RESEARCH PAPERS

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FAILURE AND DEFORMATION OF FULL-SCALE LIGHT FRAME POPLAR AND SPRUCE WALLS UNDER VERTICAL COMPRESSION AND SHEAR LOAD

Wood frame houses are usually made of softwood species in Europe and North--America. But many countries have a great population of different hardwood species, e.g. poplar. This study investigates the possibility of using poplar, instead of pine for wall frame construction of lightweight houses focusing on the mechanical properties of the panels. Vertical compression tests and cyclic lateral shear tests were done using full scale light frame panels. Results show that the poplar panels have unexpected strength; the compression failure force of poplar panels [316.02 kN] is no more than 6% lower than spruce panels [335.00 kN] and no significant displacement was found at the moment of breaking. Poplar panels show more flexibility and lower deformation values during the lateral shear cyclic and the moment of failure, although the failure force was slightly higher in the case of spruce panels.

Keywords: light frame building, spruce, poplar, wall compression test, wall shear test

Introduction

Wood is a traditional and renewable building material due to its favourable features and its better effect on the environment compared to most other building materials. In addition, it is carbon-neutral, which means that by using wood for building purposes, carbon is sequestered to balance the carbon that has been released into the atmosphere. Furthermore, the energy demand for preparing wood into building materials is much less compared to metal, plastics, and silicate-based materials. In light frame buildings, wood is the most widely used material for studs and plates instead of metal. There have been numerous studies that have investigated different aspects of light frame buildings, e.g., thermal

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properties, energy efficiency and durability or mechanical strength. Mechanical strength is one of the most important aspects of wood because of the increased stress caused by storms, hurricanes, tornadoes or earthquakes.

Some studies have focused on the strength of wooden construction frames, as well as how to strengthen walls by optimizing fastener positions, doubling sheathing boards, and reinforcing the walls with other materials such as diagonal steel strips, diagonal carbon or high-strength synthetic fibre strips. These studies have found that a smaller stapling distance gives the best value for money [Dobrila and Premrov 2003; Premrov et al. 2004; Premrov and Kuhta 2009]. Other authors [Dujic et al. 2006] have looked into improving the current Eurocode 5 [EN 1995-1-1:2004 [E] Eurocode 5 Design of timber structures] and EN 594 standards [CSN EN 594 Timber structures – Test methods – Racking strength and stiffness of timber frame wall panels] have been used to examine timber frame walls by considering more precise influences of boundary conditions. At the end of their paper they have offered a more comprehensive test method.

Several papers examined miniature models of light-frame wood houses mainly in 1/3 or 1/50 scales and focused on the roof to wall and wall to foundation connections [Datin et al. 2010; Prevatt et al. 2010]. Additionally, a design methodology called "Database Assisted Design" [DAD] was validated by comparing its results with the results of a 1/3 scale light-frame wood structure. A good correspondence was established in comparing the DAD method and ASCE 7-05 [Minimum Design Loads for Buildings and Other Structures] method related to the main wind force resisting system [Mensah et al. 2011]. Several other studies dealt with the effect of wind load [St. Pierre et al. 2003; Surry et al. 2005; Zisis et al. 2009]. The roof structure and the strength of the roof – wall connections was also investigated [Riley and Sadek 2003; Noory et al. 2005; Chowdhury et al. 2007; Jacklin 2013]. There were remarkable studies in the field of earthquake experiments and lateral load review papers [Foliente et al. 2000; Paevere 2002; Filiatrault et al. 2010].

In Europe and North America wood frame houses are usually made of softwood species. On average, the coniferous round wood production is 67.69% [FAO 2007-2011] of the total round wood production in Europe. But many other countries have a great population of different hardwood species, e.g. the fast--growing poplar. In descending order: Canada, the Russian Federation, the United States, China and Germany have the biggest reported natural poplar stands out of the International Poplar Commission (IPC) member countries. China, India, France, Turkey and Italy have the biggest reported planted poplar areas in decreasing order of IPC members [Ball et al. 2005; Working Paper IPC 2012]. Some of these countries have high quality poplar plantations that can produce materials with dimensions appropriate for building frame elements.

The possibility of using poplar in exterior structures was already examined [Cassens et al. 2008]. According to other researchers' opinions, the most

promising utilization of poplar is in structural panel industries [Balatinecz and Kretschmann 2001; Balatinecz et al. 2001; Kurt 2010]. Fraanje [1998] examined the possibilities of using poplar wood as purlin. The elementary strength and durability of different poplar clones have been widely investigated [Tóth 2006; Benke and Pásztory 2012; Komán 2013] and will not be dealt with in this study.

This study investigates the effect on the mechanical strength of substituting coniferous wood for poplar when producing light frame panel studs. The examinations compared full-scale wood frame panels made of poplar and spruce under vertical compression, and shear load. The primary goals of the test are: to investigate the responses of two types of panels to a series of increasing loads and to compare the performance of the panels under various loading levels. The secondary goals are to determine the main reasons for failure of the structures. The results of comparative analyses can help to determine whether poplar can be used as a good substitute for spruce for light framing construction as a frame element.

Materials and methods

Two types of small panels for light construction walls were investigated. One type of panel was made of spruce (*Picea abies*), the other was made of a poplar hybrid cultivar (*Populus* ×e*uramericana* cv. *Pannonia*). Spruce is commonly used for this purpose, but poplar is not. Although there are several hybrid poplar species that are potentially good for construction purposes, only those with 400 kg/m^3 or higher density wood are suitable for building materials. There is a close relationship between density and strength. The higher the density, the better the strength. The Pannonia poplar has a density of 410 kg/m^3 [Tóth 2006] which makes it a good candidate as a construction material. The mechanical strength of the stud elements made of spruce and poplar has been tested and the results are shown in table 1.

Table 1. Comparison of mechanical properties of poplar and spruce

	Poplar	Spruce
	$\sigma_h - u_{12\%}$ (N/mm^2)	$\sigma_h - u_{12\%}$ (N/mm^2)
Tensile strength	52.49	90
Compressive strength	38.51	50
Shear strength	5.24	6.7
Bending strength	57.04	78
Impact strength	3.65	4.6

The other reason this clone was chosen is because it is widely found throughout Europe, e.g. more than half of the hybrid poplar plantations are *Populus ×euramericana*. In Hungary it is called the Pannonia poplar. The Pannonia hybrid is a special Hungarian poplar clone bred for local conditions.

The knots have less adverse effects on the strength of the poplar, as well as spruce [Komán 2013].

In compression tests, four panels were prepared for both species. The authors are aware that the number of specimens is not enough to make a deep statistical analysis; the cost of the samples limited the opportunity for testing. The panels had these dimensions at the compression test: 120 mm thick and 1250 mm wide, 2500 mm high. The compression specimens were made with a double top plate and a single base plate. A third stud is in the middle between the two single end studs (fig. 1A). The inner sheathing boards were OSB3-EN300-E1 with dimensions of $15 \times 1250 \times 2500$ mm and the exterior boards were OSB3--EN300-E1 measuring $12 \times 1250 \times 2500$ mm. The nails joining the wooden frames to each other were 3.8×130 mm steel nails and the staples fastening the OSB to the wood frame were 45 mm long "U" shape nails (fig. 1B). The stapling distance was 80 mm. Similar covering plates and fasteners were used for both types of panels. The frame element cross section was 120×60 mm in the case of vertical studs, two planks of 120×45 mm (double top plates) on the top and one 120×70 mm plank made the bottom plate. The vertical studs were 625 mm apart on centre spacing. The panels were assembled by the manufacturer using the same process for both poplar and spruce.
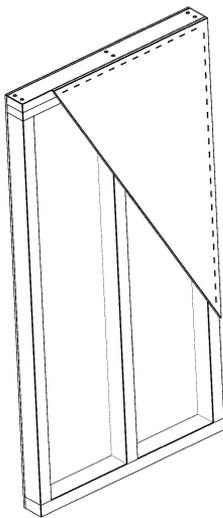

Fig. 1A. Schematic drawing
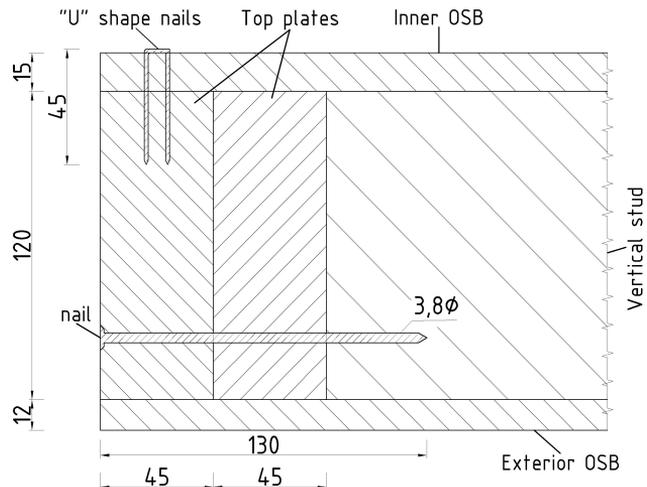**of the compressed specimen**

Fig. 1B. Schematic drawing of connections

In the compression test, there were three levels of vertical loading at gradually increasing weights. The first and second levels had an application of 100 kN and 200 kN. For the third level, the panel was subjected to increasing force until the frame suddenly broke or until the OSB came off the frame or when the intensified twist of the panels threatened the machine. This level of compression will be referred to here as the "failure" level.

The panels' upper and bottom plates were supported to avoid them buckling out of the loading plane. Two steel trundles were connected to the OSB side at the lower level of the upper horizontal frame element at about the middle point of the panel width. It did not hinder the vertical compression of the panels. The bottom part of the panel was fixed between two iron beams that were bolted to the floor to avoid lateral movement.

Actuator cylinders can register the applied force as well as the position data. These Hydraulic Power Units are a servo-hydraulic system, with an accuracy of 0.001 mm and 0.01 kN. In the compression test, two 250 kN maximum load capacity actuator cylinders were applied, positioned to the middle of the half width of the panel. The even distribution of the load was ensured through a thick glued laminated oak beam (fig. 2).

Fig. 2. Compression test measuring position

The lateral buckling of the panel was also recorded by an inductive displacement meter (WA/100) with an accuracy of 0.001 mm. The displacement meter was placed perpendicular to the OSB surface at the intersection of the diagonals and registered two directional buckling effects.

With the shear test panels, only the differences (compared to compression test panels) were determined; all other parameters and materials were the same. The panels had three modules of small panel with 3×1250 mm. The overall width of the shear test panel was 3750 mm, the thickness and the height and assembling methods were unchanged. The bottom plate of the panels was firmed bolted to the ground (fig. 3) and one of the lower corners was attached to a down bolted support to the structure from sliding due to the compressing force.

Fig. 3. Fastening to the strong floor

The actuator cylinder worked at the opposite corner of the panels as shown in figure 4. Two steel trundles were connected to the OSB next to the actuator to let it exert the force. A cyclic protocol was applied as follows; 4 cycles of 10 kN, 4 cycles of 20 kN, 4 cycles of 30 kN and 1 cycles of 70 kN. After reaching the maximum force, it was sustained for one minute.

The panels made of spruce are called S1, S2, S3, and S4 while the ones made of poplar P1, P2, P3, and P4 in the compression test. The spruce samples are called SS1, SS2, SS3, and SS4 and the poplar are SP1, SP2 SP3, SP4 in the shear test.
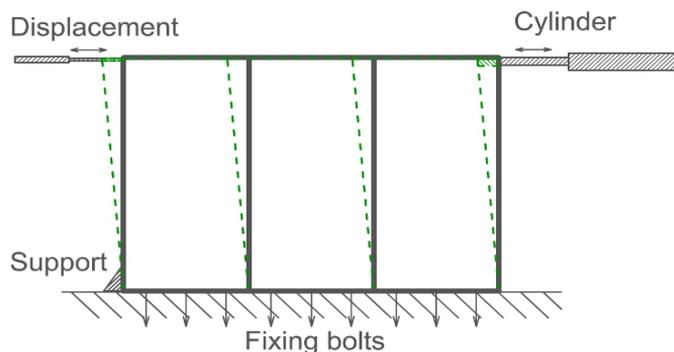

Fig. 4. Schematic view of the shear test

Results and discussion

Results of compression test

In the compression tests, data indicate that the failure of the spruce panels occurred on average at higher levels of compression force (335.00 kN) than the poplar panels (316.02 kN). The results prove that the difference in mechanical strength between the spruce and poplar species – which is significant otherwise – is less in a panel structure. Accurate results are shown in (table 2) below. On average, the spruce panels could withstand about 1.9 tons more than the poplar panels (table 1).

Table 2. Deformation and failure force data of the compression test

Panel	Compression load at failure [kN]	Deformation at failure load [mm]	Deformation at 270 kN load [mm]	Permanent deformation after the load step of 200 kN [mm]	Buckling under 270 kN load [mm]
S1	355.13	11.958	9.569	0.769	1.667
S2	326.74	13.184	10.887	2.610	1.432
S3	338.85	13.528	10.106	2.023	2.799
S4	319.27	12.662	10.211	1.848	1.434
Average	335.00	12.833	10.193	1.812	1.833
Standard deviation	15.66	0.68	0.54	0.77	0.65
P1	315.98	12.750	10.161	1.963	2.190
P2	278.76	11.588	10.892	2.017	4.616
P3	320.35	13.204	10.915	2.777	2.934
P4	348.98	14.630	10.868	2.535	0.555
Average	316.02	13.043	10.709	2.323	2.574
Standard deviation	28.83	1.26	0.37	0.40	1.69

Table 2 indicates two extremely high failure force values for spruce panel 1, and poplar panel 4 (referred to as S1 and P4).

S1 had a very good response to the compression tests. Two divergent poplar values are apparent: P2 had a very low value, which is explained below, while P4 had a very high value and was almost comparable to S1. Results show that in general (excluding P2) the responses of the poplar are comparable to those of the spruce panels. The average response is 335.00 kN for spruce and 328.44 kN (without P2) for poplar. A possible explanation for the weak value for P2 could

be related to a potential assembly error. For instance, the OSBs for P2 very slightly extended over the wood frame; this could have caused the OSB to initially bear the entire burden from compression and so, it broke down rapidly as the compression force was increased. Consequently, the wood frame got a sudden strike. In many cases, the failure starts from the joining point of the vertical and upper studs. In this place the upper studs lay on the butt surface of the vertical stud. This connection area is three times 7200 (120 mm by 60 mm) square mm which is quite a small surface regarding the load of over 300 kN. Because the compression strength of spruce and poplar is higher along the grain than perpendicular to the grain, the upper stud fibres break (fig. 5).

Fig. 5. Deformation of the upper stud

The compression at the failure force shows a few differences between the two species. The average deformation of spruce at the failure force (12.833 mm) is lower by 0.21 mm than that of the poplar panels average (13.043 mm). If the deformation data at the failure moment is divided by the failure force, it results in a specific deformation number which is 0.0383 mm/kN in spruce panels and 0.0412 mm/kN in poplar panels. The lower the specific deformation value, the better the performance. To make the deformation data comparable, it must be examined at the same load; e.g. 270 kN is the highest load value, where none of the panels reached its failure level. Very few differences can be seen between the data of spruce and poplar panels (excluding the S1). The average deformation at 270 kN load for spruces specimens is 10.193 mm, which is only 5.0% lower than that of the poplar panels 10.709 mm. The deformation differences at 100 kN and 200 kN were 1.1% and 3.8% respectively between spruce and poplar panels.

Another important issue is the permanent deformation shown in the fifth column of table 2. It would seem appropriate to determine this kind of deformation at the second biggest load step (200 kN), because it does not make sense at the moment of breaking. After the end of loading, when the actuator was released, the panel sprang back, but it could not reach its original zero point. This difference was measured as a permanent deformation. The poplar panels suffered 0.51 mm more permanent deformation (2.323 mm) compared to the spruce panels (1.813).

The lateral buckling of panels was measured as a displacement perpendicular to the OSB surface. Results from the right and left side buckling of the panels were measured using the inductive displacement meter. The spruce test panels have less standard deviation and a lower average (1.833 mm) of these impacts at 270 kN load and thus are more stable than poplar panels (2.574 mm). The values of the poplar panels have greater differences in their performance. P2 had the highest (4.616 mm) while the P4 had the lowest value (0.555 mm).

Results of shear test

In the shear test the panels were twisted or shifted without a spectacular break of the frames or OSB boards. The cyclical loads made the panel rather exhausted without a break. Each force in a given cycle was sustained for one minute and then the actuator force decreased to zero. The panels suffer some deformation from the load. Up to a certain point it is elastic and beyond that, deformation is permanent. Figure 6 shows the cumulative average deformation data of the panels. The different values between the peaks at the same load level mean the worn effect of the panels, i.e. the same force caused higher displacement. The graph proves that the spruce panels suffered higher deformations at all load levels and the failure deformation was also higher than poplar panels.

To determine the failure force is not as simple as in the compression test. There is no spectacular effect or loud crackling sound. By checking the displacement and force graphs, there is a visible breaking point on the function (fig. 7). The failure level is determined where the angular coefficient becomes milder and after this point, the acting force can cause higher displacement. Practically speaking, this point is the yield point. The curve can be divided into two parts and both parts are linear; the fitting lines have an R square over 0.99. The intersection of these lines was determined as the failure displacement point and the failure force was determined by the function value at the displacement (fig. 7). For comparison, this method offers the simplest most objective way to determine the failure point, according to the primary goal of this study.

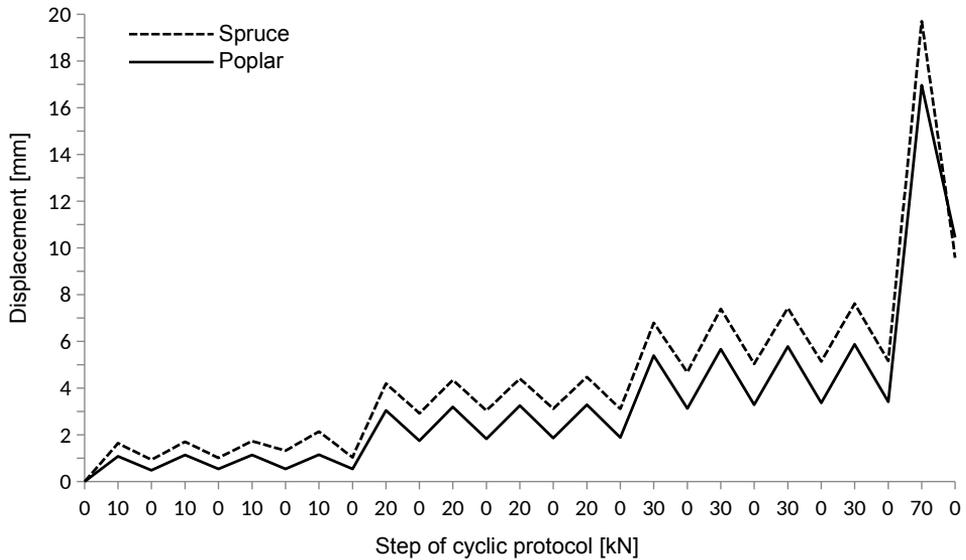Table 3 summarizes the data related to the failure force and displacement of the shear test panels.

Fig. 6. Average displacements of spruce and poplar panels
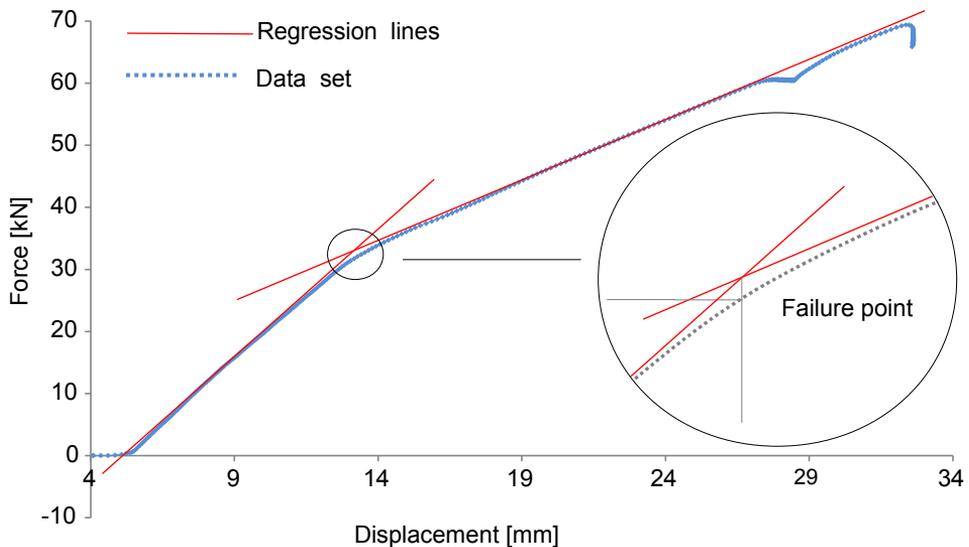

Fig. 7. Determination of failure point

On the average the spruce panels perform better at 3.66 kN which means 0.366 tons of surplus burden (table 3). The SS1 panel shows an extremely high value. Without the SS1 panel, the average value of spruce panels would be 34.05 kN which is only slightly higher than the poplar average of 33.07 kN. The difference between the spruce and poplar panel is very small which can be explained because the shear strength of the structure is derived from the joints and not from the mechanical properties of its studs.

Table 3. Deformation and failure force data of the shear test

Panel	Lateral load at failure [kN]	Deformation at failure load [mm]	Displacement at 4th cycle of 30 kN	Permanent deformation at 4th cycle of 30 kN
SS1	44.78	25.533	9.287	7.310
SS2	33.76	13.175	6.922	4.353
SS3	34.24	15.442	8.162	5.059
SS4	34.15	12.341	6.112	3.918
Average	36.73 (34.05)*	16.623	7.621	5.160
Standard deviation	5.37	6.08	1.39	1.51
SP1	33.28	11.441	4.980	2.953
SP2	32.04	13.002	7.263	4.128
SP3	31.90	26.175	4.690	2.722
SP4	35.07	13.245	6.547	3.836
Average	33.07	15.966	5.870	3.410
Standard deviation	1.47	6.85	1.24	0.68

*Average value without SS1.

Higher lateral strengths mean stronger walls. If we focus on the whole building and also examine its connections, then the smaller deformation can be more advantageous because the connections are not so strongly loaded. With the shear test, the lower the displacement value the better the performance. The majority of data is between 11.4 and 15.5 mm; one spruce (SS1) and one poplar (SP3) stand out significantly from the others. The average failure displacement data of the panels are very close to each other 16.623 mm and 15.966 mm for spruce and poplar respectively. Poplar panel average is lower by 3.95% than the spruce which means the poplar panels suffer less deformation than spruce panels; nevertheless, the failure force of spruce is higher.

To make the data comparable, it must be examined at the same load cycle. The last 30 kN load cycle was chosen, because this is the highest load value, where none of the panels reached its failure level; however, in some cases it is not far off. The average deflection at a 30 kN lateral load for spruce walls is 7.621 mm what is over 20% higher than the poplar average of 5.870 mm. In this aspect, poplar shows better performance than spruce.

After the end of loading, when the actuator was released, the panel sprang back, but it could not reach its original zero point. This difference was measured
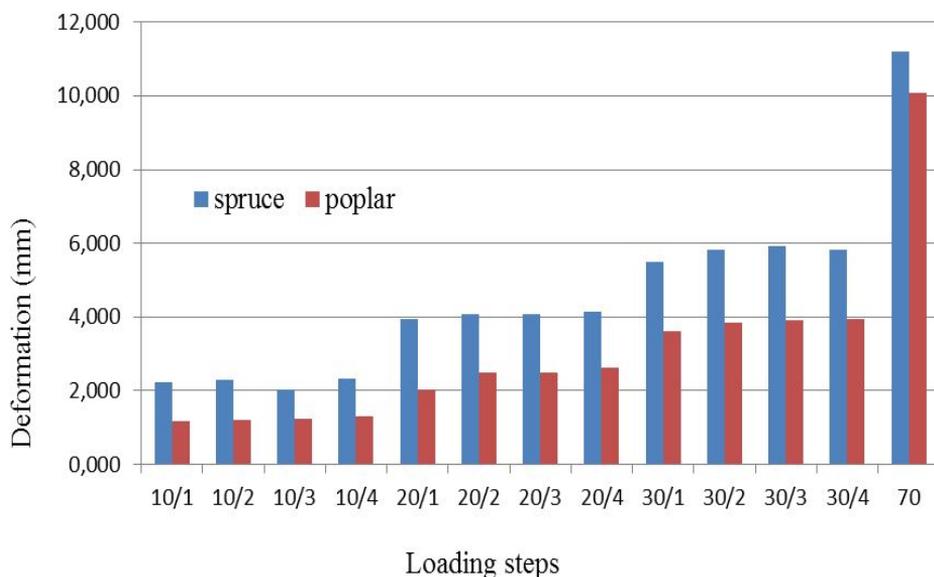
Fig. 8. Permanent deformations after every load

as a permanent deformation. Similarly, to the displacement, the permanent deformation of poplar panels was smaller after the 4th 30 kN load. The average is 5.160 mm for spruce and 3.410 mm for poplar. The poplar panels reach only 66% of spruce panel deformation, which is remarkable because the failure force is almost the same. Figure 8 shows the permanent deformation averages after every load step. The poplar panels had better stability during the shear test.

Conclusions

This study investigated the effect of the mechanical strength of light frame panel studs when substituting poplar for coniferous wood. The main objective was to compare the vertical compression strengths and horizontal shear strengths of panels made of spruce and poplar to determine whether poplar can be used instead of spruce. The secondary goal was to determine the main reasons for failure of the structures. Full scale wood frame panels were prepared and tested. Four test panels were prepared for each of the two species (spruce and poplar) used in the compression test and the vertical shear test.

    In the compression test, the performance of the panels was examined under three compression loads which were 100 kN, 200 kN, and the load leading to the panel failure. The main parameters were the failure force, deformation at the failure force and under the 270 kN load the permanent deformation after the 200 kN load and lateral buckling of one side of the panels.

- Compression strengths of spruce panels were only 6% higher than the average of poplar panels.
- The deformation was 12.833 mm and 13.043 mm in spruce and poplar panels at the moment of breaking. Practically speaking, the difference is almost negligible regarding the two tenths millimetre. The 13 mm compression is one half percent related to the panel height.
- The break started at the connecting surface of the vertical and upper studs where the upper stud fibres broke in many cases. The compression strength of wood is higher in the direction of the grain than perpendicular to the grain. The compression strength of poplar, perpendicular to the grain, is lower than spruce which can partially explain why the poplar panel had a lower compression strength.
- The poplar panels suffered significantly higher (28.2%) permanent deformation after 200 kN load than that of the spruce panels. The deformation of 1.812 mm and 2.323 mm is only 0.07% and 0.09% regarding the whole panel height (2500 mm).
- The panels buckled on average 1.833 mm in the spruce panels and 2.574 mm in the poplar. This shows that the panels made of spruce are more stable in the horizontal direction.

The spruce panel is definitely stronger in compression and more stable than the poplar panel. Although the strength difference between them is less than expected and less than the difference of mechanical properties of the two species. Poplar would be an appropriate substitute stud material (especially for vertical studs) because of its compressive strength as long as its specific gravity is higher than 400 kg/m^3.

The shear test was also carried out on spruce and poplar panels, using a repeated shear load protocol (4 cycles of 10 kN, 4 cycles of 20 kN, 4 cycles of 30 kN, and 70 kN). Meanwhile the failure force, the displacements and the permanent deformation were determined. The results were:

- Failure force shows a very small advantage for panels made of spruce.
- The displacement at the moment of failure proves that the poplar panels have almost the same stability, having a 15.966 mm shift against spruce panels 16.623 mm. The difference between them is 4.1% which is almost negligible.
- The displacement at the last 30 kN cycle was less in poplar panels than spruce panels. The average shift value of spruce panels is 7.621 mm while the average poplar panel shift is 5.870 mm which is 29.8% less. From this aspect the poplar seems to be more advantageous.
- Likewise, the cumulative permanent deformation is lower after the last 30 kN load in poplar panels. The results show that the spruce panels on average have 51.3% higher permanent deformation which is a significant difference. The poplar panels provide more flexible properties, which is a very important component of a building.

Though the shear strength of spruce panels is higher, the deformation parameters of poplar panels are remarkably advantageous. Poplar can provide the same stability and higher flexibility than spruce panels because of its response to shear stress. Thus, poplar could be substituted for spruce frame elements.

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