Feasibility of Tall Timber Buildings
Master Thesis - Structural Engineering
Supplement on Thesis

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Tall Timber Buildings

Additions to Feasibility Study

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October 2011

Picture on the cover: cover picture of att. zuschnitt Vielgeschossiger Holzbau im urbane Raum Dokumentation Forschungsprojekt 8+ proHolz Austria [5]
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1 Simplified Model

1.1 Introduction
The calculation in this chapter will serve as verification on the complicated 3D-models used in the thesis. In addition this approach will also evaluate the feasibility but also the necessity of the refined method that was used in the thesis.

1.2 Manual Beam Model
The simplified model consists of a cantilever beam model as shown in figure 1.1. The cross section of the beam consists of outer walls that create a rectangular hollow section.

![Schematics and Section of the Beam Model](image1.png)

**figure 1.1: Schematics and Section of the Beam Model**

1.2.1 Model geometry
A cantilever beam of which the schematics are shown in figure 1.1 will be assumed to represent the tube structure of the building. The openings in the tube structure for windows are arranged equidistant in both horizontal and vertical directions as shown in figure 1.2 for one half of the tube structures. In figure 1.3 the principle dimensions of the geometry are visible. All windows are the same in size. Openings influence the section properties and the self-weight of the structure.

![Geometry of the tube structure](image2.png)
![Principle of Geometry](image3.png)

**figure 1.2: Geometry of the tube structure**

**figure 1.3: Principle of Geometry**
1.2.2 Material Properties

The assumed material consists of cross laminated timber of a wood base consistent with strength class D70. This material was named D70-CLT in the thesis. The walls that create the tube-structure are about 300 mm and therefore 7 layers thick. The strength properties of the base material are:

<table>
<thead>
<tr>
<th>$f_{m,k}$</th>
<th>$f_{t,0,k}$</th>
<th>$f_{t,90,k}$</th>
<th>$f_{c,0,k}$</th>
<th>$f_{c,90,k}$</th>
<th>$f_{v,k}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>42</td>
<td>0,6</td>
<td>34</td>
<td>13,5</td>
<td>6</td>
</tr>
</tbody>
</table>

The stiffness properties of the base material are:

<table>
<thead>
<tr>
<th>$E_{0,mean}$</th>
<th>$E_{0,05}$</th>
<th>$E_{90,mean}$</th>
<th>$G_{mean}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>16,8</td>
<td>1,33</td>
<td>1,25</td>
</tr>
</tbody>
</table>

The effect on the modulus of elasticity of the 7 cross layer build-up is incorporated with:

$$\frac{E_{eff \parallel}}{E_0} = \frac{m+1}{2 \cdot m} + \frac{m-1}{30 \cdot m}$$

$$\frac{E_{eff \parallel}}{E_0} = \frac{7+1}{2 \cdot 7} + \frac{7-1}{30 \cdot 7} = 0.60$$

The effective modulus of elasticity then becomes:

$$E_{eff} = E_{mean} \cdot \frac{E_{eff \parallel}}{E_0}$$

$$E_{eff} = 20 \cdot 10^3 \cdot 0.60$$

$$E_{eff} = 12000 \frac{N}{mm^2}$$

1.2.3 Section Properties

All the walls of the hollow beam section are assumed to cooperate, uninterrupted by joints. The section properties of the hollow beam section are calculated below. The openings for windows are incorporated through subtraction of their equivalent Steiner part. The assignment of symbols are defined in the left side of figure 1.1.

Assumed is:

$$b_o = h_o, \quad b_i = h_i$$

Second moment of inertia is calculated with:

$$I_{sy} = I_{y,e} + \sum A_i \cdot y_i^2$$
The contribution of the walls without taking openings into account is:

\[ I_{yx,e} = \frac{1}{12} \left( b_o \cdot h_o^3 - b_l \cdot h_l^3 \right) \]
\[ I_{yx,e} = \frac{1}{12} \left( 29100^4 - 28500^4 \right) \]
\[ I_{yx,e} = 4,77 \cdot 10^{15} \text{mm}^4 \]

The cross section area of openings in the tube structure are equal in size, the following is therefore true:

\[
\{ A_i^n \}_{i=1} = A_{\text{open}} \\
A_{\text{open}} = b \cdot t = 900 \cdot 300 \\
\{ A_i^n \}_{i=1} = 270 \cdot 10^3 \text{mm}^2
\]

The section is symmetrical and the distance of openings to the center of gravity is equal for ten windows in one flange wall and for each set of two windows in the web walls. The calculation of the internal lever arm is therefore:

\[
\sum y_i^2 = 2 \left( 10 \cdot 14400^2 + 2 \cdot (14400 - 2250)^2 + 2 \cdot (14400 - 4950)^2 + 2 \cdot (14400 - 7650)^2 + \\
2 \cdot (14400 - 10350)^2 + 2 \cdot (14400 - 13050)^2 \right)
\]
\[
\sum y_i^2 = 5,35 \cdot 10^9
\]

Second moment of inertia and the section modulus are then:

\[
I_{xy} = 6,37 \cdot 10^{15} - 270 \cdot 10^3 \cdot 5,35 \cdot 10^9 \\
I_{xy} = 6,37 \cdot 10^{15} - 1,44 \cdot 10^{15} \\
I_{yy} = 3,32 \cdot 10^{15} \text{mm}^2 \\
W_y = \frac{2 \cdot I_{yy}}{h} = \frac{2 \cdot 3,32 \cdot 10^{15}}{28800}
\]
\[
W_y = 231 \cdot 10^9 \text{mm}^3
\]

Cross section area with reduction of openings is:

\[
A = b_o \cdot h_o - b_l \cdot h_l - n \cdot A_{\text{open}} \\
A = 29100^2 - 28500^2 - 40 \cdot 270 \cdot 10^3 \\
[A = 23,76 \cdot 10^6 \text{mm}^2]
\]
1.2.4 Support Stiffness

The rotational support stiffness of the model is calculated below using the scheme shown in figure 1.4.

The general equation of rotational stiffness of the support is deduced as follows:

\[ M = \sum_{i=1}^{n} F_i \cdot a_i \]
\[ F_i = k_i \cdot u_i \]
\[ u_i = \varphi \cdot a_i \]
\[ M = \sum_{i=1}^{n} k_i \cdot a_i^2 \]
\[ k_i = \sum_{i=1}^{n} k_i \cdot a_i^2 \]

The spring stiffness of the piles is calculated with:

\[ k = \frac{EA}{l} \]

In which:

\[ E = 20 \cdot 10^9 \, \frac{N}{m^2} \]

\[ A = \frac{1}{4} \cdot \pi \cdot D^2 = \frac{1}{4} \cdot \pi \cdot 1,50^2 = 1,77 \, m^2 \]

\[ l = 25m \]

The spring stiffness of the piles is:

\[ k = \frac{EA}{l} = \frac{20 \cdot 10^9 \cdot 1,77}{25} = 1,42 \cdot 10^9 \, \frac{N}{m} \]

The layout of the foundation allows for the following assumption:

\[ k_1 = k_2 = \cdots = k_i = \cdots = k_{n-1} = k_n = k \]
Based on the layout of the foundation shown in figure 1.5 the rotational spring stiffness is:

\[ k_r = \sum_{i=1}^{n} k_i \cdot a_i^2 = 2 \cdot 5 \cdot 1,42 \cdot 10^9 \cdot 14,4^2 + 2 \cdot 2 \cdot 1,42 \cdot 10^9 \cdot 7,2^2 \]

\[ k_r = 3,23 \cdot 10^{12} \text{ Nm/ rad} \]

1.2.5 Load Cases

The load cases are defined below based on calculations and assumptions made in the thesis.

Self-Weight Main-Structure (LC1)

The gravity of load of the structure is calculated as the density of the material times the cross section area. Based on figure 1.3 the cross section area is reduced with a factor for openings as follows:

\[ A_g = \left(1 - \frac{A_{open}}{A_{wall}}\right) \cdot (b_y \cdot h_y - b_i \cdot h_i) \]
\[ A_g = \left(1 - \frac{900 \cdot 1800}{2700 \cdot 3500}\right) \cdot (29100^2 - 28500^2) \]
\[ A_g = 28,64 \cdot 10^6 \text{ mm}^2 \]

The Self-Weight of the structure then becomes:

\[ q_g = \rho_k \cdot g \cdot A_g \]
\[ q_g = 900 \cdot 10 \cdot 28,6 \]
\[ q_g = 258 \text{ kN/m}^3 \]
Permanent Vertical Loads (LC2 + LC3)
The permanent vertical loads originate partially from the architectural façade and partially from the floor structure. It is assumed that a building core is present to carry vertical loads, but not lateral wind loads (horizontal loads). The value of the permanent floor load is multiplied by the floor area supported by the tube structure according to figure 1.6. The load is divided over the floor height in order to create a uniform distributed load. The permanent façade load is multiplied by the perimeter of the building. The value of the permanent load cases is therefore as follows:

\[
q_p = \frac{1}{h_f} \cdot (G \cdot A_f + P \cdot p) \\
h_f = 3.50 \text{m} \\
q_p = \frac{1}{3.50} \cdot (4.45 \cdot 363 + 2.45 \cdot 115) \\
G = 4.45 \text{ kN/m}^2 \\
A_f = 363 \text{ m}^2 \\
q_p = 542 \text{ kN/m}^2 \\
P = 2.45 \text{ kN/m}^2 \\
p = 4 \cdot 28.8 = 115 \text{ m}^2
\]

![Figure 1.6: Area of Floor Load](image)

Variable Vertical Loads (LC4)
The value of the variable floor load is multiplied by the floor area supported by the tube structure according to figure 1.6. The load is divided over the floor height in order to create a uniform distributed load. The value of the variable floor load case is therefore as follows:

\[
q_v = \frac{1}{h_f} \cdot Q_f \cdot A_f \\
h_f = 3.50 \text{m} \\
Q_f = 2.50 \text{ kN/m}^2 \\
A_f = 363 \text{ m}^2 \\
q_v = \frac{1}{3.50} \cdot 2.50 \cdot 363 \\
q_v = 259 \text{ kN/m}^2
\]
Equivalent Wind Load (LC6)
The value of the wind load case is based on the equivalent and uniform wind load. The width over which this load acts is adapted (increased) to match the imposed wind load on the 3D-models of the thesis. The representative wind load value is calculated with:

\[ q_w = Q_w \cdot b_m \]

In which:

\[ Q_w = 2.18 \frac{kN}{m^2} \]
\[ b_m = 30.00 + 7.20 = 37.20m \]

The representative wind load value is therefore:

\[ q_w = 2.18 \cdot 37.20 \]
\[ q_w = 81.1 \frac{kN}{m} \]

Summery
The loads cases that were calculated above are summarized here:

<table>
<thead>
<tr>
<th>q_g</th>
<th>q_p</th>
<th>q_q</th>
<th>q_w</th>
</tr>
</thead>
<tbody>
<tr>
<td>258</td>
<td>542</td>
<td>259</td>
<td>81.1</td>
</tr>
</tbody>
</table>
1.2.6 Verification of Load Values

In this paragraph the values of the calculated loads for the manual beam model are compared to the loads and reactions of the equivalent 3D-model, which is the NO-CORE model of Variant 3: 2D-Open as described in paragraph 6.5 of the thesis.

Total loads - Manual beam model

The total representative permanent load is:

\[ F_{x,p} = \sum q_{p,rep} \cdot l = (258 + 542) \cdot 112 \]

\[ F_{x,d} = 89600 \text{ kN} \]

The total representative variable load is:

\[ F_{x,v} = q_{v,rep} \cdot l = 259 \cdot 112 \]

\[ F_{x,v} = 29008 \text{ kN} \]

The total of the representative horizontal wind load on the simplified beam model is:

\[ F_{y,p} = q_{w,rep} \cdot l = 81.1 \cdot 112 \]

\[ F_{y,w} = 9083 \text{ kN} \]

The representative moment due to horizontal wind loading at the support of the cantilever beam model is:

\[ M_d = \frac{1}{2} \cdot q_{w,d} \cdot l^2 = \frac{1}{2} \cdot 81.1 \cdot 112^2 \]

\[ M_d = 509 \cdot 10^3 \text{ kNm} \]

All load values and the resulting moments of the simplified beam model are approximately equal to the loads that are used in the 3D-model(s), except for the variable floor load.

The difference in variable floor load does not influence the deflection at the top for the simplified beam model. The variable floor load is also small in comparison to the total permanent loads. It can therefore be assumed that this not to influence the results significantly.

1.2.7 Load Combinations

The load combinations are based on the Eurocode in combination with assumptions made in the thesis.

<table>
<thead>
<tr>
<th>Load Combinations</th>
<th>LC 1</th>
<th>LC 2</th>
<th>LC 3</th>
<th>LC 4</th>
<th>LC 5</th>
<th>LC 6</th>
<th>LC 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description:</td>
<td>Self-Weight</td>
<td>Perm. Floor</td>
<td>Var. facade</td>
<td>Var. Floor</td>
<td>Var. Wind</td>
<td>Eq. Wind</td>
<td>Dynamic Wind</td>
</tr>
<tr>
<td>C0: ULS</td>
<td>1,20</td>
<td>1,20</td>
<td>1,20</td>
<td>1,50</td>
<td>-</td>
<td>1,50</td>
<td>-</td>
</tr>
<tr>
<td>C1: ULS</td>
<td>1,35</td>
<td>1,35</td>
<td>1,35</td>
<td>0,75</td>
<td>-</td>
<td>0,75</td>
<td>-</td>
</tr>
<tr>
<td>C3: SLS</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
<td>-</td>
<td>1,00</td>
<td>-</td>
</tr>
<tr>
<td>C4: ULS Fire</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
<td>-</td>
<td>0,20</td>
<td>-</td>
</tr>
</tbody>
</table>
1.3 Calculation

The calculation of forces is based on the principles of classical mechanics. In this paragraph the limit state calculations and verifications are given.

1.3.1 Ultimate Limit States

The forces in the beam are calculated here to conduct a verification of the ultimate limit state.

The used section properties are:

\[
W_y = 231 \cdot 10^9 \text{ mm}^3 \\
A = 23,76 \cdot 10^6 \text{ mm}^2
\]

The representative load values are:

\[
q_g = 258 \text{ kN/m} \\
q_p = 542 \text{ kN/m} \\
q_q = 259 \text{ kN/m} \\
q_w = 81,1 \text{ kN/m}
\]

The maximum axial force in the beam under vertical loading is for load combination C0 and C1 respectively:

\[
F_{x,d} = \sum q_{i,d} \cdot l = (1,20 \cdot 258 + 1,20 \cdot 542 + 1,50 \cdot 259) \cdot 112 \\
F_{x,d} = 151 \cdot 10^3 \text{ kN}
\]

\[
F_{x,d} = \sum q_{i,d} \cdot l = (1,35 \cdot 258 + 1,35 \cdot 542 + 0,75 \cdot 259) \cdot 112 \\
F_{x,d} = 143 \cdot 10^3 \text{ kN}
\]

The maximum bending moment in the beam under wind loading is for combination C0:

\[
M_d = \frac{1}{2} \cdot q_{w,d} \cdot l^2 = \frac{1}{2} \cdot (1,50 \cdot 81,1) \cdot 112^2 \\
M_d = 762 \cdot 10^3 \text{ kNm}
\]

The force acting on a plate strip of 1,00 m wide at the location of the highest stress (base) is:

\[
F_{d,1} = \left( \frac{M_d}{W_y} + \frac{F_{x,d}}{A} \right) A_i \\
F_{d,1} = \left( \frac{762 \cdot 10^9}{231 \cdot 10^9} + \frac{151 \cdot 10^6}{23,76 \cdot 10^6} \right) \cdot 1000 \cdot 300 = (3,30 + 6,35) \cdot 300 \cdot 10^3 \\
F_{d,1} = 2896 \cdot 10^3 \text{ kN}
\]

Ultimate limit state verification for the plate strip consists of a buckling calculation. The buckling calculation of a 7 layer thick plate can be found in paragraph 3.9 of the thesis, of which the buckling resistance is:

\[
F_{\text{buc}} = 3154 \cdot 10^3 \text{ kN}
\]
This implies for the unity check:

\[ UC := \frac{2896 \cdot 10^3}{3154 \cdot 10^3} = 0.92 \]

### 1.3.2 Serviceability Limit State

The deflection at the top of the beam is calculated to conduct a verification of the serviceability limit state.

The used section properties are:

\[ I_{yy} = 3.32 \cdot 10^{15} \text{mm}^4 \]

The effective modulus of elasticity is:

\[ E_{eff} = 12000 \frac{N}{\text{mm}^2} \]

The deflection of the beam itself is:

\[ w = \frac{1}{8} \cdot \frac{q \cdot l^4}{E I} = \frac{1}{8} \cdot \frac{(1,00 \cdot 81,1) \cdot (112 \cdot 10^3)^4}{12000 \cdot 3,32 \cdot 10^{15}} \]

\[ w = 40 \text{mm} \]

The deflection at the top due to rotation of the foundation is:

\[ w_f = l \cdot \varphi \]

\[ \varphi = \frac{M}{k_r} \]

\[ M = \frac{1}{2} \cdot q \cdot l^2 = \frac{1}{2} \cdot 81,1 \cdot 112^2 = 508 \cdot 10^3 \text{Nm} \]

\[ \varphi = \frac{508 \cdot 10^6}{3,23 \cdot 10^{12}} = 1,57 \cdot 10^{-4} \text{ rad} \]

\[ w_f = 112 \cdot 10^3 \cdot 1,57 \cdot 10^{-4} \]

\[ w_f = 18 \text{mm} \]

The total deflection at the top is:

\[ \sum w = 40 + 18 = 58 \text{mm} \]

The elaborate equivalent of this manual beam model is the NO-CORE model of variant 3: 2D-Open as described in §6.5 of the thesis. The deflection of this equivalent 3D model is 130,9 mm and thus 126% higher than the simplified model. This is due to shear deformation of the section. To proof that theory a simple finite element beam model is produced using GSA Oasys software.
1.4 FEM Beam Model

Because a simplified Euler beam model does not incorporate the shear deformation, a simple finite element beam model is produced using GSA Oasys software to verify the deflection at the top. The beam model is actually the same as the cantilever beam model shown in figure 1.1 as shown in figure 1.7.

1.4.1 Model Geometry

A picture of the beam model is shown in figure 1.7. The beam consists of two nodes and one element. The nodes are numbered node 1 and node 2 as shown in the picture. The coordinates of the nodes are:

<table>
<thead>
<tr>
<th>Node</th>
<th>X</th>
<th>Y</th>
<th>Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0.12</td>
</tr>
</tbody>
</table>

The element is spaced between node 1 and node 2 as shown in the picture. The element is fixed to both nodes in all translational and rotational directions (beam releases are fixed).

Node 1 is clamped, i.e. fixed in all translational and rotational directions.

Openings of windows will be incorporated using the modified moment of inertia calculated earlier and a modified shear modulus coming up in the next paragraph.

1.4.2 Material Properties

The modulus of elasticity is taken equal to the manual beam model and is therefore:

\[ E_{eff} = 12000 \text{ N/mm}^2 \]

The effect on the shear modulus of the 7 cross layer build-up is incorporated with:

\[ \frac{G_{FE-FIT}}{G} = 0.79 \]

The effect on the shear modulus of the openings is incorporated with:

\[ \frac{G_{WALL}}{G} = 0.52 \]

The effective shear modulus then becomes:

\[ G_{eff} = G_{mean} \cdot \frac{G_{FE-FIT}}{G} \cdot \frac{G_{WALL}}{G} \]
$G_{eff} = 1250 \cdot 0.79 \cdot 0.52$

\[ G_{eff} = 514 \, \frac{N}{mm^2} \]

### 1.4.3 Section Properties

Because the shear modulus already takes into account the effect of the openings the cross section area is:

\[ A = b_o \cdot h_o - b_i \cdot h_i \]

\[ A = 29100^2 - 28500^2 \]

\[ A = 34.56 \cdot 10^6 \, mm^2 \]

The second moments of inertia are taken equal to the manual beam model and are therefore:

\[ I_{yy} = I_{zz} = 3.32 \cdot 10^9 \, mm^3 \]

The torsional constant of the section is:

\[ J = \sum A \cdot a^2 \]

\[ J = n \cdot b \cdot t \cdot a^2 \]

\[ J = 4 \cdot 28800 \cdot 300 \cdot 14400^2 \]

\[ J = 7.17 \cdot 10^9 \, mm^4 \]

The section properties are entered in the beam section table of GSA Oasys as:

<table>
<thead>
<tr>
<th>Property</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
<th>J</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
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<td>Material</td>
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<tr>
<td>Axes for Stiffness Calculation</td>
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<tr>
<td>Area</td>
<td>$I_{yy}$ or $I_{xx}$</td>
<td>$I_{zz}$ or $I_{yy}$</td>
<td>$J$</td>
<td>$K_y$ or $K_u$</td>
<td>$K_z$ or $K_v$</td>
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<td>Exploded View Areas</td>
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<tr>
<td>3.456e+007</td>
<td>3.222e+015</td>
<td>3.333e+015</td>
<td>7.1893e+00</td>
<td>0.41868</td>
<td>0.41868</td>
<td></td>
<td></td>
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</tbody>
</table>

Note the factors $K_y$ and $K_u$, these are defined as $[\text{Shear Area}] / [\text{Total Area}]$. For rectangular hollow sections it is common to only take into account $\frac{2}{3}$ of the web areas as effective shear area. The factors $K_y$ and $K_z$ are therefore calculated as:

\[ K_y = K_z = \frac{A_y}{A_{tot}} \]

\[ A_y = \frac{5}{6} \cdot h \cdot 2 \cdot t = \frac{5}{6} \cdot 28800 \cdot 2 \cdot 300 \]

\[ A_y = 14,40 \cdot 10^6 \, mm^2 \]

\[ K_y = K_z = 0.4166 \]
1.4.4 Load case
The only load case imposed on the model is the uniform distributed wind load and is taken equal to the simplified beam model and is therefore:

\[ q_w = 81.1 \text{ kN/m} \]

1.4.5 Load combinations
The only load combination is the serviceability load combination C3 SLS which is taken equal to the manual beam model.

1.4.6 Calculation Results
The output of the GSA Oasys software resulted in a maximum deflection of 108 mm for the FEM beam model as it is described above.

By keeping all other input of in the model the same and either adjust the shear modulus (G) or the shear reduction factors \( K_y \) and \( K_z \), it can be clearly seen that most deformation is due to the shear stiffness of the system. The maximum deflection can range from 57.2 mm to 211 mm by changing the K-factors from 1.0 to 0.1 respectively. This is explainable because the shear stiffness is build up out of the shear modulus (G) and the shear area (A_s). The combination is described as a perfect match with the 3D model(s) because:

\[ \sum w = 108.8 + 18 = 127 \text{ mm} \]

1.5 Conclusion
The calculations of the manual beam model show results that are different from the more complicated 3D finite element model. The manual beam model results in equal stresses and forces at the base, but a smaller deflection at the top. The additional FEM beam model was decisive proof that the deflection is significantly influenced by the shear stiffness.

The equivalent of the simplified beam model is the 3D model of variant 3: 2D-Open as described in §6.5 of the thesis. The deflection under loading of this equivalent 3D model is 130.9 mm and therefore 172% higher than the manual beam model, but is almost equal to the FEM beam model.

The maximum occurring compression force is however almost equal of the manual beam model when compared to the equivalent 3D model, as shown in graph 6.7 of the thesis.

From these two observations the flowing conclusion can be drawn, namely: the manual beam model is sufficiently accurate to calculate the occurring forces, but is not representative for the deflection because the model does not incorporate the shear stiffness in the calculation.

It can also be concluded that the ratio between the shear modulus (G) and the modulus of elasticity (E) is of influence for tall timber buildings and is unique because this ratio is small compared to steel or concrete. It is therefore that a manual beam model works for concrete building structures and steel frames where a small shear lag factor (10%) is sufficient, while for timber building structures a much larger reduction of the bending stiffness can be necessary.

The influence of the following factors on the deflection and the stress distribution could not be investigated by the simplified beam models:
- Stiffness of the joints
- The tube structure geometry
- The static indeterminate interaction between the foundation supports and the tube structure.

It was necessary to make calculations with different equivalent models, otherwise these issues would not be revealed. Summarizing, the influence of: the shear modulus; the joint stiffness; the geometry and the foundation can only be investigated in with more elaborate models.
A Wind Load Moment

The wind loads as they are imposed on the equivalent 3D-model are given in figure A.1. Wind load vectors are vertically spaced with a distance of 7 m and each row consists of 5 vectors.

![Figure A.1: Wind loads on the 3D-model of Variant 3: 2D-Open](image)

The calculation is therefore as in the table below:

<table>
<thead>
<tr>
<th>i</th>
<th>$F_i$ [kN]</th>
<th>$L_i$ [m]</th>
<th>$F_i \times L_i$ [kNm]</th>
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<tr>
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<td>85</td>
<td>7</td>
<td>595</td>
</tr>
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$105189 \times 5 = 525945$ kNm