + 6,6 \cdot 5,4 = 139,32 \text{ m}^2$$

$$q = \frac{1}{A} \cdot \sum(H_i \cdot m_i) = \frac{1}{139,32} \cdot (15 \cdot 27373,5) = 2947 \text{ MJ/m}^2$$

So the fire load is >500MJ/m², and the assumed fire resistance of 60 minutes is appropriate.

Besides the fire resistance, according to the BD it is required to mitigate the propagation of fire from one house to the other for at least 60 minutes. The burning-in rate of the CLT slab was previously set at 0,7 mm/min, resulting in a total burning-in of 42 mm in 60 minutes. To include the possibility of local burning through of some parts of the CLT, additional measures could optionally be taken.

In order to achieve total protection, a fire resistant (inflammable) layer should be applied in the cavity between the load bearing walls, so the fire from one house cannot set an attached house ablaze. An easy solution would be the application of e.g. a gypsum layer between the load-bearing walls. When including the sustainability, an inflammable layer drops out of the options due to the non-natural resource of the materials. Ultimately, fire propagation resistance is desired through the use of bio-based materials. The terms inflammable and bio-based do not concur since bio-based materials have the general characteristics of being degradable and combustible. The solution lies not in a totally inflammable layer, but rather in the application of materials contributing to the fire control. E.g. cellulose or lignin, two of the most abundant biopolymers, could provide a solution as they possess fire retardant characteristics (Costes, Laoutid, Brohez, & Dubois, 2017). The application of a layer of a fire



retardant between load-bearing walls should delay the propagation of a fire to the extent that the fire can be controlled and extinguished before any adjacent structure catches fire.

Easycell, located in the Netherlands, will be considered as the supplier of cellulose for fire safety purposes. The reaction to fire of Easycell cellulose has been determined through NEN-EN 13501-1 as B-s1-d0. This insinuates:

- 1) Class B:
 - a) EN ISO 11925-2: When exposed to an edge flame attack with exposure time of 30 seconds, the flame spread is <150 mm vertically from the application point within 60 seconds of time of application.
 - b) EN 13823: Lateral flame spread (LFS) is not occurring, fire growth rate (FIGRA) is <120 W/s, total heat release (THR_{600s}) is < 7,5 MJ.
- 2) s1 (additional classification for smoke production):
 - a) EN 13823: Smoke growth rate (SMOGRA) is $\leq 30 \text{ m}^2/\text{s}^2$, total smoke production (TSP_{600s}) $\leq 50 \text{ m}^2$.
- 3) d0 (additional classification for flaming droplets):
 - a) EN 13823: No flaming droplets occur within 600 seconds.

Assume a minimum thickness of 100mm required to achieve this reaction to fire class.



A.5 Thermal insulation measures

The thermal resistance requirement from the Building Decree was mentioned in Ch.4.5.2 of the main report as 4,5; 6,0 and 3,5 m²K/W for the walls, roof and ground floor respectively. The thermal conductivity of the applied timber was determined at 0,13 W/mK.

The first step in designing the insulation system of the building is the material selection. Currently, most applied insulation layers are composed of non-renewable materials like plastic-based fibres and foams. To stay within the main aspiration of this project, a sustainable, bio-based material must be selected to replace these conventional synthetic options. Several bio-based resources can be considered to function as insulation, such as animal- (sheep wool) and plant fibres (straw).

For fire safety measures, cellulose was proposed as a fire retardant between adjacent houses. Coincidentally, cellulose is also a bio-based, renewable material with a relatively low thermal conductivity of 0,038 W/mK. The required thicknesses of the insulation layers are equal to:

$$R_{\lambda,wall} = \frac{t_{CLT,wall}}{\lambda_{CLT}} + \frac{t_{ins,wall}}{\lambda_{ins}} \geq 4,5 \rightarrow t_{ins,wall} = \lambda_{ins} \left(R_{\lambda,wall} - \frac{t_{CLT,wall}}{\lambda_{CLT}} \right) \geq 0,038 \cdot \left(4,5 - \frac{0,098}{0,13} \right) = 0,142 \text{ mm}$$

$$R_{\lambda,roof} = \frac{t_{CLT,roof}}{\lambda_{CLT}} + \frac{t_{ins,roofl}}{\lambda_{ins}} \geq 6,0 \rightarrow t_{ins,roof} = \lambda_{ins} \left(R_{\lambda,roof} - \frac{t_{CLT,roof}}{\lambda_{CLT}} \right) \geq 0,038 \cdot \left(6,0 - \frac{0,145}{0,13} \right) = 0,186 \text{ mm}$$

$$R_{\lambda,floor} = \frac{t_{CLT,floor}}{\lambda_{CLT}} + \frac{t_{ins,floor}}{\lambda_{ins}} \geq 3,5 \rightarrow t_{ins,floor} = \lambda_{ins} \left(R_{\lambda,floor} - \frac{t_{CLT,floor}}{\lambda_{CLT}} \right) \geq 0,038 \cdot \left(3,5 - \frac{0,160}{0,13} \right) = 0,086 \text{ mm}$$

An insulation layer of 150 mm and 200 mm is assumed for respectively the walls and roof. The insulation underneath the building and the composition and layout of the foundation are left outside of the scope of this thesis.

On the inside of the parapet, there is a CLT cover of 345 mm between in- and outdoor. This means an additional insulation is required of:

$$R_{\lambda,roof} = \frac{t_{CLT,par}}{\lambda_{CLT}} + \frac{t_{ins,par}}{\lambda_{ins}} \geq 6,0 \rightarrow t_{ins,par} = \lambda_{ins} \left(R_{\lambda,roof} - \frac{t_{CLT,par}}{\lambda_{CLT}} \right) \geq 0,038 \cdot \left(6,0 - \frac{0,345}{0,13} \right) = 0,127 \text{ mm}$$

So a layer of 130 mm is applied on the inside of the parapet. The same is done for the top of the parapet with a CLT cover of 645 mm:

$$R_{\lambda,roof} = \frac{t_{CLT,par,t}}{\lambda_{CLT}} + \frac{t_{ins,par,t}}{\lambda_{ins}} \geq 6,0 \rightarrow t_{ins,par,t} = \lambda_{ins} \left(R_{\lambda,roof} - \frac{t_{CLT,par,t}}{\lambda_{CLT}} \right) \geq 0,038 \cdot \left(6,0 - \frac{0,645}{0,13} \right) = 0,039 \text{ mm}$$

So a layer of 40 mm is applied on top of the parapet.



A.6 Air tightness measures

Stated in the main report, CLT without edge bonding of the boards requires the addition of an airtight envelope around the structure to assure sufficient controllability of the indoor climate. Sheeting materials, like EPDM, are obvious choices to fulfil this function, yet are often unsustainable and non-breathable. Applying such an air tight layer would therefore imply the application of mechanical ventilation systems to control the humidity inside the structure, being undesirable for sustainability reasons, as these systems raise the total energy consumption during exploitation.

A bio-based, breathable foil is desired. Research in the field of bio-plastics (bio-based and/or biodegradable plastics) could provide a suitable solution, yet this research has mostly been directed towards food packaging. More research in terms of the structural application of products like polylactic acid (PLA) or bio-polybutylene succinate (Bio-PBS) is required to assure their performance in terms of airtightness, breathability and biodegradability in their structural application. PLA for example provides breathability, but is only industrially compostable for a thickness of up to 3.2 mm (van den Oever, Molenveld, van der Zee, & Bos, 2017), for which needs to be investigated whether it can provide the required air tightness in a structure.

Since the bio-based solutions are uncertain for direct application at this point in time, alternatives need to be investigated. Breathable, watertight membranes are available, which are not bio-based, but provide the exact characteristics required for this structure.

A.7 External cladding

Challenges in the selection of the material for the external cladding are again the biological resource and overall sustainability while still fulfilling the main requirements that come with the function of cladding. The cladding is the last applied barrier to assure a controlled environment inside the building. As this layer is located at the outer edge of external separations, it provides the first protection layer for the structure against external conditions like rain. It is therefore also subject to the most factors that may influence the durability and appearance of the cladding. To assure no moisture penetrates through the cladding into the insulation material, a small cavity of 20 mm is created between the cladding and the insulation.



Ideally, considering the desire for the structure to be created with only timber, this would also be desirable for the external cladding. Platowood from Arnhem, the Netherlands, could be considered as the supplier of durable timber without the addition of (harmful) chemicals. The characteristics of the FSC-certified timber are amplified through a patented hydro-thermolysis modification process, allowing for application in outdoor conditions. In the process, the timber is heated by steam under increased pressure, and afterwards dried and cured. This means energy consumption is inevitable in the production of the product, but the usage of bio-energy and considering the improvements in maintenance of the cladding means this application is a sustainable solution with a natural appearance. Connecting the cladding to the structure is the last challenge in this part of the structural design. Platowood prescribes assembly by the use of steel fasteners. Any other type of connection is not documented, and given the relatively large connection distance (insulation and cavity), steel is most likely the only option. Between the timber cladding and the ground a distance of 300mm is recommended to mitigate the effects of splashing dirt and water.

In 2015, the Wageningen University started the BPM2 DISCOVER project, to develop sustainable and durable roofing solutions, since most flat roofs are covered with (synthetic) materials produced from petroleum, like bitumen. Bio-based solutions are desired to decrease the fossil fuel consumption and the total environmental impact for roofing. The main challenge is the hydrophilic characteristic of biomass, making it more penetrable for moisture than fossil fuel roofing solutions. This research has not yet led to new applications of bio based roofing systems. Icopal, a research partner in the BPM2 DISCOVER project, till now only offers two “sustainable” roofing systems, which are not sustainable due to their resource, but instead compensate in terms of either air filtering (Noxite) or energy consumption during exploitation of the building (Cool Roof). As long as there are no bio-based roofing products on the market, these options are a suitable option.

To avoid water accumulation within the parapet of the building, a gradient should be applied in the finishing layer of the roof. The gradient should run towards an outlet channel, likely going through the parapet and down the side of the building.



A.8 Acoustic measures

The acoustic requirements from the Building Decree have been stated in Ch.4.7 of the main report. The first important realization in the acoustic performance of these structures is that adjacent houses are structurally uncoupled. Each house provides its own stability, so apart from possible flanking routes through the external cladding, direct sound transmission is not an issue.

The first acoustic measure is to apply a soundproofing profile between all structural members, which can for example be composed of a polyurethane mixture. Such a mixture is free of volatile organic compounds and harmful substances. This profile acts as an acoustic damper, mitigating the propagation of structure-borne noise (vibrations) e.g. from the floor to the walls to adjacent structures.

Internal:

In terms of internal acoustics, only the floors need to be checked for the propagation. They need to assure a sound difference for airborne noise of at least 32 dB, and may propagate a structure-borne sound level of 79 dB at most. The first natural frequency of a 160 mm CLT floor was calculated with a value of 12,31 Hz. The mass law equation, which gives the transmission loss (TL), adjusted for an assumed angle of incidence of the sound waves between 0° and 72°, is:

$$m = \rho_{CLT} \cdot t_{CLT} = 500 \cdot 0,16 = 80 \text{ kg/m}^2$$

$$TL = 20 \log(fm) - 47 \text{ dB}$$

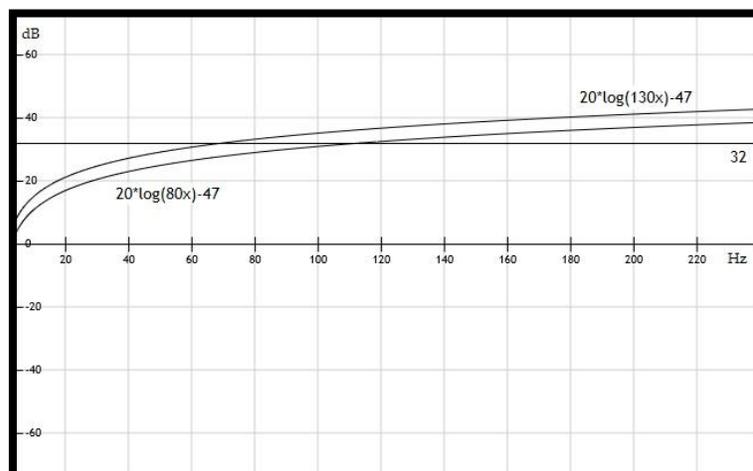


Figure A32 – transmission loss for 160mm CLT



As shown in Figure A32, a transmission loss of 32 dB is reached for a sound frequency of around 120 Hz. The human hearing frequency range is 20 to 20.000 Hz, so this transmission loss for the bare floor of 160 mm is not sufficient. However, if the additional weight from the calculations of 50kg/m² for internal walls and furniture is included, also incorporating the typical frequency range of human speech at a minimum of 80 Hz, and the given exception in the Building Decree for connected spaces (Ch.4.7), the slab of 160mm does satisfy the requirements. Even more favourable becomes the situation when considering a floor topping is required to fit installations inside the floor panels. This layer provides assurance that a floor weight of 130 kg/m² can be reached in the structure, by the addition of ballast underneath the topping. An extra check is found at dataholz.eu, a website providing information on different structural compositions and timber elements. Intermediate floor type GDMTXN01 is composed of similar elements as the floor in this building. The CLT is 130mm thick instead of 160mm, and the added weight is 90 kg/m², but the attenuation of airborne sound is determined at 62 dB, greatly exceeding the required 32 dB. It is safe to assume the 160mm slab with 50 kg/m² additional weight fulfils the minimum requirement.

To mitigate the structure-borne sound transmission of the floor, some type of damping is required. The flanking routes have already been covered by applying the profile between the structural members, but the direct structure-borne sound is easily propagated through the bare 160mm CLT slab. Consider the application of an under screed foil. These foils are applied between the CLT and the floor topping, and act as a damper to attenuate impact sound. Such products are not bio-based, but can be made out of recycled polymers that would fit perfectly into the concept of the circular economy. A theoretical estimate of the impact sound attenuation is 33,5 dB. With the internal structure-borne sound requirement of 79 dB, this means a sound source of 112,5 dB could be permitted, being a relatively high sound pressure level, comparable to a jack hammer. These types of sound levels are not conventional within a building, and thus the floor with under screed foil is assumed to fulfil the internal structure-borne sound requirement.



External:

An air-borne transmission loss of 52 dB is required for the external partitions. This value is mostly relevant for the (load-bearing) house-separating walls, since these partitions form a direct connection between two different houses. The basic composition of this system is two wall slabs of CLT with a thickness of 98 mm, separated by a cavity filled with cellulose. The cellulose contributes to the sound transmission loss through two principles; it adds weight to the separating system and it acts as a damper in between the two relatively stiff CLT slabs. The noise reduction coefficient, also known as the absorption coefficient, of a 65mm thick slab of cellulose has a minimum value of 0,59 (Yeon, Kim, Yang, Kim, & Kim, 2014). So 52 dB transmission loss due to only the absorbance of 65mm of cellulose is guaranteed for noise higher than $52/0,59 = 88$ dB. The actual applied thickness is 100mm due to fire safety. This means the actual absorbance is even higher. Additionally, the CLT slabs increase the transmission loss with a value given by the mass law previously described:

$$m = \rho_{CLT} \cdot t_{CLT} = 500 \cdot 0,098 = 49 \text{ kg/m}^2$$

$$TL = 20 \log(49f) - 47 \text{ dB}$$

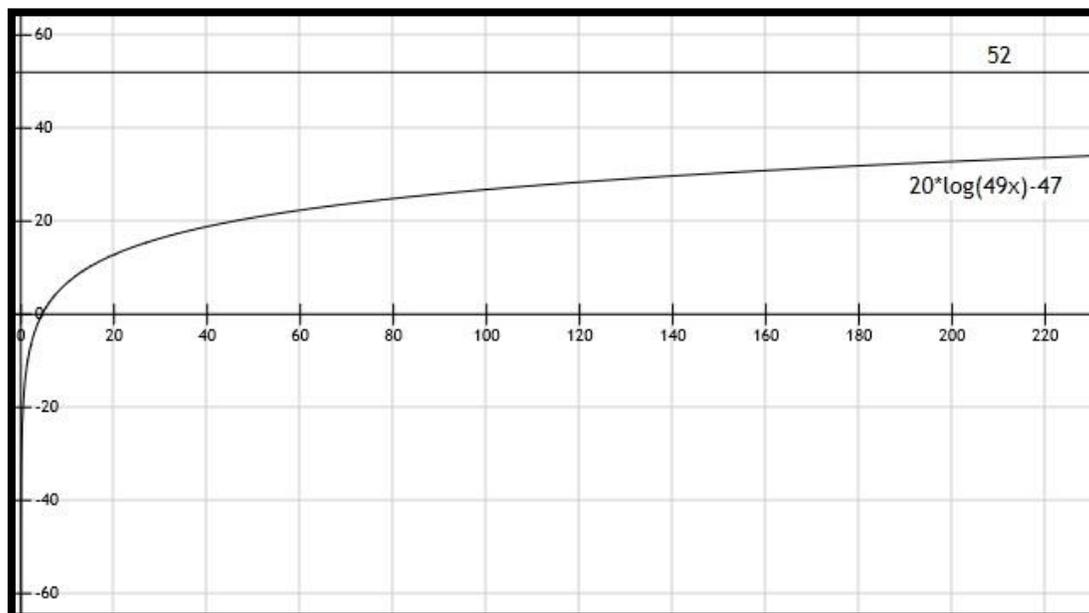


Figure A33 – transmission loss for 98mm CLT

So the transmission loss for a 98mm thick CLT slab for the lowest speaking frequency of human speech (80 Hz) is approximately 25 dB (Figure A33). Considering the partition is composed of two of these slabs, totalling for a transmission loss of 50 dB at 80 Hz, including the absorbing sound insulation



characteristics of the 100mm thick layer of cellulose in-between, this is sufficient to pass the minimum airborne noise cancellation requirement. Dataholz.eu confirms a sound reduction index (R_w) for two 94 mm thick CLT slabs with 30 mm insulation in-between of 48 dB (compartment wall type TWMXXO03A-02).

Regarding the attenuation of structure-borne noise for external partitions, a maximum sound level of 54 dB may be measured at the perceived end. The two CLT slabs in the house-separating composition are structurally uncoupled. This means the contact sound from one slab is first damped by the insulation material and then has to pass through the second CLT slab, for which a sound reduction index of 25 dB was previously determined. This value is confirmed to a certain extent by dataholz.eu, with R_w given as 33 dB for a CLT slab of 100 mm thick (internal wall type IWMXXO01A-01). This means a sound level of 79 is allowed to reach the second CLT slab. Considering the minimum absorption coefficient of the cellulose of 0,59 this leads to an allowed contact sound from the first slab of approximately 190 dB, which is relatively high (Figure A34). Therefore it can be concluded that the house-separating composition satisfies the structure-borne noise requirement.

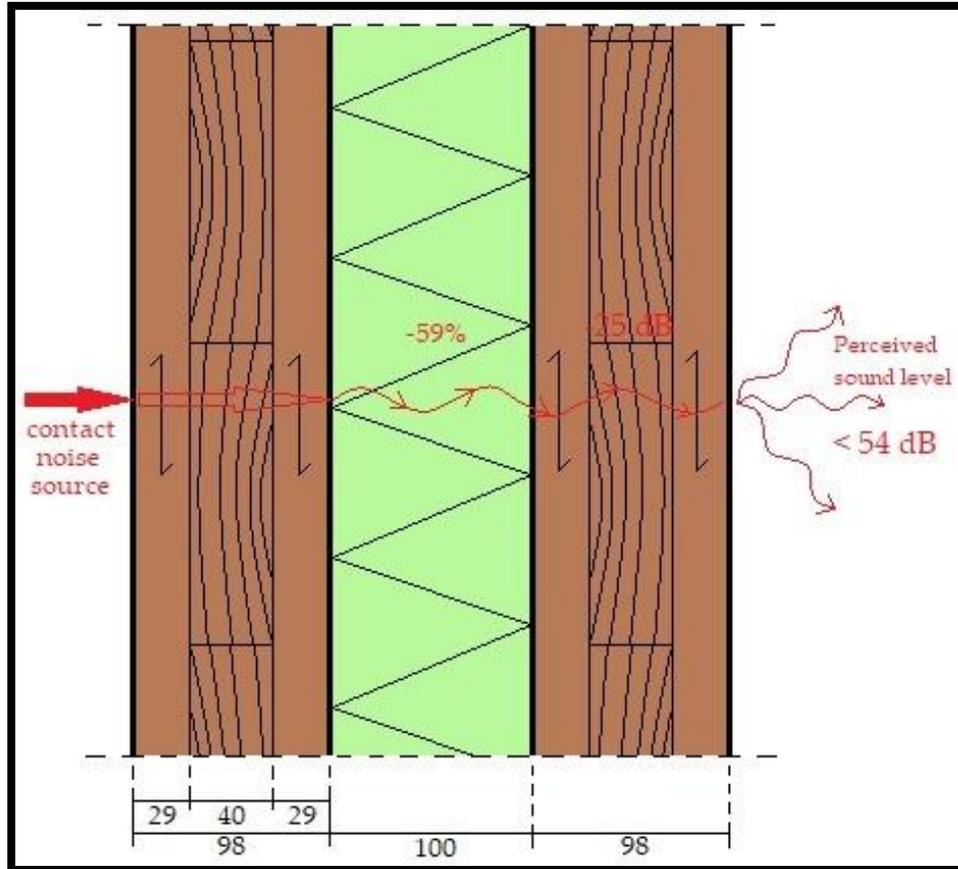


Figure A34 – Contact noise propagation through house-separating wall composition

Finally the sound propagation through the front- and back facade needs to be checked. This wall is composed of 98 mm CLT, 150 mm cellulose insulation, a breathable water resistant membrane, a cavity and 20 mm of timber cladding. The transmission loss for the CLT slab was determined previously for human speech at about 25 dB. The insulation is assumed to absorb 59% of the sound pressure, which has to absorb a minimum of 27 dB. Therefore the sound level must be above 46 dB when it reaches the insulation layer, and thus the wall attenuates enough noise for sound levels above 71 dB.

The wall composition is comparable to external wall type AWMOH003A-02 on dataholz.eu, with a sound reduction index of 43 dB, but a CLT- and cellulose thickness of respectively 100 mm and 200 mm. This sound reduction is too low compared to the requirement, so the applied external wall should be subjected to further testing to assure correct attenuation of sound.



A.9 Ventilation measures

The ventilation requirement for residencies was set at $0,7 \text{ dm}^3/\text{s}$ per square meter of floor area inside a room. Considering no inner walls inside the building: the floor area is equal to $5,4 \times 9,6 \approx 52 \text{ m}^2$, so the total ventilation per floor inside the building needs to be $36,3 \text{ dm}^3/\text{s}$. The ventilation is desired to be regulated naturally through the breathability of the timber facades.

The majority of information on breathability only covers qualitative aspects like the presence of vapour permeability, hygroscopicity and capillarity in a material, as mentioned in Ch.3.7. Quantifying the impact of building materials on the ventilation capacity of partitions proves to be complex, since it is closely linked to the (de-)humidification of a room, but does not fully relate to the ventilation, as the unity given in Ch.3.7 does not correspond with the unity of the requirement.

The CLT structure, which is left in sight in the building, allows vapour permeation and thus controls the moisture content of the air within the building. To prevent the moisture from entering the insulation layer, an additional layer is applied between the CLT and the cellulose. The terms vapour-tight and bio-based do not go hand-in-hand, meaning the barrier will have a negative impact on the total sustainability of the structure.

To fulfil the additional requirements for the kitchen and bathroom areas, mechanical ventilation is probably unavoidable. The assumption is made for the kitchen that it is located underneath the natural ventilation shaft on the ground floor in the centre of the house. The bathroom is located right next to the ventilation shaft on the first floor of the house. Due to these locations it is sensible to use part of the natural ventilation shaft as the drain for the kitchen- and bathroom extractor hoods. Allowing the mechanical ventilation to run on the self-generated energy from e.g. the solar panels means this system does not imply a negative effect on the overall sustainability of the building.



Appendix B: Variant – Hollow core CLT application

Although CLT has proven to be a good building product, guaranteeing quality and relatively fast production rates, appendix A has shown some timber elements over-perform in their relevant structural checks. Research has brought up an interesting variant of hollow core CLT slabs which may result in more favourable performance (van Aken, 2017). This variant will be investigated as a replacement for the CLT in structural elements through a qualitative assessment. This appendix will contain the elaboration of the structural design according to the safety requirements. The other building criteria are discussed in Ch.5 of the main report.

B.1 Assumptions

For this design, the basic assumptions, boundary conditions and requirements are similar to those of the CLT design in Appendix A. The structural composition, including the slab dimensions, will be assumed to remain equal. The connection principles, designed in Appendix A, will therefore not differ between this variant and the CLT design.

B.2 Principle of structural performance

Before elaborating the requirements for hollow core CLT, it is first important to realize why this product could provide a suitable alternative for regular CLT. Considering the bending strength calculation for a certain cross-section, both the accumulated area of a material and the distance from the neutral axis of the total cross section to the central axis of different parts of that material play an essential role. When attempting an improvement in structural performance, applying hollow cores can therefore allow a reduction in material usage for bending members.

Additionally, the creation of hollow cores could allow for the integration of measures within the slab, affecting the different building criteria mentioned throughout this report. The hollow cores could incorporate required additives like installations, acoustic measures and ballast.

The design of the hollow core slabs consist of 2 layers of cross-laminated boards, separated by vertically stacked boards. This leads to ample possibilities to choose from, and selection of an optimum is very complex and differs per required functionality of a structural member. The starting position in the decision-making is the realization that the total timber area should be less for hollow core CLT than



for regular CLT. When mitigating the production steps of the hollow core, one could opt to apply two pre-produced CLT slabs and attach those with GLT beams. These slabs should have a thickness of less than half the thickness of the designed solid CLT member.

The GLT beams need to assure a sound connection between the slabs. They need to be checked on normal and shear stresses, and additionally the spacing needs to be sufficient for the top slab to remain functional and not fail due to a local point load between the beams.

B.3 Hollow core CLT structural design

Floor:

Appendix A has shown a minimum CLT thickness of 160 mm was needed to fulfil the floor requirements. To assure the hollow core design is more material efficient, the thinnest available CLT slabs (for a certain supplier) are applied as the flanges of the slab. Different GLT thicknesses will be considered to inspect the impact on the structural performance.

Spacing:

The minimum CLT thickness produced by a certain chosen supplier is 63 mm with layer composition 20-23-20. With this composition, the spacing (s) of the GLT beams can be determined (Figure B1). The moment of inertia in the weak direction of the top layer (per running meter) is:

$$I_{net,top,weak} = \frac{1}{12}bh^3 = \frac{1}{12}1000 \cdot 23^3 = 1,014 \cdot 10^6 \text{ mm}^4$$

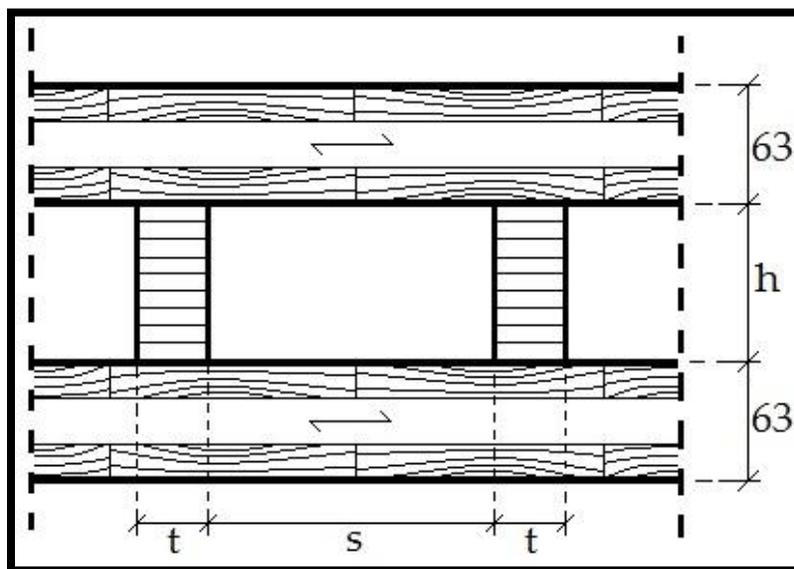


Figure B1 – Hollow core CLT composition



Assuming a GLT width of 80 mm, and given a maximum imposed point load from the national annex of NEN-EN 1991 of 3 kN, this leads to the following checks for the spacing; strength (ULS) and deformation (SLS):

$$f_{m,d} \leq k_{mod} \frac{f_{m,k}}{\gamma_M} = 0,8 \cdot \frac{24}{1,25} = 15,36 \text{ N/mm}^2$$

$$M_{max} \leq \frac{f_{m,d} I_{net,top,weak}}{\frac{1}{2}Z} = \frac{15,36 \cdot 1,014 \cdot 10^6}{\frac{1}{2} \cdot 23} = 1,35 \cdot 10^6 \text{ Nmm}$$

$$s_{max,ULS} \leq \frac{4M_{max}}{1,5 \cdot F_{imp}} = \frac{4 \cdot 1,35 \cdot 10^6}{1,5 \cdot 3 \cdot 10^3} = 1200 \text{ mm}$$

$$w_{max} = \frac{1}{48} \frac{F_{imp} s_{max}^3}{EI_{net,top,weak}} \leq \frac{s_{max}}{300}$$

$$s_{max,SLS} \leq \sqrt{\frac{48EI_{net,top,weak}}{300F_{imp}}} = \sqrt{\frac{48 \cdot 11000 \cdot 1,014 \cdot 10^6}{300 \cdot 3000}} = 771 \text{ mm}$$

So a spacing of 750 mm will be applied. With this spacing, several GLT beam heights are investigated.

Strength:

First the strength requirements for the total cross-section over the span of 5,4 m and for an imposed load of 1,75 kN/m are checked. The minimal height of the GLT is 120 mm, but to achieve a minimal design, smaller heights will be investigated as well. The self-weight consists of the timber cross section and an additional load for toppings and inner walls of 500 N/m. The moment of inertia of the GLT needs to be adjusted to conform to the calculation per running meter:

$$q_d = 0,9 \cdot (0,89 \cdot 1,35 \cdot G_{k,j} + 1,5 \cdot 1,75) = 2,625 + 1,08G_{k,j}$$

Type	h [mm]	I _{CLT} [mm ⁴]	I _{CLT,steiner} [mm ⁴]	I _{GLT} [mm ⁴]	I _{net} [mm ⁴]	A _{net} [mm ²]
63-80-63	80	2,028E+06	5,129E+08	4,551E+06	5,194E+08	1,345E+05
63-100-63	100	2,028E+06	6,536E+08	8,889E+06	6,645E+08	1,367E+05
63-120-63	120	2,028E+06	8,128E+08	1,536E+07	8,302E+08	1,388E+05
63-160-63	160	2,028E+06	1,186E+09	3,641E+07	1,225E+09	1,431E+05
63-200-63	200	2,028E+06	1,633E+09	7,111E+07	1,707E+09	1,473E+05

Table B1 – Cross sectional properties hollow core CLT

Appendix B: Variant – Hollow core CLT application

Type	$G_{k,j}$ [N/m]	q_d [kN/m]	M_{ed} [kNm]	$\sigma_{m,d}$ [N/mm ²]	UC
63-80-63	1160	3,88	14,13	2,80	0,18
63-100-63	1170	3,89	14,18	2,41	0,16
63-120-63	1181	3,90	14,22	2,11	0,14
63-160-63	1202	3,92	14,30	1,67	0,11
63-200-63	1223	3,95	14,38	1,37	0,09

Table B2 – Strength check characteristic load situation, hollow core CLT

The unity checks (Table B2) following from the cross sectional properties (Table B1) of the engineered slabs show a significant margin between the current designs and failure, even for the minimal producible CLT and GLT dimensions.

Deformations:

The long-term deformation requirements consist of the instantaneous and the final deflection.

$$q_{d,inst} = G_{k,j} + 1,75$$

$$w_1 + w_3 = \left(\frac{5}{384}\right) \left(\frac{q_{d,inst} l^4}{EI}\right)$$

$$q_{d,fin} = G_{k,j} + 0,3 \cdot 1,75$$

$$w_2 = \left(\frac{5}{384}\right) \left(\frac{q_{d,fin} l^4}{EI}\right)$$

The maximum deflection is $w_{max} \leq 5400/250 = 21,6$ mm.

Type	I_{net} [mm ⁴]	$G_{k,j}$ [N/m]	$q_{d,inst}$ [kN/m]	$w_1 + w_3$ [mm]	$q_{d,fin}$ [kN/m]	w_2 [mm]	w_{max} [mm]	UC
63-80-63	5,194E+08	1160	2,91	5,64	1,68	3,26	8,90	0,41
63-100-63	6,645E+08	1170	2,92	4,42	1,70	2,57	6,99	0,32
63-120-63	8,302E+08	1181	2,93	3,55	1,71	2,07	5,62	0,26
63-160-63	1,225E+09	1202	2,95	2,43	1,73	1,42	3,84	0,18
63-200-63	1,707E+09	1223	2,97	1,75	1,75	1,03	2,78	0,13

Table B3 – Deflection check characteristic load situation, hollow core CLT

Fire safety:

The strength under fire conditions needs to be determined. The load combination has been given in Appendix A.

$$q_d = G_{k,j} + 0,5 \cdot 1,75 \text{ kN/m}$$

$$M_{Ed} = \frac{q_d l^2}{8} = (0,875 + G_{k,j}) \frac{5,4^2}{8} \text{ kNm}$$



The moment of inertia under fire conditions needs to be adjusted according to the part of the cross-section that gets burnt in the first 60 minutes of a fire. This results in the charring of 42 mm of the CLT slab (orange area in Figure B2). This means one layer in strong direction and one layer in weak direction lose their strength and are excluded in the calculations.

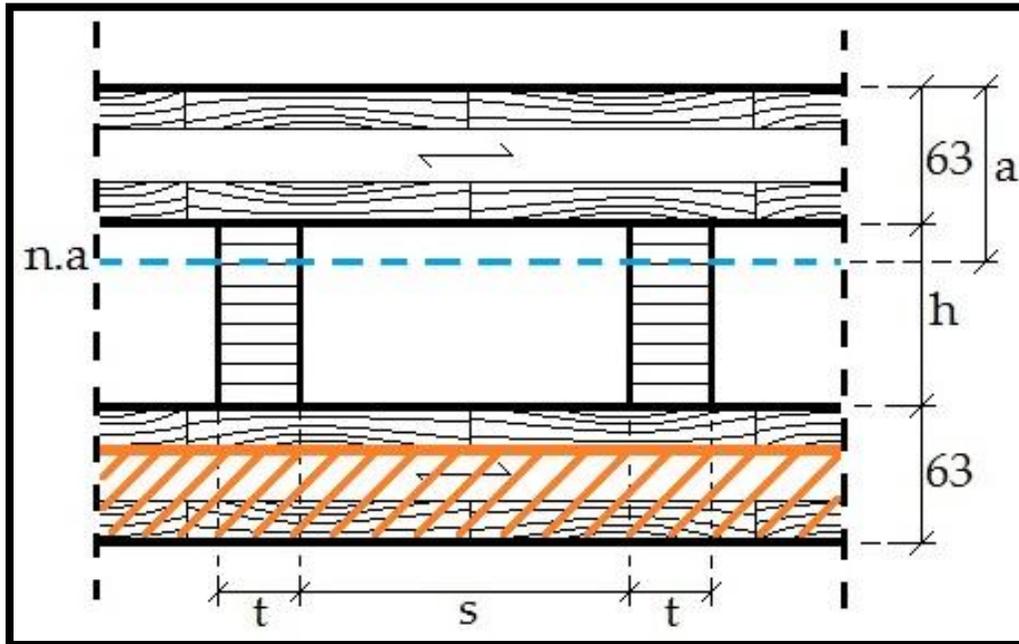


Figure B2 – Hollow core cross-section under fire conditions

The charring causes the neutral axis to shift from its original location in the middle of the slab (blue line in Figure B2). The location of this new neutral axis (for $b = 1000\text{mm}$) is determined as:

$$a = \frac{20 \cdot 1000 \cdot \left(63 + h + \frac{1}{2} \cdot 20\right) + h \cdot t \cdot \left(\frac{1000}{750}\right) \cdot \left(\frac{1}{2}h + 63\right) + 20 \cdot 1000 \cdot 63}{20 \cdot 1000 + h \cdot t \cdot \left(\frac{1000}{750}\right) + 2 \cdot 20 \cdot 1000}$$

Type	h [mm]	a [mm]	$I_{\text{net,fire}}$ [mm ⁴]	q_d [kN/m]	M_{Ed} [kNm]	$\sigma_{m,d}$ [N/mm ²]	UC
63-80-63	80	75,86	2,291E+08	2,03	7,42	2,46	0,16
63-100-63	100	83,85	3,070E+08	2,05	7,46	2,04	0,13
63-120-63	120	91,96	3,986E+08	2,06	7,49	1,73	0,11
63-160-63	160	108,48	6,244E+08	2,08	7,57	1,32	0,09
63-200-63	200	125,38	9,102E+08	2,10	7,65	1,05	0,07

Table B4 – Strength check under fire conditions, hollow core CLT

The examined slab compositions are all able to meet the requirements from the Eurocode, and given the unity checks, they perform too well. To further optimize the engineered slab, thinner or weaker plating material than 63mm CLT should be considered. This has a significant impact on the moment of



inertia of the slab. This investigation is not the main goal of this research, and should be an interesting topic for future research.

Considering A_{total} in Table B1, an equivalent timber thickness can be calculated for the cross section by dividing the value by 10^3 . The 63-80-63 application for all floors would imply a total material saving of:

$$V_{timber,saved} = 3 \cdot 5400 \cdot 9600 \cdot (160 - 134,5) = 3,965 \cdot 10^9 \text{ mm}^3$$

Roof:

Given the fact that a larger CLT thickness was required in Appendix A for the floor than for the roof, the observed hollow core CLT slabs will suffice for the roof slab as well. This implies a material saving of:

$$V_{timber,saved} = 5400 \cdot 9600 \cdot (145 - 134,5) = 0,544 \cdot 10^9 \text{ mm}^3$$

Walls:

The wall slabs for the minimum CLT design have a thickness of 98 mm. The flanges of the hollow core CLT would need to be very thin to decrease the material usage, and less than the minimal available thickness of 63 mm. The decrease of flange thickness by application of a product such as plywood would negatively influence the cross-sectional properties with regard to e.g. fire safety and acoustics.

Additionally, the walls have a stabilizing function. This requires sufficient strength in two directions, whereas the formation of hollow cores increases the strength in the parallel direction, but decreases it in the perpendicular direction. Hollow core CLT is therefore not an option for the wall slabs.

Total structure:

The total material saving accumulates to approximately 7,5% of the total timber usage in the conventional CLT structure.