ABSTRACT

Post-tensioned timber joints have been studied at the Institute of Structural Engineering at the ETH Zurich. A post-tensioned beam-column timber joint has been developed using glued laminated timber with local hardwood reinforcement. Only a single straight tendon is placed in the middle of the beam and post-tensioned to restrain the rotation of the joint. No additional steel elements are used. The developed joint is characterised by a high degree of pre-fabrication and easy assembly on site. The glue-laminated timber moment-resisting frame featuring this joint shows great potential for timber frame structures especially for multi-storey buildings.

Robust performance-based design criteria are a necessity for a successful market implementation of the proposed system. A simplified analytical model was developed in order to predict the structural performance of the post-tensioned timber connection and facilitate preliminary performance-based design. This model was implemented in OpenSees using a bi-linear rotational spring approach. The model was verified against an analytical model and validated against tests on a post-tensioned timber joints under gravity loading and under horizontal loading. Moreover, a complex numerical model was developed using OpenSees to check the accuracy of the proposed preliminary design model.

The preliminary design model was used to design fictitious moment-resisting post-tensioned glue-laminated timber frame structures using the loads prescribed in the Swiss code. The design process showed that neither the gravity loads nor the seismic load controlled the design. The design was governed by the lateral deformations due to wind. Based on this finding it is recommended to focus further research on increasing the connection stiffness or on adding additional structural elements to address the seismic performance of post-tensioned timber frames in regions of high seismicity.

KEYWORDS

Post-tensioned, timber frame, performance-based design.

INTRODUCTION

In order to compete with structures made of concrete, timber structures have to be safe, economical, assembled quickly, and flexible. Timber frame structures fulfil the mentioned requirements. A disadvantage is the complicated connection between beam and column that are often bolted and reinforced with screws. These connections are able to resist a moment, which is however often neglected (Lam et al. 2010) and the beams are often designed as simply supported (Kolb 2008). A system that has been developed in New Zealand at the University of Canterbury is suitable for timber frames. The system uses the post-tensioning technique to provide for moment resistance and quick assembly (Buchanan et al. 2008, Newcombe 2011, Smith et al. 2011, Sarti et al. 2012, van Beerschoten 2013, Moroder et al. 2014). The system developed in New Zealand has been adapted at ETH Zurich to fit the European market. The result is a post-tensioned timber connection shown in Figure 1.
The beam-column timber joint is made of glued laminated timber (Norway spruce, *picea abies*) with local strengthening of the column in the connection region using hardwood (European ash, *fraxinus excelsior*). No steel elements are required. Only a single straight tendon is placed horizontally through a cavity in the middle of the beams and the column to form the moment-resisting timber joint. The post-tensioned beam-column timber joint is characterized by a high degree of pre-fabrication and easy assembly on site.

The proposed system was evaluated in a series of large-scaled tests, i.e. tests under vertical loading on a post-tensioned timber connection (Figure 2 left, Wanninger and Frangi 2014b) and pushover tests on a post-tensioned timber frame (Figure 2 right, Wanninger 2015).

The tests demonstrated the self-centering behaviour of the proposed system under gravity loads as well as under horizontal loads. Practically no damage was observed during the test under gravity loads even for loads much higher than the design loads. A premature failure in a finger-joint occurred during the pushover-tests. However, this failure had no influence on the load bearing capacity of the frame, and only resulted in a deterioration of its lateral stiffness (Wanninger 2015).

**MODELLING**

Different models were used to evaluate the structural behaviour of the post-tensioned timber structural frames. An analytical model, a numerical model, as well as a simplified numerical model using rotational springs were developed and validated against the results obtained in the tests.

**Analytical Model**

An analytical spring-based model was introduced by the author and is described in more detail in Wanninger and Frangi (2014a) as well as in Wanninger (2015). The model assumes that the softer column can be replaced with springs embedded in the stiffer column. The model allows for calculating the connection behaviour with simple equations that can be derived using equilibrium conditions.
Numerical Model

A model, based on the analytical approach described above, was developed using the OpenSees finite element modelling framework (McKenna and Fenves 2006) to simulate the behaviour of the connection specimen, which was subjected to vertical loading as shown in Figure 4. This model was subsequently used to simulate the pushover tests on the post-tensioned timber frame shown in Figure 5. Both models were created using these elements:

- Quadrilateral shell elements for the timber parts and also the steel plates (anchorage for the tendon)
- Corotational truss elements for the tendon (1D element)
- Zero length elements for the connection area to model the gap between the column and the beams

The material properties assigned to the model are identical to the properties used in the analytical model:

- J2-plasticity for the joint area of the model, which takes into account embedment failure perpendicular to the grain in the column (controlling design criterion)
- Elastic orthotropic material for the columns of the model, which takes shear deformations into account (pushover tests). The Poisson’s ratio $\nu_{RL}$ was set to 0.059 (Green et al. 1999).
- Elastic isotropic material for the beams
- Uniaxial initial strain material for the tendon in combination with uniaxial material “Steel02”. The initial strain is used to apply the tendon force
- Elastic isotropic material for the steel plates at the end of the beams (anchorage)

**Model Comparison**

The obtained results from the analytical model (labelled “model”) as well as for the numerical model (labelled “opensees”) are shown in Figure 6 for the connection tests under gravity load and in Figure 7 for the frame pushover tests together with the experimental results.

Both modelling approaches match the test results under gravity load well (Figure 6). The numerical model also includes yielding of the tendon, which the analytical model does not.

![Graphs showing model comparison](image)

**Figure 6** Results obtained from tests on the post-tensioned timber connection as well as analytical model (model) and numerical model (opensees). Shown are the Moment M, tendon force P, position of the neutral axis x, the stresses at the interface $\sigma_{int}$ over the connection rotation $\theta$.

The results for the pushover-tests (Figure 7) were predicted well with the analytical model whereas the numerical model predicts a softer behaviour than actually measured during the tests on the frame specimen. The difference is a result of the simplifications made with the analytical model; the analytical model assumes rigid beams, whereas the numerical model takes their stiffness into account.
Figure 7 Results from the pushover tests with the analytical prediction (model) as well as the numerical prediction (opensees). The horizontal force $F$ is plotted with the measured horizontal displacement of the frame.

**PRELIMINARY DESIGN**

A preliminary design, presented in this section, can be performed using OpenSees or any similar software that is able to model a frame structure. The connection between the columns and beams is modelled using a rotational spring. The spring properties can be derived using either with the analytical or the numerical model presented in the previous section. The design is demonstrated on the ETH House of Natural Resources (Leyder et al. 2015), a building with a two-storey post-tensioned timber frame situated on the ETH Hönggerberg campus. The design is performed according to the Swiss SIA standards (SIA260, SIA261, SIA265).

**Structure**

The structural system of the building consists of a post-tensioned timber frame and two different slab systems. The frame has a height of 6 m (2.3 m stories and three 6.5 m span bays) in both directions (Figure 8). The beams have a cross section of $720 \times 280$ mm and are made of glulam GL24h (SIA 265 2003). The hardwood columns have a cross section of $380 \times 380$ mm and are of the strength grade D40 (EN 338 2010). A tendon force of 500 kN was chosen to achieve the building design objectives. This force has to be guaranteed over the lifetime of a building; thus, the initial tendon force was set to 700 kN, i.e. nearly 30% tendon force losses are accounted for (Wanninger et al. 2014c).

The slabs on the first storey are timber-concrete composite slabs, made using beech LVL (laminated veneer lumber) as reinforcement (Boccadoro and Frangi 2013). The slabs on the roof are also timer-concrete composite slabs made with beech LVL. The design of the slabs is more conventional; a 70 mm concrete slab is placed on beech LVL beams with a cross section of $240 \times 80$ mm. Under the beams are the 40 mm thick LVL plates acting as tension reinforcement (Figure 8). The design of the slabs will not be discussed herein. The span of the slabs was altered within each field of the frame to guarantee a uniform load distribution from the slabs to the frame.

The columns are modelled with pinned connections at the bottom to simplify the boundary conditions. The actual column support conditions are semi-rigid in order to facilitate the transfer of both gravity and horizontal loads.

Figure 8 Post-tensioned timber frame (left) and slab systems (right)
The gravity loads are summarised in Table 1. The weight of the frame itself is not listed in the gravity loads but it is taken into account in the calculations. The dead load of the 1st storey includes the weight of the slabs as well as the weight of the underlay and the floor covering. In addition to the dead load, 0.5 kN/m² were added to cover non-structural elements such as office partition walls. The dead load of the 2nd storey includes the weight of the slabs as well as the weight of the entire roof structure. The live loads are given by the standards for an office building (1st storey) and for a non-accessible roof (2nd storey). The snow load was calculated for the building’s location in Zurich. The horizontal loads are also summarised in Table 1. The global wind force was calculated for the given building’s geometry and its location. The ground acceleration is chosen according to the 2003 Swiss standards for a site in Zurich. The soil at the site is classified into seismic class C.

Table 1: Characteristic loads for the design of the post-tensioned timber frame based on the Swiss standards SIA

<table>
<thead>
<tr>
<th>Gravity loads</th>
<th>1st story</th>
<th>2nd story</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load slabs</td>
<td>g_k</td>
<td>6.4 kN/m²</td>
</tr>
<tr>
<td>Live load</td>
<td>q_k</td>
<td>3.0 kN/m²</td>
</tr>
<tr>
<td>Snow loads</td>
<td>q_k,s</td>
<td>-</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Horizontal loads</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind force</td>
<td>q_k,w</td>
<td>1.0 kN/m²</td>
</tr>
<tr>
<td>Ground acceleration</td>
<td>a_g,d</td>
<td>0.6 m/s²</td>
</tr>
</tbody>
</table>

Design for Gravity Loads

The analysis was performed using OpenSees. The frame was modelled with elastic “BeamColumn-elements” as shown in Figure 9 (left) whereas the connection between beams and columns was modelled using rotational springs. The spring characteristic is shown in Figure 9 (right) and was derived with the analytical model described above. The spring used in OpenSees model is a trilinear-spring, therefore only approximating the moment-rotation behaviour obtained from the analytical model as can be seen in Figure 9 (right).

![OpenSees model of the frame](image)

Figure 9: OpenSees model of the frame (left) and rotational spring characteristics for the beam to column connection (right)

The frame is loaded uniformly according to the characteristic loads given in Table 1. The loads for the design are calculated according to the Swiss standards for the first story:

\[
q_{ld,1} = (1.35 \cdot 6.4 \text{ kN/m}^2 + 1.5 \cdot 3.0 \text{ kN/m}^2) \cdot \frac{6.5}{2} \text{ m} = 42.7 \text{ kN/m} 
\]

(1)

and the second story:

\[
q_{ld,2} = (1.35 \cdot 4.7 \text{ kN/m}^2 + 1.5 \cdot 1.3 \text{ kN/m}^2) \cdot \frac{6.5}{2} \text{ m} = 27.0 \text{ kN/m} 
\]

(2)

The moments due to gravity loads are shown in Figure 10.
The design limit for the connection can be estimated with the analytical model described previously. The controlling design criterion is the compression strength perpendicular to the grain in the column, which is estimated at 8.6 MPa, including a factor for the load distribution. The design limits are summarised in Table 2 and compared to the values resulting from the gravity loads.

Table 2: Design under gravity loads and design limit according to Wanninger (2015)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1st story</th>
<th>2nd story</th>
<th>Design limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
<td>q</td>
<td>42.7 kN/m</td>
<td>27.0 kN/m</td>
</tr>
<tr>
<td>Stresses interface</td>
<td>$\sigma_1$</td>
<td>6.5 MPa</td>
<td>4.5 MPa</td>
</tr>
<tr>
<td>Moment interface</td>
<td>M</td>
<td>60 kNm</td>
<td>95 kNm</td>
</tr>
</tbody>
</table>

**Seismic Design**

The design is performed using three uniform hazard spectra (Giardini et al. 2013) with a return periods of 73, 475 and 2475 years as well as the design spectra derived from the Swiss standards (SIA 261 2003) for a soil class C (“Deposits of naturally consolidated and uncemented gravel and sand and/or moraine with a thickness exceeding 30 m”) and a structural damping value of 5% (Figure 11). The design spectrum coincides with the UHS spectra with a return period of 2475 years (i.e. the maximal credible earthquake) for spectral displacements larger than 0.1 m.

The pushover curve for the building is generated with the same OpenSees model that was used for the gravity loads. The spring characteristics had to be altered in order to account for the shear panel deformations (Wanninger 2015). The gravity loads were added to account for the P-Delta-effect. The pushover curve for the two-story frame was transformed to a pushover-curve for an equivalent SDOF-system (Chopra 2007) and is plotted in Figure 11.

Based on the data in Figure 11, the building should withstand the design earthquake without collapsing; damage however can occur according the performance based design principles (Priestley 2000). The design earthquake leads to a roof displacement of approximately 0.02 m for the equivalent SDOF-system. This value translates to a roof displacement of 0.025 m for the two story frame (Wanninger 2015).
The maximal moments resulting from the design earthquake are shown in Figure 12. The maximum connection moment is 58 kNm at the outer columns and 42 kNm at the inner columns.

The moments from the pushover-analysis (Figure 12) have to be added to the moments resulting from the gravity loads. The moments due to gravity loads acting on the structure during the design earthquake are plotted in Figure 13.

The maximum combined moment in a connection is 95 kNm (53 kNm due to gravity loads and 42 kNm due to the horizontal force). This moment is the same as for the design under gravity loads (see Figure 10). Therefore, the frame response remains in the elastic range for the design-level earthquake. Note that this earthquake is also the maximum credible earthquake as shown in Figure 11.

The frame would in theory be able to withstand larger earthquakes as the design earthquake In Zurich. To demonstrate this, the response of the frame to the East-West component of the Patti Gulf magnitude M=5.5 earthquake recorded at the station “Patti” on the 15th of April 1978 in Sicily. This earthquake is stronger than the design earthquake for Zurich and would lead to the moments that are plotted in Figure 14 (Wanninger 2015). The maximum connection moment of 124 kNm would lead to an exceedance in stresses perpendicular to the grain in the column without taking into account the 38 kNm from the gravity loads. However, the pushover tests that were performed on the post-tensioned timber frame specimen exceeded this moments without suffering much damage. The moments during these pushover tests exceeded 146 kNm (Wanninger 2015).
An unloaded frame (i.e. a frame carrying only horizontal loads) would therefore be able to withstand the Patti shock without any serious damage. If the gravity load has to be accounted for, the maximum moment would add up to 162 kNm, which exceeds the maximum moment obtained from the pushover tests by approximately 15%.

Wind Loads

The wind loads lead to smaller loads compared to the design earthquake. However, since the timber frame is a relatively soft system, the deformations under wind have to be checked.

The wind loads are applied to the structure as indicated in Figure 15, assuming that the façade is fixed at the top and the bottom of each story. It is assumed that the building has a height of 7 m including non-structural elements on the roof.

The loads for the top story are:

\[ Q_{w,1} = 1.0 \text{ kN} / \text{m}^2 \cdot 6.5 \text{ m} \cdot \frac{3.5}{2} \text{ m} = 11.4 \text{ kN} \]  

and for the first story:

\[ Q_{w,2} = 1.0 \text{ kN} / \text{m}^2 \cdot 6.5 \text{ m} \cdot 3.5 \text{ m} = 22.8 \text{ kN} \]

The wind forces in combination with the gravity loads load induce a roof displacement of 9 mm. This corresponds to a drift of H/666. However, the inter-story drift in the first story reaches a value of H/500 (i.e. 6 mm horizontal displacement). This corresponds to the limit given by the Swiss standard (SIA 260, 2003) for brittle installations and non-structural elements.

CONCLUSIONS

The design for gravity loads can be performed with a simple hand calculation or with the aid of software that is able to represent the non-linear moment-rotation behaviour of the post-tensioned timber connection using rotational springs.

The controlling design criterion is the stress in the column perpendicular to the grain. However, the tests under gravity loads lead to the conclusion that the joint can be loaded beyond the design value for the strength perpendicular to the grain without suffering from noticeable damage. Complying the requirements according to
the standards lead to large cross sections of the beams in order to distribute the load over a larger area and therefore reducing the stresses. This usually leads to beam sizes that would allow the beams to carry the applied gravity loads as simply supported beams.

The seismic design for areas with low seismic hazard, such as Switzerland, is not problematic. Displacement limits under wind loads control the design process, especially if brittle elements have to be accounted for leading to small allowable horizontal deformations of the frame. It is therefore recommended to install non-structural elements that are able to follow the frame in its deformations, i.e. that the deformation limits are less strict (H/300 instead of H/500). Moreover, it is advised to model the connection between the column and the foundation adequately so that its stiffness can be accounted for. Pinned connections are not suitable if the frame has to be designed for horizontal loads.

OUTLOOK

The pinned connections modelled at the bottom of the columns lead to a large horizontal displacement of the bottom story, which are controlling for the design (deformations under wind load). It is therefore necessary to estimate the actual stiffness of the connection at the bottom of the column. Tests on different kind of connections are planned and will deliver the needed data. Furthermore, the OpenSees model will be modified so that the same model can be used for horizontal and vertical load cases.

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