THE SHEAR STRENGTH, AND FAILURE MODES, OF TIMBER JOISTS OBTAINED FROM THE TORSION TEST METHOD

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ABSTRACT: This paper presents details of the experimental method and test results from a series of torsion tests undertaken to evaluate the shear strength of timber joists. The failure modes and the correlation of shear strength and torsional shear modulus were also studied. Test results obtained indicate that shear strength of tested joists was higher than the published values in EN 338 and in the USDA Wood Handbook. The test joists fractured mostly at the middle with cracks propagated towards either supports or edges. However, combined tension shear and crushing failure modes were sometimes observed at supports. A good correlation was found between torsional shear strength and the shear modulus, but, it appeared that knots do not have substantial influence on the shear strength. The recent revision of the testing standard EN408 includes the torsion testing approach to obtain the shear modulus of timber. It is proposed that a torsion test also be adopted as a method for evaluating the shear strength of timber.

KEYWORDS: Torsion test, Shear strength, Shear modulus, Design values, Failure mechanism

1 INTRODUCTION

The shear strength parallel to grain or "shear strength" is a fundamental mechanical property of wood and is used in general timber structural design. The shear strength can be determined by testing small clear wood blocks ("shear blocks") as recommended by testing standards such as EN408 [1] and ASTM D 143-94 [2] .The shear block test method allows the shear strength values to be obtained free from influence of wood defects and, therefore, the test procedure underestimates the heterogeneous nature of wood.

To account for the possible influence of wood defects and heterogeneity of wood, full size structural lumber can be tested under bending (three or four point) or in torsion [3] to obtain the shear strength. In practical terms, the application of the shear strength values in EN338 [4] is on the basis of the bending strength of timber, both via the allocation of timber to a grade in the strength grading process and by the use of the rules in EN384 [5]. The current version of the standard uses the following equation to calculate shear strength from the bending strength, obtained by testing full size structural timber in four point bending:

$$\begin{aligned} f_{v,k} &= 0.2\left(f_{m,k}\right)^{0.8} \\ f_{v,k} &\leq 3.8 \end{aligned} \tag{1}$$

Where $f_{v,k}$ is the characteristic shear strength and $f_{m,k}$ is the characteristic bending strength. The upcoming revisions of EN338 [6] and EN384 [7] apply a slightly different relationship, but to the same end: shear strength is obtained on the basis of bending strength.

A short-span flexural test might be close to the real-life loading condition but would not provide a simple to analyse state of shear due to the interaction of tensile, perpendicular compressive and shear stresses that take place. On the other hand although a torsion test does not represent an actual real-life loading condition it does produce a purer and more uniform system of shear stresses in the specimen allowing measurement of the pure shear strength. However, until recently, very little attention has been paid to use the torsion test method.

Various researchers have examined different test methods for obtaining the shear strength, and compared shear strength values with bending strength. Norris [8] introduced a panel shear test method. Mandery [9] and Keenen [10] obtained shear strength of Douglas-fir beams by testing them under three point bending tests. Rammer *et al.* [11] used a four point bending test approach on Douglas-fir and compared the results with the shear block shear strength values. Cofer *et al.* [12] used a finite element approach to evaluate the

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performance of three and five point bending tests to obtain shear strength of wood beams.

To evaluate the applicability of torsion testing, Riyanto and Gupta [13] conducted research to compare shear block, bending and torsion test approaches for attaining the shear strength. They concluded that the torsion test is a better approach than the other methods. Later, Gupta *et al.* [14,15] used experimental and finite element approaches and concluded that the torsion test method was the more applicable test method to the shear block test method.

The recent draft of EN408 [13] included the torsion test method to obtain the shear modulus of wood in place of the previous bending method (alongside the option of measuring shear strain directly during bending). Certainly there is evidence to support the rejection of the old bending method (*e.g.* [17]).

This study investigated the applicability of the torsion test method to obtain the shear strength of timber joists and proposes this also be included in the test standard. The main objective of presenting this paper is to describe the application of the torsion test to obtain the shear strength values, and to compare the results with the published values from EN338 [4] and in the Wood Handbook [18]. The secondary objectives were to examine the failure mechanism of wood under torsion, the correlation of torsional shear strength with shear modulus, and torsional shear modulus with modulus of elasticity.

2 MATERIAL AND METHODS

Sitka spruce (*Picea sitchensis*) and Norway spruce (*Picea abies*) joists of nominal cross section of 45 × 100 mm were tested. Sitka spruce timber of C16 strength class was cut into four different lengths of 1.0 m, 2.0 m, 2.8 m and 3.6 m with 15, 10, 12 and 25 samples, respectively selected for each length (denoted here SP). Norway spruce (NS) wood of strength class C16 and C24 was cut into 2.4 m lengths with 14 and 12 specimens respectively. Before testing, all samples were conditioned in a controlled-environment room (21°C and 65% relative humidity) until they attained constant mass (approximately 12% moisture content). A 1 kN-m torsion testing machine (Tinius Olsen, Pennsylvania USA) was used to test the timber joists under torsion and to measure the twisting displacement of the timber inclinometers with a range of ± 30° were attached to the upper edge (45 mm dimension) of each sample, as shown in Figure 1.

The mounting positions for the inclinometers depended on the length of sample being tested, but in all cases inclinometers were mounted at least 100 mm from the clamps to avoid end effects. For 1.0 m long samples, two inclinometers, each located 200 mm from the end clamps, allowed displacement to be measured on a 600 mm central span. Figure 2 gives the positions of inclinometers for each length. The main purpose of mounting inclinometers was to obtain the relative twist of the span free from machine and clamp distortion to calculate shear modulus (G). The shear strength was calculated on the basis of the maximum torque.

All test specimens were tested at 4°/min [3] until they fractured. The shear strength and G of each test specimen was calculated on the basis of Saint-Venant torsion theory for rectangular sections as follows:

$$\textit{Shear Strength} = \frac{\textit{Maximum Torque}}{\left(d\, t^2\, k_2\right)} \qquad (2)$$

$$G = \frac{\textit{Stiffness}}{\left(d\, t^3\, k_1\right)} \qquad (3)$$

In Equations (2) and (3), d is the depth (major cross-section dimension) and t is the thickness (minor cross-section dimension) of the test specimen and k_1 and k_2 are constants that depend on the depth thickness ratio (see *e.g.* [19]).

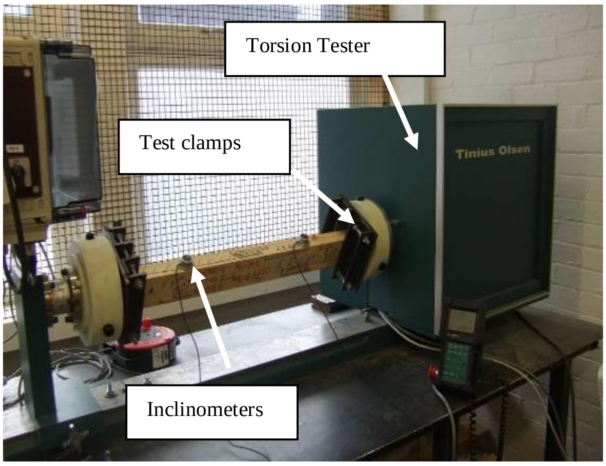

Figure 1: *Torsion testing apparatus1.0 m sample*

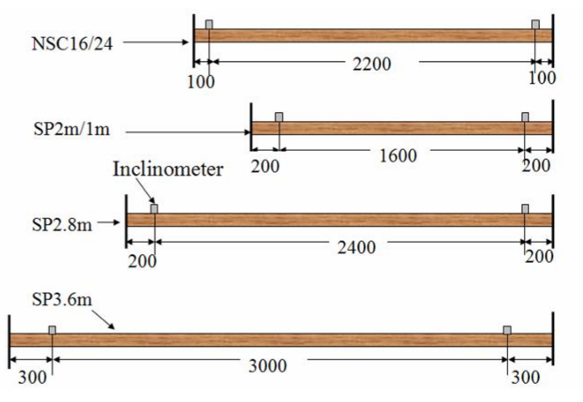

Figure 2: *Shear strength test arrangements for SP and NS joists (lengths in mm)*

Shear strengths were calculated on the basis of the maximum applied torque, as shown in Figure 3. The stiffness was obtained by conducting linear regression analysis of the applied torque and the relative twist per length within the elastic region as shown in Figure 3.

For most of the tested specimens the elastic region was found between 3% and 30% of maximum applied torque, and therefore, linear regression analysis was conducted between 5% and 25% of the maximum applied torque to obtain the stiffness.

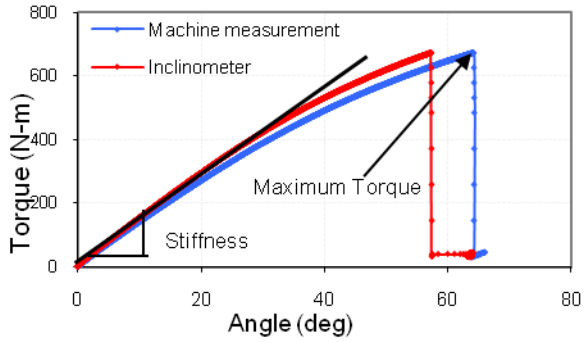

Figure 3: *A typical applied torque and relative twist of 2.8m test sample*

3 RESULTS AND DISCUSSIONS

3.1 DESIGN STANDARDS AND TORSIONAL SHEAR STRENGTH VALUES

Table 1 provides the mean shear strength values of both Sitka Spruce and Norway spruce beams. For C16 Sitka spruce, the mean shear strength of 7.2 MPa was attained and that was about 15% lower than Norway spruce of the same grade, and 22% less than the C24 Norway Spruce. C24 class timber has the highest shear strength (9.3 MPa), which agreed with expectations that the higher strength class would have higher shear strength values. For the C16 of Norway spruce the shear strength was about 9% lower than C24 of the same species. It was found that different species has different shear strength values. This is perhaps because the different species have different ratios of shear and bending properties.

In the current EN338 [4], the characteristic shear strength values for C16 and C24 are 1.8 MPa and 2.5 MPa respectively. These values are calculated on the basis of bending strength of full size structural timber beams tested under four point bending test in accordance with EN408 [1,5] (Equation 1) although EN408 also provides a method to obtain the shear strength by testing a 32×55×300 mm wood block. Much higher characteristic shear strength values of 4.8 MPa (166% higher) of C16 (combined SP and NS) and 7.5 MPa (200% higher) of C24 were achieved when joists were tested under torque. The revision of EN338 [6] has raised values of characteristic shear strength for these grades (3.2 MPa and 4.0 MPa) but these are still substantially less than those observed experimentally in this study.

Based on shear block tests of clear samples, the Wood Handbook [18] provides the mean shear strength values for Sitka spruce and Norway spruce of 6.7 MPa and 7.4 MPa respectively. From this research, mean shear strength of SP was 7.2 MPa (8% higher) and for NS was 8.5 MPa (13% higher), even though the material tested was of structural size and not clear wood. Similarly, Riyanto and Gupta [13] have shown that in comparison to the shear block tests, the torsional shear strength values of Douglas-fir were about 18% higher than the shear strength values of tested shear blocks and about 20% higher than the published values in the Wood Handbook [18].

Table 1: *The mean shear strength values of tested joists*

Group	Strength Grade	Length (m)	No. of Joists	Max. Applied Torque (N-m)	Mean Shear Strength (MPa)
SP	C16	1.0	15	485	7.8
	C16	2.0	10	460	6.7
	C16	2.8	12	550	7.7
	C16	3.6	25	475	6.7
	C16	Overall		490	7.2
NS	C16	2.4	14	390	8.5
	C24	2.4	12	410	9.3

This comparison shows that relatively higher shear strength values were achieved when the torsion test approach was used, even in comparison to tests on clear timber. Although it should be noted that only two species were tested in this research, a marked difference in shear strength was found compared with values given in EN338 [4,6].

This suggests that the assignment of shear strength values according to the results of bending tests may be over-conservative, at least for lower grades where knots affect bending strength, but not so much the shear strength (which may even be improved by knots). It may be appropriate that the torsion test procedure be adopted as a standard method to obtain the shear strength values, especially in light of its inclusion as a method to obtain shear modulus.

3.2 FAILURE MECHANISM UNDER TORSIONAL LOADING

All test specimens were fractured when tested under torsion. Samples of shorter length (1 to 2.4 m) fractured within the range of 30° per meter twist, while longer samples (2.8 m and 3.6 m) fractured within the range of 20 to 30° per meter. This amounts to a high value of total twist for long specimens. One of the 3.6 m joists was twisted to 110° (31° per metre) before it broke as shown in Figure 4. Throughout the tests, small cracking noises were heard and it was noticed that small horizontal hair-type cracks appeared in the test samples while torque was still applied on specimens. During tests, most of the joists fractured with large bang sound and a puff of wood dust in air around the location of failure was seen. It was found that failure cracks, in many cases, were initiated within the clear wood even though a number of large knots were present in test joists.

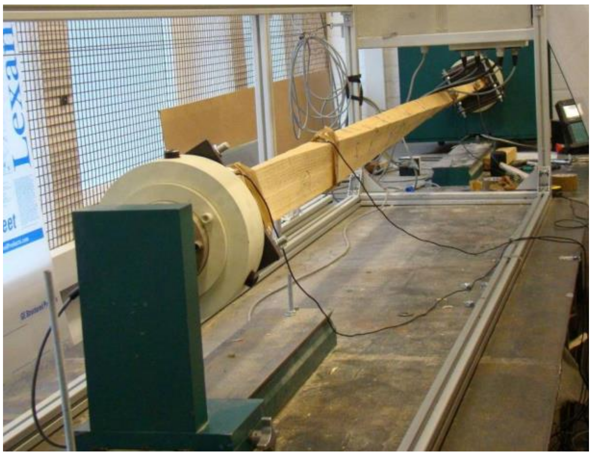

Figure 4: *A typical 3.6 m joists with large rotational deformation before the fracture*

Four different types of failure modes (*viz* crushing (40% of tested specimens), shear (25%), combined tension shear failure (12%) and horizontal shear failure (23%)) were observed as described below.

3.2.1 Crushing Failure

The crushing failure is defined here as a failure that occurs at the supports triggered mainly by clamps crushing the wood material. It was noticed that 40% of SP and NS specimens were fractured either at loading or reaction clamps with crushing failure mode. The main reason behind crushing of wood was because, in addition of shear stresses, the test clamps induced compressive stresses on the cross sectional area and the combined shear and compressive stresses caused small cracks in growth rings which, in turn, caused crushing failure. The cracks began in the earlywood zone in RT plane (Figure 5) and propagated along LR plane (long side), as shown in Figure 6. In Figure 6, the relative torque-twist graph is given and the ordinate of the graph represents the applied torque and the abscissa represents the rotation from torsion tester.

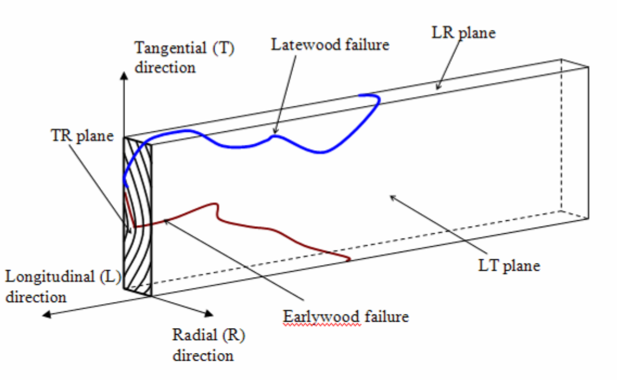

Figure 5: Schematic diagram of timber joists showing grain direction

It was observed that for the crushing failure, the fracture was occurred within the initial plastic zone range of torque-twist relationship. The cracks usually started from growth rings and ran horizontally along the LT plane ending near the middle of the span depending upon the length of the test joists. In some cases cracks were started in the latewood zone and travelled towards first the LT plane (short side) and then propagated towards the LR plane ending near middle of joist span. It was observed that presence of knots, inner bark or pith was causing the discontinuation of the cracks and such battens failed immediately within the linear torque-rotation zone as a brittle failure.

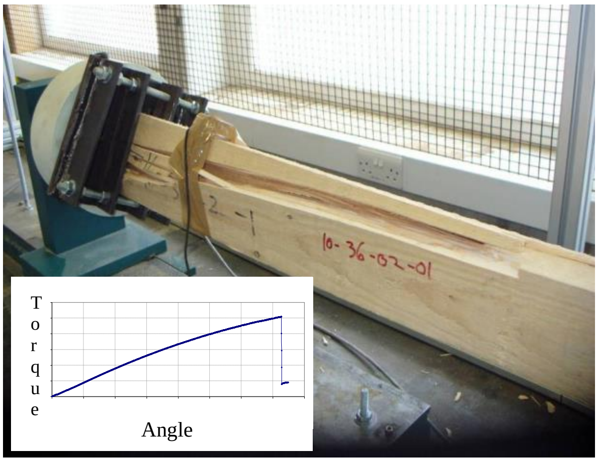

Figure 6: *A crushing failure of 3.6m batten and its torque-twist relationship*

3.2.2 Combined Shear Tension Failure

Another type of failure mode observed was the combined shear-tension failure and this occurred mostly in Sitka spruce joists. Nine out of 46 SP joists fractured with combined shear tension failure mode. The applied torque produces shear stresses and these stresses were dominant in causing this type of fracture. In the case of clear wood, the shear crack initiated from the middle of the LT plane and due to tension propagated towards, and was ended, in the LR plane. This may be because the grain angle might not be parallel to the longitudinal axis and, therefore, grains were fractured locally in tension and the failure travelled diagonally along the grain direction.

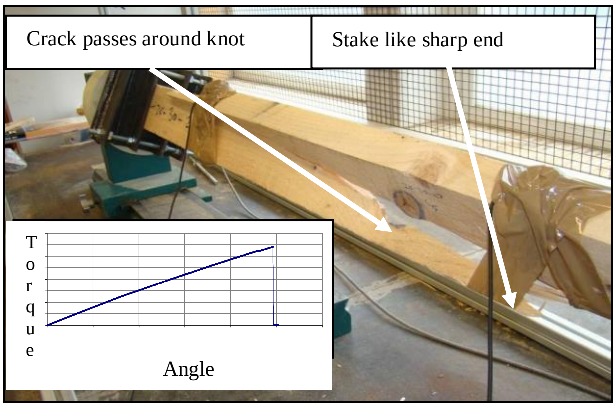

Figure 7: *A combined shear tension failure occurred in 2.8m joist and crack passed through knot and ends up with a sharp end*

It was also observed that when a crack approaches a knot it travelled around the knot rather than pass through it. Thus, this indicates that knots may provide some resistance to the shear failure. Figure 7 shows a combined shear tension failure, and it can be seen that the crack passed around the knot and produced a stake shaped end. In some samples, however, it was observed that combined knot and grain deviation on the LR plane and knot fissures initiated the shear failure and that test joists were fractured within their elastic range as a brittle member.

3.2.3 Shear Failure

Another type of failure that occurred was the shear failure, which was also mainly seen in the Sitka spruce joists. About 18 out 46 SP joists fractured with shear failure mode. It was observed that shear stresses were the main cause of initiating the cracks for this failure mode, and that these cracks were usually started at either the top or bottom side, due to a knot, and then propagated as a diagonal crack along the long side to rupture the specimen in shear due to the knot at the other edge. This failure takes place because edge knots are usually surrounded by cross grain and this cross grain breaks locally in shear to initiate the failure (Figure 7).

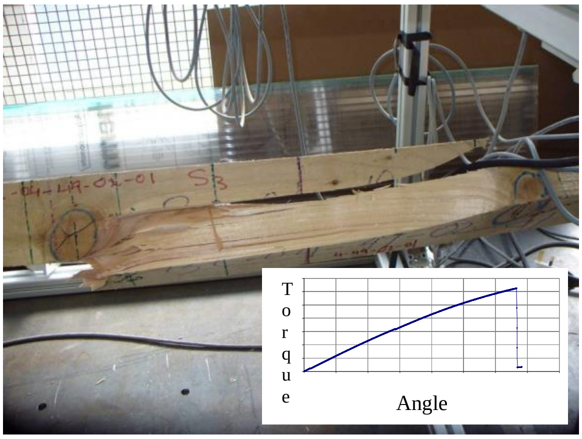

Figure 7: *A typical shear failure occurred in 2.8m specimen due to top and bottom edge knots*

It was found that in this type of the failure the cracks passed around the knots that were present in the long side of the battens. This shows that knots are not the weaker plane along the long side of the battens.

It was also seen that wood defects, especially of combined knot and grain deviation on the LR plane, also initiated brittle shear failure and that test joists were failed within their elastic range as a brittle material, as shown in Figure 8. An existence of knot in the middle of the LT plane was also found to be crucial under torsion. Although it has been seen that most of joists were fractured within clear wood, in some test specimens it was noticed that knots at the middle of long side caused the fracture. A closer look reveals that actually the knot fissures initiated the crack which travelled horizontally for a short distance and then travelled towards the edges, as shown in Figure 9. In this type failure, the failure occurred within initial plasticity of the joists and most of failure curves were like a straight line with a slight bend at the end.

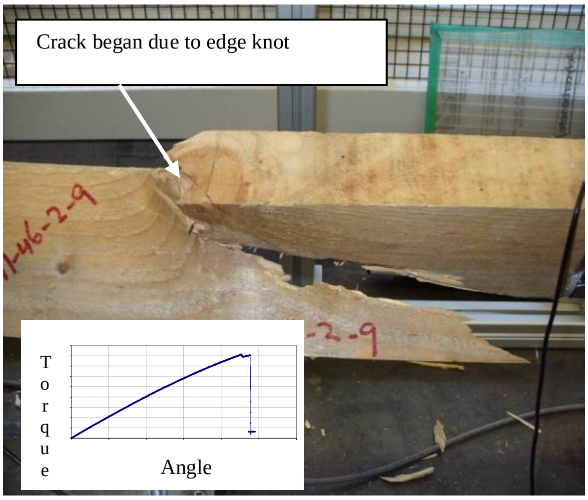

Figure 8: *A typical shear failure began due to an edge knot in 3.6m joist*

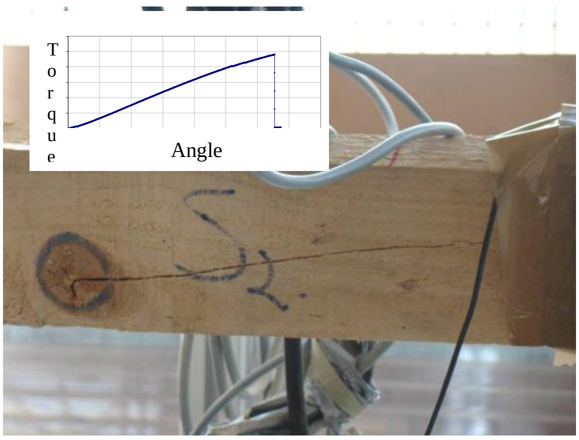

Figure 9: *A typical shear crack started from knot fissure*

3.2.4 Horizontal Shear Failure

This type of failure was only observed in Norway spruce specimens. In this type of failure, the shear cracks were usually initiated from clear wood within the LR plane and travelled parallel to the longitudinal direction towards end supports, as shown in Figure 10. 16 out of 26 Norway spruce battens fractured with horizontal shear failure mode. The term horizontal shear failure is given here because the shear cracks ran horizontally along the length of the joists. It was also noticed that some secondary cracks were also developed accompanied with the major cracks.

It is thought that this type of failure occurred because the NS specimens had grain that was close to parallel to the longitudinal axis along the joist span. Therefore, when failure occurred the cracks travelled through the grain parallel to the length. Secondly, it was observed that the knots diverted the crack path in Sitka spruce specimens but the Norway spruce test joists had no large knots (diameter > 25mm) that could have diverted the crack direction. It was observed that most joists were failed within their plasticity with arch-type failure. In some joists of C24 it was seen that two or three major cracks developed along the long side but did not initiate through-fracture of the sample, as shown Figure 11.

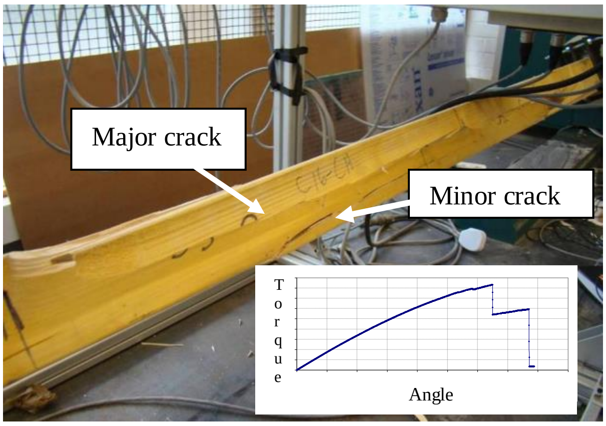

***Figure 10:** A typical NS C16 batten with a large horizontal shear crack and minor cracks*

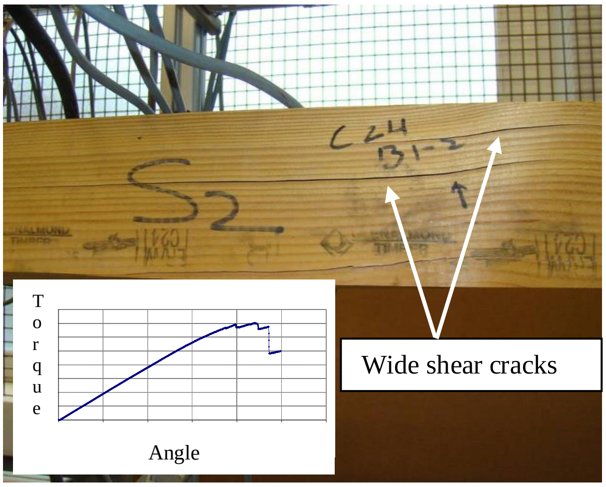

***Figure 11:** A typical wide shear cracks occurred in C24 timber joists under torsion loading*

3.3 RELATIONSHIP BETWEEN SHEAR STRENGTH AND SHEAR MODULUS

3.3.1 Torsional Shear Modulus Values

Table 2 presents the mean, maximum and minimum shear modulus of each length group. It was clearly observed that C24 joists have the higher shear modulus values of 760 MPa, and it was expected that higher strength class would have the higher shear stiffness values. For the C16 Norway spruce, *G* was about 20% lower than for C24 of the same species. For Sitka spruce. the mean shear modulus of 520 MPa attained, was about 17% less than of C16 Norway Spruce and 32% less than of C24 Norway spruce. Just as the Sitka spruce joists were found to have lower shear strength, they also have lower shear modulus.

***Table 2:** The mean shear strength values of tested joists*

Group	Grade	Length (mm)	No. of Specimens	G (MPa) Mean	Max	Min
SP	C16	1.0	15	490	750	300
	C16	2.0	10	500	560	430
	C16	2.8	12	530	630	410
	C16	3.6	25	560	715	430
		Overall		520	750	300
NS	C16	2.4	14	610	760	515
	C24	2.4	12	760	1100	600

The characteristic values of the mean shear modulus for C16 and C24 in EN338 [4,6] are given as 500 and 690 MPa respectively. These are determined from of *E*:*G* ratio of 16:1 [5,7]. The *E* represents the mean modulus of elasticity of timber obtained on four-point bending tests [1]. In this study, the overall mean *G* for C16 (including both SP and NS species) of 560MPa was obtained, about 13% higher than in EN338. For C24, torsion tests produced a mean *G* of 760MPa which was 10% higher than in EN338.

3.3.2 Shear Strength and Shear Modulus Correlation

A linear correlation between the shear strength and *G* of Sitka spruce and Norway spruce joists was developed, as shown in Figure 12. The R-squared values were calculated without including the outlying higher shear strength values of the Norway spruce test specimens. This is because only two higher shear strength values were obtained and this their inclusion would unduly bias the correlation of shear strength and *G*. It is thought that the slightly higher correlation for Norway spruce was obtained because most of these specimens were free of wood defects and joists failed within clear wood. The Sitka spruce specimens, on the other hand, contained more knots resulting in some specimens failing prematurely in a brittle manner. However, it was also noted in this study that knots have very little influence on *G* and on shear strength overall. Rather, in some Sitka spruce specimens it was found that knots initiated the failure and caused a low shear strength values but had no major affect on *G*, which may weaken the correlation.

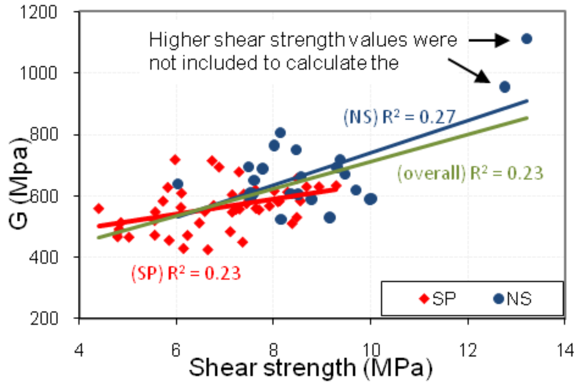

***Figure 12:** Linear relationship between G and the shear strength of SP and NS joists*

3.4 CORRELATION OF SHEAR MODULUS AND MODULUS OF ELASTICITY

A relationship of *G* and *E* was developed to observe if both mechanical properties have any correlation. The *G* of 600 mm and 1800 mm portions of some of the C16 and C24 joists was obtained using torsion tests and compared to measurements of local and global *E* from bending tests. Furthermore, the *G* of the whole spans (2.8 m and 3.6 m) was compared to dynamic *E* determined from resonance acoustic tests. More details can be found in [20] and [17]. Linear correlation results are shown in Figures 13, 14 and 15

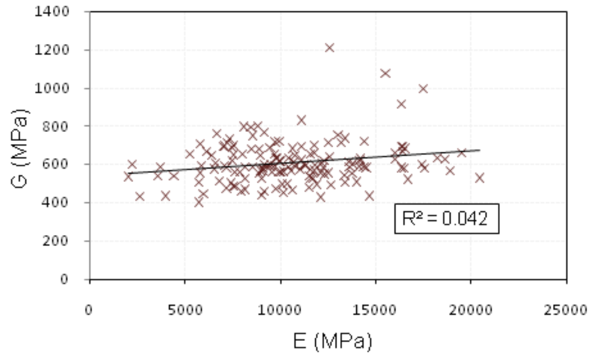

***Figure 13:** Linear relationship between G and the (local) E of 600 mm sections*

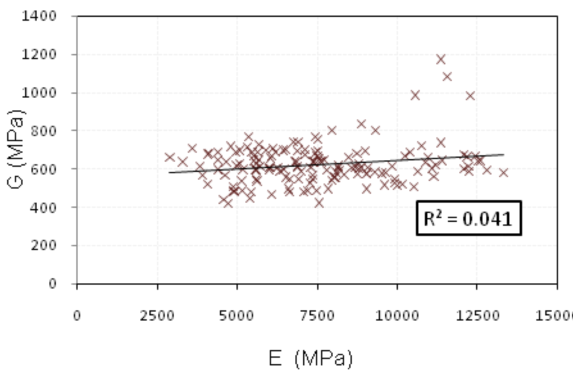

***Figure 14:** Linear relationship between G and the (global) E of 1800 mm sections*

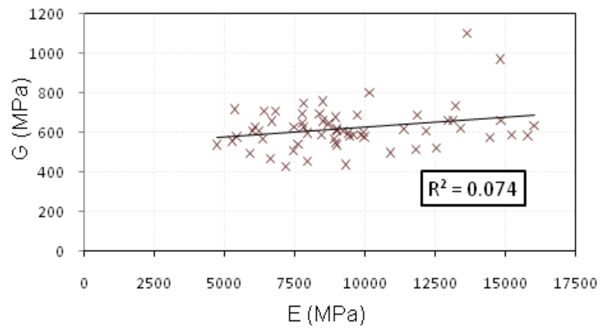

***Figure 15:** Linear relationship between G and the (dynamic acoustic) E at span level*

No correlation was found between *G* and *E* and this suggests that *G* is independent of *E* within the C16 to C24 range. As mentioned earlier that in EN338 [4] the mean *G* values were determined from *E*:*G* ratio of 16:1. However, this study has found no evidence of any correlation between *E* and *G* within tested grade. Therefore, it becomes more important to obtain *G* values using the torsion test method (or direct shear strain measurement) as in the revised EN408 [16].

4 CONCLUSION

This paper has presented the outline results of a series of torsion tests to determine the shear strength of Sitka spruce and Norway spruce joists. The torsion test procedure has been seen to produce higher strengths than values published based on bending and shear block tests.

In the test method, it was found that samples fractured within the long side where shear stresses are presumed to be maximum under applied torque (except in the cases where fracture was initiated by crushing at the support).

It was noticed that the cracks were commonly initiated within clear wood and caused shear failure, but that, in some specimens, cracks started due to shear and then propagated as a tensile failure. Support conditions were found to be important. It was noticed that testing clamps induced additional compressive stresses which lead to crushing of the wood at the supports and premature failure for some specimens. Therefore, this is important to design such testing clamps so that they minimise localised compressive stresses.

In this paper shear modulus values were also obtained from torsion tests and are presented. It was found that test values based on torsion tests were slightly higher than the shear strength values in EN338. It was also found that within the grades C16 and C24 the torsional shear modulus and shear strength are correlated to a degree similar to that between MoR and MoE in bending. It was found that *G* and *E* are not correlated with each other within the range of grades tested.

The recent draft revision of the testing standard EN408 [16] includes the torsion testing approach to obtain the shear modulus of timber. In this study it was found that both shear strength and shear modulus can be obtained from torsion tests and it is proposed that EN408 allow the torsion test to be used to determine shear strength as well. However it is also noted that the torsion testing approach requires careful application to avoid premature failure due to crushing at the specimen grips.

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