THE EVALUATION OF SHEAR PROPERTIES OF TIMBER BEAMS USING TORSION TEST METHOD

by

AAMIR MUHAMMAD KHOKHAR

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The Centre for Timber Engineering
School of Engineering and the Built Environment
Edinburgh Napier University
Edinburgh, Scotland, United Kingdom
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ABSTRACT

The use of shear properties including the shear modulus and shear strength of timber joists in current timber design is becoming increasingly important, especially to provide an adequate torsional stability and to avoid vibrational serviceability problems. No proper test method has been established to evaluate the shear properties and the standard testing agencies have recommended the determination of shear properties from modulus of elasticity of timber joists. A torsion test approach can be used to obtain the shear properties as it creates a purer state of shear in the timber joists. However, little research has been conducted to address the proper use of torsion and this requires an urgent detailed investigation on torsion.

This research was primarily conducted to provide a better understanding the use of torsion test approach to evaluate the shear modulus and shear strength of timber joists. Full-scale laboratory torsion tests were conducted on structural size Sitka spruce (*Picea sitchensis*) and Norway spruce (*Picea abies*) joists and on small clear wood and shear properties were obtained. It was found that torsion test produced considerably higher shear modulus and shear strength of timber joists than design values provided from the testing agencies. This signifies that torsion test may be a better approach to evaluate the shear modulus and shear strength of timber joists.

This project also investigated the variation in shear modulus along the length of joists and influence of knots on shear modulus. A substantial variation, as much as 30%, was found in shear modulus along the length of individual joists. However, it was observed that knots did not cause any variation of shear modulus.

This study also details the fracture mechanism of timber under torsional loading and recommends four general failure modes that can be used as a guideline for future investigations on torsion. In general, it was noticed that the torsion test yields predominantly shear failure in joists and that the fractures were commonly initiated

within clear wood and propagated parallel to the long side of joists where shear stresses were presumed to be maximum under applied torque.

This research also investigated the relationship between shear modulus and modulus of elasticity. No correlation between shear modulus and modulus of elasticity was found when the relationship was developed from small clear wood to full structural size. This raises serious concerns to whether determination of shear modulus based on modulus of elasticity is an appropriate approach.

Overall, the work conducted here will provide an in-depth and broader understanding of use of torsion. The recent revision of the testing standard EN408 includes the torsion testing approach to obtain the shear modulus of timber. This work endorses the inclusion of torsion testing and proposes that the torsion test also be adopted as a method for evaluating the shear strength of timber.

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1. INTRODUCTION

1.1 Introduction

The use of timber as a structural material in construction industry, especially for residential buildings, is long known. Timber beams are the most significantly used structural member like other timber members such as columns and struts, trusses and shear walls. Timber beams, often referred to as a “joists”, have traditionally been used as solid sawn lumber, as shown in Figure 1-1. With advent of engineering wood products, the use of engineered wood I-joists, open web trusses and glued laminated timber joists (Figure 1-1) has become more frequent.

Timber joists should be designed in such a way that they meet stiffness and strength requirements. The bending, shear, tensile and compressive properties are the basis of the design of simple solid timber joists to large dimensional glued laminated timber beams. However, under normal loading conditions, bending and shear properties becomes more crucial for the design approaches. The bending properties including modulus of elasticity (E) and the bending strength can be evaluated from flexural tests. In this regard, European Committee for Standardization ‘CEN’ (EN408:2003, 2003) and American Standard of Testing Materials ‘ASTM’ (ASTM-D198-94, 1996) have recommend tests of full-size timber joists under two transverse point loads at one-third distance of span from supports (four-point bending test).

Less attention has been paid to evaluate the shear properties of joists and generally they are determined from clear wood or bending properties of joists. The test standards (EN408:2003, 2003 and ASTM-D143-94, 1996) recommended evaluation of shear strength from tests of clear wood blocks, “shear blocks”, under compressive loads. However, the use of shear block test has potential limitations such as the extent to which shear blocks can take account of anisotropic behaviour of wood and the influence of wood defects. Full-size structural joists can be tested under bending to take the possible influence defects and orthotropy into account. Considering this, the design shear strength values in (EN338:2008, 2008) are determined from bending strength of full-size joists.

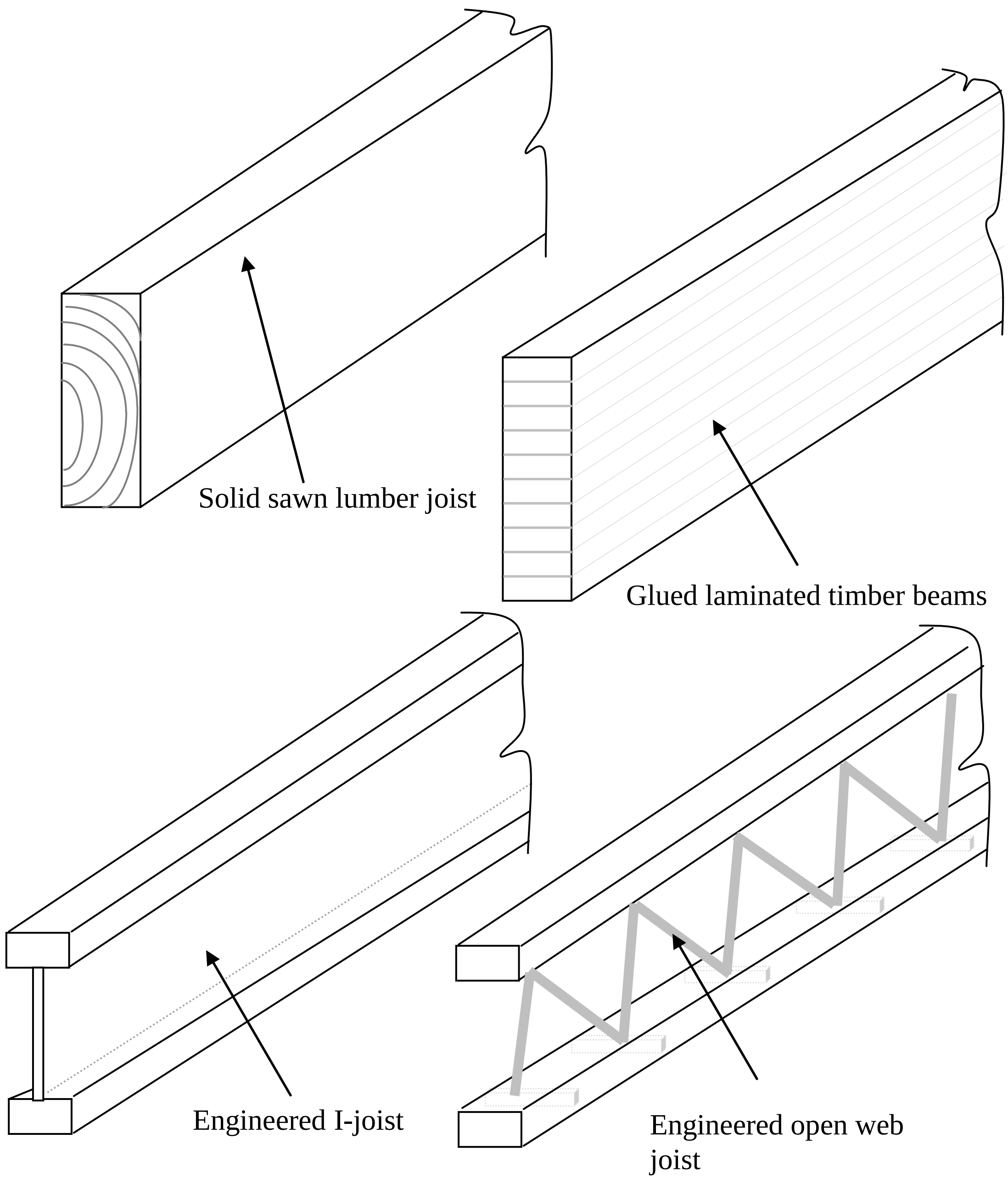


Figure 1-1 Different types of timber joists use for structural purposes.

No published standard test procedures are given to determine the shear modulus (G) of timber joists. However, CEN (EN408:2003, 2003) and ASTM (ASTM-D198-94, 1996) provided an analytical approach to obtain the shear modulus incorporating the modulus of elasticity of joists obtained from three and four point bending tests. The analytical approach has been rarely used due to its complex procedure and, therefore, shear modulus is often calculated from the modulus of elasticity using E to G ratio of 16:1 (Bodig and Goodman, 1973). The published design values of shear modulus in CEN (EN338:2008, 2008) and Woodhand book (USDA, 1999) are also determined from E:G of 16:1. However, no research investigations have been conducted to determine if shear modulus and modulus of elasticity are correlated with each other and that the above approaches are applicable to evaluate the shear modulus.

Evaluation of shear modulus and shear strength from the modulus of elasticity and bending strength may not be an appropriate approach. Although a joist under bending might be close to the real-life loading condition it would not provide a simple approach to analyse state of shear due to anisotropic of wood and the interaction of tensile, perpendicular compressive and shear stresses that take place. A torsion test approach can be used to attain the state of pure shear. Although a torsion test does not represent an actual real-life loading condition, it does produce a purer and a clearer shear stress distribution in the specimen allowing measurement of the pure shear properties. However, until recently, very little attention has been paid to use of the torsion test to evaluate the shear modulus and shear strength. To this end, very recently, the CEN test standard (EN408:2009, 2009) has included the torsion test approach to determine the shear modulus of timber joists.

A very few research works were conducted to consider torsion test for shear properties. Riyanto and Gupta (1998) were first to assess the applicability of torsion test to evaluate the shear strength of solid timber joists. They found that the torsion test is more appropriate than bending tests when compared with failure modes and shear strength values. Gupta et al. (2005a, 2005b) studied the applicability of torsion test to determine the shear strength of solid timber and structural composite lumber (SCL) joists using both experimental and finite element approaches. Burdzik and Nkwera (2003) employed torsion test to obtain the shear modulus of solid timber joists to examine the validity of E:G ratio of 16:1. Hindman et al. (2005a, 2005b) used torsion test to obtain the torsional rigidity of solid timber and SCL solid and I joists. Harrison (2006) used torsion test method to obtain the shear modulus and to assess the accuracy of E:G ratio 16:1 to predict shear modulus of timber beams.

As stated above, the torsion test may be a better approach to evaluate the shear properties of timber joists as it induces only shear stresses in the member and gives a pure state of shear. It is apparent that the effectiveness of torsion test mainly depends on proper use of test procedure. However, previous investigations were mainly focused on the determination of shear properties and paid less attention on the appropriate use of test procedure. In addition to this, CEN (EN408:2009, 2009) does not adequately address the test method in great details. Therefore, there is a need for a systematic investigation that provides a proper use of torsion test to evaluate shear properties of joists and gives a better understanding of the fundamentals of torsion tests. This thesis is oriented towards this goal.

1.2 Objectives

The main objective of this research is to examine the applicability of the torsion test to obtain the shear modulus and shear strength of timber joists. The other key intent of conducting this work is to look at the relationship between the shear modulus and the modulus of elasticity. The other purposes of investigations are:

1. To evaluate the shear modulus of solid timber joists of various lengths and to observe the influence of clockwise and anti-clockwise torque on shear modulus of timber joists as an orthotropic material.

2. To examine if shear modulus is constant or varies along the length of joist and if torsion is a repetitive test.

3. To evaluate shear strength of solid timber joists of different lengths and to develop a correlation between shear modulus and shear strength obtained from torsion tests.

4. To examine the failure modes within joists under maximum torsional loading and to observe and define the relationship between shear modulus and fracture.

5. To examine the correlation between the shear modulus and modulus of elasticity.

1.3 Summary of Methods

To achieve the above objectives, an extensive experimental study was conducted. A torsion tester was used to induce torque in the test specimens and relative twists were obtained directly from the tester. However, the embedment of wood and machine gearing system may alter the twist. Therefore, inclinometers were used to measure the actual twist of joists. Sitka spruce (Picea sitchensis) and Norwegian spruce (Picea abies) solid joist of structural grades of C16 and C24 were tested. To evaluate the shear modulus, joists were tested within elastic range by inducing torque in both clockwise and anti-clockwise directions. The relative twists of joists were measured by mounting inclinometers near the reactional supports. The Saint-Venant torsion theory was employed to determine the shear modulus.

One of the main objectives of this research was to examine if shear modulus varied along the length of timber joists. To attain this, tests were conducted by mounting inclinometers at various locations along the length of joist. This facilitated the determination of the shear modulus of several of sections of joists and demonstrated the variation in shear modulus within joists. To attain the shear strength, the same joists were tested until they were fractured under applied torque. A relationship between shear modulus and shear strength was developed to examine if both properties had correlation when obtained from torsion. Observations were also made to examine the location of initiation and type of fractures. A correlation between the fracture location within joist section and the shear modulus value of the same section was inspected.

This investigation was mainly focussed on the shear properties of timber joists. However structural size joists may contain various wood defects which may cause some influence on shear properties. Therefore, small clear wood specimens were tested under torsion to determine shear properties of free from defect wood.

As mentioned, in the past, shear modulus primarily obtained from modulus of elasticity and from an E:G ratio of 16:1. Therefore, the other most important objective of this investigation was to examine if shear modulus and modulus of elasticity has any correlation. The correlation was established from full-size structural joist to small clear wood. The modulus of elasticity of the test joists was obtained using four point bending tests. The four point bending allowed attaining the modulus of elasticity of 600mm sections and 1800mm sections of test joists. The modulus of elasticity of joist span and clear wood specimens was obtained using acoustic method. The shear modulus values of the same joist sections, joist spans and clear wood was determined using torsion tests and correlation between the two properties was developed.

1.4 Scope and Contribution of Research

This research will provide a better understanding and proper use of torsion test to evaluate the shear properties and will assist further research on torsion test for engineered wood joists. CEN (EN408:2009, 2009) have recently proposed the use of torsion to determine the shear modulus of timber joists. The outcome of this study will endorse the recommendation of torsion test by CEN and will also form the basis of adopting the torsion as standardize test for shear strength. This investigation will also contribute the information on the fracture mechanism of joists under torsion. This will facilitate in setting out the general failure modes of joists under torsion as there is no published information available on the failure modes. The work will also assess the correlation between shear modulus and modulus of elasticity and will direct if determination of shear modulus from modulus of elasticity is applicable.

1.5 Organisation of the Thesis

The structure of the thesis is as follows:

The first, basic principles of Saint-Venant torsion theory of various sections are briefly discussed in Chapter 02. Then a comprehensive review of available standard test procedures on shear properties of timber is presented. Earlier research investigations on torsion tests to determine of shear properties of timber joists are also reviewed.

Chapter 03 and Chapter 04 present the experimental investigation of full-scale torsion tests to determine the shear modulus of joists. Chapter 03 starts with the description of detailed experimental set-up followed by test procedure for shear modulus with focus on clockwise and anti-clockwise torque loading. Chapter 04 primarily characterizes the test method to examine the variation in shear modulus. This chapter also describes the applicability of torsion in relation to repetitive testing and influence of knots on shear modulus.

Chapter 05 describes the full-scale laboratory torsion test to evaluate the shear strength of timber joists. At first, test arrangements of the test method are discussed thoroughly, followed by the test results. The chapter also discusses the causes of fractures and type of failure modes of joists under torsional loadings. Furthermore, correlation between fracture and shear properties also presented.

To understand better the torsion test approach, small-scale laboratory tests were conducted on clear wood and presented in Chapter 06. The test procedure, the test results and failure mechanism of small clear wood under torsion is explained in detail.

Chapter 07 is the penultimate chapter and presents the most significant investigation that was conducted to examine the relationship between modulus of elasticity and shear modulus. The correlation between the two fundamental properties of wood was studied in a broad range from a full-size structural joist to small clear wood. The chapter also discusses the modulus of elasticity and shear modulus ratio obtained from the study and its comparison with the E:G of 16:1.

The final Chapter summarizes the thesis and details the conclusions of the test results of this thesis and presents the recommendations for further studies.

2. LITERATURE REVIEW

2.1 Introduction

Timber joists have been used extensively as a structural member in construction from small to very large structures. The design of a timber joists mainly depends upon its stiffness and strength properties. The shear, bending, tensile and compressive strength properties are the basis of ultimate design for elements from simple timber joists to very large glued laminated timber beams. The shear properties, shear modulus and shear strength, become more essential when joists are designed for lateral torsional stability and complex design and control of wood floor vibration. The different agencies have provided standard test methods, including of shear block test, torsion test, etc., to attain the shear properties of timber joist. On the other hand, researchers conducted studies on the applicability of standard test approaches and have been trying to introduce different testing methods to evaluate the shear properties more accurately and effectively. The torsion test approach can be used to evaluate the shear properties of timber. However, it is found that not much attention has been given to use of torsion test.

This research is conducted to emphasis comprehensively use of the torsion test method and its applications to evaluate the shear modulus and shear strength of timber joists. In this regard, this chapter presents the previous research works that were conducted on the use of torsion test method to attain the shear properties of joists. A summary of available standard tests and other test approaches for shear properties is also presented in this chapter. This chapter also discusses the application of St. Venant torsion theory to obtain the shear modulus and shear strength as this research took account of torsion theory to obtain the shear properties.

This chapter, therefore, discusses relevant work under the following four categories:

1. Torsion theories
2. Standard test methods for shear properties
3. Past researches on use of torsion test for shear properties
4. Past proposed test methods to evaluate shear properties

2.2 Torsion Theory

When a member of any cross sectional shape is subjected to a torque along its longitudinal axis, the torque tends to produce a rotation in the member with respect to its longitudinal axis. This rotation causes twist the in member and this state is known as torsion.

2.2.1 Basic Torsion Theory

Elementary torsion theory was limited to the circular members and was developed on the following assumptions:

- The cross section must be circular, without taper, no stress concentrations and the axis of rotation of the bar must be straight.
- Torque must be applied by shear stresses that vary linearly with the same distance from the axis.
- Angle of twist must be small and varies linearly along the longitudinal direction.
- Plane cross sections of the bar do not change after angular deformation and all radii must remain straight: cross sections do not warp.
- The material must be homogenous, linearly elastic and isotropic.

Based on the above assumptions, the relationship between the applied torque (T) and the angle of twist (φ) of circular bar is described in Equation (2-1):

$$\frac{T}{\varphi} = \frac{G\,J}{L} \qquad (2\text{-}1)$$

Where G is the shear modulus of the material; J is the polar moment of inertia. The maximum shear stress (τ_{max}), occurs at the outer surface, can be accounted as,

$$\tau_{max} = \frac{Tr}{J} \tag{2-2}$$

Where

$$J = \frac{1}{2}\pi r^4 \tag{2-3}$$

r is the radius of the circular member.

2.2.2 Saint-Venant Torsion Theory

In order to develop the torsional behaviour of non-circular cross section, Saint-Venant made the following assumptions:

- The member is straight, has constant cross section without taper and.
- The load is pure torque and produced by the shear stresses distributed over the end cross sections.
- Each cross section of member rotates approximately as rigid body and rotation of each cross section varies linearly along the longitudinal direction.
- Angle of twist must be small and for small deformation and that warping must be small and the same for each cross section.
- The member must be homogeneous, isotropic and linearly elastic

On the basis of the above assumptions and by applying strain displacement relationship and Hooke's law, Saint-Venant developed the following equations for solving torsion of non-circular cross sections:

$$\frac{\partial^2\psi}{\partial x^2} + \frac{\partial^2\psi}{\partial y^2} = 0 \tag{2-4}$$

$$\tau_{xz} = G\theta\left[\frac{\partial\psi}{\partial x} + y\right] \tag{2-5}$$

$$\tau_{yz} = G\theta \left[\frac{\partial\psi}{\partial y} - x\right] \quad (2\text{-}6)$$

τ_{xz} and τ_{yz} represent shear stresses in relative planes to applied torque and ψ is the warping function relates to angle of twist per unit length of member (θ) and axial displacement in the direction of applied torque. Later in 1903, Ludwig Prandtl suggested that the τ_{xz} and τ_{yz} can be taken as a magnitude of a slope of stress function surface to their perpendicular planes and Equations (2-4) to (2-6) can be written as:

$$\frac{\partial^2\emptyset}{\partial x^2} + \frac{\partial^2\emptyset}{\partial y^2} = -2G\theta \quad (2\text{-}7)$$

Where $\emptyset$ is a stress function represents surface over the cross section of the torsion member. Also twice the volume between $\emptyset$ and the plane of cross-section characterizes the torque and can be written as:

$$\mathrm{T} = 2\iint_A \emptyset\, dxdy \quad (2\text{-}8)$$

The stress function can be determined by using an elastic membrane analogy approach. The equation of membrane analogy can be derived by applying a uniform lateral pressure (p) on the opening of the plane, covered by homogenous elastic membrane, such as soap film. The opening has the same shape as of the cross-section that is under torque. The applied lateral pressure causes a lateral displacement (z), and an initial tension (S) as shown in Figure 2-1, and that membrane bulge out. If the pressure is small then slope of membrane will also be small and the equation for membrane analogy can be derived as follows:

$$\frac{\partial^2 \mathrm{z}}{\partial x^2} + \frac{\partial^2 \mathrm{z}}{\partial y^2} = -\left[\frac{p}{S}\right] \quad (2\text{-}9)$$

Therefore, the stress function ($\emptyset$) in Equation (2-7) can be represented mathematically equivalent of displacement of membrane (z) in Equation (2-9).

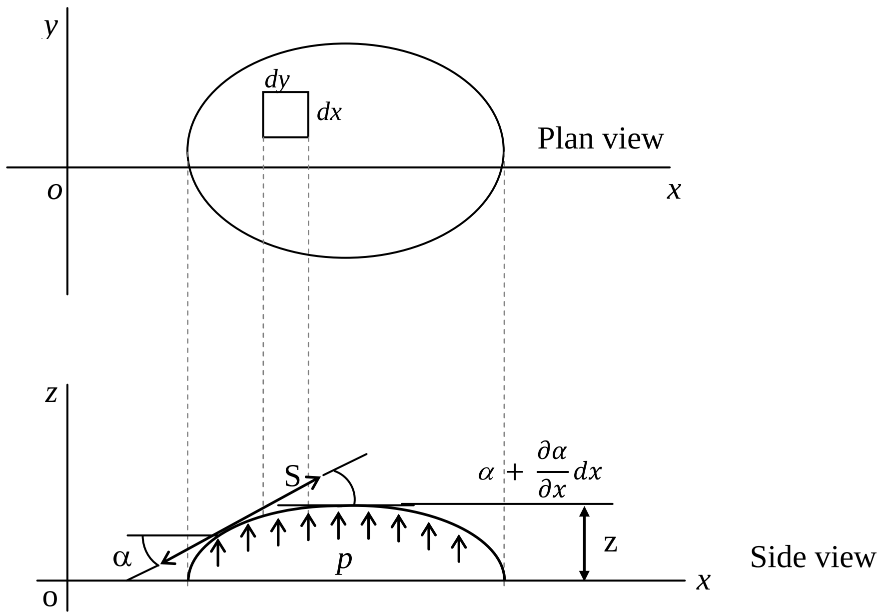


Figure 2-1 Deformation of pressurized elastic membrane

The stress function and the membrane analogy can also be employed to obtain the torsional behaviour of rectangular and I-shaped beams. For narrow rectangular cross section such that width << depth, the membrane analogy can be obtained as:

$$\frac{\partial^2 z}{\partial x^2} + \frac{\partial^2 z}{\partial y^2} = -\frac{2z_o}{b^2} \tag{2-10}$$

Where z_o is the maximum deflection of member and y is boundary for thickness (b), as shown in Figure 2-2. The stress function can be simplified as follows:

$$\emptyset = G\theta h^2 \left(\frac{b}{2}\right)^2 \left[1 - \left(\frac{y}{b/2}\right)^2\right] \tag{2-11}$$

By substituting Equation (2-11) into Equation (2-8),

$$\frac{T}{\theta} = \frac{1}{3} G\,(d)(b)^3 \qquad (2\text{-}12)$$

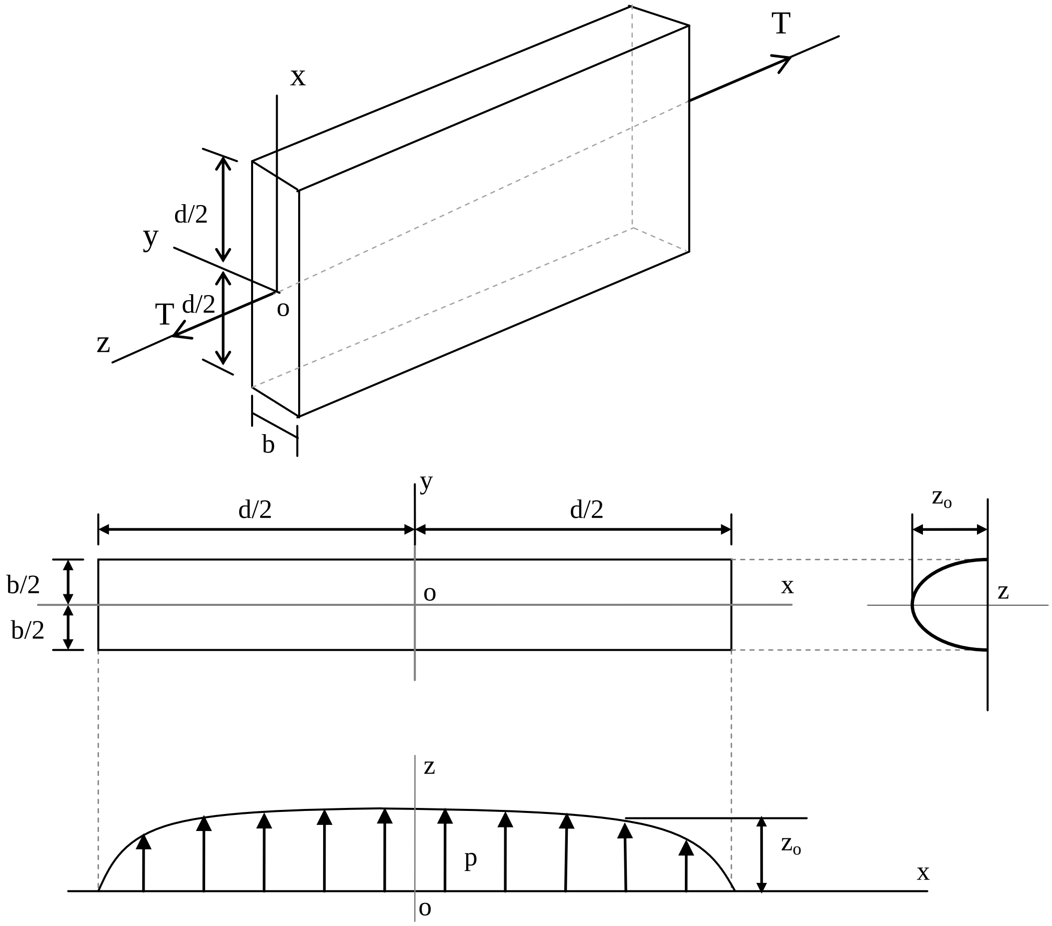


Figure 2-2 A rectangular cross-section member under action of torque and membrane on cross-section

For normal rectangular cross-section, the stress Equation (2-11) can be expanded using the Fourier series and the Equation (2-12) can be solved to torque-rotation relationship can further be simplified by:

$$\frac{T}{\theta} = G k_1\,(d)(b)^3 \qquad (2\text{-}13)$$

The maximum shear stress ($\tau_{\max}$) in the cross-section can be obtained as:

$$\tau_{max} = \frac{T}{k_2(d)(b)^2} \tag{2-14}$$

The values of k_1 and k_2 can be obtained on the basis of the depth and thickness ratio given in Table 2-1.

Table 2-1 Torsional parameters for isotropic rectangular cross-sections

d/b	1.0	1.5	2.0	2.5	3.0	4.0	6.0	10	∞
k_1	0.141	0.196	0.229	0.249	0.263	0.281	0.299	0.312	0.333
k_2	0.208	0.231	0.246	0.254	0.267	0.282	0.299	0.312	0.333

2.2.3 Torsion Theory of Orthotropic Bars

An anisotropic body is defined as a body in which elastic properties, modulus of elasticity, shear modulus are different in various directions drawn through a given point. Orthotropic property is a special type of anisotropic property where elastic properties of material are different in three different orthogonal directions. For the orthotropic rectangular member bar, the stress function equation (Equation (2-11)) can be written as (Lekhnitskii, 1963):

$$\left(\frac{G_{LT}}{G_{LR}}\right)\frac{\partial^2 \emptyset}{\partial x^2} + \frac{\partial^2 \emptyset}{\partial y^2} = -2G_{LT}\theta \tag{2-15}$$

G_{LT} and G_{LR} are the shear modulus in Longitudinal-Tangential (LT) plane (longside) and Longitudinal-Radial (LR) plane (shortside), respectively of the member, as shown in Figure 2-3.

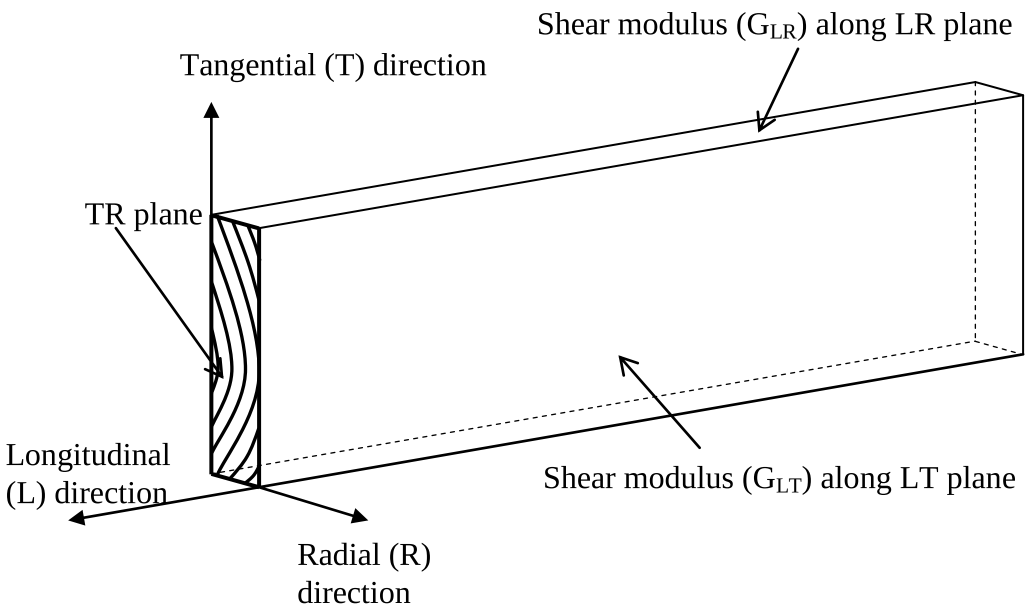


Figure 2-3 Schematic diagram of an orthotropic rectangular member

By applying Equation (2-11), torque-rotation relationship can be resolved and simplified into the following form:

$$\frac{T}{\theta} = G_{LT}(d)(b)^3\beta \qquad (2\text{-}16)$$

The maximum shear stresses can be obtained in shortside and longside as follows:

$$\tau_{\max(Shortside)} = \frac{T}{(d)(b)^2}k_{A1} \qquad (2\text{-}17)$$

$$\tau_{\max(Longside)} = \frac{T}{(d)(b)^2\left(\sqrt{\frac{G_{LT}}{G_{LR}}}\right)}\ k_{A2} \qquad (2\text{-}18)$$

The values of β, k_{A1} and k_{A2}, depending on cross section and shear modulus of rectangular bar, can be obtained by using Table 2-2, given by (Lekhnitskii, 1963).

Table 2-2 Torsional parameters of an orthotropic rectangular bar

$\frac{d/b}{\sqrt{G_{LT}/G_{LR}}}$	β	k_{A1}	k_{A2}
1	0.141	4.803	4.803
2	0.229	4.065	3.232
5	0.291	3.430	2.548
10	0.312	3.202	2.379

2.3 Standard Test Methods for Shear Properties

The testing standard agencies, such as European Committee for Standardization (known as CEN) and American Standard of Testing Methods (ASTM) have recommended different test methods to evaluate shear modulus and shear strength of timber. The test methods include testing of small clear wood and of full-size structural timber under flexural and torsion loadings. In addition to this, test standards used bending strength and an approach of modulus of elasticity to shear modulus ratio of 16:1 (E:G ratio 16:1) to provide design values for shear strength and shear modulus. The following sections are detailing the test procedures recommended from CEN and ASTM.

2.3.1 Test Methods for Shear Strength

2.3.1.1 Test of Small Clear Wood

The CEN (EN408:2003, 2003) recommends that shear strength can be evaluated by testing a 32×55×300mm clear (free of defects) wood by inducing compressive load along the grain direction. To avoid any compression failure, the load can be induced at 14° on 10mm steel plates that glued to the specimens, as illustrated in Figure 2-4. The shear strength can be calculated from:

$$f_v = \frac{F_{max}\cos(14^\circ)}{l\ b} \tag{2-19}$$

f_v defines the shear strength, F_{max} defines the maximum applied load, l denotes length and b represents the width of specimen.

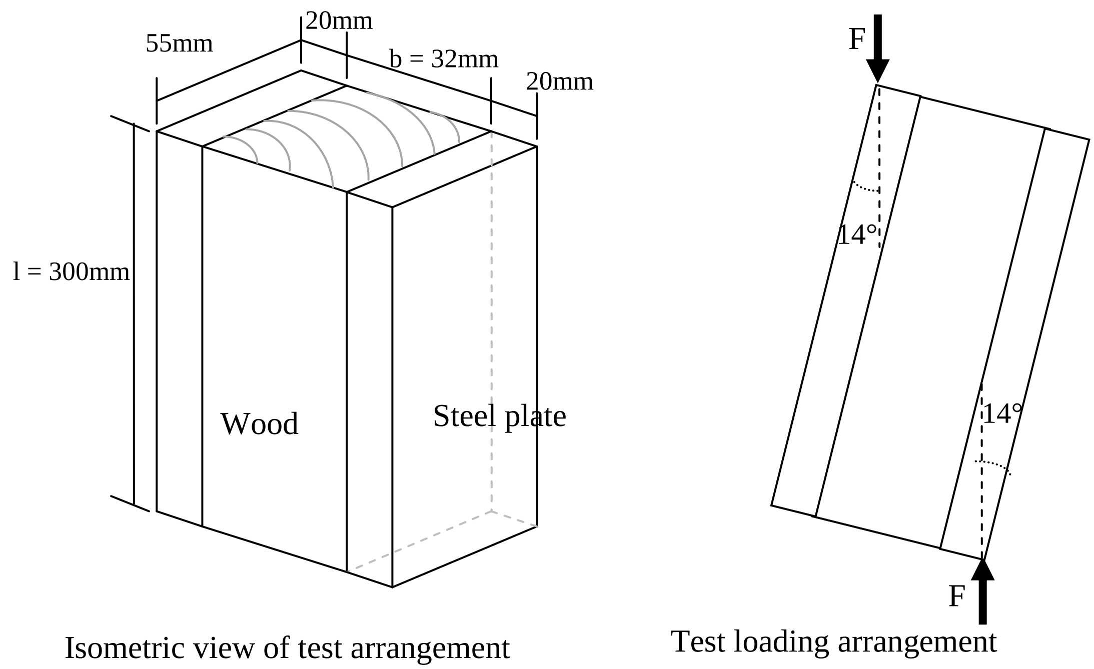


Figure 2-4 Test setup for determination shear strength recommended by (EN408:2003, 2003)

It is found that the above test method is rarely used due to complexity of the method. However, the small clear block test (ASTM-D143-94, 1996) is most often used for evaluting the shear strength due to simplicity of the procedure. In the test procedure, 51×51×64mm one end notched clear wood block is tested by inducing load at notched end, as shown in Figure 2-5. The test procedure is commonly known as "shear block test method" as applied load only develops shear stresses in the wood block and fails mainly due to shear. The shear strength can be calculated by dividing the maximum applied load by the shearing area.

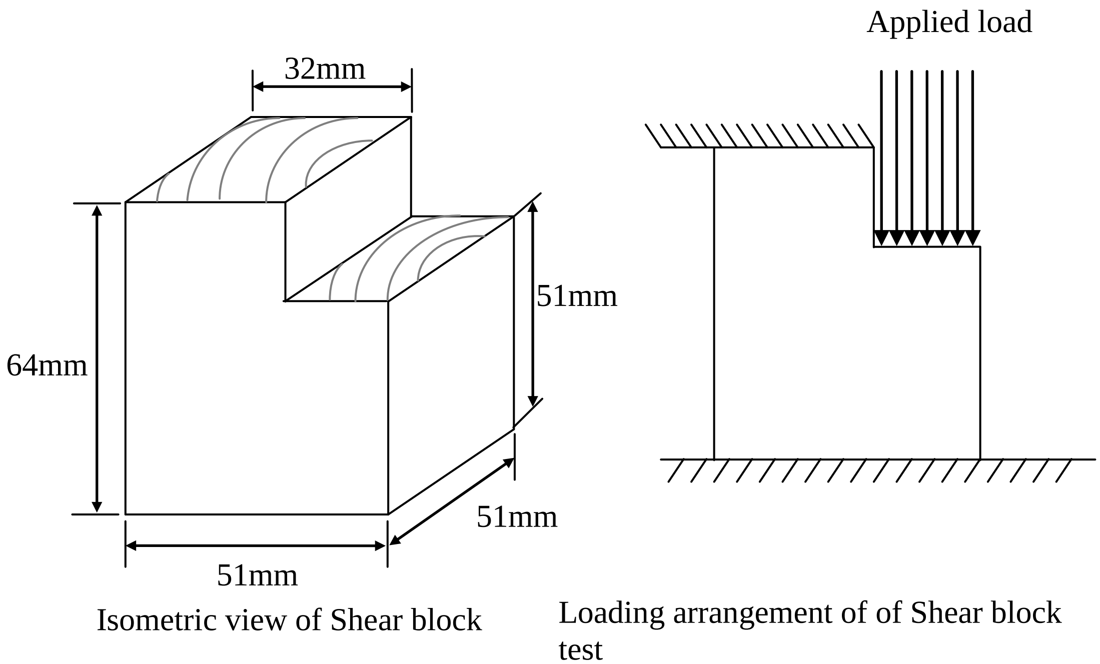


Figure 2-5 Test setup of shear blocks test recommended by (ASTM-D143-94, 1996)

2.3.1.2 Tests of Full-size Structural Timber

The CEN is only limited to test small clear wood and no test method has been suggested to evaluate the shear strength of timber joists. ASTM (ASTM-D198-94, 1996) has proposed torsion and four-point bending test methods to attain shear strength of full-size structural lumber. In the four point bending test, a structural size timber specimen can be tested by applying two symmetrical transverse loads at one third distance from either supports, as shown in Figure 2-6, until the specimen failed. In the method, it is assumed that shear stresses are dominant along the shear span and that bending stresses are less influential. To allow a higher percentage of shear failure, the distance between reaction and nearest loading point usually considered as six times of the depth. The shear strength can be calculated by using elementary beam theory as given in Equation (2-20):

$$\tau_{max} = \frac{3}{4}\frac{F_{max}}{b\,d} \qquad (2\text{-}20)$$

Where,
τ_{max} = maximum shear stress

F_{max} = maximum applied load

b = width of the member

d = depth of the member

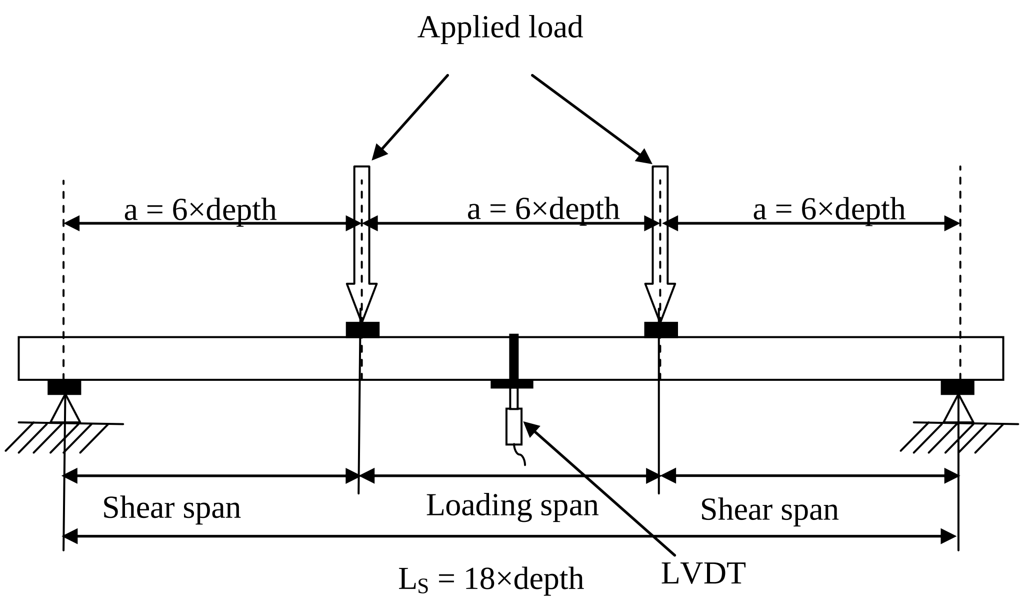


Figure 2-6 Test setup of four-point bending test for shear strength

In the torsion test approach (ASTM-D198-94, 1996), shear stresses can be induced in test specimens by applying moment couple nearer the specimen ends. The induced stresses cause twisting distortion in the member and produce shear failure within the span of the member. A vise-like shape clamps can be used to avoid any possible slippage, damage or stress concentrations. The test method also suggests that the test specimens should be at least eight times larger than its depth (major cross section). The torque must be induced with constant rate of speed so that maximum torque must be achieved within ten minutes and not less than five minutes and nor over twenty minutes. The shear strength can be calculated along the longside and shortside of the member using following equations:

$$\tau_{\max\,(Longside)} = \frac{8\,\gamma\,T}{\mu\,d\,b^2} \tag{2-21}$$

$$\tau_{\max\,(Shortside)} = \frac{8\,\gamma_1\,T}{\mu^3} \tag{2-22}$$

Where,

T = maximum applied torque

d = depth (major cross section) of the member

b = thickness (minor cross section) of the member

μ, γ and γ_1 = Saint-Venant constant

2.3.2 Testing Methods for Shear Modulus

2.3.2.1 Torsion Test Method

In recent proposed version of CEN (EN408:2009, 2009), the torsion test method has been included to obtain shear modulus (G) of timber. In the test procedure, a structural timber of length of 19 times of the depth is tested by inducing torque within elastic range. The relative rotational displacements can be measured nearer the end of specimens at a distance two to three times the depth of the test specimen. The shear modulus (G_{Tor}) can be obtained by using following equation.

$$G_{Tor} = \frac{K_{Tor}\,l_1}{\eta\,d\,b^3} \tag{2-23}$$

K_{Tor} defines rotation stiffness and can be determined by conducting regression analysis of applied torque within proportional limit and relative twist of the member.

l_1 = gauge length

d = depth of the member

b = width of the member

η = shape factor, depends on the depth to width ratio

The torsion test is also recommended by the ASTM (ASTM-D198-94, 1996) to obtain the shear modulus. The shear modulus can be obtained by inducing torque within elastic range at strain rate of 4º/metre/minute by using following approach:

$$G = \frac{16\,L\,T'}{w\,b^3\left[\left(\frac{16}{3}\right) - \lambda\left(\frac{b}{d}\right)\right]\Theta} \tag{2-24}$$

Where:

T' = applied torque at proportional limit

L = gauge length

Θ= total angle of twist

b= thickness of member

d = width of member

λ = Saint-Venant constant

2.3.2.2 Flexural Test Methods

Earlier to the proposed torsion test, CEN (EN408:2003, 2003) suggested single and variable span methods to attain the shear modulus. The variable span method is also recommended by the ASTM (ASTM-D198-94, 1996). In both test methods, the shear modulus is calculated from modulus of elasticity. In the single span method, the shear modulus is calculated from apparent modulus of elasticity ($E_{m,app}$) and local modulus of elasticity (E_{Local}). The E_{local} can be determined by testing test specimen under four point bending as shown Figure 2-6. The $E_{m,pp}$ is achieved by testing the same specimen under three point bending by applying load at centre, as illustrated in Figure 2-7. $E_{m,pp}$ actually represents both induced shear and bending stresses and E_{Local} only accounts the pure bending for the loading length (L_L), as shown in Figure 2-7. Hence the analytical model (Equation (2-25)) determines the shear modulus in the specimen by deducting the pure bending from combined bending and shear stresses as follows*:*

$$G = \left[\frac{K_G \, h^2}{\frac{1}{E_{m,app}} - \frac{1}{E_{Local}}} \right] \qquad (2\text{-}25)$$

Where K_G is constant value of 1.2 for rectangular cross-section and h is the depth of the member.

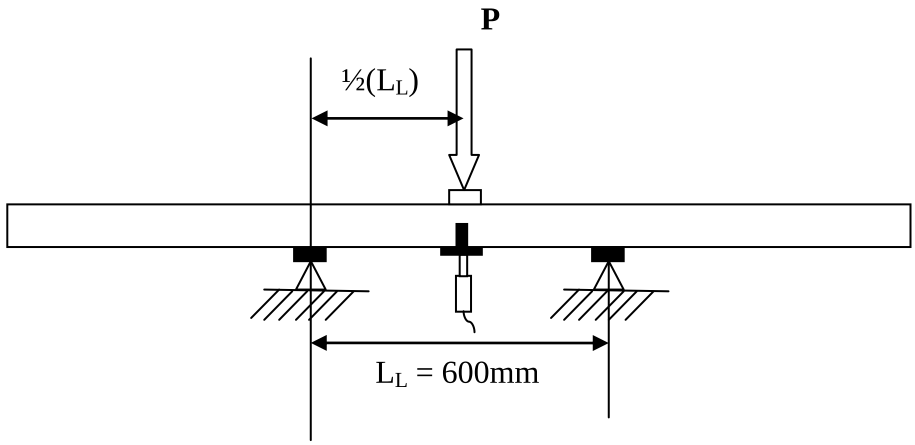


Figure 2-7 Test arrangement for apparent modulus of elasticity (EN408:2003, 2003)

In the variable span method, shear modulus is calculated from apparent modulus of elasticity and depth (h) to span ratio$\left[\frac{h}{L_s}\right]$. In the calculation procedure, the shear modulus is obtained from slope (K_1) of correlation between the reciprocal of apparent modulus ($1/E_{app}$) and square of depth to span ratio, as shown in Figure 2-8. The shear modulus then calculated from the following equation:

$$G = \frac{K_G}{K_1} \qquad (2\text{-}26)$$

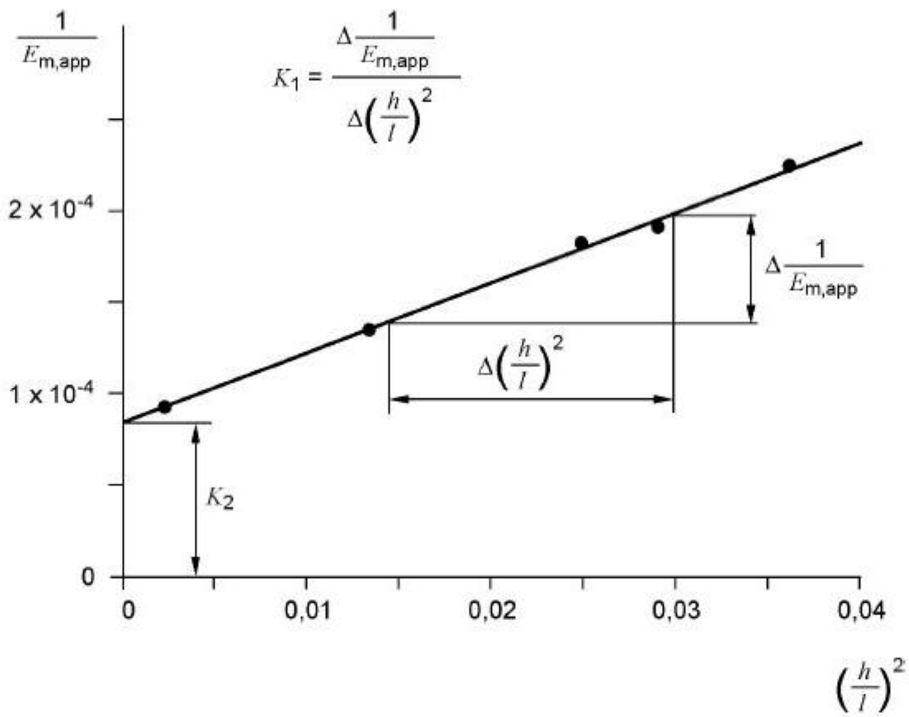


Figure 2-8 Determination of shear by variable span method as described by (EN408:2003, 2003) and (ASTM-D198-94, 1996)

2.3.3 E:G Ratio of 16:1 Approach

Apart from torsion test, the shear modulus is mainly evaluated from the modulus of elasticity of the wood. The modulus of elasticity can be evaluated easily and is well documented wood material property. Due to complexity of test approaches for calculating the shear modulus, an E:G ratio of 16:1 is used to obtain the shear modulus, especially for design equations of torsional rigidity and lateral torsional stability of timber joists (USDA, 1999). The design shear modulus values provided in EN 338 (EN338:2008, 2008) are also obtained from E:G ratio of 16:1. The E:G ratio was originated from by Bodig and Goodman (1973) and Gunnerson et al. (1973) on the basis of testing clear wood under plate twisting method. Although the E:G ratio has been considered as constant of 16:1. Researchers (Hindman et al., (2001), Chui, (1991) and Divos et al. (1998)) have shown that the ratio varies for different species, especially for structural composite lumber.

2.4 Past Researches on Torsion Test for Shear Properties

2.4.1 Shear Strength

A torsion test produces a purer and more uniform system of shear stresses in the specimen allowing measurement of the pure shear stiffness and strength. However, until recently, very little attention has been paid to use the torsion test. In the early Seventies, Vafai and Pincus (1973) used torsion test to investigate the behaviour of square and circular cross-sectional timber joists. The main objective of their work was to obtain the shear strength and failure modes of timber beams using torsion test. The shear strength was also computed from the orthotropic approach (Lekhnitskii, 1963) and from strain gauges mounted on the longside (Longitudinal-Tangential (LT) plane) and along the shortside (Longitudinal-Radial (LR) plane) of beams. The beams were also subjected to combined torsion and bending, and bending only loads to examine the failure patterns.

A 1400kN Universal testing machine was used in such a way that torsional, bending and combined torsion bending loads were induced. 100×100mm square and 100mm diameter circular beams of 2440mm long Douglas-fir and Red wood species were tested till fracture. They noticed that square timber beams failed at torque from 450 to 750Nm, and circular were failed from 400 to 800Nm. In their work, most specimens fractured along the LT plane under torsion and combined torsion and bending. In bending tests most specimens failed at the tension side or around knots. Vafai and Pincus (1973) found a good agreement between shear strength obtained from torsion test and shear strength obtained from strain gauges and from the orthotropic approach. They also found that longitudinal normal stresses were negligible under torsion and that specimens were fractured under pure shear.

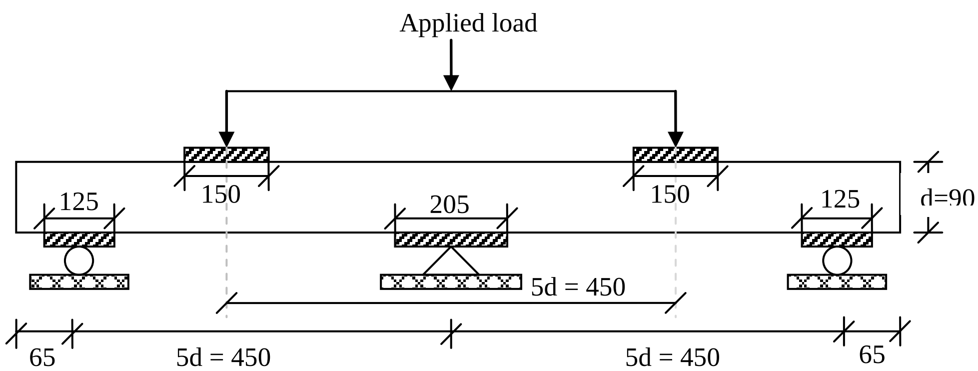
However, the shear strength from bending tests had a poor agreement with the shear strength obtained from strain gauges.

In late Nineties, Yoshihara and Ohta (1997) used torsion tests to develop shear stress and shear strain relationship for rectangular wood bars. In their work, Sitka spruce short beams of different depth to thickness (aspect) ratios were tested. The specimens were tested by a manual torsion test device and shear strains were measured from strain gauges mounted on specimens. The test specimens were tested by inducing torque in either longitudinal-tangential (LT) or longitudinal-radial (LR) direction until they fractured. They found that shear stresses were 5 to 7% different in the LT and LR planes but aspect ratio did not have substantial influence on shear stresses both in the LT and LR directions.

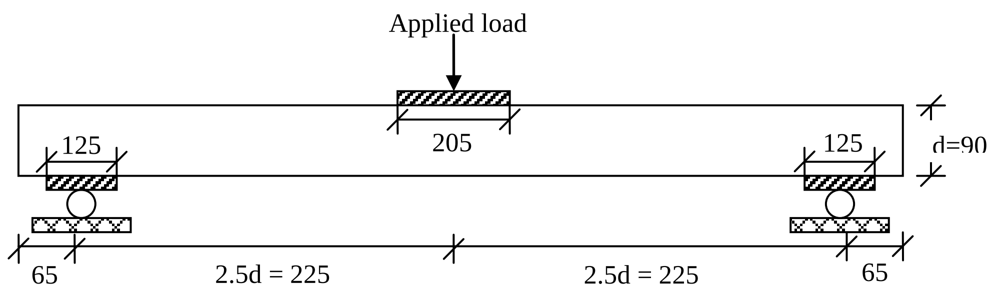
Riyanto and Gupta (1998) were the first to propose the torsion test approach to evaluate the shear strength of structural size timber. In their detailed research work, they used torsion test, bending (three point, four point and five point) tests and shear block tests to find suitable method to determine the shear strength. Figure 2-9 illustrates each test configurations used by Riyanto and Gupta (1998). For each test method, 76 test specimen of Douglas-fir were tested with different span depending on depth, as shown in Figure 2-9. A 7kN-m torsion tester was used for torsion tests and a 30kN universal test machine was used for bending tests and as accordance of ASTM (ASTM-D198-94, 1996). The shear blocks were obtained from the tested specimens and were tested as accordance of ASTM (ASTM-D143-94, 1996).

To evaluate the applicability of test method, they compared the failure modes and average shear strength values obtained from all test methods. They observed that in torsion test, all 76 specimens failed due to only shear and that the shear plane was parallel to grain direction. In most specimens the failure occurred in clear wood and that crack started from the mid span of longside and propagated towards the supports. This suggests that wood beams fractured in pure shear when tested under torsion as shear stresses are maximum along the middle of the longside and on shortside and caused fracture in the member. They noticed that torsion test accounts for the slope of grain as specimens with high slope of grain failed at a lower torque than the specimens with straight grain.

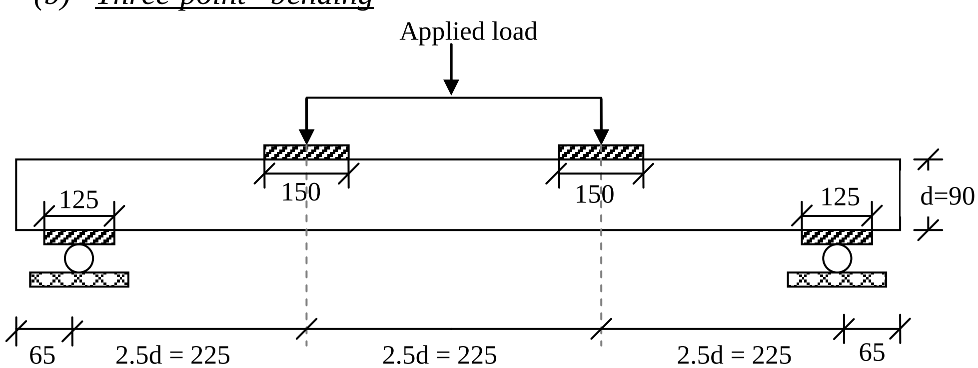
In bending tests, they noticed that the three-point produced the highest number of shear failures (33 out of 76 specimens) and that only six specimens were failed in shear when specimens tested under four-point bending. 21 out of 76 specimens were fractured in shear when tested in the five-point bending tests. They also found that the torsion test produced the highest average shear strength of 12.7MPa, about 60% higher than average shear strength (7.9MPa) obtained from shear block tests. Also, the four-point tests produced the lowest average shear strength value of 6.5MPa and the three and five-point provided shear strength of 10.1MPa and 11MPa respectively. Based on failure behaviour and average shear strength values, they recommended that torsion test method is a better approach for obtaining the shear strength full-size structural lumber.



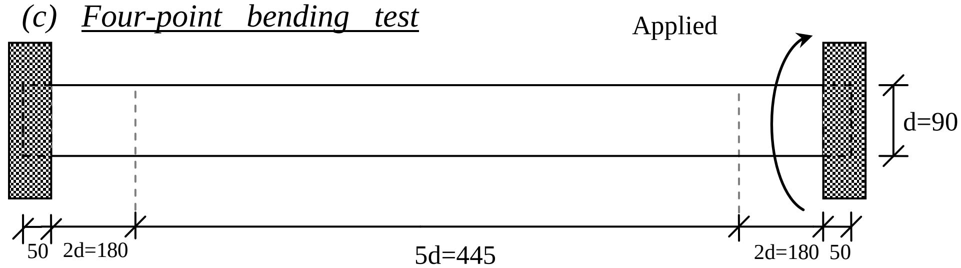
(a) *Five-point bending test*



(b) *Three-point bending*



(c) *Four-point bending test*



(d) Torsion test setup

Figure 2-9 Various test methods for evaluation of shear strength of wood used by (Riyanto & Gupta, 1998) (units = mm)

In continuation of applicability of torsion test method, Gupta et al. (2002a) used torsion test to examine if the length and depth of timber joists have any influence on shear strength. 38m thick Douglas-fir joists of depths of 90, 140, 190 and 250mm were tested. For each cross-section, lengths of 700, 1100, 1500, 1900 and 2300mm were used. Each specimen was tested by inducing toque at 4º per metre per minute (ASTM-D198-94, 1996) till specimens were fractured. The shear strength of both the longside and shortside of specimens was obtained using following equations:

$$\tau_L = \frac{\gamma T}{\mu h_r b_r^2} \tag{2-27}$$

$$\tau_S = \frac{\gamma_1 T}{\mu b_r^2} \tag{2-28}$$

Where

τ_L = maximum shear stress at the middle of the longside

τ_S = maximum shear stress at the middle of the shortside

h_r = ½ specimen depth

b_r = ½ specimen width

μ, γ and γ_1 = Saint-Venant constants

They found that length and depth do not have substantial influence on shear strength. Regardless of any cross section and for all lengths they noticed that shear strength values ranged between 10.2 to 10.9MPa and that for all depths the values varied from 9.3 to 10.5MPa. They observed that most specimens were fractured along the longside and that cracks begun at middle of centre of span and propagated along the length toward the end supports. They noticed in a length study that torsion tests produced 27% higher shear strength values (10.6MPa) than the shear block shear strength values (8.3MPa). For the depth, torsion test produced average shear strength of 9.9MPa, 11% higher than shear strength (8.9MPa) from shear blocks. They also examined the correlation between the shear strength of full size beams obtained from torsion and that of shear blocks. The correlation was developed by including shear area, depth and specific gravity and conducted by simple linear and multiple regression analysis. They found a moderate evidence of good relationship of shear block and torsion based shear strength. This, thus, indicates that torsion test is an appropriate method to obtain the shear strength as values obtained from torsion testing linearly correlate with the shear strength obtained from shear blocks.

Further to their experimental work, Gape et al. (2002b) used finite element (FE) approach to enhance the validity of the torsion test to attain the shear strength of timber beams. FE analysis was also conducted to understand the shear stress distribution, influence of shear span and failure mode of timber beams under torsion. Three dimensional rectangular specimens were modelled using eight node element with each node having six degrees of freedom. To simulate the torsion test method, a force couple was applied at one end while the other end was constrained to simulate the test clamps. A triangular load distribution was used to account for more bearing on the corners, as in the laboratory, the applied torsional loading to a rectangular specimen impacted on the opposite side corner more than the centre of the rectangle. Both isotropic and orthotropic material properties of Douglas-fir were included in the models.

They noticed that within the shear span, all shear stresses were nearly zero at the cross sectional centreline along longitudinal axis of the rectangle. They found that the maximum shear stress occurred at the middle of the longside along longitudinal direction and that shear stresses were almost uniform along both long and shortside. The maximum shear stress that caused shear failure was in the same orientation of the longside and this shows that specimen subjected to torsion loading was in state of pure shear. They recommended that torsion test be considered as a standard test method for determining shear strength of full size structural lumber as member is in pure shear and yields 100% shear failure.

Later, Gupta and Siller (2005a) employed torsion test method to evaluate the shear strength of full-size structural composite lumber (SCL). 44×140 ×1500mm laminated veneer lumber (LVL), parallel strand lumber (PSL) and laminated strand lumber (LSL) of 30 specimens each were tested under torsion. Addition to this, 76 shear blocks for each material were tested (ASTM-D143-94, 1996). The shear strength along longside (LT plane) and shortside (LR plane) was calculated using both isotropic equation (2-14) and orthotropic analytical equations (2-17) and (2-18) (Lekhnitskii, 1963).

They found that orthotropic behaviour of wood material does not have substantial influence on shear strength along the longside of the joists. As shear strength of LVL and PSL joists did not vary much and an increase of 2 to 4% was found when predictions were shifted from isotropic and orthotropic approach. However, a higher increase of 9.5% (11.6 to 12.9MPa) in shear strength was seen for LSL joists for the same. They observed that SCL joists behave more orthotropic along the shortside when predictions were conducted from isotropic to orthotropic theory. As for LSL, the shear strength was reduced considerably from 8.6 to 6.4MPa (27% decreased), 5.83 to 4.9MPa (16% decrease) for LVL and 5 to 4.5MPa (9% decrease) for PSL.

This was also seen when they conducted finite element analysis of the SCL joists (Gupta and Siller, 2005b). They found that shear strength was higher in the longside (LT plane) than in the shortside (LR). For LVL and PSL specimens, the fracture usually occurred at the mid span longside and along shortside of LSL joists. Based on their work, they recommended that the torsion test should be adopted as the test method for shear strength. As for solid lumber and SCL joist, the torsion test produced higher shear strength values and that it accounted for the orthotropic behaviour of joists. Also test specimens fractured at the locations where shear stresses were presumed to be the maximum.

2.4.2 Shear Modulus

Burdzik and Nkwera (2003) used torsion test to evaluate the shear modulus of 35×110×2800mm lumber South African pine of grades S4 and S5. The torque was induced by a hydraulic cylinder at one end through a lever arm and relative twist was measured by one tiltmeter mounted at the end of the member with applied torque. The shear modulus was calculated by conducting linear regression analysis within elastic region of applied torque and twist measured. The correlation between shear modulus and modulus of elasticity and approach of E:G ratio of 16:1 was examined. The apparent, global and dynaminc modulus of elasticity were obtined using the four-point bending and the vibration test methods. They found that the shear modulus was not much varied for different grades as the shear modulus of 780 and 790MPa was obtained for both grades. They noticed that shear modulus and modulus of elasticity had a poor correlation. Values for S4 (R^2 =0.45) and for S5 (R^2=0.35) were obtained when linear regression analysis was conducted. They also found that for both S4 and S5, the E:G ratio was slightly lower of 12:1 and 13.5:1 was obtained, respectively.

Hindman et al. (2005a) used torsion tests to obtain torsional rigidity (GJ) of solid timber and SCL joists. 38×241mm solid sawn lumber (SSL) and SCL members of parallel strand lumber (PSL), laminated veneer lumber (LVL) and laminated strand lumber (LSL) were tested. A 226N-m torsion machine was used to apply the torque to maintain 2º per metre rotation and relative angular rotations were measured from the change in arc length using Linear Variable Differential Transducers (LVDT). In all tests, specimens were loaded until an angular rotation of two degrees was attained. The GJ was obtained from the slope of the applied torque versus angular rotation curve multiplied by specimen length. They also used an E:G ratio of 16:1, isotropic Equation (2-29) by (Timoshenko & Goodier, 1930) and orthotropic Equation (2-30) by (Lekhnitskii, 1963) to compare with the torsional GJ.

$$GJ = G_{LT}\left[\frac{b^3}{3}\left(1-\frac{2}{\pi}\left(\frac{b}{d}\right)\right)\right] \quad (2\text{-}29)$$

$$GJ = G_{LT}\left[\frac{b^3 d}{3}\left(1-\frac{192b}{\pi^5 d}\sqrt{\frac{G_{LT}}{G_{LR}}}\right)\right] \quad (2\text{-}30)$$

Where,

G_{LT} = shear modulus along Longitudinal-Tangential (longside) plane

G_{LR} = shear modulus along Longitudinal-Radial (shortside) plane

From torsion tests, they found SCL joists produced higher GJ than SSL joists. The SSL joists produced GJ of 3100 Nm^2 and, LVL, PSL and LSL produced 2430 Nm^2 (21% higher than SSL), 3730 Nm^2 (22% higher) and 4160 Nm^2 (35% higher), respectively. The higher GJ for SCL was expected as the joists were fabricated from timber with less wood defects. They also noticed that torsional GJ was slightly higher (3 to 5%) for solid timber joists when compared with the predicted GJ from isotropic (2960 Nm^2) and for orthotropic (2850 Nm^2) approaches for the same joists. This, thus, indicates that orthotropic property of wood has very small influence on the shear rigidity of solid joists. However, for SCL members, both isotropic and orthotropic equations predicted 4 to 13% higher GJ compared to torsional GJ of the same joists. They also found that E:G ratio of 16:1 predicted substantially higher (17 to 42%) higher GJ values in comparison to tested values. They concluded that E:G ratio of 16:1 approach may not be appropriate for predicting GJ as it produced higher values. Later, Hindman et al. (2005b) also concluded that an E:G ratio of 16:1 and isotropic analytical approach is not appropriate method to predict torsional rigidity for solid and composite I joists.

Harrison (2006) used torsion test method to obtain the shear modulus and to assess the accuracy of E:G ratio 16:1 to predict shear modulus of timber beams. In her work, shear modulus was also obtained from three-point and five point bending tests to examine the applicability of torsion test method. The modulus of elasticity obtained from four-point bending tests was used in E:G ratio. 50×200×3600mm LVL joists and 24 machine-stress-rated (MSR) Southern pine joists of the same dimensions were used for each test. In the torsion test setup, a 560 Nm torsion machine that could twist members to ±50° was used to induce toque and relative angular displacements were measured by 20° range duel axis clinometers. Figure 2-10 shows the torsion test setup used by Harrison (2006). Each specimen was tested at 3.5° per minute and was tested up to 7° to avoid any damage to the specimen. The clinometers were mounted at 500mm distance from end clamps to eliminate any possible end effects.

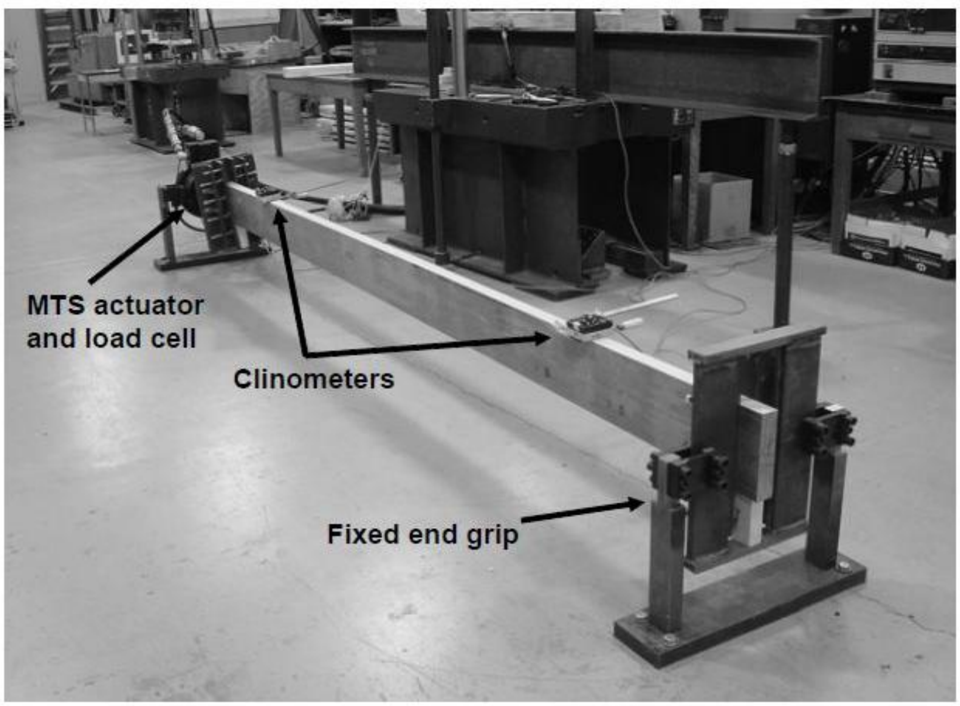


Figure 2-10 Torsion test setup used by Harrison (2006).

She found that solid timber torsion tests gave the highest average shear modulus value of 1160MPa, 32% higher than five point shear modulus (790MPa) and 21% higher than three point bending test shear modulus of 900MPa. However for PSL joists, five point bending produced slightly higher shear modulus values than torsion tests, as shown in Table 2-3:

Table 2-3 Shear modulus and modulus of elasticity values obtained by Harrison (2006)

Properties	Joist	Torsion test	3-Point bending	5-Point bending
Shear modulus (MPa)	MSR	1160	900	790
	LVL	1040	750	1140
Modulus of Elasticity (MPa)	MSR		17400	17550
	LVL		19600	20100

Harrison (2006) noticed a higher variation in E:G ratio predicted from tests and that found that the E:G ratio was not a constant value. For LVL, three and five point tests produced 27:1 (70% higher than 16:1) and torsion test gave E:G ratio of 19:1 (18% higher). For MSR, both bending tests also produced higher E:G ratio of 19:1 to 22:1 (24to 39% higher). Torsion test gave a slightly lower E:G ratio of 15:1. Since E:G ratio has significant variation within test methods, therefore she recommended to not to use the E:G ratio of 16:1.

Brander et al. (2007) investigated the applicability of torsion test, single span and variable span methods (EN408:2003, 2003) to evaluate the shear modulus of glued laminated timber beams. The modulus of elasticity was achieved by four point bending tests. In torsion tests, 5.70m long LVL were tested by inducing torque at speed of 4°/min (ASTM-D198-94, 1996) and relative rotations were recorded using extensometers. The extensometers were placed at 1600, 2950 and at 5000 mm and rotations were measured with end distance of 380mm. The material was considered as isotropic and shear modulus was calculated by using Equation (2-23).

An average shear modulus of 615MPa with a co-efficient of variation (COV) of 4% was attained when torsion test method was used. They noticed that a small increase of 600MPa to 630MPa in shear modulus was found with an increase in length from 1600mm to 5000mm. In comparison to torsion test, both single and variable span methods produced higher shear modulus of 790MPa (COV=12%) and 750MPa (COV=10.7%), respectively. They related the higher shear modulus values from bending tests to interacting of shear modulus in different directions, different loading situation and influence of shear warping. They recommend the torsion test approach for to determine the shear modulus as it is simple and cost efficient.

2.5 Past Proposed Test Methods to Evaluate Shear Properties

2.5.1 Past Investigations on Shear Blocks and Flexural Tests

Shear block test method has been investigated since 1950 to assess its applicability to obtain the shear strength of wood. This is because the shear block test method does not account the influence of anisotropy of wood and influence of wood defects. Therefore, the test procedure underestimates the heterogeneous nature of wood. Radcliffe and Suddarth (1955) proposed to test short timber beams to attain the shear strength. In their work, shear strength values obtained from their proposed short beam test and from ASTM shear block tests were compared. In their proposed test method, a simply supported 360 mm long notched beam, as shown in Figure 2-11, was tested by applying load at middle. They found that proposed beam test produced about 20 to 40% higher shear strength of different species than shear block tests. They noticed that most beam specimens were fractured within web and the stress distribution was uniform along length. They recommended short notched beams can be used for obtaining the shear strength of wood.

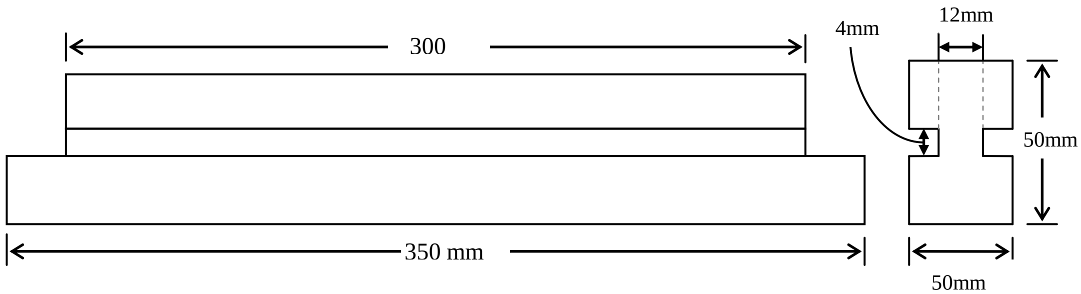


Figure 2-11 A test sample tested by Radcliff and Siddhartha (1955)

Meadows (1956) tested solid and laminated small I-shaped beams to obtain the shear strength of wood. The specimens were fabricated with assumptions that full width at top and bottom contributes the bending strength and that reduced width at neutral plane contributed the shear stresses only. 11 specimens of 460mm long Southern pine beams were fabricated by gluing two 50mm radial grain direction sections together and 11 were fabricated by gluing tangential grain direction wood sections. Figure 2-12 provides the schematic diagram of I shape beams tested by Meadows (1956). Meadows (1956) also tested shear blocks that were obtained from tested I shape beams. He discovered that specimens with radial grain direction produce 25% higher average shear strength (12.8 MPa) than specimens with tangential grain direction (9.7 MPa). However, he noticed that shear block testing does not account for the influence of grain direction as it produced about the same shear strength values when shear blocks tested in both directions. He suggested that the shear block test method is not an appropriate method to determine the shear strength as it does not account for the influence of grain directions. He recommended the I-shaped test method for shear strength as it accounts for the influence of radial and tangential direction.

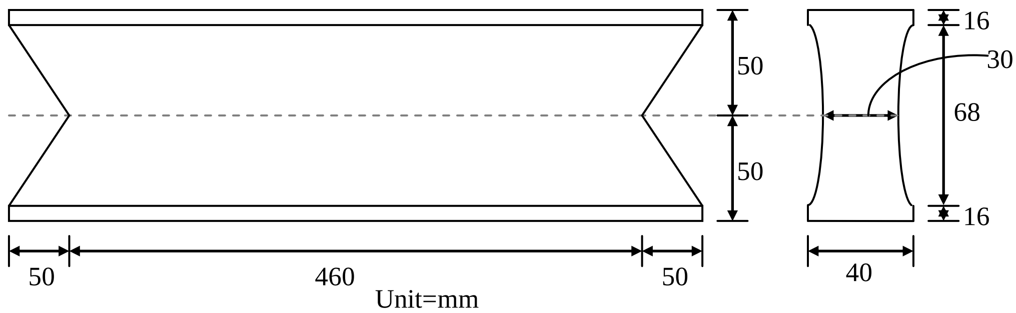


Figure 2-12 Short I shaped beams tested for determining of shear strength by Meadows (1956)

Norris (1957) introduced a panel shear test method to obtain the shear strength. In the test method, a cross-shaped wood panel was tested by inducing load in the direction of wood block edges in the way that central square part of the specimen was subjected to pure shear stresses. He assumed that with the setup the applied load will create only uniform shear stresses and that shear strains will be uniformly distributed throughout the volume of the specimen. Wood panel and shear blocks of Douglas-fir, Yellow-poplar and Yellow parch and were tested. He found that the shear strength obtained from both test procedures had good agreement and that shear strength was not much different from both test methods.

Mandery (1969) found that compressive stresses perpendicular to the grain and the shear stress parallel to grain were linearly related and that an increase in compressive stresses caused increase in shear stresses. Based on testing 12×12×140mm kiln dried Douglas-fir as a beam by forcing shear failure at adjacent the supports and shear block tests, he concluded that shear stresses are mainly depends on compressive stresses perpendicular to grain and that shear block test can be used for the shear strength.

Later, Keenen (1974) found that evaluation of shear strength not only depends upon the induced compressive stresses but also on the shear span and the shear area. In his investigation, he examined the influence of shear span (a, distance between applied load and nearer support) to depth (d) ratio on shear strength. For this, 68, 25×25×100mm specimens with radial planes of obliquity and 16 of tangential obliquity were tested under compression parallel to grain. In addition to this, 33 ASTM shear blocks with tangential and radial obliquity were tested by applying various levels of transverse compressive stresses to investigate the influence of compressive stresses on shear strength. In addition to this, Keenan also used finite element approach to investigate the shear and transverse compressive stress distribution in relate to a shear span to depth ratio.

Keenen noted from shear blocks tests that tangential plane shear strength was much less sensitive to compressive stresses than is the radial plane. This shows that shear block test approach may not be appropriate as it does not account the influence of grain directions. This was also observed by Meadows (1956). Keenen also found that decreasing the shear span to depth ratio from 6 to 1 had no substantial effect on transverse compressive stresses and on shear strength as the both stresses were not distributed in which they have an overall effect on the beam. He found that shear span and shear strength were not correlated with each other. However, he found that both the shear strength and shear area (beam width times shear span) were correlated very well. Foschi and Barrett (1975) carried out finite element analysis and found that shear strength mainly depends on the shear area of the member.

However, Longworth (1977) found that shear strength mainly depends not only on shear area but also on the volume of the member. In his investigation, four groups of strength grade C24 Douglas-fir, as shown in Table 2-4, were tested under four-point bending tests. The beam length was equal to six times the beam depth plus 130mm. Shear blocks taken out from the tested beams were also tested. He observed that shear strength was affected by beam shear area and volume under shear. He found a decrease of up to 45% in shear strength when the shear area was increased from 690 to 7240mm^2 and the volume under shear from 6170 to 412620mm^3, as given in Table

2-4. He also reported that the shear block test method is not suitable to obtain the shear strength as in his result, the shear block test predicted lower for the A and B group and substantially higher values for C, D, and E. Based on his investigation, he proposed the following equations for determining the shear strength based on shear area and volume under shear:

$$\tau_{max} = 2351 - 704 \log A_S \tag{2-31}$$

$$\tau_{max} = \frac{3280}{\sqrt[6]{V_S}} \tag{2-32}$$

Table 2-4 Details of test specimens and the relative shear strength values(Longworth, 1977)

Test series	Specimen No.	Width (mm)	Depth (mm)	Length (mm)	Shear span (mm)	Sheared area $\times 10^3$ $(mm)^2$	Volume under shear $\times 10^6$ $(mm)^3$	Mean shear strength (MPa)
A	30	76	115	800	230	18	7	5.8
B	30	76	265	1700	530	40	342	7.1
C	30	220	265	1700	530	115	991	5.6
D	30	76	725	4500	1450	110	2480	4.7
E	30	130	725	4500	1450	188	4240	4.9

Soltis and Rammer (1994a and 1994b) used five-point bending test to evaluate shear strength of timber beams and examine the influence of shear area and shear volume on the shear strength. They used the five point test arrangement because they assumed it produces high shear forces between the load point and at middle support which allows specimens to fracture in shear. 130×150mm to 130×760mm glued laminated beams were used in their research work. The length under high shear stresses (2.5d) times the width of the beam was considered as the shear area and shear volume was taken as the shear area times beam depth. They found that shear area affected the shear strength as increase in the shear area results in decrease of shear strength. The shear blocks, obtained from tested specimens, were also used and they produced 10 to 15% higher shear strength to the beams. They proposed an analytical approach (Equation (2-33)) to obtain the shear strength based on ratio of beam shear strength to ASTM shear block strength and correlation of shear strength with shear area and shear volume as follows:

$$\tau_{SR} = \frac{1.3 C_f \tau_{ASTM}}{(A_s)^{\frac{1}{5}}} \qquad (2\text{-}33)$$

Where

τ_{SR}= beam shear strength (Soltis & Rammer, 1994a)

C_f = 2, the stress concentration factor to adjust th ASTM shear block

τ_{ASTM}= shear strength values obtained by shear block test method

A_s = sheared area

Rammer et al. (1996) further extended their investigation and used four point and five point bending tests to examine the influence of shear area, volume and beam depth on the shear strength. The other purpose of their study was to verify if Equation (2-33) is valid for solid sawn material. 160 Douglas-fir beams were tested under five point bending and 120 samples of the same species were tested for four point bending. Cross sections specimens were ranging from 40×90mm to 100×350mm were used. In addition to this, 150 ASTM shear blocks were used to obtain the shear strength.

They found that in five point bending about 99 of 160 specimens were failed due to shear but in four point bending, most specimens were failed either in tension or compression parallel to the grain. They observed that the shear strength of smaller cross section beams obtained from both bending tests was 10 to 25% higher than the shear strength obtained from shear blocks. However, increase in cross sections caused decrease in shear strength and was 15% to 40% lower of larger cross sections members than of the shear blocks. They concluded that shear strength of timber beam is not a constant value but varies according to beam size and that increase in beam size causes decrease in the shear strength. Furthermore the decrease was higher if the shear area is small. They proposed that shear strength can be calculated incorporating ASTM shear block strength and shear area as given in Equation (2-33).

Cofer et al. (1997) conducted study to evaluate the validity of three and five point bending tests for shear strength using finite element approach. The shear strength was predicted for timber beam varying from 20×40mm to 40×120mm of length of 7 times of the depths. Finite elements models consisted of eight-node, bi-quadratic plane stress elements for both three and five point bending tests. Material properties of Southern pine were taken from Wood Handbook (USDA, 1999) and shear strength values and were compared with experimental test results conducted earlier. They found that five point bending test always produced higher numerical shear strength from 5% to 10% to those obtained from three point bending configuration. However, they did not found any effects of beam size on shear strength when achieved from FE modelling found that was observed in experimental work.

Yoshihara and Furushima (2003) undertook three-point bending and asymmetric four-point loading method to obtain the shear strength of short 18 ×18 × (75 to 300mm) solid beams. They found that shear strength decreased with an increase in span to depth ratio for asymmetric four-point and three-point tests. They observed short length of (75 to 125mm) beams fracture due to horizontal shear along the neutral axis when tested in asymmetric four-point bending. However the larger beams fractured due to simple tension in both bending tests. They concluded that both bending test approaches are not applicable to determine the shear strength wood beams.

2.5.2 Past Research on E:G Ratio of 16:1

Bodig and Goodman (1973) used plate bending and plate twisting method to evaluate shear modulus and modulus of elasticity of wood in longitudinal, tangential and in radial directions. Eighteen different softwood species from lighter to heavier density were used in their work. The test specimens were fabricated in such a way that small wood pieces of 8×82×106mm were glued to fabricate 8×165×320mm long specimens having six different orientations. In the plate test methods, point loads were applied at various locations and deflections at different locations were measured. The loads were applied in such a way that the plate was subjected to a constant bending moment at centre sections and that the plate had zero bending moments at the ends due to rotation of supports. The modulus of elasticity and shear modulus in LR, RL, LT, TL, TR and RT were calculated using beam deflection at centre and shear distortion (Biblis, (1965) and Gunnerson et al., 1973). They were first to document the modulus of elasticity and shear modulus values of most wood species and the resulting data became the source of the E:G ratio of 16:1 commonly used in design equations for wooden beams.

Chui (1991) used a free-free ends vibration test method to evaluate the shear modulus and the influence of knots on shear modulus. 40×40×320mm knotty and clear white spruce specimens were tested. The test specimens were suspended in air and free-free vibration mode was induced by exciting with instrument hammer along the length and responses were measured by accelerometers. The first and second natural frequencies were obtained and shear modulus and modulus of elasticity were obtained. He found that knots did not have any influence on shear modulus but rather 45% of knotty specimens had 5 to 8% higher shear modulus than clear specimens. Although knots had substantial influence on modulus of elasticity as presence of knots reduces it to three times. This may results in that knots may enhance the shear modulus though it mainly depends upon the growth and characteristic of knots.

Chui (1991) also examined the correlation between modulus of elasticity and shear modulus and found that both properties were independent of each other. He also found that for both knotty and clear wood the E:G ratio was not constant but varies from 8:1 to 47:1 with average of 20:1. He proposed to use free-free vibration as a non destructive method for shear modulus and modulus of elasticity as the method accounts the influence of knots.

Gorlacker and Kurth (1994) also used a vibration test method to attain the shear modulus. They also noticed that E:G ratio is not a constant value but it varies with the increase in modulus of elasticity. In their work, 30×150×150mm glued laminated short beams were tested by inducing longitudinal vibration by tapping light-weight hammer at one cross-sectional end and measuring vibration time using GrindoSonic equipment. On basis of testing of 1200 specimens, they found that increase in modulus of elasticity increases the E:G ratio. They recommended that E:G ratio of 16:1 is very conservative approach and cannot be used for wood design.

Divos et al. (1998) also recommended the torsional vibration technique to evaluate the shear modulus of timber. They found that the torsional vibration method based on testing of 55×110×(1300 to 2100)mm spruce battens using variable span method (ASTM-D198-94, 1996), free-free vibration method (Chui, 1991) and torsional vibration method (Gorlacker & Kurth, 1994). They found that shear modulus obtained from torsion and from variable method correlate very well and torsional vibration can be used with installed lumber in structure. Also, Burdzik and Nkwera (2003), Hindman et al. (2001), Hindman et al. (2005a) and Harrison (2006) have shown that E:G ratio of 16:1 approach is not applicable to predict shear modulus and shear rigidity of timber as for most species the ratio is not a constant values and varies from10% to 60%.

2.6 Summary

In this chapter a detailed discussion has been provided about the torsion theories, recommended test methods to evaluate the shear properties and past research works related to obtaining shear properties. Also, the objective of this research is to understand the use of torsion test method for timber joists. Therefore, more attention was given to the past research works that were related to the use of torsion test approach. Most investigations were attempted primarily to compare the torsion test to flexural and shear block tests to examine the applicability of torsion test for evaluating the shear properties of timber. Less attention has been paid to how torsion test method can be more informative in relate to determine the shear properties.

Riyanto and Gupta (1998), Gupta et al. (2002a, 200b) and Gupta and Siller, (2005a, 2005b) were those of a few researchers who used torsion test to evaluate the shear strength. Although in their work tests were conducted accordance to (ASTM-D198-94, 1996), at the strain rate that speciemens were tested was not clearly expressed, especially of composite lumber. Also, they reported that shear cracks occured along the middle of the longside of joists and ran and ended at supporting clamps but there was a lack of clear demonstration on whether the cracks began from the middle or test clamps initiated the cracks. This is important because test clamps may induce additional compressive stresses into test specimens. Also, little attention has been paid to any other failure mode apart from shear failure as deviation in grain direction, knots and other defects may results in other failure modes.

Hindman et al. (2005a, 2005b) used torsion test to obtain shear rigidity but his investigation was limited to twisting of joists within very small rotational displacements. These small rotaional displacements may not represent the actual shear rigidity as they do not create adequate distortion so that structural timber will resist it. Harrison (2006) used torsion test to evaluate shear modulus of the actual joist and did not measure the shear modulus of various along the length of joists.

This was demonstrated by Brandner et al. (2007) for glued laminated timber, yet no study has been attemped to investigate if there is a variation in shear modulus along the length of joists. In torsion tests, torque can be induced in either clock-wise or an anti clock-wise direction and that it may be possible that the spiral grain direction of grain may have a substantial influence on shear modulus. Yet, no any research has been conducted to examine if torque inducing in different directions have cause any influence on shear modulus.

The above issues have not been addressed and there is a need for an in-depth investigation of the torsion test method that clearly demonstrate the above issues and provide information the use of torsion test for evaluation of shear properties of timber.

3. EVALUATION OF TORSIONAL SHEAR MODULUS OF TIMBER

3.1 Introduction

This Chapter presents the experimental investigation that was conducted to determine the shear modulus of joists using torsion test. A comprehensive details of torsional experimental setup is also discussed which could be useful for any future investigations on torsion. Sitka spruce and Norwegian spruce structural joists of different lengths were used. The shear modulus was obtained from applied torque within elastic range and relative twist of specimens. The specimens were also tested under cyclic torsional loading by inducing torque in clockwise and in anti-clockwise directions. This allowed observing any difference in shear modulus due to different toque directions. The shear modulus and density of both species were also compared to examine correlation. Further details have been provided in the following sections.

3.2 Evaluation of Torsional Shear Modulus

3.2.1 Test Set-up and Equipment

3.2.1.1 Torsion Tester

A torsion tester (Tinius Olsen, Pennsylvania USA) was used to test specimens. The machine is capable of inducing up to 1 kN-m torque and can rotate specimens as many revolutions as required. The test data, in the form of measured torque and rotation can be obtained from the machine using Test Navigator software with measurement accuracy of ± 0.5% of the applied torque and ± 0.05° of the rotation. The machine consists of two parts, a driving house that includes of a motor and a gear box, applies the torque, and a detached unit contains torque sensor and works as a reaction unit. The detached or reaction unit is adjustable to change its position to accommodate testing of specimens of different lengths, as shown in Figure 3-1 and Figure 3-2.

Figure 3-1 The driving house and the reaction bench of the torsion tester.

3.2.1.2 Testing Bench

A 5.6 m long test bench was designed and constructed to support the torsion tester. The bench allowed the reaction unit to be adjusted for testing joist lengths from 0.5m to 4.5m. The flexibility of this design enabled testing to achieve the shear modulus and shear strength of timber joists of different lengths. Figure 3-2 illustrates the possible adjustments of reaction unit for testing purpose. The bench, the machine and the specimens was a self balance system. The torque, generated by the motor, was applied to specimens and balanced by the extension bar and was then transmitted back to the motor through a main beam (rectangular hollow section RHS 100×200×5 along the span at middle of the bench).

3.2.1.3 Testing Clamp

The chucks of the machine were limited to testing small specimens, up to 40 mm diameter or 30×30 mm cross section, therefore, a pair of testing clamp was fabricated for testing structural size joists. The angles were adjustable part of the clamps and can facilitate testing specimens of thickness from 5mm to 60mm and of depth from 10mm to 120mm. The clamp restrains specimen in perpendicular direction by contacting surfaces and allows the occurrence of the unrestrained warping in other two directions which leads avoid additional warping stresses along the length of the specimens. The clamps were fabricated to allow easy installation of the specimens and yet grip the specimen ends with large contact area as to avoid excessive embedment

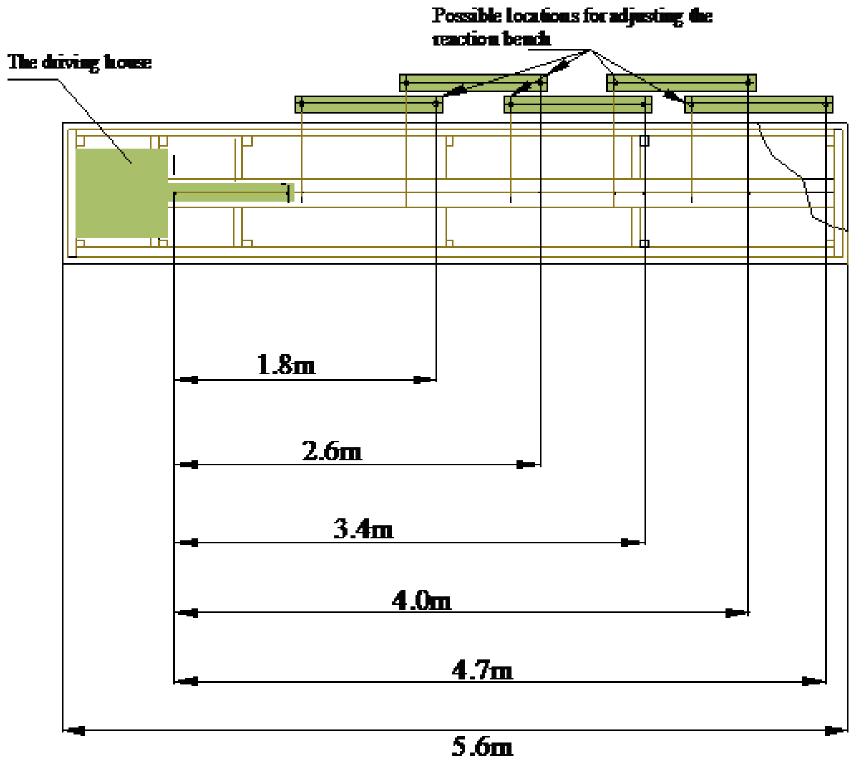


Figure 3-2 Possible locations for the reaction unit for different lengths

3.2.1.4 Inclinometers

The rotations that were achieved from the machine may include small slippage between the dowel and the teeth of chucks, embedment of test specimens in clamps, twisting of the clamp and gearing system of motor. To eliminate these factors and to measure the actual rotation of the specimen, dual axis inclinometers (Model IS-2-30, Level Developments, UK) of measuring range of ±30° with accuracy of ± 0.05° were used. The inclinometers can be mounted on the surface of specimens by means of screws and the relative twist in direction of applied torque (Y direction in this research) and twist in perpendicular direction of applied toque (in X direction) can be measured. The twists of joist can be recorded using inclinometer software developed by the authors. The data achieved from Tinius Software and Inclinometer software can be recorded synchronically.

3.2.2 Test Material

Sitka spruce (*Picea sitchensis*) and Norway spruce (*Picea abies*) joists of nominal cross section of 45 × 100 mm were tested. Sitka spruce (SP) joists were obtained from Strategic Integrated Research in Timber (SIRT) research project, conducted at Edinburgh Napier University. In SIRT, Sitka spruce logs were obtained from five different locations of Baronscourt Estate in County Tyrone, Northern Ireland. In each location, the trees were grown with different spacing for research study of effect of spacing between trees on the mechanical properties. All logs were cut into three parts of bottom, middle and top or crown with each part was approximately 3.7m long and were machine graded to C16. Samples were considered to be good representative for C16 population with clear trace of biological history. For this research, test joists were chosen from middle part only with assumptions that mechanical properties tends to be more consistent in compare to the bottom or crown part where the mechanical properties could be highly varying.

Specimens of four different lengths (1.0m, 2.0m, 2.8m and 3.6m) of 15, 10, 12 and 25 replicates were used, respectively. Table 3-1 provides the further details of the test material that was used in this research. It is noted that the 1.0 and 2.0m samples were obtained from tested joists for modulus of rupture. A care was taken such that 1.0 and 2.0m specimens were cut from farthest side of destruction section. It should be noted that 1.0m samples were obtained from a different locations other than the Baronscourt location.

Seven, 45×100×4800mm Norwegian spruce (NS) joists of grade C16 and C24 were obtained from a local commercial timber industry. Each joist was then cut in to two pieces of length of 2.4m. The main purpose of testing C24 was to obtain a higher value of the shear modulus (G) and the modulus of elasticity (E) to cover a wider spectrum correlation (will be explained in Chapter 07). Prior to tests, specimens of SP and NS were conditioned in the testing laboratory, a self controlled-environment room (21°C and 65% relative humidity), until they attained approximately 12% moisture content (a constant mass).

Table 3-1 Details of the test material used in this study

Test Specimen	Species	Strength grade	Length	No. Of replicates
Full-size structural joists	Sitka spruce	C16	1.0m	15
			2.0m	10
			2.8m	12
			3.6m	25
	Norway spruce	C16	2.4m	14
		C24	2.4m	12

3.2.3 Test Procedure

Each test specimen was tested by mounting in torsion tester and applying torque, using displacement control approach. The measurements of twist from the torsion tester were not used for data analysis and only used to control the application of the torque as they included other components of twist in addition to the twist of the specimen itself. To measure the actual rotational displacement, two inclinometers were mounted on the topside (45mm) of specimen at distance of twice of depth of joist (2d) from either clamp end, as shown in Figure 3-3.

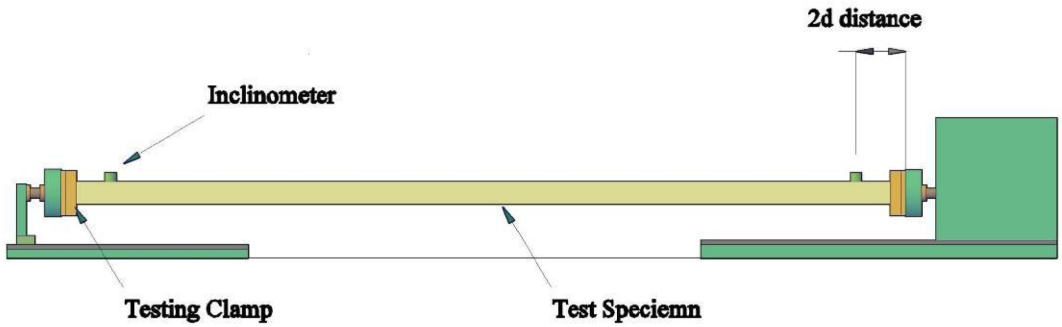


Figure 3-3 Schematic diagram for test setup of 2.8m joist

This provides span of 0.8m, 1.6m, 2.4m and 2.2m for 1.0m, 2.0m, 2.8m and for NS specimens, respectively. For 3.6m, inclinometers were connected on topside at 3d distance which allowed 3m span for tests of 3.6m joists. The 2d distance was employed to minimize the accounting of additional stresses that might create due to clamps. Gupta et al. (2005a) showed that 2d is adequate distance to minimize the end effect. All the test specimens tested using displacement control at speed rate of 4°/minute as accordance of ASTM (ASTM-D198-94, 1996) until specimen fractured by inducing torque in anti-clockwise direction. This should be noted that before conducting above tests, all samples were tested under elastic loading for influence of in clockwise and ant-clockwise torque and to attain the shear modulus of various sections along the length of specimens (Chapter 04).

3.2.4 Results and Discussion

The shear modulus of each tested specimen was obtained by using Saint-Venant torsion theory of rectangular section as follows:

$$Shear\ modulus\ =\ \frac{Stiffness\ \times L}{(d\ t^3\ k_1)} \qquad (3\text{-}1)$$

In the Equation (3-1), L is the length of the sample, d is the depth (major cross-section dimension) and t is the thickness (minor cross-section dimension) of the test specimen and k_1 is a constant value, depends on the depth thickness ratio (see e.g. Boresi & Schmidt, 2003). The stiffness in Equation (3-1) can be determined by conducting linear regression analysis of the applied torque (T) in N-m and the relative twist (θ) in degrees within the elastic region as shown in Figure 3-4.

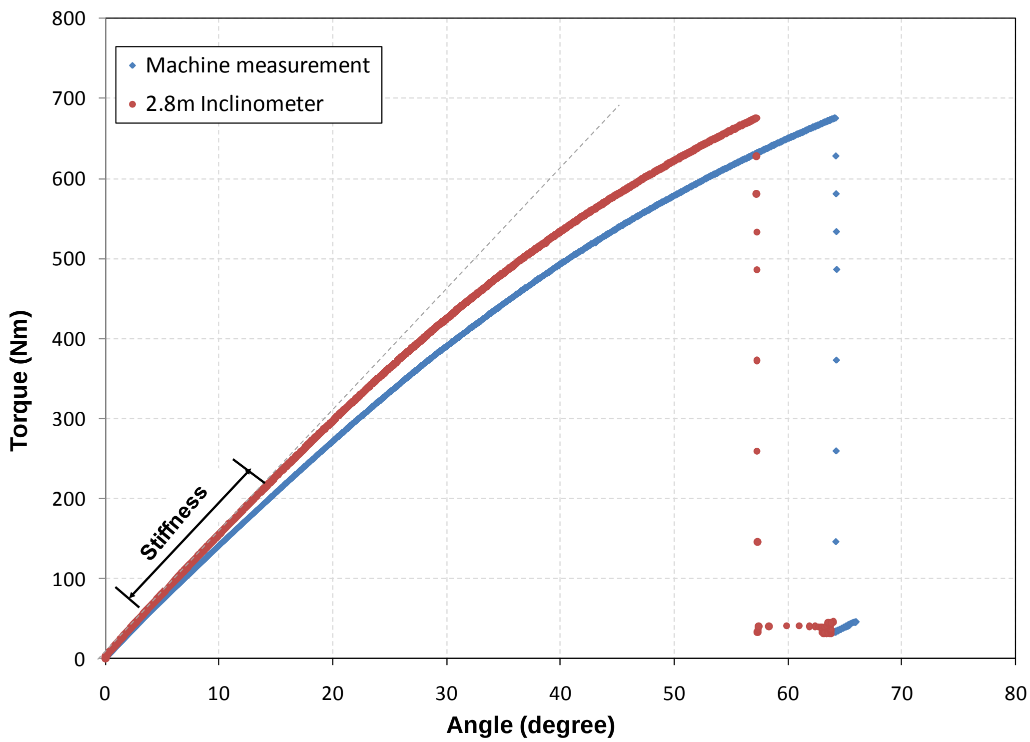


Figure 3-4 A typical torque-twist response and the tangent stiffness within elastic range for 2m specimens.

In this research the elastic region was between 3% and 30% of maximum applied torque for most of the tested joists. Therefore, linear regression analysis was conducted between 5% and 25% of maximum applied torque to obtain the stiffness. The maximum applied torque is defined as the ultimate applied torque at which test joists were either fractured or reached at their maximum strain hardness point. The calculated the mean, the maximum and the minimum shear modulus of all tested group are presented in Table 3-2.

Table 3-2 The average shear modulus values for all tested lengths.

Group	Grade	Length (mm)	Specimen	Shear modulus (MPa)			Density kg/m^3
				Mean	Maximum	Minimum	
SP	C16	1.0	15	490±95	750	300	
		2.0	10	500±40	560	430	380±25
		2.8	12	530±75	630	410	385±40
		3.6	25	560±70	715	430	390±45
		Average*		520±70	660	390	385±35
NS	C16	2.4	14	610±60	760	515	430±45
	C24	2.4	12	760±145	1100	600	465±70

± represents standard deviation
* represents the average of the mean of each length

Table 3-2 reveals that 1.0m specimens have the lowest mean shear modulus of 490 MPa of 19% coefficient of variation (CoV). For 2.0m, 2.8m and 3.6m specimens, the mean shear modulus values increases slightly from 500MPa to 560MPa, about 3% to 6% increment. This implies that longer specimens may have slightly higher shear modulus values than shorter specimens. This is because longer specimens have a greater probability of large knots than shorter specimens but each large knot takes up a smaller proportion of the total length. This allows more consistent shear flow in longer specimens than shorter specimens. Also, as it was expected, the shear modulus of C24 (NS) has the highest mean shear modulus of 760MPa, about 20% and 30% higher than NS C16 and SP respectively. Also, NS specimens of C16 have 15% higher mean shear modulus than of SP joists. This is because that it may be possible that different species has different shear modulus values and such that Norwegian spruce has higher shear modulus values in compare to the Sitka spruce species.

In addition to different species, the joists that were obtained from SIRT contained large number of knots in compare to NS specimens. Therefore, knots may have some influence on shear modulus and this could be the other reason of a lower shear modulus of SP joists. In this regard, influence of knots on shear modulus was taken into account and is detailed in Chapter 04. In addition to knots, wood defects such checks, slope of grain and small bark may have substantial influence on the shear modulus. However, this research was mainly conducted to evaluate the shear properties of timber joist and influence of other defects was not accounted. Therefore it becomes important to conduct future investigation to examine the influence of defects on the shear modulus. A good correlation between shear modulus and density was achieved. As shown in Figure 3-5, it can be seen that R^2 values of 0.56 (NS) and slightly lower of 0.30 (SP) were obtained based on linear regression analysis. It was also found that Sitka spruce joists have slightly lower density values (385 kg/m^3) than of (10% and 17% less) NSC16 and NSC24, as shown Table 3-2.

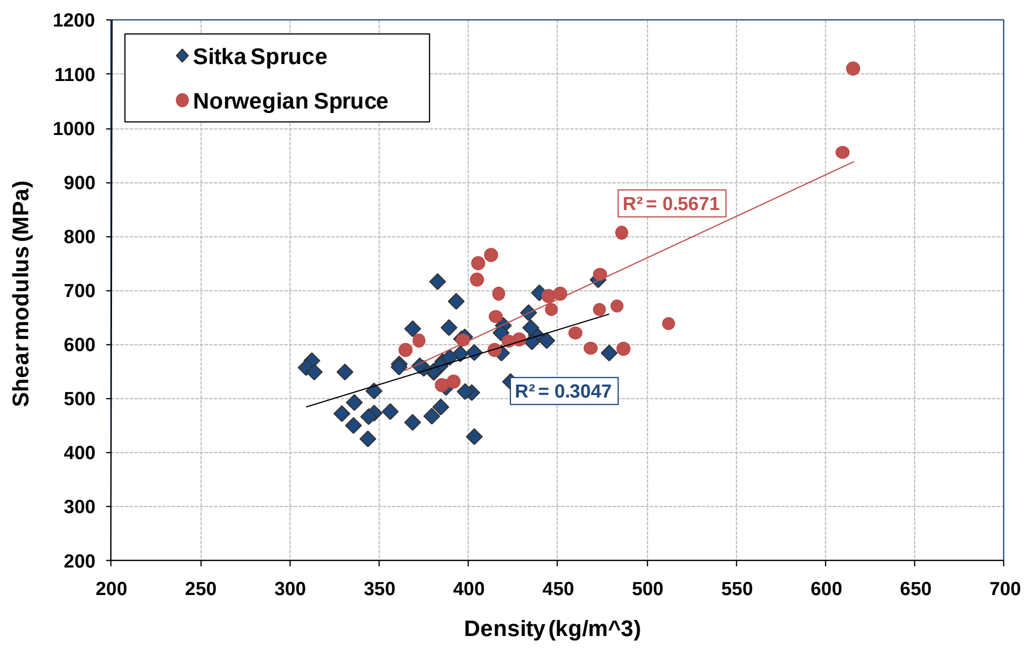


Figure 3-5 The correlation of shear modulus and relative density of Sitka spruceand Norwegian spruce joists.

CEN (EN338:2008, 2008) provides the design shear modulus values of 500 and 690 MPa for structural grade C16 and C24, respectively. The values were calculated on the basis of modulus of elasticity to shear modulus (E:G) ratio of 16:1 and modulus of elasticity was obtained from four-point bending tests. In this research, the mean C16 (including both SP and NS species) shear modulus of 560MPa was attained, about 13% higher than CEN published design values. For C24, torsion tests produced 10% higher mean shear modulus (760MPa) than design values in CEN. Although it should be noted that only two species were tested in this research, the difference in shear modulus suggests that estimation of shear modulus obtained from modulus of elasticity may underestimates the actual shear modulus property of timber.

This suggests that shear modulus of timber joist must be determined from appropriate approach such as torsion test. The recent draft CEN (EN408:2009, 2009) included the torsion test to obtain the shear modulus of timber. In this study it was found out that actual shear modulus can be obtained from torsion tests and, therefore, it is concluded that torsion provides an effective way to obtain shear modulus.

3.3 Influence of Cyclic Loading

3.3.1 Test Procedure

The main intention of conducting cyclic loading tests was to observe whether inducing torque in clockwise or in anticlockwise direction influences the shear modulus. It was presumed that by applying torque in clockwise and anti-clockwise direction may lead to find the influence of spiral direction grain on the shear modulus. For this purpose, 1.0m (14 replicates) and 2.0m (10 replicates) Sitka spruce joists were tested. For this, each sample was loaded within its elastic range by applying a clockwise torque, and then unloaded at the same strain rate before being loaded at a constant rate of 4°/min (ASTM-D198-94, 1996). Then the specimen was loaded in anticlockwise direction with the same speed rate and was then unloaded.

Two cycles of clockwise and anti-clockwise torque was applied on each specimen and rotational displacements were measured by inclinometers, mounted at 2d distance from end clamps. To locate the elastic range, preliminary tests were performed to locate the upper limit of the region of elastic behaviour for each sample length. This was achieved by testing five samples of each length until they fractured or began to exhibit non-linear behaviour under torsion. Based on these tests, it was determined that 1.0m and 2.0m samples could be twisted by up to 10°/m before they began to yield. Therefore a maximum displacement of 8°/m was used in all subsequent tests on 1.0m and 2.0m samples.

3.3.2 Result and Discussion

Figure 3-6 represents a typical diagram of a cyclic loading pattern for 1.0m joist. Table 3-3 and Table 3-4 provide average shear modulus of obtained for all four directions for 1.0m and 2.0m joists, respectively. By observing Table 3-2 and Table 3-4, it is very clear that twisting joists in either clockwise or anti-clockwise direction do not have substantial influence on the shear modulus. This is because the grains of tested joists may have a uniform direction and there was no presence of grains with spiral direction. It was also observed that loading and unloading in one direction does cause only a little influence on the shear modulus. The same observation has been made from Table 3-3 as there was a small difference was found in shear modulus by twisting 2.0m joists in either direction.

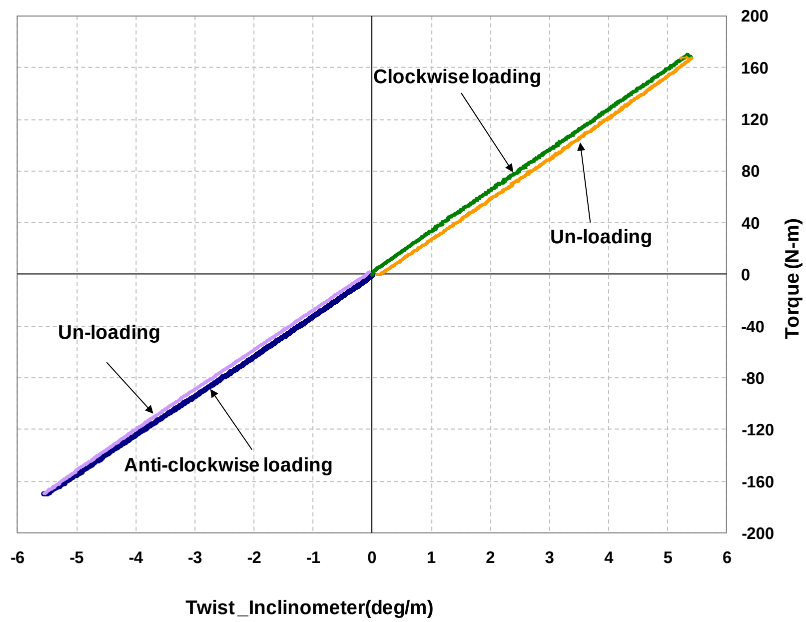


Figure 3-6 A typical clockwise and anti-clockwise torque-twist relationship of 1.0m joists.

Table 3-3 Shear modulus of 1.0m joists in clockwise and in anti-clockwise direction

Specimen	Average shear modulus (MPa)			
	Clockwise direction		Anti-clockwise direction	
	Loading	Un-loading	Loading	Un-loading
1	420	420	415	430
2	435	435	420	435
3	310	315	310	315
4	445	455	450	455
5	535	540	525	535
6	540	540	545	545
7	440	445	430	445

8	490	480	485	490
9	725	705	740	705
10	565	565	560	570
11	515	530	520	535
12	400	400	390	395
13	540	540	540	540
14	470	490	465	470

Table 3-4 Shear modulus of 2.0m joists in clockwise and in anti-clockwise direction

Specimen	Average Shear modulus (MPa)			
	Clockwise direction		Anti-clockwise direction	
	Loading	Un-loading	Loading	Un-loading
1	555	560	540	545
2	435	460	440	445
3	470	490	480	480
4	420	440	425	430
5	515	515	500	520
6	530	545	525	540
7	455	465	445	460
8	505	520	500	515
9	485	490	470	485
10	555	575	570	560

It can be observed that inducing torque in different direction did not cause any substantial variation in shear modulus values. From both Tables, the highest difference was found in specimens 09 and 04 of 1.0 m joists and was about 4% and 6% when loaded and unloaded in anticlockwise direction, other than that the difference was about 1%. Also, it was found that shear modulus have a very strong correlation as R^2 was found about 0.98, as shown in Figure 3-7. This may indicate that shear modulus is less sensitive when obtained in either direction and there was no substantial influence on shear modulus while inducing torque in either direction.

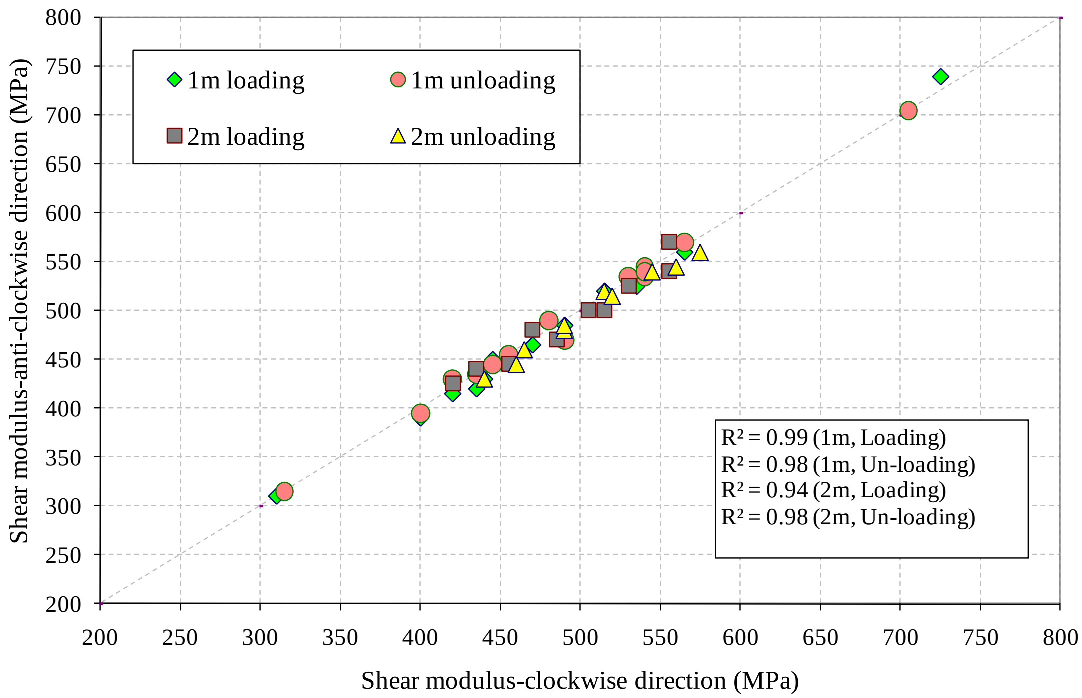


Figure 3-7 A correlation of shear modulus obtained from loading and unloading in clockwise and anti-clockwise direction.

Also, a closer look shows that shear attained from unloading the specimen is higher (2% to 4%) in all the tested specimens than shear modulus attained when specimens were loaded. This can be seen in specimens 01, 03, 04, 05, 07 and 11 of 1.0m and 01, 04, 06, 07, 08 and 09 specimens of 2.0m. This may be because when joists were in loading, wood fibres were compressed due to applied torsional load and the void spaces between fibres were minimized. When joists were unloaded, the packed wood fibres might provide a little more resistance to the load and, therefore, a trend of higher shear modulus was found while unloading.

The main purpose of conducting above test was to achieve any change in shear modulus by twisting joists in clockwise or anti-clockwise direction. This was also allowed to observe the influence of possible presence of spiral direction of grain on shear modulus. However, there was no significant difference found in shear modulus which presumably suggests that test joists may not contain any spiral direction grains. Therefore, this leads that shear modulus most likely will be same in either direction if the joists are tested within elastic torque.

3.4 Summary

In this chapter emphasis was placed on evaluating the shear modulus of timber joists using torsion test method. The torsion test procedure was discussed in detail and the procedure could be adopted for future research on torsion test. In the test procedure, two species of timber, Sitka spruce of C16 and Norway spruce of C16 and C24, of lengths from 1m to 3.6m were tested. A torsion tester was used to induce torque and inclinometers were used to measure the relative twists of the test joists. The shear modulus was obtained on the basis of applied torque within elastic zone and twist per length. A comparison have been conducted between the test values and published design values and it was found that test values based on torsion tests were higher than the published values that were obtained from modulus of elasticity. This suggests that torsion test approach must be adopted in test standards to evaluate the shear modulus of timber. The results from this research also support the recent draft of CEN (EN408:2009, 2009) which included the torsion test method to obtain the shear modulus of timber.

It has been found in this research that the elastic zone of applied torque lies between 5% to 25% of ultimate applied torque. It has been found out that inducing torque in both clockwise or in anti-clockwise direction does not have any influence on shear modulus. This may lead that either the test specimens did not have grains in spiral direction or it may be possible that spiral grains have little influence on shear modulus. However, a further investigation is required to observe influence of spiral grains on shear modulus by testing spiral grain timber.

4. VARIATION IN SHEAR MODULUS AND KNOT INFLUENCE

4.1 Introduction

The one of the main objectives of this research was to examine the behaviour of shear modulus along the length of timber joist and is presented in this chapter. To attain the objective, Sitka spruce and Norway spruce joists were tested under torsion loads within elastic range and the shear modulus of various segments along the length were obtained. In particular, this will allow examining any variation in shear modulus along the joist length. This also assists in determining the correlation between shear modulus and modulus of elasticity of the various segments (will be discussed in Chapter 07).

In this chapter, presence of knots was taken into account and influence of knots on shear modulus was investigated. This was conducted by comparing the total knot area ratio within a segment of the joists and the shear modulus of the same segment. Also, repeated elastic loading tests were conducted to assess the validation of the testing procedure and to determine if loading history has any effect on the shear modulus. The following sections are describing the behaviour of shear modulus along the length of joists, the influence of knots and the effects of repetitive loading.

4.2 Variation in Shear Modulus

4.2.1 Test Set-up and Material

The torsion test setup, described in Chapter 03, was used. The same C16 Sitka spruce (SP) and C16 and C24 Norway spruce joists, detailed in Chapter 03, were tested. For SP, 10, 12 and 25 replicates of 2.0m, 2.8m and 3.6m lengths were used, respectively. For 2.4m long NS C16 and C24 14 and 12 specimens were tested, respectively.

4.2.2 Test Procedure

Since the test procedure involves in testing specimens within their elastic behaviour. Therefore, it became very important to locate the upper limit of elastic region for each length. For this purpose, preliminary tests were conducted and five samples of each length were tested until they fractured or began to exhibit plastic behaviour. Based on these tests, it was found that 2.0m samples could be twisted by up to 10°/m before they began to yield. Therefore, a maximum displacement of 8°/m was used in all subsequent tests on 2.0m samples. Subsequently, 2.8, 3.6 and NS 2.4m joists were tested with maximum displacement of 5.5°/m, 4.5°/m and 5°/m, respectively.

Four consecutive tests were conducted on each 2.0m joist and shear modulus of four 400mm segments along the length was obtained. Since only two inclinometers were available, therefore, first inclinometers were mounted at 200mm (2d) distance from loading end clamp to test the first segment (S1). The test was conducted by inducing torque in anti-clockwise direction at speed rate of at 4°/min and then was un-loaded at the same speed rate. In all subsequent tests the torque was induced in anti-clockwise because it was found that application of torque in any direction does not have influence on shear modulus. Multiple tests were conducted on other three segments (S2, S3, and S4) accordingly. Figure 4-1 illustrates the test arrangement for the 2.0m joists.

It becomes essential to examine the applicability segment arrangement of joist to evaluate the shear modulus along the length. For this, additional tests were conducted on each 2.0m joist such that twists were measured of sections by extending the distance between mounted inclinometers, shown in Figure 4-2. In these additional tests named as overlapping tests, first shear modulus of middle section (Region 1 (R1)) of 400mm was obtained. Then, subsequent tests were conducted for 800mm (R2), 1200mm (R3), and 1600mm (R4) by extending the distance between inclinometers.

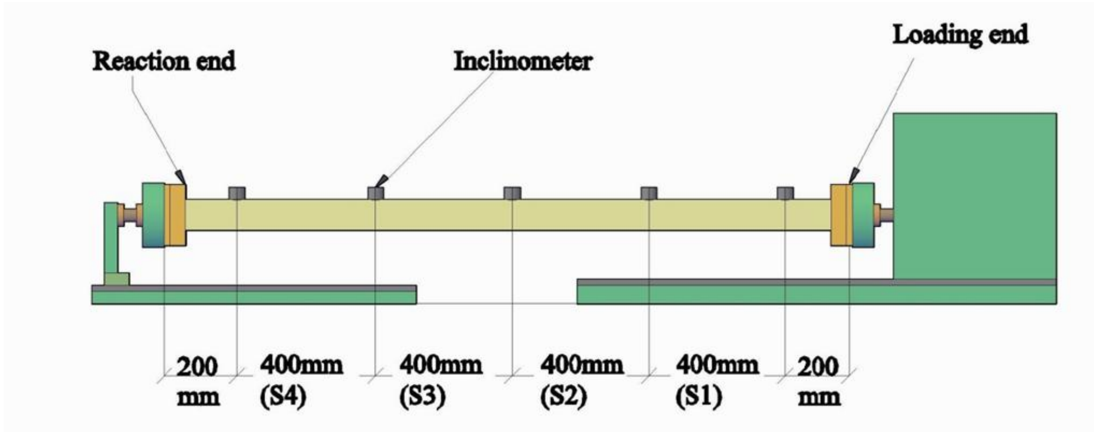


Figure 4-1 Test arrangement for 2.0m joist to determine the shear modulus of various segments.

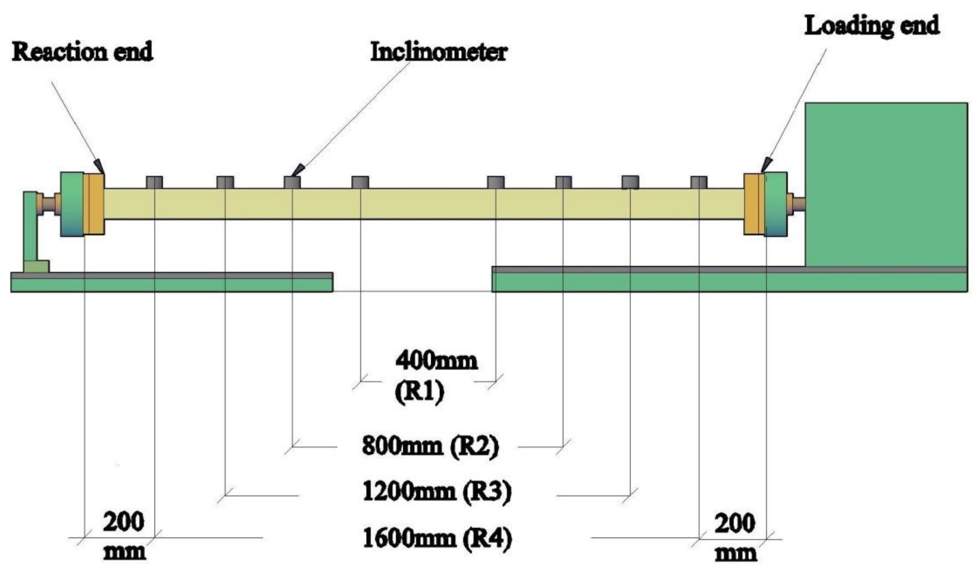


Figure 4-2 Testsetup (2) for 2m joists for attainting the G of overlapping sections.

Two elastic test sequences were conducted on 2.8m joists with the same arrangement (as shown Figure 4-1) by partitioning into four 600mm segments with 200mm end distances. At first, joists were tested within elastic torque and shear modulus of four consecutive segments were obtained and the joists were placed in the testing laboratory for 28 days and retested. The shear modulus of 600mm segments will also assist in developing the correlation of shear modulus and modulus of elasticity as bending tests allows to obtain modulus of elasticity 600mm sections, will be discussed in Chapter 07. Two more inclinometers were acquired for 3.6m joist tests. Three elastic test series (28 days gap par test) were conducted on 3.6m joists by partitioning into five 600mm segments with 300mm end distance. For 3.6m joists, four inclinometers facilitated to obtain the shear modulus of first three segments in first and the shear modulus of other two segments from the second test par test series.

2.8m and 3.6m joists were obtained from logs that taken from four different plots, named as Plot-A to Plot-D. In each plot, trees were grown with different spacing such that in Plot-A, trees were grown with the widest spacing and was considered to have the lowest mechanical properties, Accordingly, for Plot-D logs trees were grown with the narrowest spacing and was assumed to have the highest stiffness. More details are provided in Lyon et al. (2007). Each log was about 12 to 13m long, therefore, tests joists were obtained from the middle part of the logs. This is because it was assumed that stiffness will not be differing within the middle part of log in compare to top end or bottom end. This should be noted that although top end sections generally have greater stiffness properties but stiffness varies suddenly. 2.8m joists were tested by marking segment 04 near the bottom end and S3, S2 and S1 were marked subsequently. S1 was assigned near the top end for 3.6m joists and other segments were labelled respectively, as shown in Figure 4-3.

NS C16 and C24 joists were also partitioned into four segments in such a way that the segments next to the loading end reaction ends were 500mm long and the two middle segments were 600 mm long which allows a 100mm end distance. Only one elastic test was conducted on each NS joist.

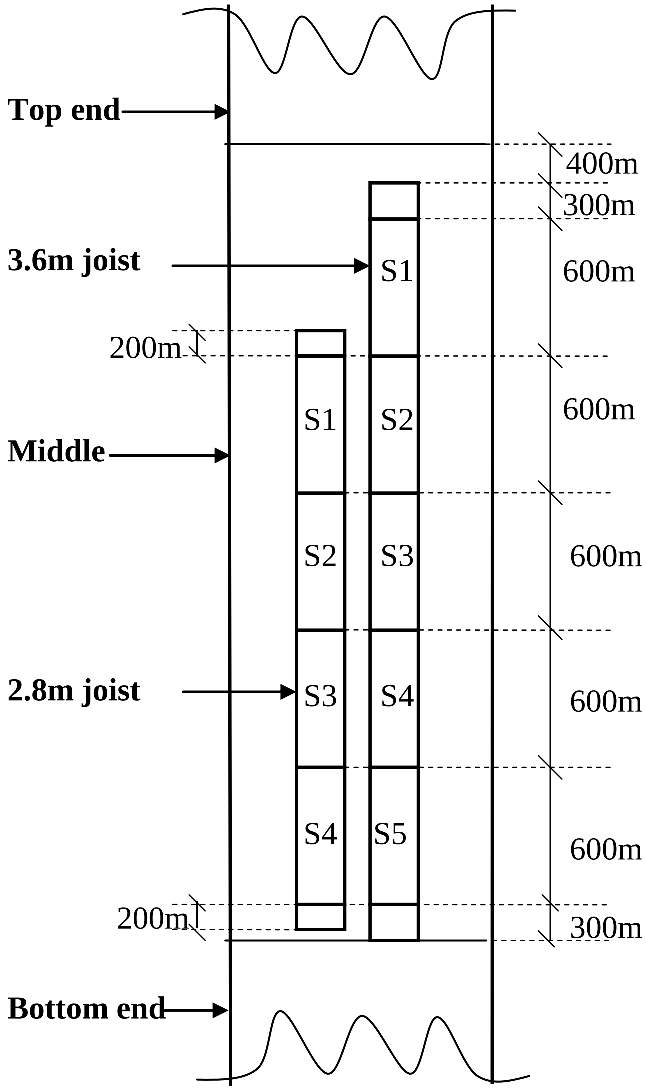


Figure 4-3 Schematic diagram for segment locations of 2.8m and 3.6m joists within the logs.

4.3 Result and Discussions

4.3.1 Variation in 2.0m Joists

Table 4-1 presents shear modulus values of each 2.0m joist. Figure 4-4 provides the percentile variation of shear modulus within the length for 2.0m joists. The shear modulus was obtained using Equation 3-1(Chapter 03). It is apparent from Table 4-1 and Figure 4-4 that shear modulus varies substantially along the length and the maximum variation was found in joist 03. In Joist 03, segment 02 (S2) has 10% higher shear modulus (545MPa) and segment 03 (S3) has 20% lower (390 MPa) shear modulus values in compare to mean shear modulus (G_{Avg}) of 490 MPa. The G_{Avg} represents the average shear modulus of all four segments of a joist and that $G_{Avg\text{-}seg}$ represents the average shear modulus of each segment of all tested joists. The same variation was also observed in joist 06 in that shear modulus of S1 was 16% lower and for S2 was 12% higher in comparison to G_{Avg}. Furthermore, S2 of joist 05 has about 12% lower shear modulus than the other three segments.

Table 4-1 The shear modulus values of four segments of 2.0m joists.

Joist No.	Shear modulus (MPa)				
	S1	S2	S3	S4	G_{Avg}
01	580	550	550	550	560
02	480	480	470	435	465
03	500	545	390	530	490
04	400	440	435	440	430
05	530	530	450	530	510
06	450	600	530	540	530
07	460	450	470	430	450
08	490	550	510	500	510
09	450	490	540	460	485
10	530	580	570	570	560
$G_{Avg\text{-}seg}$	490	520	490	500	500

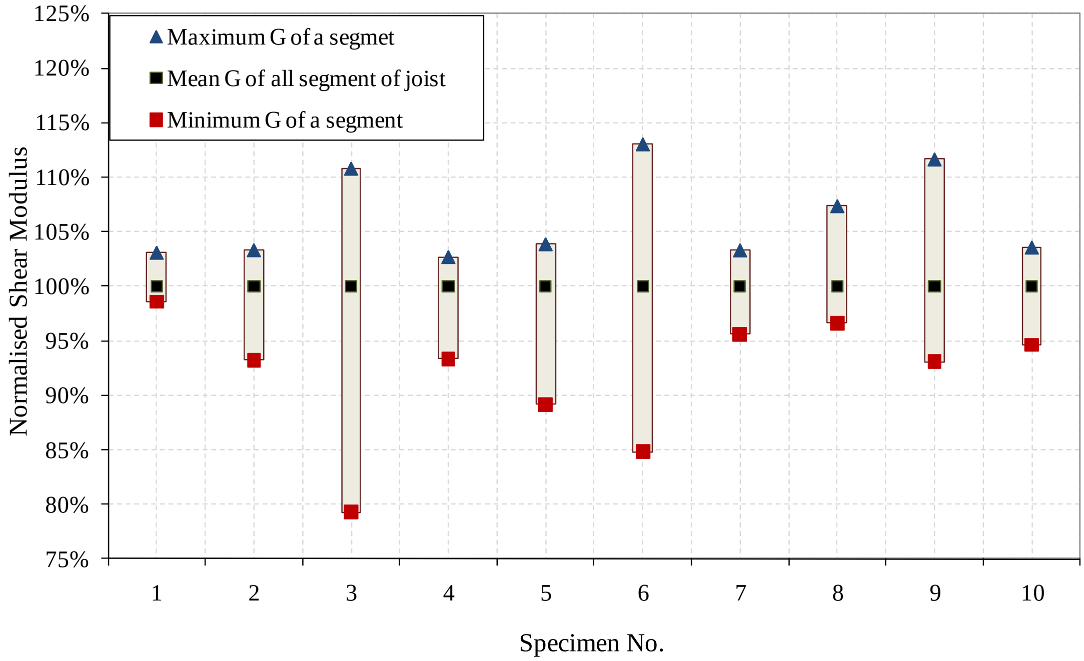


Figure 4-4 The percentile variation in shear modulus along the length of 2.0m joists.

The joists were then visually examined for any wood defects, especially of knots, in particular the segments with lower shear modulus values. It was found in joist 03 that S3 (20% lower Shear modulus) contained 20mm topside knot and S2 with higher shear modulus consisted of clear wood. Also for joist 06, S1 (16% lower shear modulus) contained a 60mm central knot but S2 of joist 09 (12% lower shear modulus) did not contain any knots. This may indicate that knots may have some influence on shear modulus and therefore, it becomes necessary to investigate the influence of knots on shear modulus.

As mentioned earlier that specimens were tested with overlapping test arrangement to examine the applicability of segment test arrangement. In this regard, Table 4-1 provides the measured shear modulus values of the four regions of each joist. It should be noted that shear modulus of R2 is equal to the average shear modulus of S2 and S3 and R4 is equal to average of all four segments (S1+S2+S3+S4). A little difference was found between R2 and (S2+S3) and between R4 and (S1+S2+S3+S4).

This can be examined by comparing shear modulus values of joists 01, 04 and 07. For joists 01, 04 and 07 the average values (S2+S3) of 550, 440 and 460 MPa, respectively were obtained. For R2 of the same joist shear modulus values of 570, 450 and 490 MPa were attained.

Also, the shear modulus of 470 MPa for R2 of joist 03 was achieved and was the same for the average shear modulus of S2 and S3. Although the shear modulus of S2 was 10% higher and 20% lower of S3 was attained of the same joists. Also for joist 06, the G_{Avg} (covers the 15% lower G in S1 and 10% higher G in S2) and shear modulus R4 were about same 540 MPa obtained. The comparison suggests that testing joists in segments is an appropriate approach to obtain shear modulus along the length of joist. This also can be observed from Figure 4-5 that shows that shear modulus obtained from both tests have quite good relationship (R^2=0.91).

Table 4-2 Test results for test setup 02 for 2.0m joists

	Shear modulus (MPa)				
Joist No.	R1	R2	R3	R4	G_{Avg}
1	565	565	565	540	560
2	520	500	525	460	500
3	520	470	465	475	480
4	435	450	435	435	440
5	490	505	510	520	505
6	530	575	555	540	550
7	490	490	460	460	475
8	530	515	510	515	515
9	460	510	485	485	485
10	540	595	565	565	565
$G_{Avg-reg}$	510	520	510	500	510

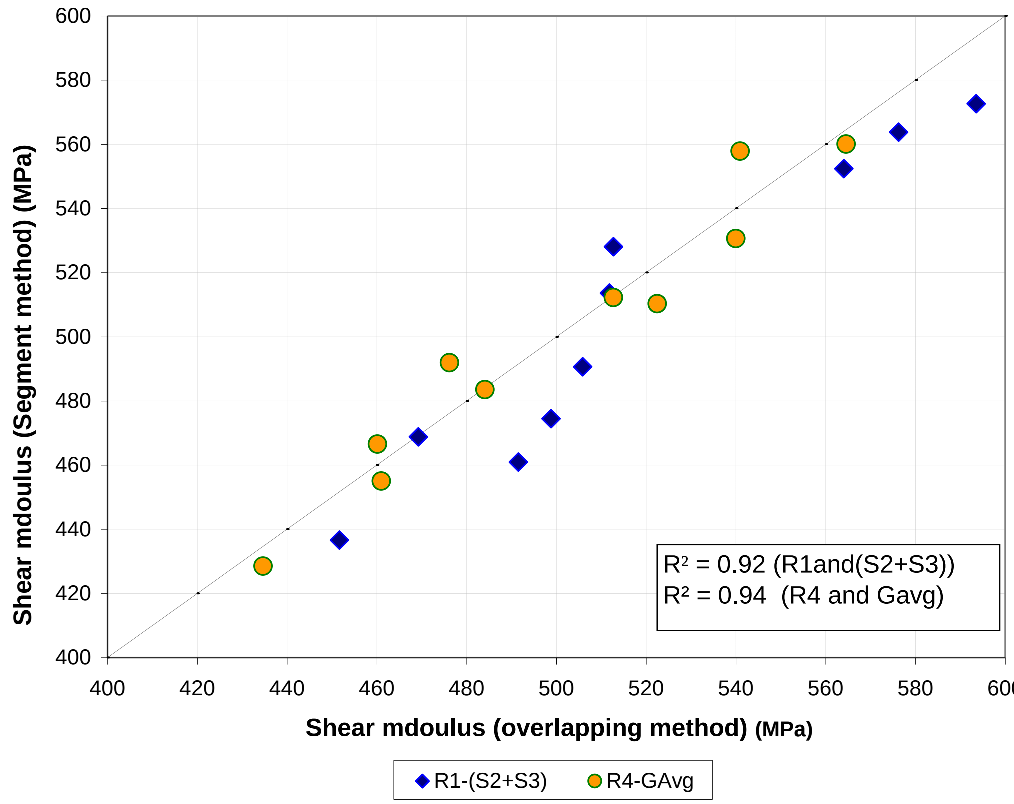


Figure 4-5 Comparison of shear modulus obtained from test setups for 2.0m joists.

4.3.2 Variation of Shear Modulus in 2.8m Joists

Table 4-3 details the average shear modulus of each test specimen of 2.8m obtained from two elastic tests. Two elastic tests were performed in such that first all specimens were tested elastically then retested after 28 days. In Table 4-3, G_{Avg} represents the average shear modulus of all four segments of each joist and $G_{Avg\text{-}sgt}$ defines the average shear modulus of each segment of each plot. $G_{2.8m\text{-}joists}$ symbolized the average shear modulus of each segment of all tested joists. Figure 4-6 shows the details of percentile variation in shear modulus within each segment of joist in relative to G_{Avg} of that joist.

Table 4-3 The shear modulus of tested 2.8m joists catagorized according plots

Plot ID	Joist No.	Shear modulus (MPa) Segment S1	S2	S3	S4	G_{Avg}
Plot-A	01	450	440	400	410	425
	02	575	620	660	590	610
	03	625	560	540	560	570
	$G_{Avg\text{-}sgt}$	550	540	530	520	535
Plot-B	04	430	440	455	470	450
	05	480	460	485	460	470
	06	585	590	720	565	615
	$G_{Avg\text{-}sgt}$	495	495	555	500	510
Plot-C	07	550	610	580	590	580
	08	610	600	655	665	630
	09	635	650	615	640	635
	$G_{Avg\text{-}sgt}$	600	620	620	630	615
Plot-D	10	550	585	590	615	585
	11	620	650	600	595	615
	12	625	550	575	520	570
	$G_{Avg\text{-}sgt}$	600	595	590	575	590
	$G_{2.8m\text{-}joist}$	560	560	570	560	560

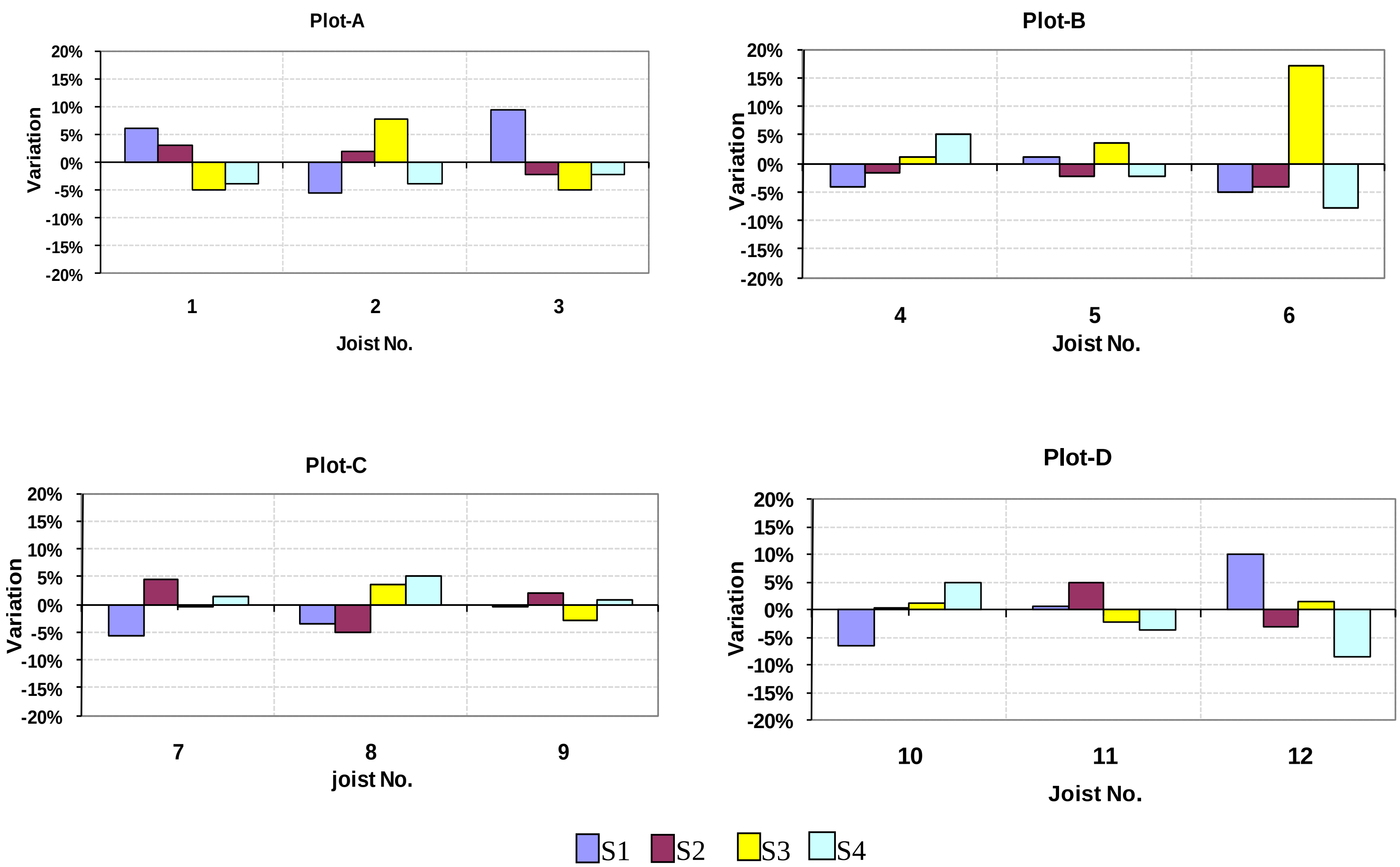


Figure 4-6 The graphical representation of percentile variation for each plot of 2.8m joists

From Table 4-3, it can be observed that the average shear modulus does not vary significantly when considered for all 2.8m and for each plot joist. However, substantial variation was found in shear modulus when each test joist was examined individually, as shown in Figure 4-6. The highest variation was observed in joist 06 as the shear modulus of segment 3 (S3) was found to be 17% higher (720 MPa) than G_{Avg} (615 MPa) of the same joist. Trend of higher stiffness was also observed in joists 02, 03 and 12 as shear modulus of S1 was noticed to be 8% to 12% higher than G_{Avg} of these joists. It was also found that shear modulus values were significantly lower within the joists. About 9% lower shear modulus (520 MPa) was attained of segment 04 in compare to G_{Avg} when joist 12 was tested. Furthermore, about 4% to 8% lower shear modulus was achieved fro S1 of joist 02, S1 of joist 07 and S2 of joist 08.

This suggests that shear modulus varies considerably along the length when individual joist were taken into account. It was found that in joists 02, 06 and 07 had a significantly higher variation in shear modulus values. Therefore, these joists were visually inspected for presence of wood defects, more specifically for knots. Two 60mm diameter knots were found on S1 (6% lower shear modulus) of joist 02 but S3 (8% higher shear modulus) of the same joist was consisted of clear wood. The same observation was made for joist 06 as S4 contained 30mm diameter centre knot and shear modulus was about 8 to 10% lower than the G_{Avg}. The above observations suggest that presence of knots may impose some influence on shear modulus but are inconclusive. Therefore, further study was be carried out in 3.6m joists regarding the influence of knots on shear modulus.

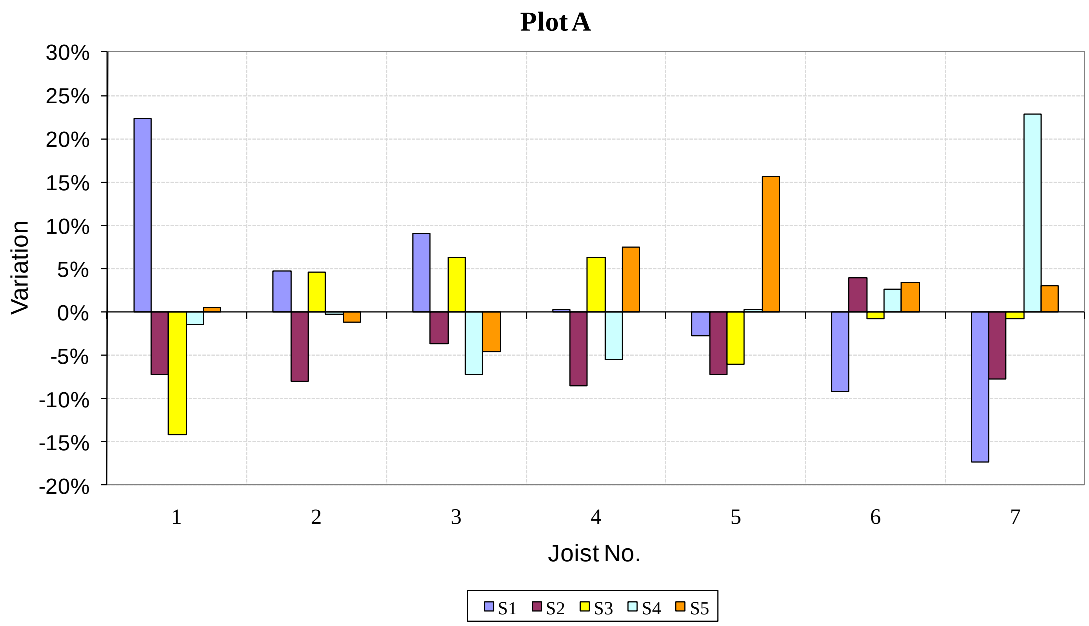
Table 4-3 shows that the mean shear modulus ($G_{2.8m\text{-}jost}$) of all tested joists does not vary significantly as mean shear modulus of 560 MPa was obtained for all tested joists. Also, a little variation in shear modulus was noticed when 2.8m joists were considered according to Plots. Maximum of 8% higher $G_{Avg\text{-}seg}$ was obtained for segment 03 of Plot-B. As described earlier that among the plots, Plot-D joists supposed to have higher mechanical properties due to the narrowest spacing between trees and subsequently Plot-A joists have the lowest mechanical properties. In this research a similar trend of higher shear stiffness was found for Plot-D and Plot-C and that Plot-A and Plot-B joists had lower shear stiffness. It was found that the Plot-B joists have the lowest $G_{Avg\text{-}seg}$ of 510 MPa, about 20% lower than $GA_{vg\text{-}seg}$ of 615 MPa of Plot-C. Plot C joists had the highest $G_{Avg\text{-}seg}$ but not much difference was seen between $G_{Avg\text{-}seg}$ of Plot-C and of Plot-D. This may indicates that spacing between trees has some influence on the mechanical properties. This should be noted that this investigation was more concentrating on the variation in shear modulus and, therefore, it does not encompass the influence of tree spacing on the shear modulus. However, the results from this study may assist the current SIRT research that is being conducted on tree spacing

4.3.3 Variation of Shear Modulus in 3.6m Joists

Table 4-4 gives the shear modulus values of all five segments for each tested 3.6m joist, average shear modulus ($G_{Avg\text{-}sgt}$) of joists of plots and average shear modulus ($G_{3.6m\text{-}joist}$) of all 3.6m joists. Figure 4-7 and Figure 4-8 detail the percentile variation within each joist for each plot. It was observed that shear modulus varied substantially within individual joist and that the variation was significantly higher than found in 2.8m joists. It was observed that more than one segments had lower shear modulus values within single joist. This can be found in joist 07as S1 and S2 have 18% (390 MPa) and 8% (430 MPa) lower shear modulus then G_{Avg} of (470 MPa), respectively. Also, S1 and S4 of joist 14 have 10% and 12% lower shear modulus values than G_{Avg} of the same joist. Moreover, shear modulus of middle segments (S3, S4) of joist 17 and (S2, S3, S4) of joist 02 were about 8 to 14% lower than the G_{Avg}. As well, shear modulus of S2 and S3 of joist 01 have 7% and 15% lower than G_{Avg}. Interestingly, S1 of the same joist have 22% higher shear modulus (670 MPa) than G_{Avg} (550 MPa) of the same joist.

Table 4-4 The shear modulus of various segments along the length of 3.6m joists

		Shear modulus (MPa)					
	Joist			Segment			
Plot ID	No.	S1	S2	S3	S4	S5	G_{Avg}
Plot-A	01	670	510	470	540	550	550
	02	500	440	500	475	470	475
	03	600	530	585	510	525	550
	04	475	430	500	445	510	470
	05	500	475	480	515	595	515
	06	510	585	560	580	580	565
	07	385	430	460	570	480	465
	$G_{Avg\text{-}sgt}$	520	485	510	520	530	515
Plot-B	08	580	530	540	570	555	555
	09	575	560	595	555	600	575
	10	620	600	585	710	645	630
	$G_{Avg\text{-}sgt}$	590	565	575	610	600	590
Plot-C	11	610	635	600	650	530	605
	12	710	535	530	490	520	555
	13	535	585	590	585	625	585
	14	450	590	525	465	570	520
	15	760	690	655	715	765	715
	16	600	560	570	505	565	560
	17	645	715	600	615	715	660
	18	615	595	650	560	720	630
	$G_{Avg\text{-}sgt}$	615	615	590	570	625	610
Plot-D	19	730	555	600	590	560	610
	20	790	760	735	670	645	720
	21	630	570	595	635	680	620
	22	655	545	555	590	605	585
	23	650	755	715	685	670	700
	24	500	500	570	590	580	550
	25	715	660	705	645	675	680
	$G_{Avg\text{-}sgt}$	670	620	645	625	630	640
$G_{3.6m\text{-}joist}$		600	570	580	580	600	590



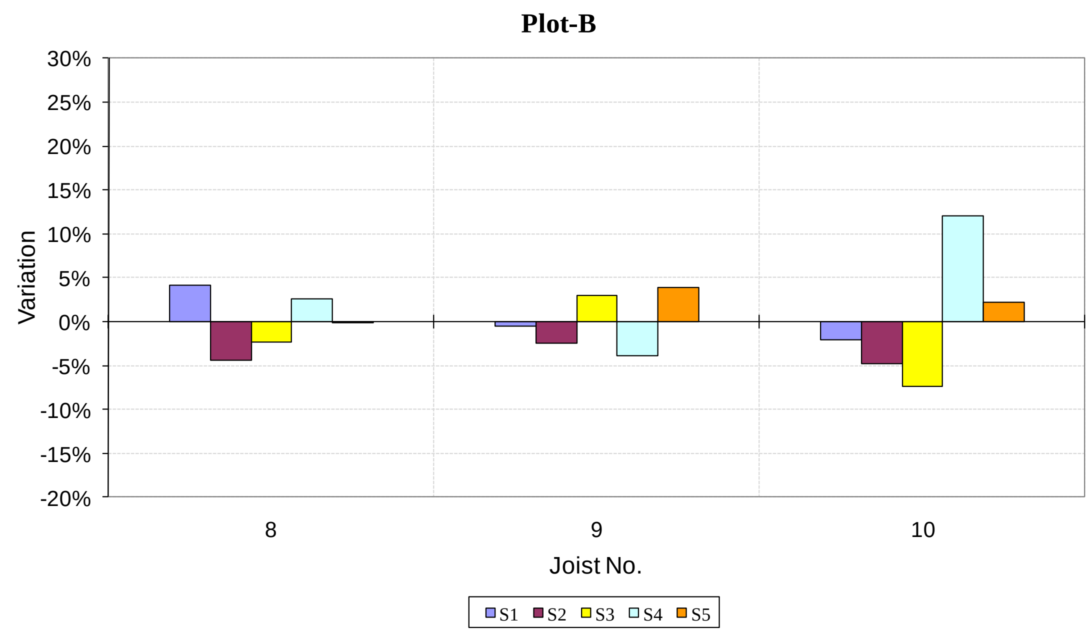
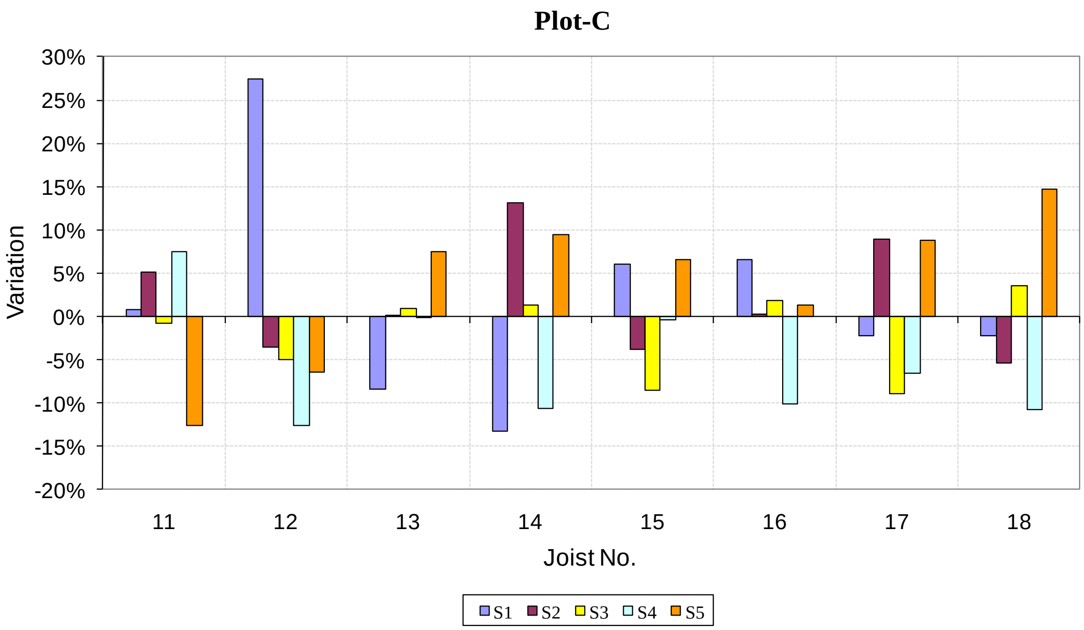


Figure 4-7 The percentile variation for 3.6m joists of Plot-A and Plot-B.



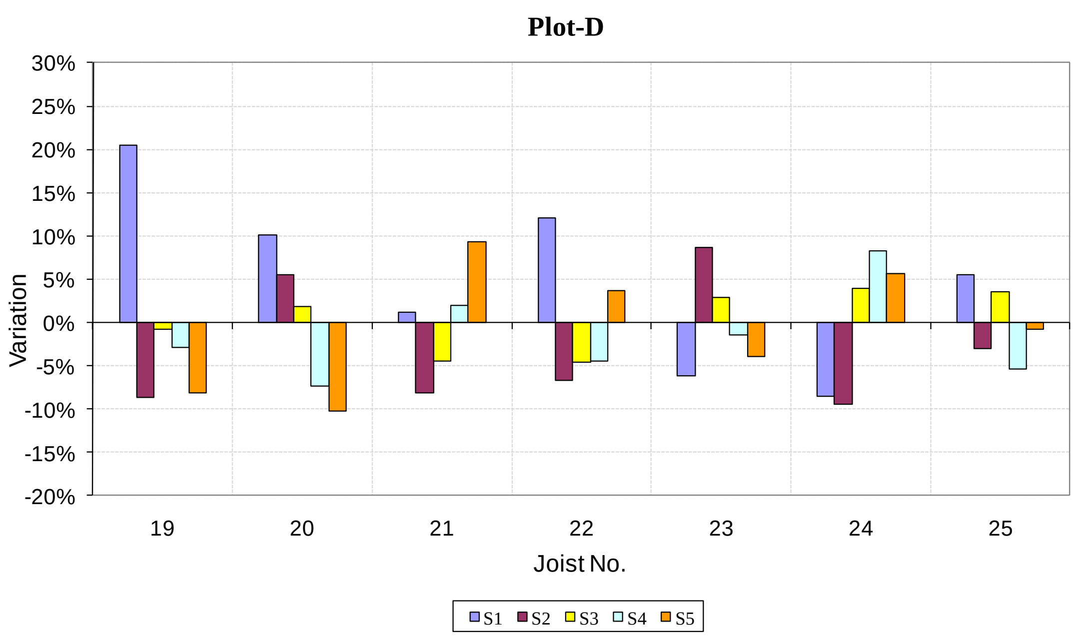


Figure 4-8 The percentile variation for 3.6m joists of Plot-C and Plot-D.

It was noticed that in some joists, segment 01 have relatively higher shear modulus. About 20% to 28% higher shear modulus values of S1 were obtained for joists 01, 12 and 19 joists and about 6% to 12% higher for joists 02, 16, 0, 22 and 25. After torsion tests, S1 of joists 01, 12 and 19 was examined for difference in density. Slightly higher values were obtained when density of S1 was compared to the density of the actual joist. The density of S1 of joist 12 of 470 kg/m^3 was obtained, about 6% higher than averaged density of 445 kg/m^3 of joist 12. The same observation was mead for joist 01 and 19 of that density of S1 was 3% to 5% higher than the density of joists. This, perhaps, suggests that density was not the factor for the higher shear modulus values. This may be indicating that timber nearer to top end relatively higher mechanical properties than the middle or bottom end as S1 was taken from top of middle section, as shown in earlier Figure 4-3.

A very small variation was found in average shear modulus $G_{Avg-seg}$ when considered for plots. About 4% variation was attained for plots A and D and approximately 6% variation was obtained for plots B and C. Not much difference was seen in average shear modulus ($G_{3.6m-joist}$) of all 3.6m joists. The influence of tree spacing on shear modulus was also examined for 3.6m joists. The G_{Avg} of Plot-A was found to be the lowest (515 MPa) and increased to 640 MPA for Plot D, accordingly. This may indicates that for trees growing with different spacing may have some influence on the shear properties.

The presence of knots was also taken into account and it was noticed in most segments of a lower shear modulus contained knots. It was seen that two knots of 30mm and 40mm knots were located in S5 of joist 07 have 18% lower shear modulus than G_{Avg}. A 40mm knot was found in S3 of joist 01 and that segment has 5% lower shear modulus. Interestingly, although S3 of joist have 10% lower shear modulus and that there was no knots were found. This was also seen that in most joists although shear modulus varies but no knots were found. This perhaps suggests it may be possible that variation in shear modulus was independent of the presence of knots. The observations that were made in regard of knot influence is not explicable, therefore, a comprehensive comparison was carried out between shear modulus and knots and will be discussed later.

4.3.4 Variation of Shear Modulus in Norway spruce

Norway spruce C16 (NSC16) and C24 (NSC24) joists were also tested elastically to obtain shear modulus all four segments along their length of 2.4m. In this regard, Table 4-5 and Table 4-6 represent the shear modulus values of all tested joists for NSC16 and NSC24. Figure 4-9 provides the percentile variation within a segment of joist in relate to the average shear modulus of (G_{Avg}) the same joist of NSC16 and NSC24. From Table 4-5, Table 4-6 and , it can be noted that shear modulus varies considerably along the length of commercially graded timber. For C16, it was found that shear modulus was 15% lower (650 MPa) of S1 of joist 01. Also, S4 of joist 11 had about 12% less shear modulus values than the G_{Avg}. The same observation was also made for C24 joist (Table 4-4) as S4 of joists 05 has 15% lower shear modulus and S3 of joist 02 has 640 MPa shear modulus (8% lower) then the G_{Avg} of the joists.

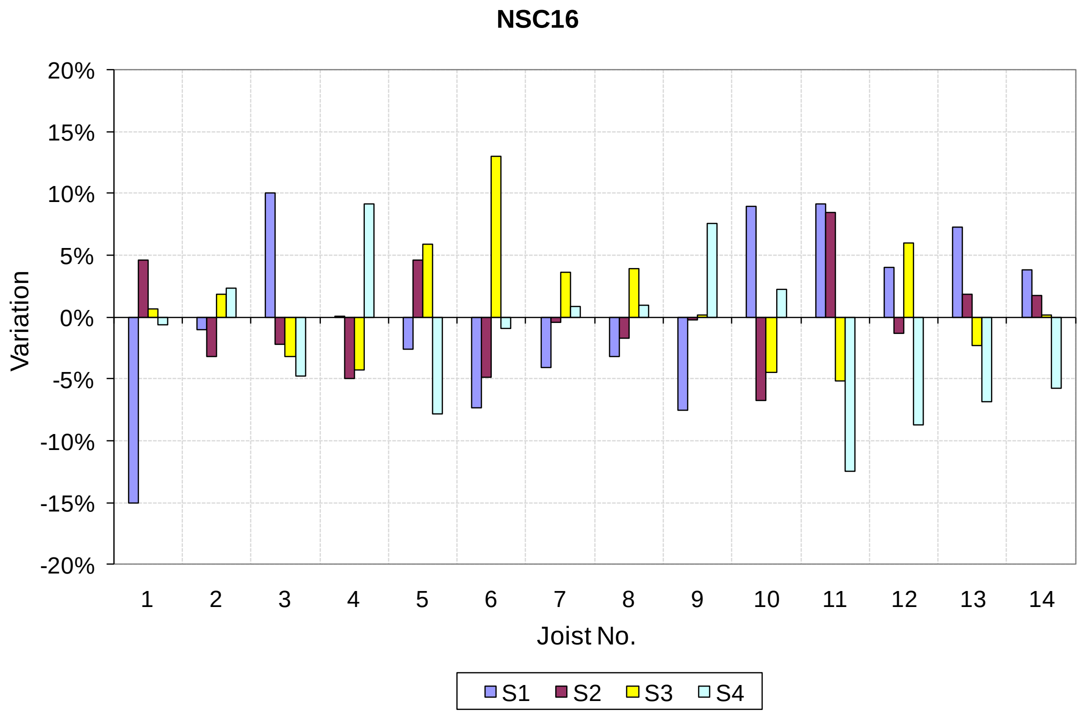
A trend of higher shear stiffness was also found in Norway spruce joists. It was observed in C16 that S1 of joist 3 and S3 of joist 6 have 10% (670MPa) and 13% (685MPa) higher shear modulus than G_{Avg} (610 and 605 MPa) of the same joists, respectively. In addition to this, S1 and S2 of the joist 05 of C24 had 10% higher shear modulus and S5 of the joist 09 had 10% higher shear modulus in compare to $G_{Avg.}$ The same inclination of higher shear modulus (5% to 10%) was also seen in S4 of joists 04, S1 of joist 10 and S2 of joist 11. Although it was found out that the shear modulus varies along the length within joists. This shows that regardless of wood species and strength grade, the shear modulus varies substantially when evaluated using torsion test method. Therefore, this research recommends the torsion test for determining the shear modulus as bending test or E:G ratio of 16:1 may not facilitate to determine the variation in shear modulus.

Table 4-5 The shear modulus Norway spruce C16 joists

	Shear modulus (MPa)				
Joist No.	Segment 1	Segment 2	Segment 3	Segment 4	G_{Avg}
01	650	800	770	760	765
02	600	590	620	625	610
03	670	595	590	580	610
04	590	560	565	645	590
05	575	615	625	545	590
06	560	575	685	600	610
07	610	635	660	645	640
08	650	660	700	680	670
09	485	525	525	565	525
10	580	495	505	540	530
11	725	720	630	580	665
12	615	585	630	540	590
13	635	600	580	550	590
14	645	630	620	585	620
G_{NSC16}	615	620	620	600	615

Table 4-6 The shear modulus of Norway spruce C24 joists

	Shear modulus (MPa)				
Joist No.	Segment 1	Segment 2	Segment 3	Segment 4	G_{Avg}
01	645	695	625	640	650
02	690	720	635	705	690
03	850	800	830	745	805
04	595	610	615	615	610
05	1210	1210	1080	945	1110
06	905	920	1000	1000	955
07	700	690	695	690	695
08	730	700	795	780	750
09	655	680	755	790	720
10	710	700	685	680	695
11	670	655	670	660	665
12	735	740	700	745	730
G_{NSC24}	760	760	760	750	755



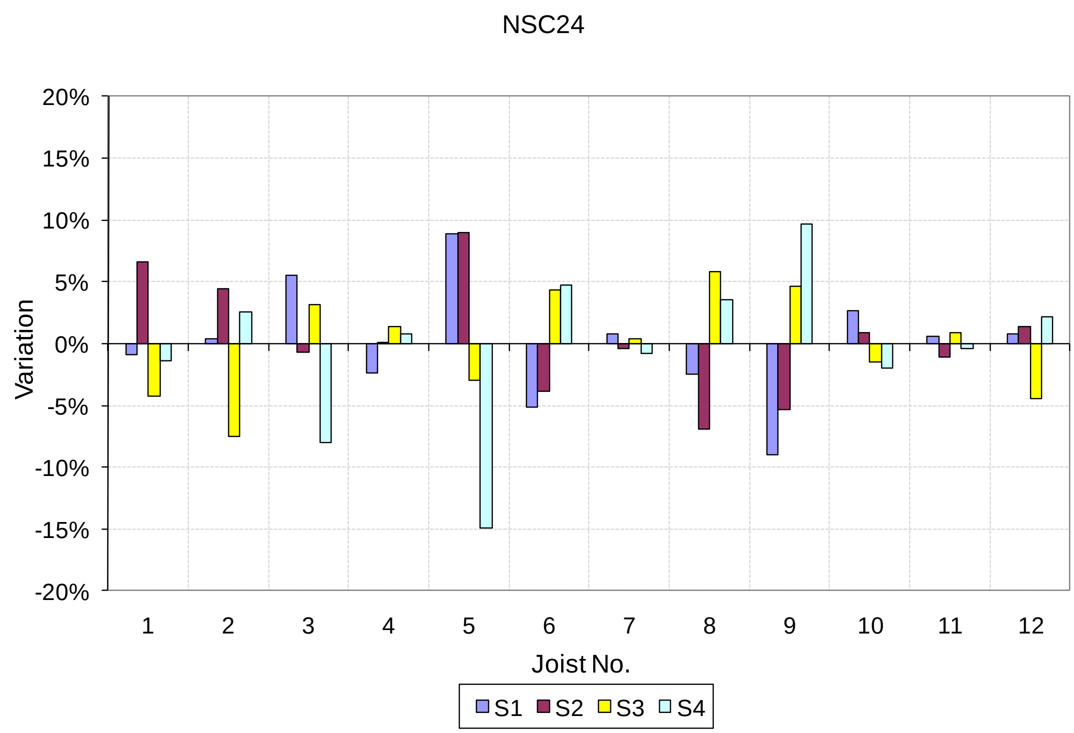


Figure 4-9 The Percentile variation in shear modulus in Norway spruce joists.

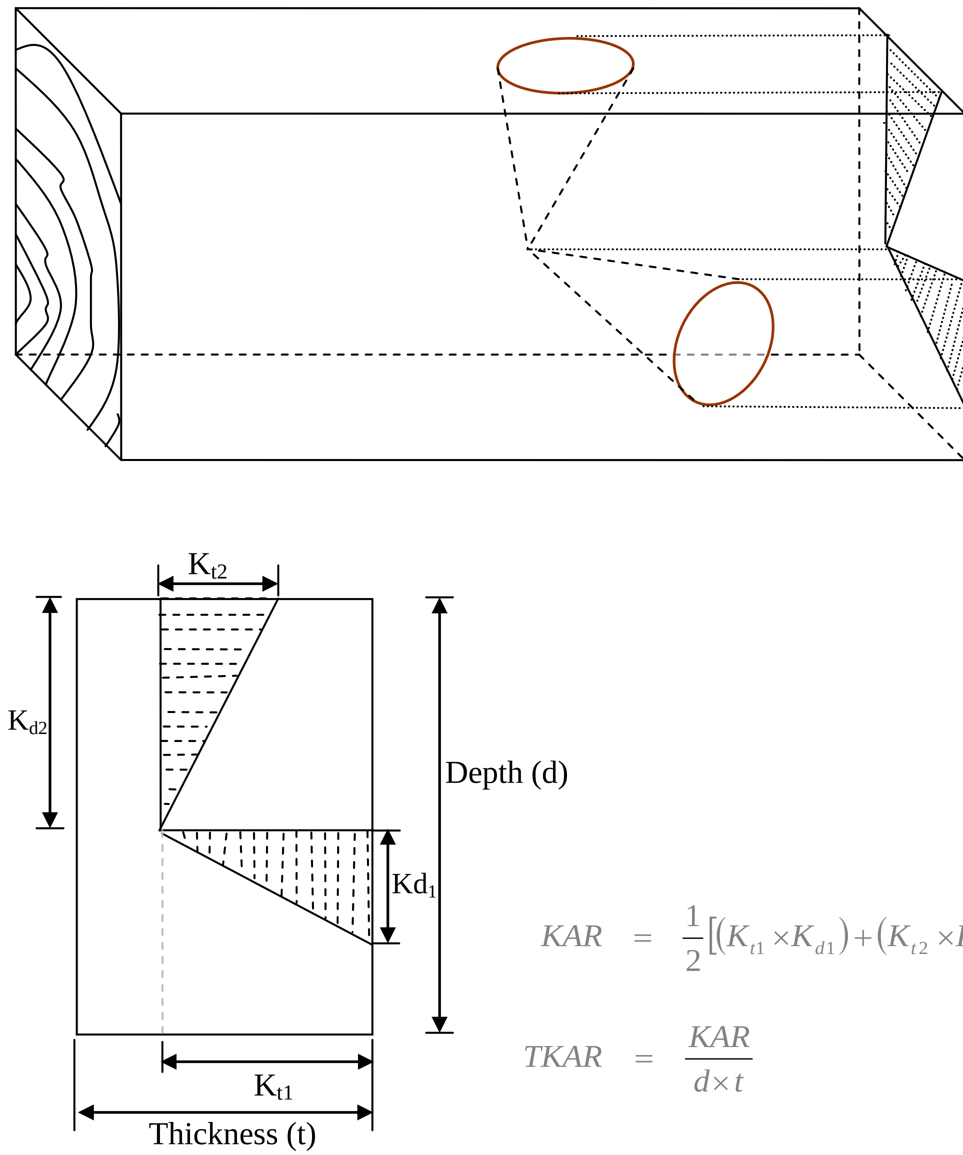
4.4 Influence of Knot on Shear Modulus

A knot is a common feature in the structure of wood. The presence of a knot in structural joists may have adverse effect on mechanical properties of as knots cause the distortion of fibres around them which, in turn, create stress concentrations and non-uniform stress distributions. In previous sections it has been noticed that the shear modulus varies along the length of joists substantially. Therefore, it becomes important to found if the presence of knots is the main source of causing this variation. This is because it was examined that in some joists the shear modulus was considerably lower within a segment that contained large knots. However, in some specimens, although shear modulus was significantly lower, the segment consisted of clear wood. As a result, it was not apparent from the above observations that if knot has any influence on shear modulus. Therefore, to determine the influence of knots, a correlation was developed between the total knot area ratio (TKAR) (BS4978:2007, 2007) of a segment and shear modulus of the relative segment and is presented in the following sections.

4.4.1 Test Material and Methods

Only Sitka spruce (2.0, 2.8 and 3.6m) joists were examined as Norway spruce mostly consisted of clear wood. The correlation between TKAR and shear modulus was acquired at the segment level. The TKAR was obtained on the basis of method described in British Standards (BS4978:2007, 2007). In the method, TKAR can be obtained by dividing the projected cross sectional area of all knots within the section to the cross sectional area of the same section. The projected cross sectional area of knot is mainly depending on the pith.

If the pith lies within the cross section then straight lines can be drawn to the pith from widest diameter of knot on either the face or edge of joist and then projection can be drawn on the cross section. Knot can be considered as cone with apex at the pith if the pith lies outside the cross section, as shown in Figure 4-10. Therefore in this study, all knots areas were measured as accordance of BS (BS4978:2007, 2007) and the TKAR was then calculated by summing the areas of knots within segments. If there were more than one knots were located in segments then TKAR was calculated on the basis of sum of areas of all knots within segments.



$$KAR = \frac{1}{2}\left[\left(K_{t1} \times K_{d1}\right) + \left(K_{t2} \times K_{d2}\right)\right]$$

$$TKAR = \frac{KAR}{d \times t}$$

Figure 4-10 Graphical representation for calculating the TKAR

4.4.2 Results and Discussion

Figure 4-11 represents the correlation between shear modulus of each segment within 2.0, 2.8 and 3.6m joists and the relative TKAR of that segment. This shows that there is very weak correlation between the knot area and the shear modulus (R^2 = 0.1087, 0.0084 and 0.035 for 2.0m, 2.8m and 3.6m respectively). This indicates that knot do not appear to have any substantial influence on shear modulus, although, 2.0m joists shows a small trace of correlation. From Figure 4-11, it can be noticed that in some joists both low shear modulus and knot were strongly correlated but it is not conclusive. It can noted that one of the segments of 3.6m joist with TKAR of 85% has shear modulus of 430MPa (15% lower) but in other segment though it has TKAR of 90% but shear modulus was bit higher at 550 MPa.

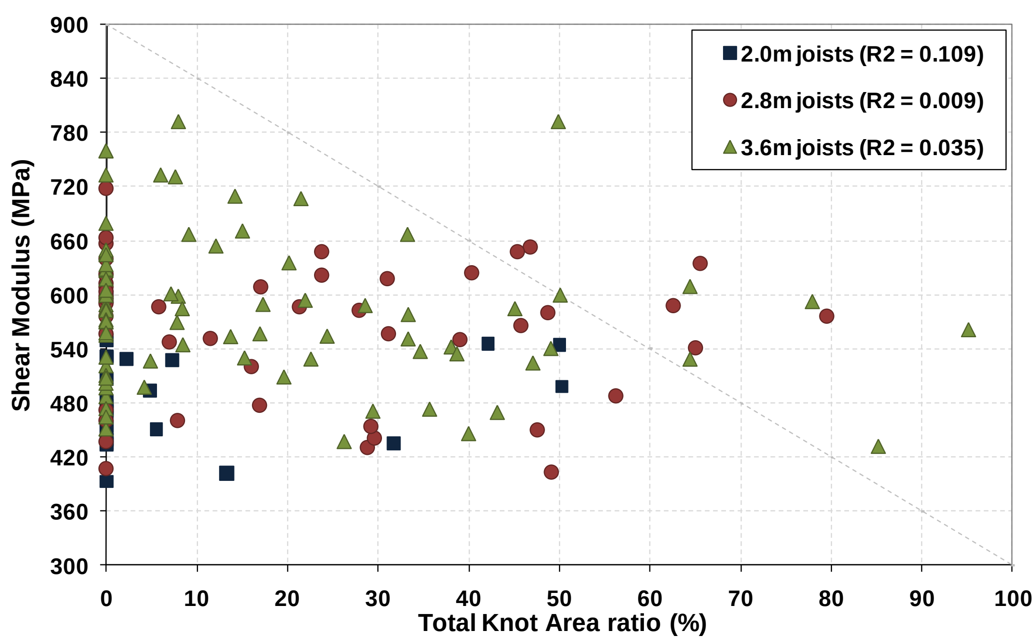


Figure 4-11 Correlation of TKAR and the shear modulus for 2.0, 2.8 and 3.6m joists

It was seen that joist 05 of 2.0m contains two central knot in S1, (32% TKAR) and in S2 (50% TKAR), a spike knot (Figure 4-12) in S3 (42% TKAR) and S4 consisted of clear wood. Yet, only S3 with spike knot had a slightly lower shear modulus value of 430 MPa (12% lower). The same observation was made for joist 09 of 2.0m as S1 has TKAR of 50% but shear modulus was 0nly 2% lower than G_{Avg}. It was also found out that S1 of joist 02 of 2.8m surrounded by three knots (80% TKAR, Figure 4-13) but shear modulus was only 11% lower than G_{Avg}. However, within the same joist, S4 has only 24% TKAR and that the shear modulus was also 11% lower than G_{Avg}. This was also found in S2 and S3 of joist 05 as they have 30% and 65% TKAR but shear modulus values were the same as of G_{Avg}.

Figure 4-12 A spike knot is positioned in segment 3 of joist 05 of 2m joists

Figure 4-13 Three knots of 80% TKAR present in the 2.8m joist.

It was also observed that in some of 2.8m joists although segments have about 50% TKAR but shear modulus values were substantially higher than the segments of clear wood within the joist. The same result of higher TKAR of a segment with higher shear modulus values were also observed in joists 04, 10 and 11 of 3.6m joists. More interestingly, S2 of joist 12 of 3.6m joist has TKAR of 95% and S3 has 78% TKAR. Yet, shear modulus values were about 2% higher than the G_{Avg} of the joist. This was also examined in S2 of joist 14 as the TKAR was 85% but shear modulus was about only 8% lower than the G_{Avg}.

From above observation, it is apparent that the presence of a knot does not have substantial influence on shear modulus. Although it was found that segments with higher TKAR have a lower G within the joists. However, in most segments with TKAR of 20% 60% has the same values of shear modulus as of the clear wood segments. It was also observed that in some segments where TKAR was about 75% to 95%. Yet, it was found that the shear modulus values of those segments were not affected. Also, in some joists it was found that segments with higher TKAR have the higher shear modulus. It was assumed that knot might have a substantial influence on the shear modulus because of due to discontinuation of grains and the grain deviation surrounding by knot region which might cause a high stress concentration and may disturb the transformation of shear.

However, from this study it can be concluded that knots do not have influence on shear modulus and the property is independent of the presence of knots. This may be because joists were tested under elastic torsional loadings, therefore, transformation of shear within the knot region was not affected at such small loading. Also as TKAR was used to measure the knot area it may be possible that the TKAR procedure may not adequate to measure the knot area. Therefore, there is a need of research investigation in this regard.

4.5 Influence of Time History and Repetitive Testing

This research also examines the adequacy of torsion testing setup in regard of its repetitive test method. Therefore, a repetitive testing study was conducted to measure the adequacy of the testing setup, described in following section.

4.5.1 Objective and Test Methods

The main purpose of conducting this study was to determine the influence of repetitive testing on the shear modulus and the adequacy of test setup for attaining the shear modulus of timber joists tested repeatedly. The other purpose was to observe any affect on shear modulus by testing joists at different time. To attain these objectives, 12 joists of 2.8m and 16 joists of 3.6m length (described in 4.2.1) were used. After conducting first elastic tests on joists (T01), all the joists were placed in a SIRT storage room for 28 days. The storage room had no facilities for controlled atmosphere, therefore, the joists were exposed to the natural atmosphere and it was assumed that joists might have attained higher moisture content.

All the joists were transported again to the testing laboratory and were conditioned at 21°C and 65% relative humidity until they attained approximately 12% moisture content. The 2.8m joists were re-tested (T02) by mounting again on torsion tester and tested under elastic torsional loading, as mentioned in 4.2.2, to achieve shear modulus of all four segments of each joist. Since, in the earlier test (T01) only two inclinometers (I_1 and I_2) were used for each segment. Therefore, in test two (T02), two more inclinometers were used (I_3 and I_4) in such way that both I_3 and I_4 were involved in attaining shear modulus for each segment of joists. Figure 4-14 provides the details positions of inclinometers for T01 and T02. The main purpose of using different inclinometers at the same segments was to eliminate any experimental error that might occur due to use of the same inclinometers.

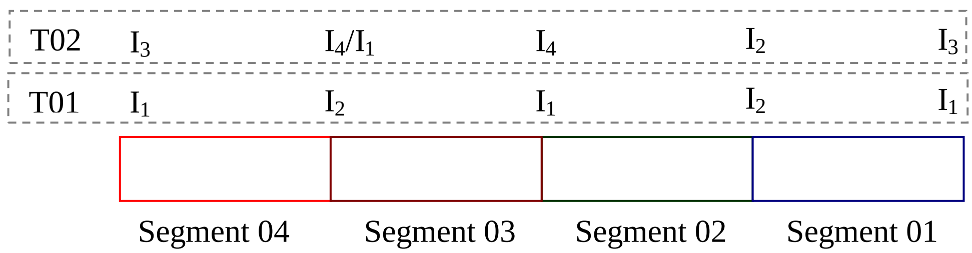


Figure 4-14 Inclinometer position for test 01 and test 02 for 2.8m joists

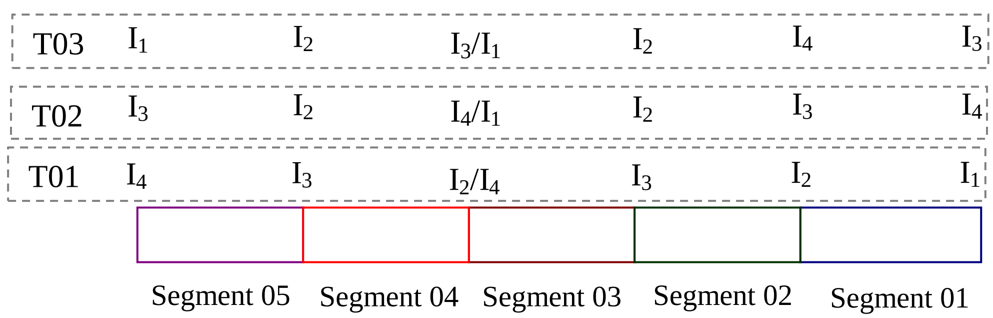


Figure 4-15 Inclinometer position for test 01, test 02 and test 03 for 3.6m joists

Along with T01 and T02 tests for 2.8m joists, the 3.6m joists were also tested by changing the position of inclinometers, as described in Figure 4-15. The 3.6m joists were then tested third time (T03) after 15 days interval. During that 2.8m joists were tested under ultimate loading which allowed 3.6m joists un-mounting and re-mounting of 3.6m joists. The mounting, un-mounting and re-mounting might allow observing possible influence of clamping of joists and motor system on test results on shear modulus.

4.5.2 Results and Discussion

Figure 4-16 and Figure 4-17 represent the average shear modulus of each segment of 2.8m and 3.6m joists, respectively. The test results show that there is a slight increase in shear modulus for Test 02 in comparison to Test 01 for both 2.8m and 3.6m joists. For 2.8m joists, highest increase of 7% was found in S1 and for Segments 2, 3 and 4 (S2, S3, and S4) an increase of 3.5 to 5% was achieved. Although there was no increase observed of S1 of 3.6m, yet 2% to 3.5% increase was attained in shear modulus for other four segments. A comparison of shear modulus was also conducted between T02 and T03 of 3.6m joists. It was found out that shear modulus of S3 was increased to 8% from T02 to T03 and shear modulus of S1, S2, S4 and S5 was increased up to 5%.

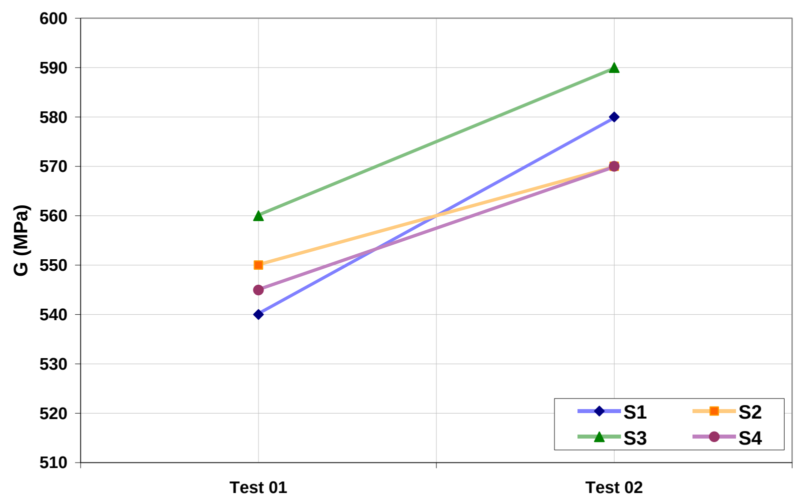


Figure 4-16 Influence of repetitive testing on shear modulus of 3.6m joists

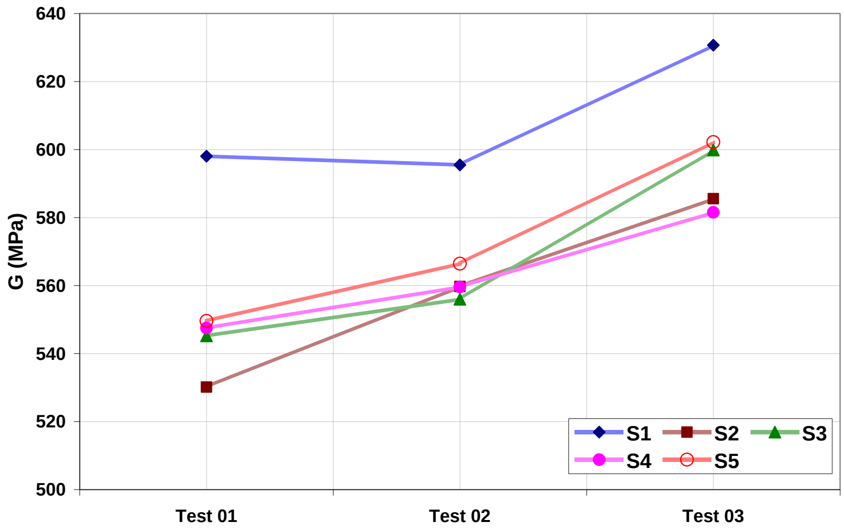


Figure 4-17 Influence of repetitive testing on shear modulus of 3.6m joists

The above results show that re-testing of timber joists under elastic torsion loading slightly increases the shear modulus. This may be because while loading joists in first test might pack wood fibres together and decrease the void spaces among them. When the joists were re-tested, the packed wood fibres would have provided slightly higher resistance to the torsional loading and, in turn, higher shear stiffness was achieved. This suggests that cyclic torsional loading has a positive influence on the shear modulus and this was also examined during conducting cyclic loading tests (Section 3.3). Although the tests were conducted with sufficient time difference, however, this seems that after conducting T1 the fibres did not went back to their original location and, therefore, time history may not have any influence on the shear modulus. The above results also imply that the testing method of attaining shear modulus in this research is adequate enough as there was no unsystematic values were achieved by conducting repetitive tests.

4.6 Summary

This chapter presents the investigation that was conducted to examine the variation in shear modulus along the length of timber joist. Sitka spruce and Norway spruce joists of lengths from 2.0m to 3.6m were used. The shear modulus was determined of 400mm and 600mm segments along the length of joists. This was achieved by conducting multiple elastic tests and measuring the twists of segments by placing inclinometers on topside of joists. It was found that shear modulus varies significantly along the length. About 30% lower shear modulus were obtained of a segment in compare to average shear modulus value of relative joist. It was assumed that the variation of shear modulus may be caused by knots. Therefore, a correlation of total knot area ratio of a segment and the relative shear modulus was developed. The correlation suggested that shear modulus was independent of the presence of knots. However, a further research is required to examine the influence of knots on shear modulus. It was also observed that shear modulus is not altered by testing at different time. However, it was noticed that shear stiffness slightly increased by retesting the timber.

5. TORSIONAL SHEAR STRENGTH OF WOOD

5.1 Introduction

This chapter presents the investigation that was conducted to obtain the shear strength of timber joists using torsion test. In previous chapters, Sitka spruce and Norway spruce joists were tested under elastic torsional loading to obtain the shear modulus. In this chapter, the same joists were also tested under torsion until either they were fractured or exhibited the pseudo-plastic behaviour. This allowed obtaining shear strength of structural timber using torsion test method. The shear strength values were calculated based on based on the maximum applied torque and using Saint-Venant torsion theory.

The shear strength parallel to grain or "shear strength" is a fundamental mechanical property of wood and is used in general timber structural design. Testing standards such as (EN408:2009, 2009) and ASTM (ASTM-D198-94, 1996) recommend to determine the shear strength of wood by testing small clear wood blocks ("shear blocks"). The published shear strength design values in the Wood Handbook (USDA, 1999) are also based on tests of shear blocks. 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 (ASTM-D198-94, 1996) to obtain the shear strength. The published design values of shear strength in CEN (EN338:2008, 2008) are calculated on basis of bending strength by testing full size structural lumber as accordance of EN (EN384:2008, 2008). 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.

Riyanto and Gupta (1998) have shown that torsion test is a better approach than bending and shear block tests. Gupta et al. (2002a, 2002b) also used experimental and finite element approaches and concluded that the torsion is more applicable test method to the shear block tests. Therefore, in this research the torsion test was used to attain the shear strength values of structural size timber joists. A comparison was also carried out between the published design values and the test values to observe the difference between shear strength values. In addition to this, a correlation between shear modulus and the shear strength was examined. This study also investigates and presents the failure mechanism of wood under torsion. This includes of correlation between fracture location and the shear modulus within the same location of joists. As discussed in Chapter 04 that knots have a very little influence on shear modulus but it was inconclusive. Therefore, in this chapter influence of knots on the shear strength and on fracture initiation was also investigated.

5.2 Test Material and Procedure

Sitka spruce (*Picea sitchensis*) and Norway spruce (*Picea abies*) joists of nominal cross section of 45 × 100 mm joists 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). The Test joists are the same samples that were tested under elastic torsion (discussed in Chapter 03). The torsion machine was used to induce torque in samples and the relative twists were measured from inclinometers, attached to the upper edge (45 mm dimension) of samples. For each length, inclinometers were mounted nearer the supports in such a way that the twist can be measured of the span of each length. Figure 5-1 shows the positions of inclinometer mounted for different length of specimens. All test specimens were tested at speed rate of 4°/minute until specimens were fractured under applied torque. The shear strength was calculated on the basis of maximum applied torque and that maximum twist of the member was not considered. This is because the maximum range of measuring rotation of inclinometers was 60° and due to this it was not possible to measure the maximum twist for joist span as they were twisted to higher rotational displacements (up to 100°). Inclinometers were used to measure the relative twist of span to calculate the shear modulus of span.

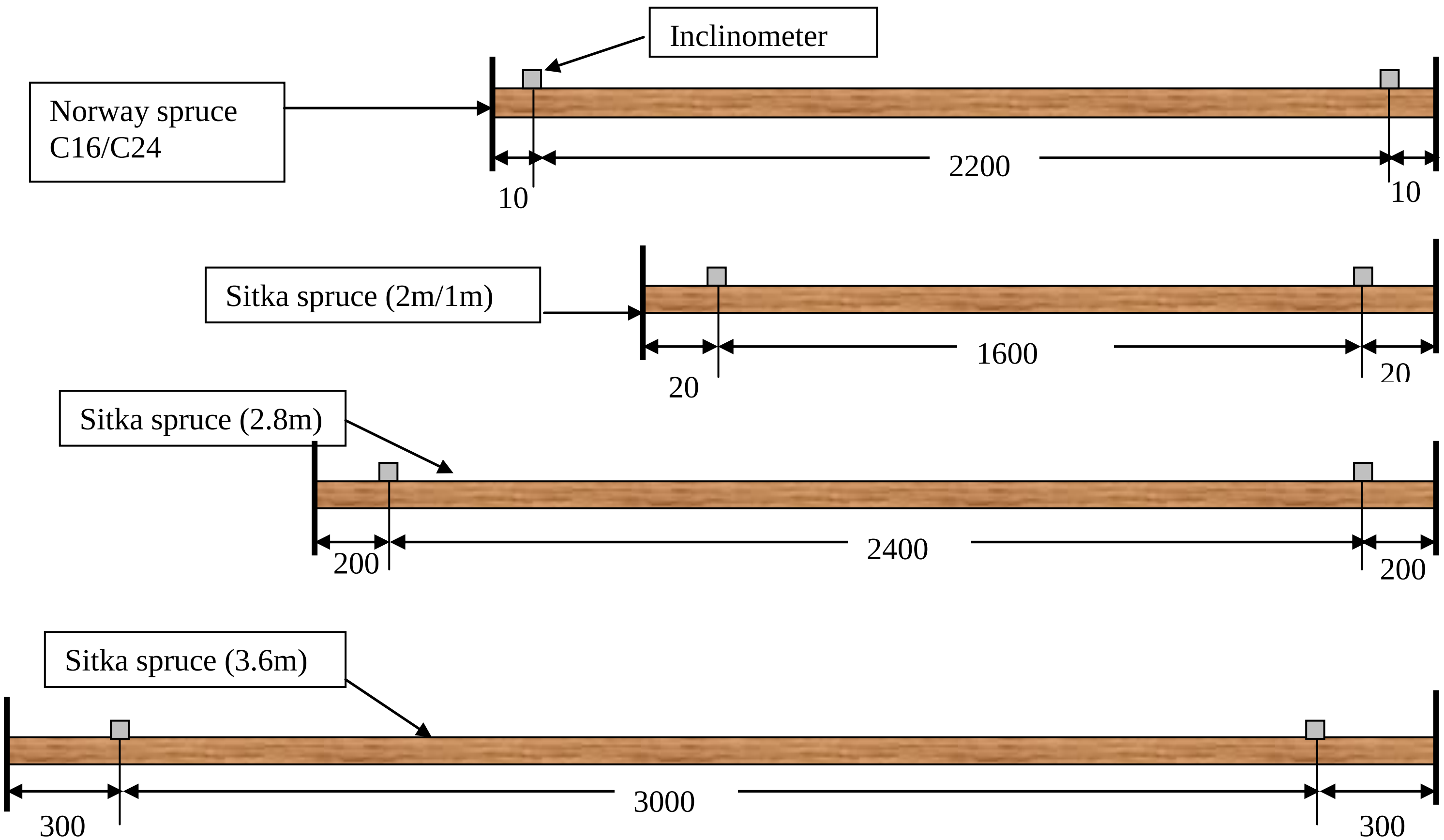


Figure 5-1 Shear strength test arrangements for Sitka spruce and Norway spruce joists (length in mm).

5.3 Result and Discussion

5.3.1 Design Standards and Torsional Shear Strength Values

The shear strength of each test joist was calculated on the basis of applied maximum torque using Saint-Venant torsion theory of rectangular section:

$$Shear\ strength = \frac{Maximum\ Torque}{(d\ t^2\ k_2)} \tag{5-1}$$

In Equation (5-1), d is the depth (major cross-section dimension) and t is the thickness (minor cross-section dimension) of the test specimen and k_2 is the torsional constants that depend on the depth thickness ratio (e.g. (Boresi and Schmidt, 2003). Shear strength was calculated on the basis of the maximum applied torque, as shown in Figure 5-2. The maximum applied torque is defined as the ultimate applied torque at which test joists were fractured. Table 5-1 represents the mean, the maximum and the minimum shear strength of all tested group.

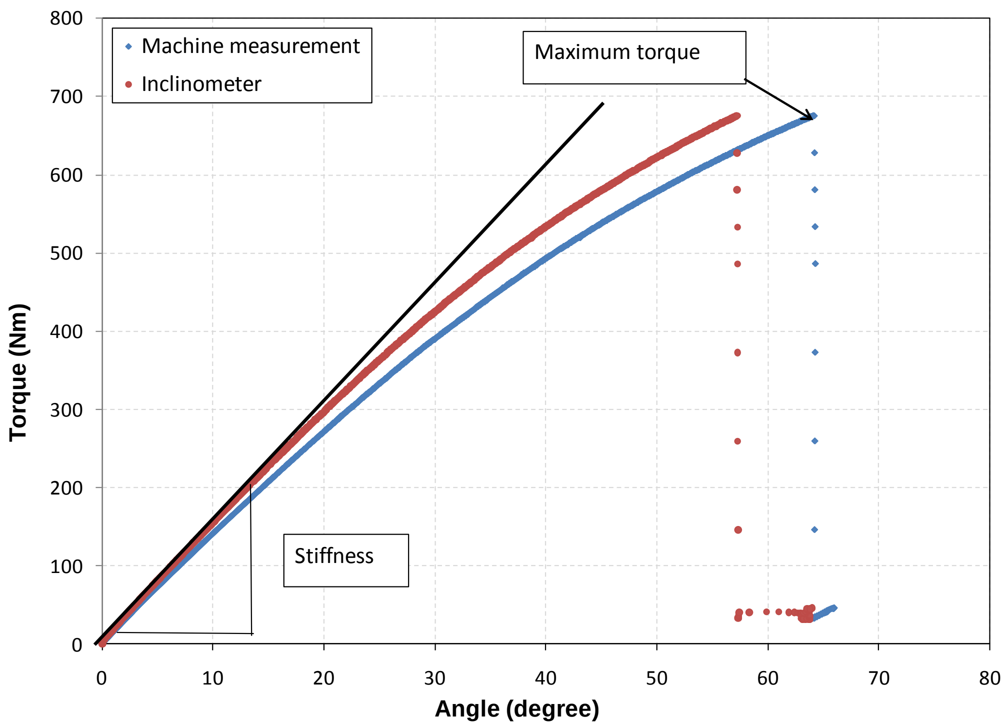


Figure 5-2 A typical relationship of applied torque and relative twist of 2.0m joist.

Table 5-1 The mean shear strength values of different tested timber species

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 average		490	7.2
NS	C16	2.4	14	390	8.5
	C24	2.4	12	410	9.3

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

In the current CEN (EN338:2008, 2008), 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 CEN (EN408:2003, 2003) (as shown in Equation (5-2).

$$f_{v,k} = 0.2\left(f_{m,k}\right)^{0.8} \qquad (5\text{-}2)$$

$$f_{v,k} \leq 3.8$$

Where $f_{v,k}$ represents the characteristic shear strength and $f_{m,k}$ is the characteristic bending strength. 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 CEN (EN338:2008, 2008) has raised the characteristic shear strength values for these grades (3.2 MPa and 4.0 MPa) but these are still substantially less than those observed experimentally in this study.

The Wood Handbook (USDA, 1999) provides the mean shear strength values of 6.7 MPa and 7.4 MPa for Sitka spruce and Norway spruce, respectively. The values were obtained on the basis of shear block tests. From this research, mean shear strength of Sitka spruce was 7.2 MPa (8% higher) and for Norway spruce was 8.5 MPa (13% higher) was obtained when torsion test was used. Similarly, Riyanto and Gupta (1998) have shown the shear strength values of Douglas-fir obtained from torsion tests 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 ((USDA, 1999). This comparison shows that relatively higher shear strength values were achieved when the torsion test approach was used. 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 CEN (EN338:2003, 2003).

This suggests that the assignment of shear strength values according to the results of bending tests may be over-conservative and this leads to conclusion if published values are applicable to design timber beam. The torsion test showed higher shear strength values and this may indicate that the method can 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.

5.3.2 Failure Mechanics 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 metre 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. It was observed that one of the 3.6 m joists was twisted to 110° (31° per metre) before it broke, as shown in Figure 5-3. 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.

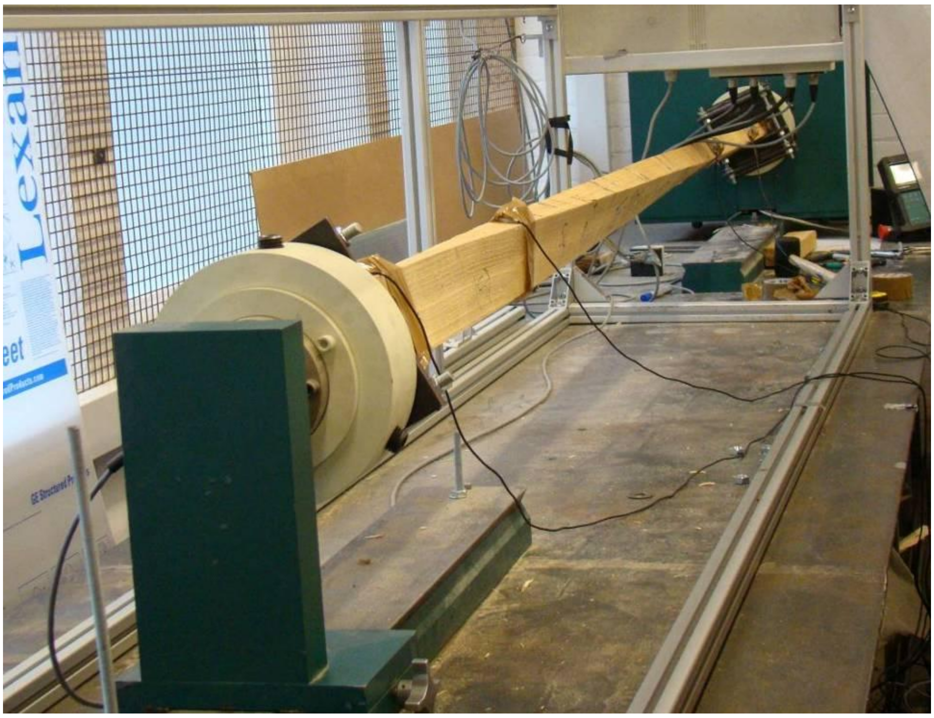


Figure 5-3 A typical 3.6 m joists with large rotational deformation before the fracture

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. The cracks were started from the middle of span and propagated towards the edges of joists or travelled towards the supports. Support conditions were found to be important. It was noticed that testing clamps induced additional compressive stresses which lead to a crushing of the wood at the supports and premature failure for some specimens.   Also, some joists were fractured suddenly as a brittle failure within the elastic zone mainly due to the inside bark or combination of inside bark and edge knots, as shown in Figure 5-4. In the Figure 5-4, relative torque-twist graph is shown as ordinate of the graph represents the applied torque in N-m and abscissa details rotation in degrees from torsion tester. It was examined that those types of specimens were fractured within low torque and produced lower shear strength. .

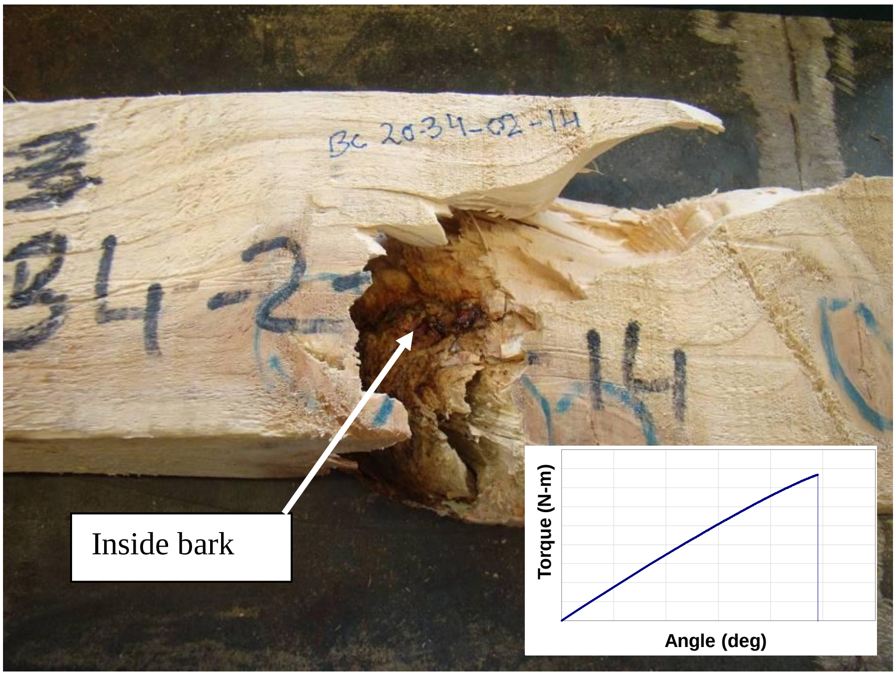


Figure 5-4 A typical premature fracture due to an inside bark.

Four different types of failure modes (viz crushing (44% of tested specimens), shear (25%), combined shear tension failure (12%) and horizontal shear failure (23%) were observed and are described below.

5.3.2.1 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 45% (32 out of 72) of Sitka spruce and Norway spruce 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 Radial-Tangential (RT) plane (Figure 5-5) and propagated along Longitudinal-Radial (LR) plane (long side), as shown in Figure 5-6.

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 Longitudinal-Tangential (LT) (short side) 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 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 joists failed immediately within the linear torque-rotation zone as a brittle failure, as shown Figure 5-7.

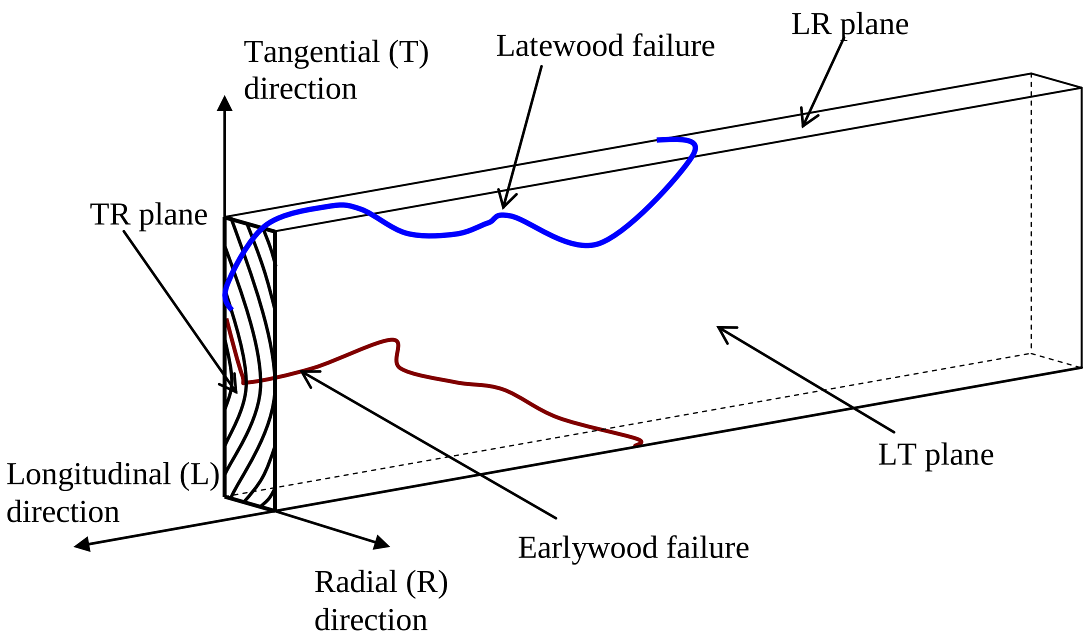


Figure 5-5 The Schematic diagram of timber joists showing grain direction.

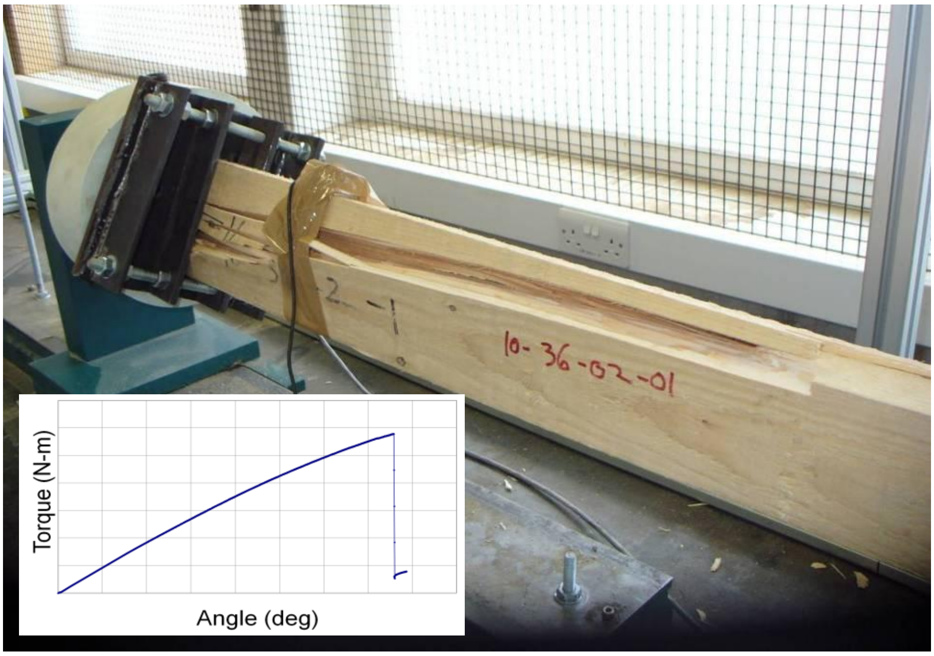


Figure 5-6 A crushing failure of 3.6m joist and its torque-twist relationship

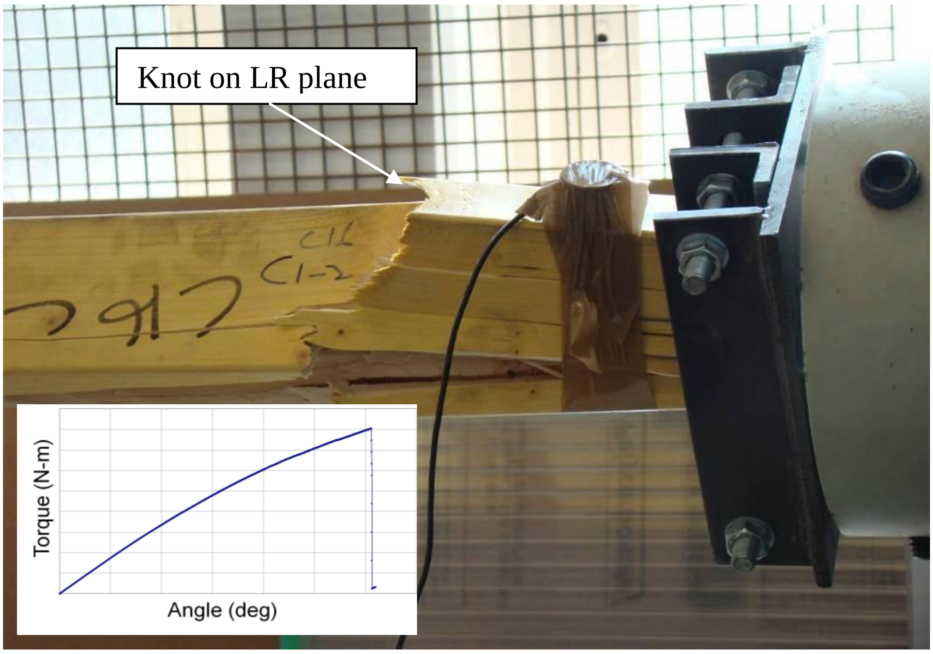


Figure 5-7 A sudden crushing failure in NS C16 joist due to a knot at LR plane

5.3.2.2 Combined Shear Tension Failure

Another type of failure mode observed was the combined shear tension failure and this occurred mostly in Sitka spruce joists. Seven out of 46 SP joists fractured with combined shear tension failure mode. The applied torque produced 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. It was also observed that when a crack approached 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 5-8 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.

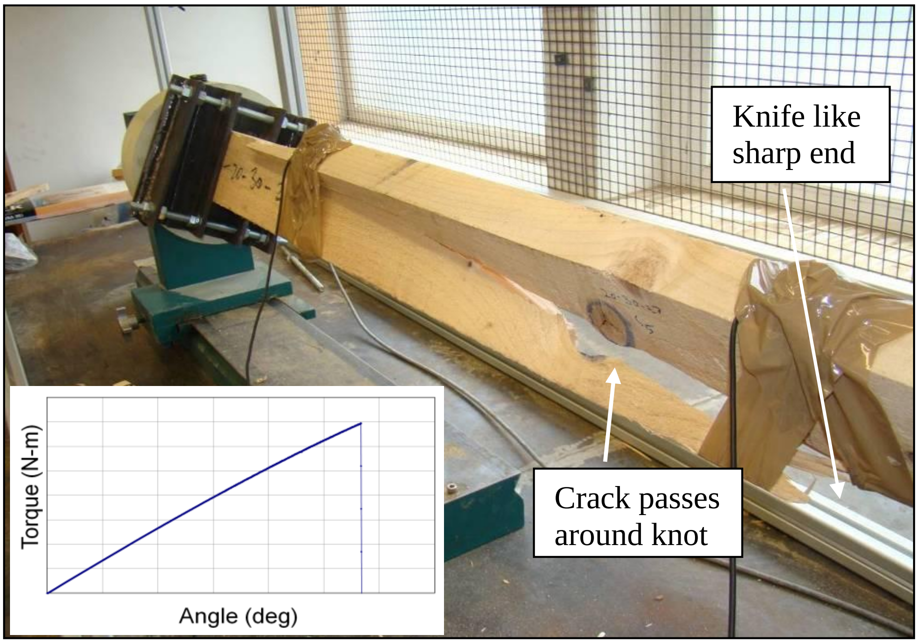


Figure 5-8 A combined shear tension failure occurred in 2.8m joist and crack passed through knot and ends up with a sharp end.

5.3.2.3 Shear Failure

Another type of failure that occurred was the shear failure, which was also mainly seen in the Sitka spruce joists. About 17 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. The 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 type of 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 5-9). It was found that in this type of the failure, the cracks passed around the knots that were present in the longside of the joists. This shows that knots are not the weaker plane along the long side of the joists.

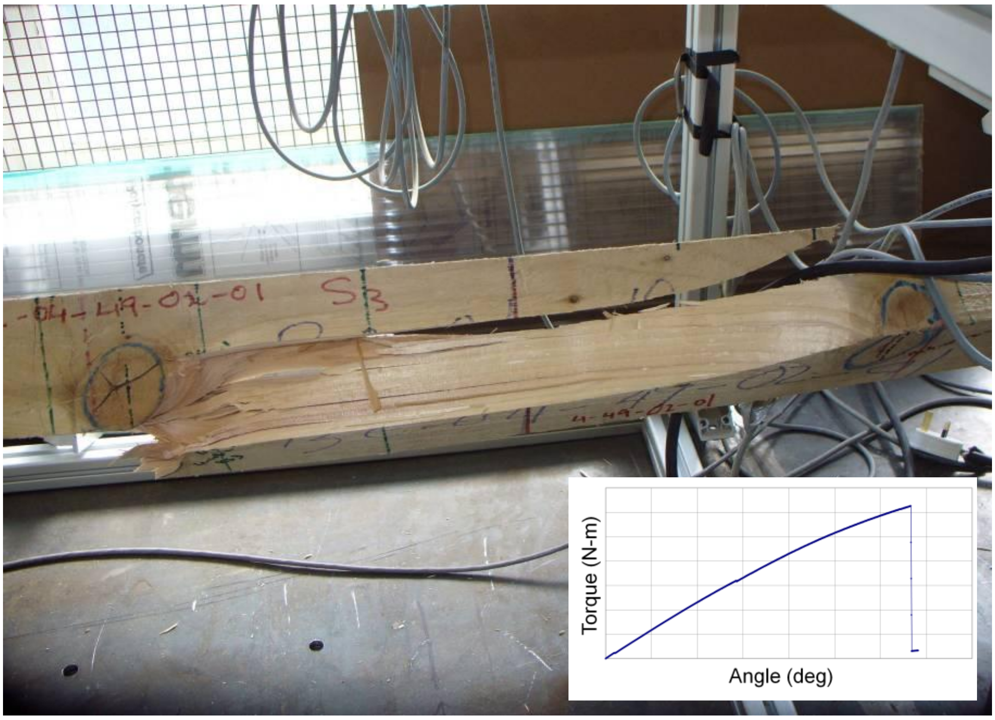


Figure 5-9 A typical shear failure occurred in 2.8m specimen due to top and bottom edge knots.

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 5-10. 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 5-11. 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.

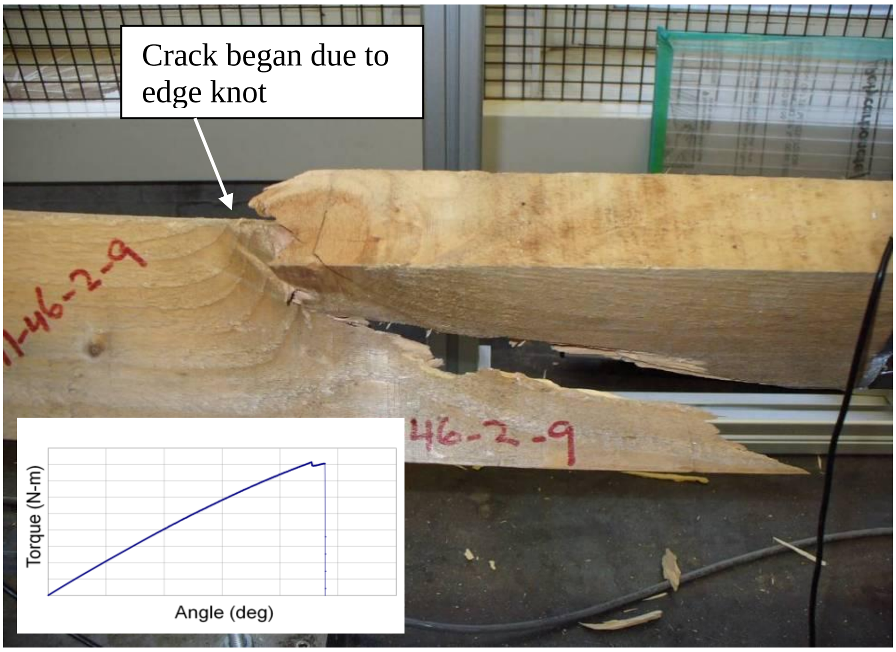


Figure 5-10 A typical shear failure began due to an edge knot in 2.8m joist

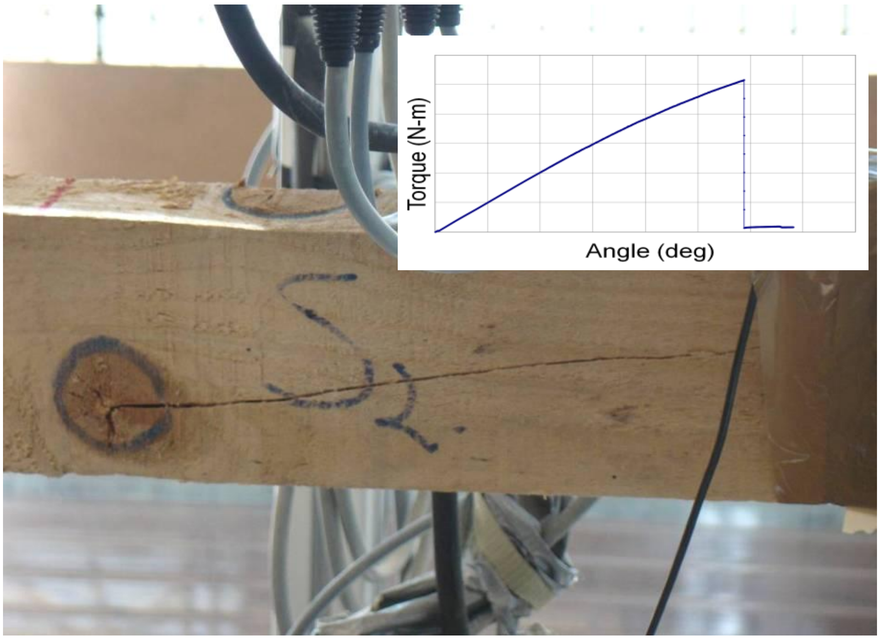


Figure 5-11 A typical shear crack started from knot fissure

5.3.2.4 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 5-11and in Figure 5-12. 16 out of 26 Norway spruce joists 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 Norway spruce specimens had grain direction 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 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 5-13.

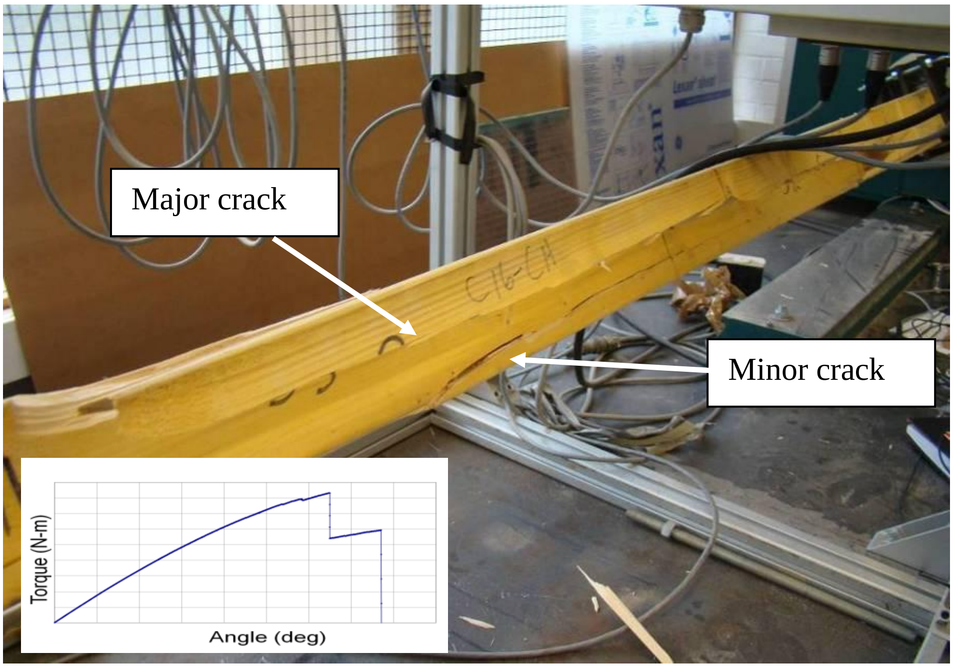


Figure 5-12 A typical Norway spruce C16 joist with a large horizontal shear crack and minor cracks

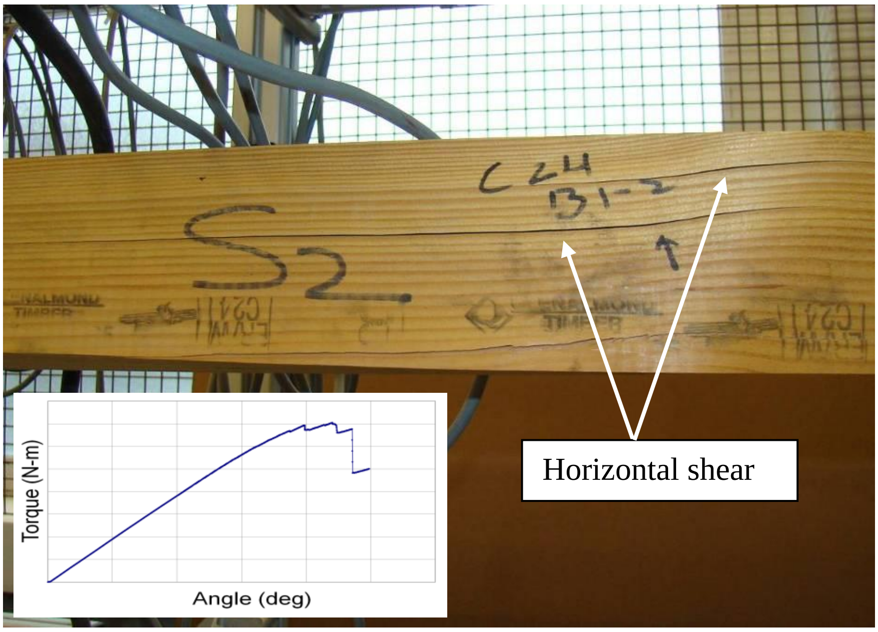


Figure 5-13 A typical wide shear cracks occurred in C24 timber joists under torsion loading.

5.3.2.5 Correlation of Failure Modes and Shear Strength

This section details the relative shear strength and the failure modes. In this investigation four failure modes were observed when joists were tested under torsion. It was seen that about 50 percent of joists were prematurely fractured due to clamps and this may have affect the actual shear strength values. Therefore, this is important to design testing clamps so that they can minimize the localized compressive stresses. It was found that joists were fractured due to clamps have a slightly lower shear strength (7.15 MPa) for Sitka spruce than shear failure mode shear strength values of 7.40 MPa, as shown in Table 5-2. The same observation was made for Norway spruce in that crushing failure mode shear strength was 8% lower than the horizontal shear failure mode values. This perhaps suggests that shear strength values may be higher if joists were fractured due to shear or horizontal shear failure modes.

Table 5-2 The failure mode type and relative shear modulus and shear strength

Failure mode	Species	No of joist	Shear modulus (MPa)	Shear strength (MPa)
Crushing failure	Sitka spruce	22	540	7.10
	Norway spruce	10	660	8.50
Shear failure	Sitka spruce	17	580	7.40
Combined shear tension failure	Sitka spruce	7	510	6.20
Horizontal shear	Norway spruce	16	690	9.10

5.3.3 Relationship of shear strength and shear modulus

The other study was conducted to examine if both shear modulus and shear strength are correlated each other. For this, a linear relationship between the shear strength and shear modulus of Sitka spruce and Norwegian spruce joists was developed, as shown in Figure 5-14. The shear modulus was calculated from the same test that was conducted for the shear strength on the basis of applied torque and the relative twist of the span as described in Chapter 03. In Figure 5-14, the R^2 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 their inclusion would unduly bias the correlation of shear strength and shear modulus. 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 shear modulus 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 shear modulus, which may weaken the correlation.

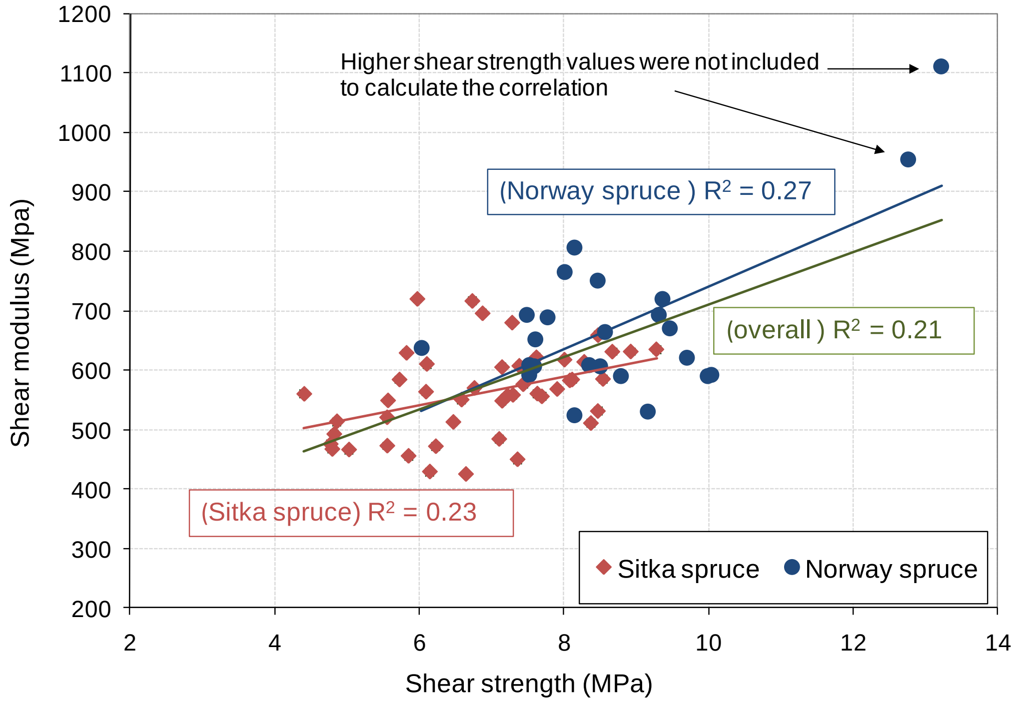


Figure 5-14 Linear relationship between shear modulus and the shear strength of Sit ka spruce and Norway spruce joists

A relationship between shear modulus and shear strength of each failure was also developed. For this, shear strength values were categorized according to each failure mode and were compared with the relative shear modulus and is presented in Figure 5-15. A weak correlation ($R^2 \approx 0.17$) was observed for shear failure and combined shear tension failure (CSTF) modes. A slightly better correlation ($R^2 \approx 0.35$) was seen for crushing and horizontal shear failure modes. It is noted that the two higher values were not included in calculating the correlation for horizontal shear failure. A low correlation was obtained for shear failure and CSTF modes because 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 shear modulus, which may weaken the correlation. Higher correlation for crushing failure modes were obtained as in both cases all the tested specimens were fractured within clear wood.

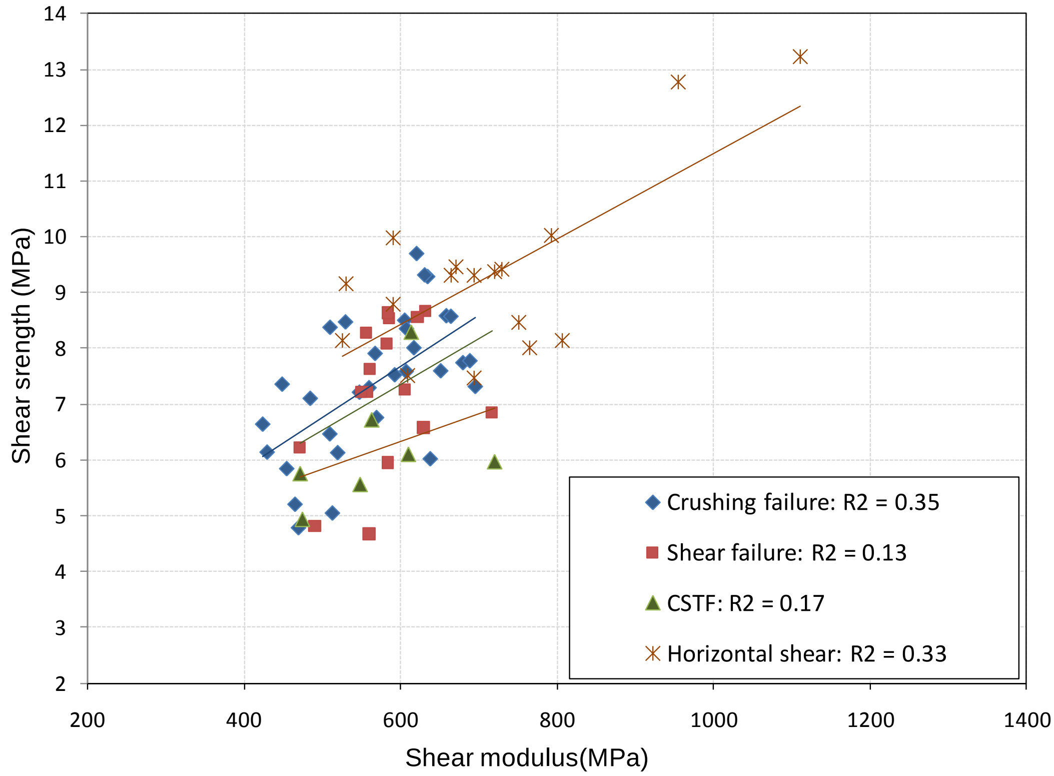


Figure 5-15 Linear relationship between shear modulus and the shear strength of various failure modes.

5.3.4 Correlation of Fracture Location and Shear Modulus

In this research an investigation was also conducted to examine the relationship between fracture location and the shear modulus at that same location of joist. It is reported that shear modulus varied along the length of joists and was considerably lower at various segments than average shear modulus (G_{Avg}) of the joist. Therefore, a relationship between the fracture location and the percentile variation in shear modulus within all segments of each Sitka spruce and Norway spruce joist was examined. In this regard, Figure 5-16 presents the percentile variation of shear modulus in relate to G_{Avg}, fracture location and knot positions along the length of 2.0m joists.

Joist No	Loading end	Segment 01	Segment 02	Segment 03	Segment 04	Reaction end
1		3	-1	-1	-1	
2		2	2	1	-7	
3		1	11	-20	8	
4		-6	2	1	3	
5		4	3	-11	4	
6		-15	13	0	3	
7		2	-1	3	-5	
8		-4	7	-1	-3	
9		-7	1	11	-5	
10		-6	3	1	1	

Central knot (along longside ≥ 25 mm diameter) Edge knot

Failure location Direction of fracture propagation

Figure 5-16 Fracture location and variation in shear modulus of 2.0m joist.

In Figure 5-16, the negative value represents lower shear modulus than G_{Avg} and vise versa. It can be noticed that eight of ten joists were fractured either at loading or reaction end. This may be because joists with shorter length have shorter shear span and fracture initiated within the testing clamps. Yet, it was seen in joist 3 that fracture was began from segment 03 (S3) and that the shear modulus of the same segment was 20% lower than G_{Avg}. The fracture was appeared from longside due to a central knot and then travelled towards and stopped by another central knot in segment 2 with shear failure mode.

The initiation of fracture within lower shear modulus section was also observed in 2.8m joists, as shown in Figure 5-17. In most test samples (1, 4, 5, 8, 9, 11 and 12), it was noticed that fracture was initiated within segments having shear modulus values were 4 to 8% less than G_{Avg}. In test specimen 5, the crack initiated from segment 2 that has 5% lower shear. It was found that the crack started within clear wood along longside, although a spike knot was located within the same segment. In specimen 9, the fracture initiated from clear wood of longside of S4 (4% lower shear modulus), passes through a spike knot and ended at shortside with combined shear tension failure (CSTF) mode. The same observation was also made for joist 11 as crack started from longside within clear wood (S4) of 8% lower shear modulus, passes around a central knot and ended at shortside as CSTF mode. From this, it can be concurred that cracks mainly initiated from clear wood sections that have low shear modulus values and that knots are less effective in triggering the fracture under torsion.

In only two out of twelve joists, the fracture was mainly triggered due to a knot located within lower shear modulus sections. In specimen 4, the fracture was began from an edge knot at end of S1 (6% less shear modulus) than travelled through segment 2 and ended in segment 3 due to another edge knot with shear failure mode. The same observation was also made for specimen 08. This thus indicates that knots are, at some extent, initiates the cracks but are not the major factors. This can be further observed in specimens 12 in that there were two edge knots located within S1 and central knots within segments 3 and 4 but the fracture was occurred at reaction support due to clamps. Only six out of twelve specimens were fractured due to clamp. This indicates that specimens with longer length fractured with shear span in compare to shorter length (2.0m length).

Joist No	Loading end	Segment 01	Segment 02	Segment 03	Segment 04	Reaction end
1		-6	0	1	5	
2		-4	-2	1	5	
3		9	-2	-5	-2	
4		-6	5	0	1	
5		-3	-5	4	5	
6		0	2	-3	1	
7		1	5	-2	-4	
8		1	-2	3	-2	
9		-6	2	8	-4	
10		10	-3	1	-8	
11		-5	-4	17	-8	
12		6	3	-5	-4	

▲ Spike knot

Figure 5-17 Fracture, shear modulus values and knot locations for 2.8m joists

Initiation of fracture within lowest shear modulus wood sections was also found in 3.6m joists, as presented in Figure 5-18. In most 3.6m joists, the crack was started from clear wood section where the shear modulus was lower than G_{Avg} and travelled though the length of the joist. In specimens 3, the crack was initiated within clear wood section of longside from segment 5 (10% less shear modulus), travelled through S4 and a central knot and then vanished in S3 with CSTF mode.

Joist No	Loading end	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Reaction end
1		27	-4	-5	-13	-6	
2		4	-4	-2	3	0	
3		10	6	2	-7	-10	
4		22	-7	-14	-1	0	
5		5	-8	5	0	-1	
6		6	-4	-8	0	7	
7		-8	0	1	0	7	
8		-13	13	1	-11	10	
9		9	-4	6	-7	-5	
10		-1	-2	3	-4	4	
11		1	-8	-4	2	9	
12		0	-9	6	-6	8	
13		-2	-5	-7	12	2	
14		12	-7	-5	-4	4	
15		1	5	-1	7	-13	
16		-6	9	3	-1	-4	
17		-3	-7	-6	0	16	
18		7	0	2	-10	1	
19		-9	4	-1	3	3	
20		-17	-8	-1	23	3	
21		-2	9	-9	-6	9	
22		-2	-5	4	-11	15	
23		9	-9	4	8	6	
24		6	-3	4	-5	-1	

Figure 5-18 Failure location, knot position and G of various segments of 3.6m joists

This was also seen in joists 2, 9 and 19 as crack began within lower shear modulus clear wood section and passed through knots and finished with CSTF mode. A more interesting fracture occurred in specimen 5 as the crack initiated from clear wood of 8% lower shear modulus section (S2), passes around two central knots located in S3 and S4 and then the crack path was ended in S5 region with shear failure mode. In specimens 11 and 18 it was noticed that the cracks path was started within clear wood sections of 8% to 10% lower shear modulus ended with shear failure mode. In three out of 24 specimens the knots initiated the fracture within lower shear modulus segments. In specimen 12, a crack was started from knot fissure in S2 (9% lower shear modulus) and then ended due to an edge knot in S3 as a shear failure mode. In specimen 15, a top edge knot initiated crack in wood section having 13% lower shear modulus (S5) and then crack was ended by a central knot in segment 3.

In some tests other wood defects also initiated the fracture. In specimen 4, the fracture was occurred in segment 5 due to a bark although two edge knots were located within wood section of 14% lower shear modulus (segment 3 and 4). The same observation was made for specimen 7 as fracture was occurred in segment 5 (7% higher shear modulus) due to a large grain deviation, even though, segment 1 of the same specimen has 8% lower shear modulus. This suggests that that wood defects initiated fracture but they are not major factor in leading a crack under torsion. As it was seen that in thirteen samples out of sixteen samples the crack was started within clear wood although the number of knots were present along the length of joist.

The initiation of fracture within section of low shear modulus values was also observed in Norway spruce test specimens. In this regard, Figure 5-19 shows the fracture location and shear modulus values for Norway spruce C16 grade joist. It can be examined that six out of eight joists were fractured within section of lowest shear modulus and remaining six joists were fractured due to clamps. In specimen 1, the crack was started in segment 1 that has 15% lower shear modulus than G_{Avg} of the joist. The crack then travelled and ended in S3 with horizontal shear failure mode. In specimen 4, fracture took place at wood section of 4% low shear modulus (S3), passed from S2 (5% low shear modulus) and S1 and ended at loading support with horizontal shear failure mode.

The same observation was made for joist 5 in that crack began from 8% lower shear modulus wood section (S4) and travelled about 1500mm and stopped at loading support as a horizontal shear failure mode. Also in specimen 10, the fracture was initiated in segment 2 (7% lower shear modulus) passes through segment 1 and ended at loading end. This was also seen that fracture started within lower shear modulus section in Norway spruce strength grade C24 joists, as shown in Figure 5-20.

Five out of nine test specimens were fractured within the section with the lowest shear modulus value and that remaining three were broken at supports. In specimen 8, the fractured started from segment 2 (7% lower shear modulus) and then travelled and ended at loading end. In specimen 5, the crack began from segment 3 (3% lower shear modulus) then travelled through segment 4 (15% lower shear modulus) and ended in reaction end. Also, in specimens 11 and 12 the cracks initiated within sections having the lowest shear modulus values and travelled towards reaction end and ended with horizontal shear failure mode.

From above correlation of shear modulus and the fracture locations it can be concluded that under torsion joists most often fractured within sections where shear modulus values found to be lower. This may allow predicting the weak plane of timber joists by examining the variation in shear modulus.

Joist No.	Loading end	Segment 1	Segment 2	Segment 3	Segment 4	Reaction end
1		-15	5	1	-1	
2		-1	-3	2	2	
3		10	-2	-3	-5	
4		0	-5	-4	9	
5		-3	5	6	-8	
6		-7	-5	13	-1	
7		-4	0	4	1	
8		-3	-2	4	1	
9		-8	0	0	8	
10		9	-7	-4	2	
11		9	8	-5	-12	
12		4	-1	6	-9	
13		7	2	-2	-7	
14		4	2	0	-6	

Figure 5-19 Correlation of shear modulus and failure location for NSC16 joists

Joist No.	Loading end	Segment 1	Segment 2	Segment 3	Segment 4	Reaction end
1		-1	7	-4	-1	
2		0	4	-8	3	
3		6	-1	3	-8	
4		-2	0	1	1	
5		9	9	-3	-15	
6		-5	-4	4	5	
7		1	0	0	-1	
8		-3	-7	6	4	
9		-9	-5	5	10	
10		3	1	-2	-2	
11		1	-1	1	0	
12		1	1	-4	2	

Figure 5-20 Correlation of shear modulus and the crack location for NSC24 joists

5.4 Summary

In this chapter an investigation is presented regarding the determination shear strength of the timber joists using torsion test approach. Sitka spruce and Norway spruce structural timber joists were tested until they fractured under applied torque. The shear strength was calculated on the basis of maximum applied torque that causes the fracture. It was noticed that higher strength grade joists produced higher shear strength values. It was found that testing timber joists under torsion produced higher shear strength values in comparison to published shear strength values in standard codes mainly obtained from shear block or bending tests. In general, specimens were fractured within clear wood along the longside of the specimen. The crack propagated towards supports running horizontally along the length of the joists. In some joists it was also noticed that knots initiated the fracture.

Various types of fractures were noticed under torsion and were categorized into four different failure modes. In shear failure mode, it was seen that the cracks were initiated within clear wood on longside, travelled horizontally and ended at the short side of the joists. It was also found that on the odd occasion knots on long or shortside initiated the fracture and that cracks were propagated towards and vanished from other knots located at opposite shortside. Combined shear and tensile forces also produced cracks that usually travelled diagonally along the longside of the test joists. Support condition found to be very critical as 40% of test joists were fractured at supports mainly due to inducing of additional stresses by testing clamps. Therefore, it is suggested that a better clamping system may be fabricated to minimize the clamp effect.

A good relationship between the fracture location and shear modulus within a joist section was found. It was observed that fracture mainly initiated from a segment have lower shear modulus in comparison to average shear modulus of joist. This indicates that a fracture location can be predicted on the basis of shear modulus values. However, it was noticed that knots or other wood defects were not the major factors in initiating the fracture.

6. TORSIONAL SHEAR MODULUS AND STRENGTH OF CLEAR WOOD

6.1 Introduction

This chapter details the study that was conducted to attain the shear modulus and the shear strength of clear wood using torsion test. The main purpose was to examine the variation in shear properties of clear wood and of structural size joists when obtained from torsion. This may also assist in determining the effects of wood defects. The structural size joists may contain wood defects such as slope of grain, knots, shakes, checks, bark and these defects may have influence on the shear properties. In previous chapters, it was found that knots do not have substantial influence on shear modulus and on shear strength. However, other wood defects may have effect on shear properties of the timber joist. To assess this, small clear wood specimens were tested under torsional loadings and shear modulus and the shear strength were obtained.

The shear strength was calculated on basis of maximum applied torque that was achieved by testing samples till they were fractured. The shear modulus was calculated within elastic range of applied torque and the relative twists. A comparison of shear modulus and shear strength values of clear wood and tested joists was conducted to observe any difference in values. It was an intention to examine if shear values of clear wood tested under torsion are appropriate to use for design of structural size joists. The other purpose of using clear wood sample was to achieve their modulus of elasticity (E) to develop a correlation of E and shear modulus. The correlation will be discussed in detail in Chapter 07.

6.2 Test Material and Methods

Fifty Sitka spruce species C16 specimens of 20×20×300mm were fabricated as accordance of (BS373:1957, 1957). A care was taken to make sure that the all specimens were free of knots, pith, slope of grain and other wood defects. The specimens were conditioned in a controlled environment chamber at 65% relative humidity and 21°C (approximately three weeks) until they attained moisture content of approximately 12%. The torsion tester was used to induce torque and inclinometers were used to measure the relative twist of specimens. The clear wood specimens were tested by mounting them in testing clamps of the torsion tester and by applying torque at 4°/min until they were fractured. Figure 6-1 illustrates the test setup of clear wood. The inclinometers were mounted on the topside of specimen at either end at distance of 70mm from testing clamp edges and this gives 160mm span of clear wood specimens.

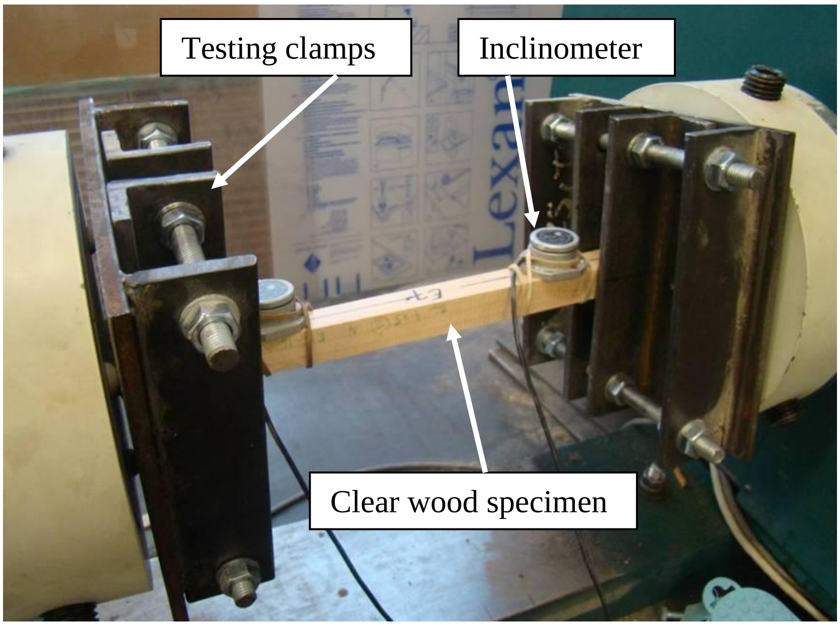


Figure 6-1 The torsion test setup of clear wood specimens.

The inclinometers that were used can measure the twist in parallel direction of applied torque (the local Y direction of the inclinometer) plus twist in perpendicular direction of applied toque (the local X direction of the inclinometer). The inclinometers were mounted using rubber bands rather than screws to avoid any possible damage to specimens. Extra care was taken to make sure that inclinometers were properly placed on specimen. This was achieved by observing displacement of inclinometers in Y direction and any displacement in X direction. In all tests, it was found that inclinometers were rotating in only Y direction and there was no displacements were observed in the X direction. This suggests that mounting inclinometers using rubber band is an adequate approach as inclinometers were measuring only the relative twist of specimens caused by applied torque.

6.3 Results and Discussions

6.3.1 Shear Modulus and Shear Strength Values

The shear modulus and shear strength of each test specimen was calculated based on Saint-Venant torsion theory of rectangular section (e.g. Bickford 1998) were:

$$Shear\ modulus\ = \frac{Stiffness\ \times L}{(d\ t^3\ k_1)} \tag{6-1}$$

$$Shear\ strength\ = \frac{Maximum\ torque}{(d\ t^2\ k_2)} \tag{6-2}$$

In Equations (6-1) and (6-2), L represents the distance between the two inclinometers, d is the depth (major cross-section dimension) and t is the thickness (minor cross-section dimension) of the test specimen,k_1 and k_2 are torsional constants depend on the depth thickness ratio The maximum applied torque is defined as the ultimate applied torque at which test specimens were fractured. The stiffness was obtained by conducting regression analysis of the applied torque and the relative twist per length within the elastic region as shown in Figure 6-2.

For most of the test specimens the elastic region lies in range of 5% to 30% of maximum applied torque, therefore, linear regression analysis was conducted from 5% to 25% of maximum applied load. Table 6-1 shows the mean maximum applied torque, mean shear modulus and the mean shear strength of clear wood specimens. The mean shear modulus of 670MPa and the mean shear strength of 10MPa were obtained when clear wood were tested in torsion. It was noticed that both shear modulus and shear strength of clear wood were about 28% and 40%

higher than average shear modulus (520MPa) and average shear strength (7.2MPa) of tested joists of the same species.

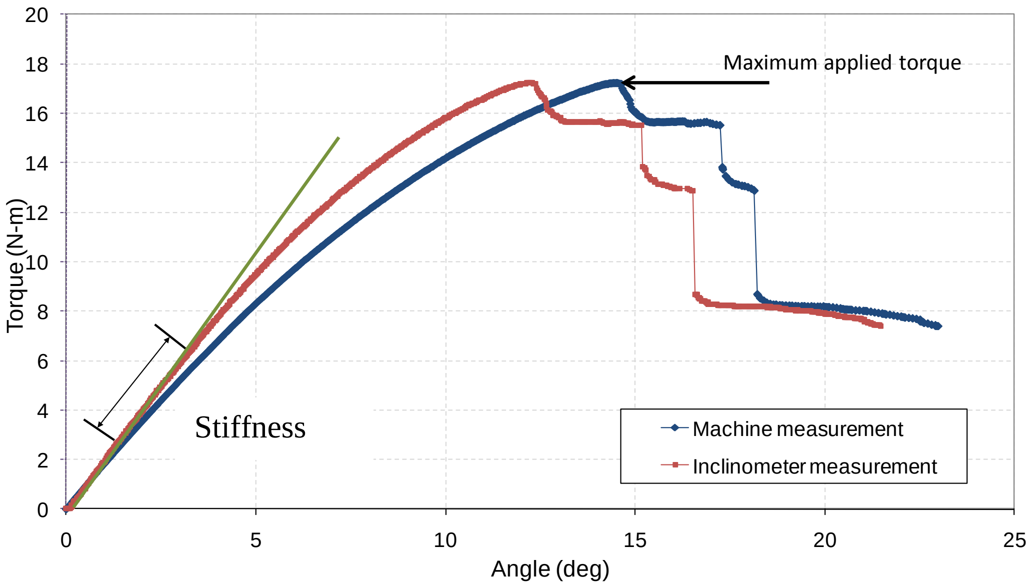


Figure 6-2 A typical torsional test for clear wood under torsion

Table 6-1 The shear modulus and shear strength values of small clear specimens

	Density (kg/m^3)	Maximum torque (N-m)	Shear modulus (MPa)	Shear strength (MPa)
Mean	395	25	665	10
StD.	60	5	185	2
CoV	15	19	28	23
Maximum	530	38	1190	16
Minimum	290	17	400	6

The higher shear stiffness and strength values of clear wood indicate that presence of various wood defects may affect the shear properties. Although in this research, it was identified that knots have very small influence on shear modulus and also have a small effect on the shear strength. Therefore, influence may caused by other factors such as, size effect, checks, shakes and slope of grain in structural size joists. This implies that shear properties determined from clear wood tested under torsion may not be appropriate to use for general design of structural size wood joists as the clear wood test method overestimates the shear properties of wood.

A correlation between shear properties and the density of clear wood was also examined to observe how both are correlated. In this regard a linear correlation of shear modulus and shear strength to density of clear wood was developed, as shown in Figure 6-3. A good correlation between shear properties and density was found. As the R^2 values for shear modulus and of shear strength of 0.38 and 0.49 were obtained, respectively. About the same R^2 values of 0.30 were obtained when correlation of shear modulus and density of Sitka spruce joists was developed. However, slightly weaker correlation (R^2 = 0.25) was found when density and shear strength of Sitka spruce joists were taken into account. It suggests that shear properties of timber are, at some extent, depend upon the density of wood.

A relationship between shear modulus and shear strength of clear wood specimens was developed, as shown in Figure 6-4. A lower correlation (R^2 = 0.30) were obtained between shear properties of clear wood. This was almost the same R^2 values (0.23) when correlation of shear modulus and shear strength of Sitka spruce joists was obtained. It should be noted that clamps triggered the cracks in thirty test specimen and caused premature fracture which could have resulted in lower shear strength values. A premature fracture can possibly be avoided by using longer clear wood specimens as it was observed that longer joists were fractured mostly with span and very few joists had premature failure in due to clamps in compare to shorter span joists.

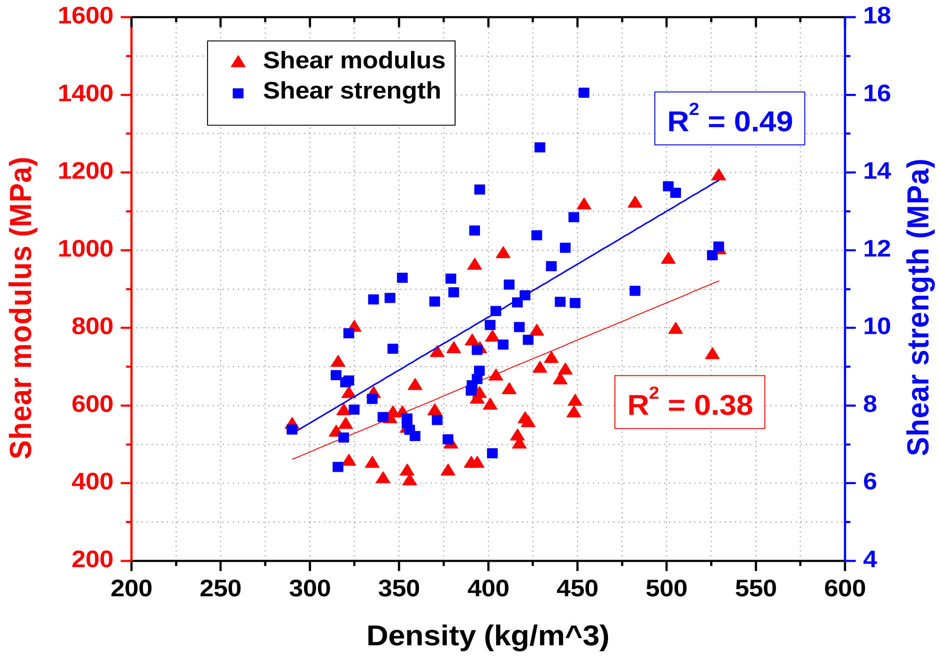


Figure 6-3 A correlation of density, shear modulus and shear strength of clear wood specimens

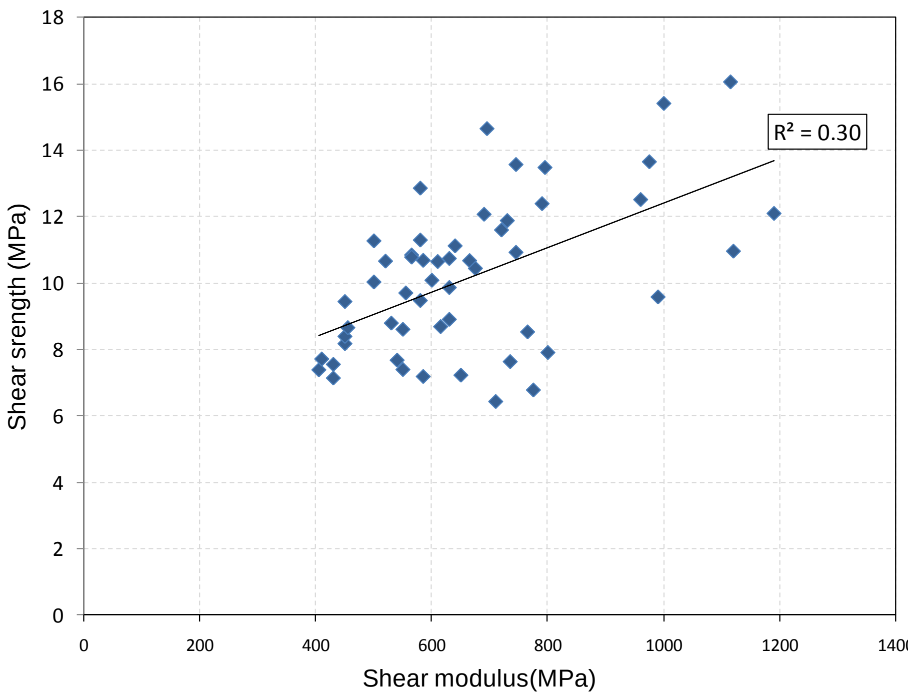


Figure 6-4 A linear correlation of shear modulus and shear strength of clear wood specimens.

6.4 Failure Mechanism of Clear Wood

It was noticed that most of the specimens were fractured at lower torque within range from 15 to 40 N-m and that specimens were twisted up to 10° per 100mm. It was also found that in most of the tests, samples did not fractured as a brittle material but they exhibit a ductile type failure and that torque-twist relationship went into pseudo-plastic region before test specimens were ruptured. At the time of failure, cracking noise was heard and that small wood pieces and a puff of dust was observed. Three failure modes, crushing failure, shear failure and combined tension shear failure, were seen when clear wood specimens were fractured under torsion. Twenty nine test samples exhibited crushing failure modes as testing clamps either at loading or reaction end triggered cracks, as shown in Figure 6-5. The fracture usually was initiated along cross-section in radial direction as ring growth direction provides less resistance than of tangential direction. The crack then travelled towards top or bottom end and stopped at the surface edges shown in Figure 6-5.

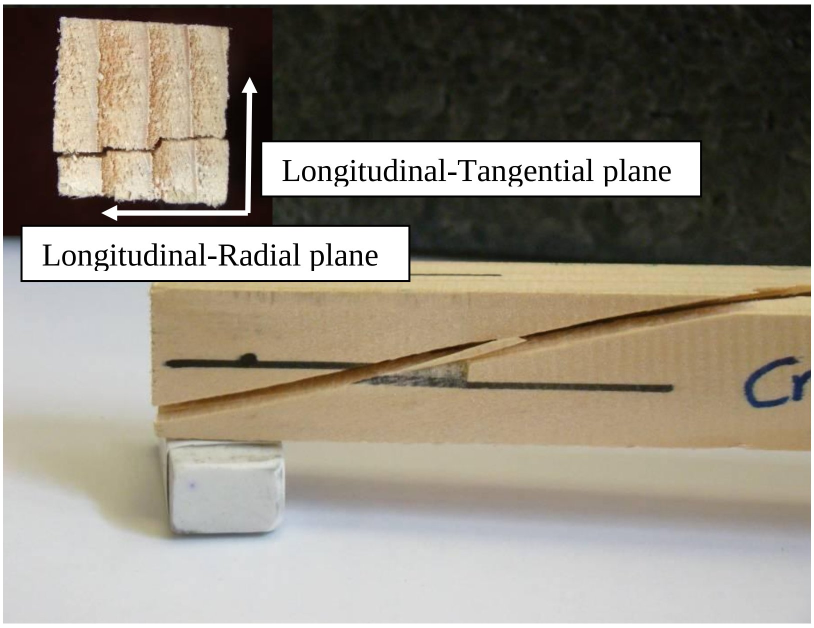


Figure 6-5 Front and cross-sectional of view of a typical crushing failure of clear wood specimen.

About 8 of 50 test specimens were fractured with shear failure mode. In this failure mode, cracks started at the middle and travelled along grain direction and ended towards top or bottom surface edge, and in some cases run towards supports. Figure 6-6 shows a typical shear failure in clear wood specimen. Another type of failure mode observed was the combined shear-tension failure and 13 out of 50 specimens were fractured with combined shear tension failure mode. The applied torque produces shear stresses and these stresses were dominant in causing this type of fracture. The shear crack initiated from the middle of the longitudinal-tangential plane and due to tension propagated towards, and was ended, in the longitudinal-radial plane, as shown in Figure 6-7. 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 6-6 A typical shear failure occurred in clear wood specimen.

Figure 6-7 A typical combined shear tension failure occurred in clear wood specimen

6.5 SUMMARY

In this chapter a work is presented that was conducted to determine the shear modulus and strength of clear wood. Sitka spruce clear wood specimens were tested under torsion until the specimens were fractured. A torsion tester was used to induce toque and relative twist of specimens was measured from inclinometers. The shear modulus was calculated from elastic region of applied torque and relative twist and the shear strength was measured using ultimate torque. A higher shear modulus and shear strength values were obtained when clear wood was tested in compare to the full size structural joists. This may suggest that different wood defects and specimen size may have influence on shear properties of wood. It was observed that specimens predominantly fractured at the clamps. However, it was also seen that cracks initiated at the middle and propagated towards supports as a shear failure mode. Also, combined shear and tension stresses caused cracks at the middle of specimens and then crack travelled towards and ended at the edges. The results also assist in developing the correlation between shear modulus and modulus of elasticity of clear wood, will be discussed in the next chapter

7. CORRELATION BETWEEN MODULUS OF ELASTICITY AND SHEAR MODULUS OF TIMBER

7.1 Introduction

The investigation presented in this chapter is conducted to examine the correlation between shear modulus (G) and modulus of elasticity (E) of timber. In timber design, the shear modulus is often calculated using E to G ratio of 16:1, especially for vibrational serviceability of wood based floors and for lateral torsional stability of timber joists. The shear modulus values in CEN (EN408:2009, 2009) and in USDA Wood handbook (USDA, 1999) are also obtained from E G ratio of 16:1. In addition to this, standard test methods recommend that shear modulus can be derived from modulus of elasticity. Therefore, this becomes essential to examine if both modulus of elasticity and shear modulus have any relationship and if it is appropriate to determine shear modulus from modulus of elasticity. For this, correlations between the two were conducted from small clear wood sections to structural size timber members. Four point bending test was used to attain the modulus of elasticity and torsion test approach was implied to determine the shear modulus of various sections within the joists and full span of the joists. The modulus of elasticity and shear modulus of clear wood was attained from acoustic tests and from torsion tests, respectively.

7.2 Test Method and Materials

Sitka spruce and Norway spruce species were used for the investigation. For Sitka spruce, strength C16 joists of length 2.8m (12 specimens) and 3.6m (25 specimens) were tested. 2.4m long Norway spruce joists of grade C16 (14 specimens) and C24 (12 specimens) were tested. For small clear wood, fifty (20×20×300mm) Sitka spruce specimens fabricated as accordance of BS (BS373:1957, 1957) were used.

7.2.1 Test Procedure for Modulus of Elasticity

Bending and acoustic tests were used to obtain the modulus of elasticity of test joists and clear wood specimens. Test joists were tested under four point bending as accordance of CEN (EN408:2003, 2003) and the modulus of elasticity of 600mm and 1800mm sections were obtained along the span. The acoustic tests were conducted to attain the modulus of elasticity of joist span and of clear wood. In bending tests, specimens were loaded under bending at two 600mm points over span of 1800mm using the Zwick Z050 universal testing machine. A maximum load of 3kN at displacement control of 2mm/minute was applied which allowed to test specimens within elastic range and that no permanent deformation occurred to the specimens. Deflection was measured of the top surface of the specimens using Linear Variable Differential Transducers (LVDT), provided at each side of the specimen. Figure 7-1 shows the test arrangement for obtaining bending modulus of elasticity.

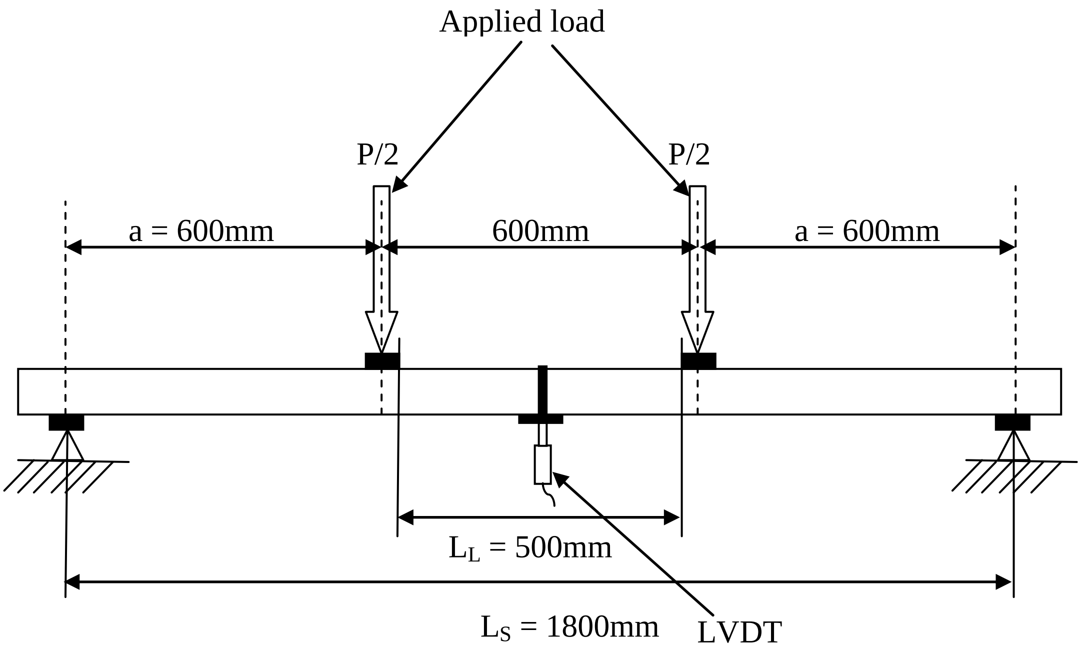


Figure 7-1 A four-point test arrangement for the modulus of elasticity of timber joist.

The local modulus of elasticity (E_{Local}, refers to 500mm middle segment) and global modulus of elasticity ($E_{Global,}$ refers to 1800mm section) were obtained using Equation (7-1) and Equation (7-2) as follows:

$$E_{Local} = \frac{aL_L^2\,(\Delta P)}{161(\Delta\delta)} \tag{7-1}$$

$$E_{Global} = \frac{L_L^3\,(\Delta P)}{td^3(\Delta\delta)}\left[\left(\frac{3a}{4L_S}\right) - \left(\frac{a}{L_S}\right)^3\right] \tag{7-2}$$

(ΔP/Δδ) was obtained by conducting linear regression analysis between initial and final applied loads (P) and the correspondent deflections (δ). In the Equations, d denotes the depth (major cross-section dimension), t represents the thickness (minor cross-section dimension) and I refers to moment of inertia. L_L, L_S and represent the distance between support and loading points, as shown in Figure 7-1. The test setup was designed in such a way that E_{Local} and E_{Global} can be obtained simultaneously of different segments along the length of joists. A detailed explanation is given in the following discussion.

Each 3.6m joist was tested three times to obtain three values of E_{Local} and E_{Global}. First, test was conducted by applying loads on segment 02 (S2) of 600mm length with reaction supports at 1800mm distance from segment 01 (S1) to end of segment 03 (S3), as shown in Figure 7-2. This allows obtaining E_{Local} of S2 and relative E_{Global} of span (S1+ **S2**+S3) where **S2** indicates the location of applied load. Consequently, tests were conducted and E_{Local} of segment 3 (S3) and segment 4 (S4) and relative E_{Global} of (S2+**S3**+S4) and (S3+**S4**+S5) were obtained. Two consecutive tests were conducted on 2.8m joists and on Norwegian Spruce C16 and C24 specimens. In both tests, E_{Local} of S2 and S3 segments and relative E_{Global} of (S+**S2**+ S3) and (S2+**S3**+S4) 1800mm were determined accordingly.

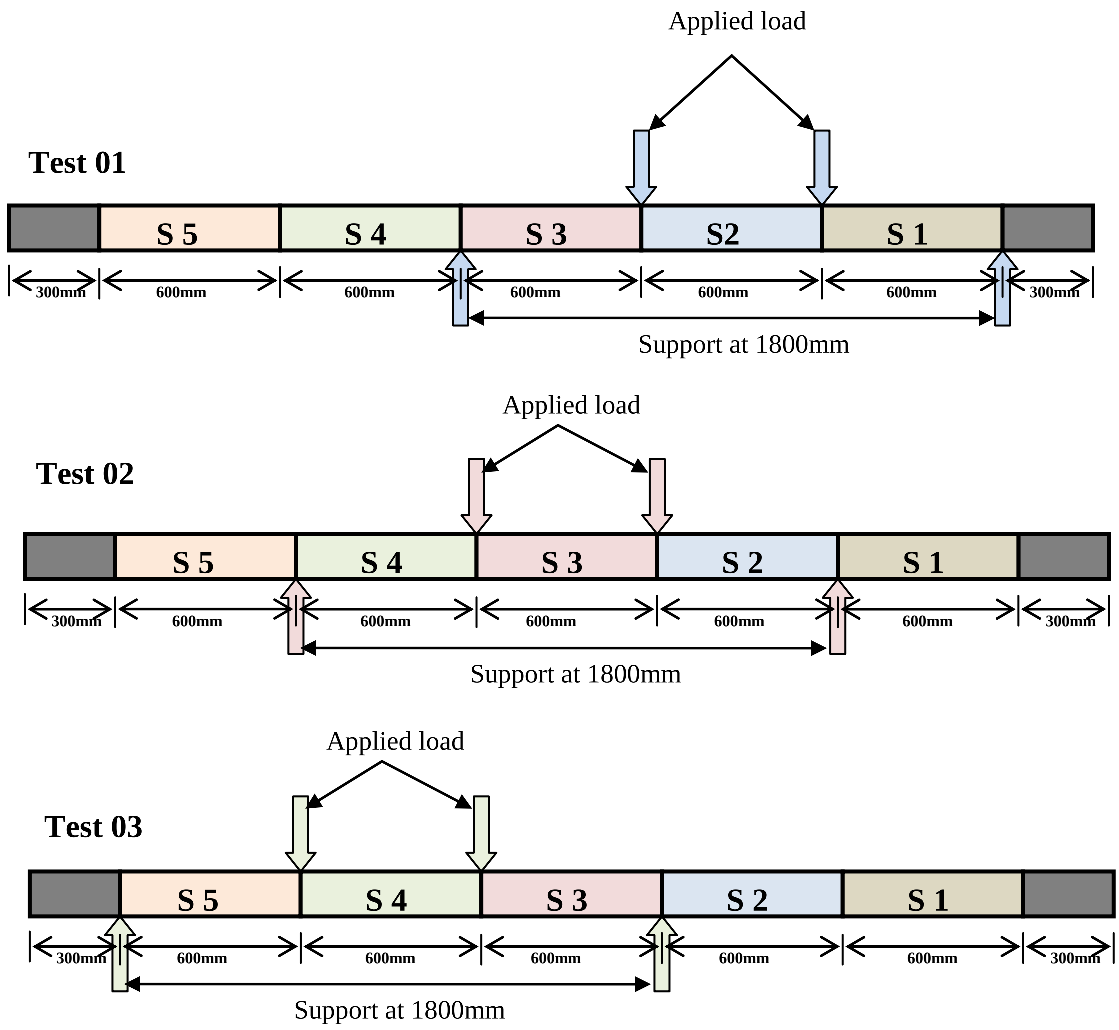


Figure 7-2 Test arrangements of 3.6m joist under four point applying loads at S2 (Test 01), S3 (Test 02) and S4 (Test 03).

The four point bending test arrangement in this study can only allow testing of 1800mm joist length. Therefore, acoustic test method was used and modulus of elasticity of overall joist span (E_{Span}) and of clear wood (E_{CW}) was obtained. For joists, stress waves were generated in the samples by tapping one cross sectional end by hammer and measuring velocities of the waves by acoustic tool Hitman HM-200 at the other cross-sectional end. The E_{Span} was then calculated from measured acoustic velocity and wood density (ρ), as given in Equation (7-3). The V represents the velocity of the wave and is given by, $V = 2\ L_{sample}f$, where f is the fundamental resonance frequency and L_{sample} is the sample length. This testing method has been found appropriate for modulus of elasticity of timber beams based on research conducted at Edinburgh Napier University (Lyon et al., 2007).

$$E_{Span} = \rho V^2 \qquad (7\text{-}3)$$

For clear wood specimens, Grindosonic MK5 instrument was used to measure the acoustic velocity. In the test method, a longitudinal stress wave was created in the sample by lightly tapping on the one end face with a ball bearing hammer. The fundamental frequency of the longitudinal wave was measured at the opposite end face of the sample using a microphone and Grindosonic. Each sample was placed on two "knife edge" supports located at 0.25 length of clear wood specimen, as shown in Figure 7-3. The E_{CW} was then calculated from fundamental frequency, total length of the specimen (L_{CW}) and wood density (ρ), as described in Equation (7-4):

$$E_{CW} = \rho \left[\frac{f}{L_{CW}}\right]^2 \qquad (7\text{-}4)$$

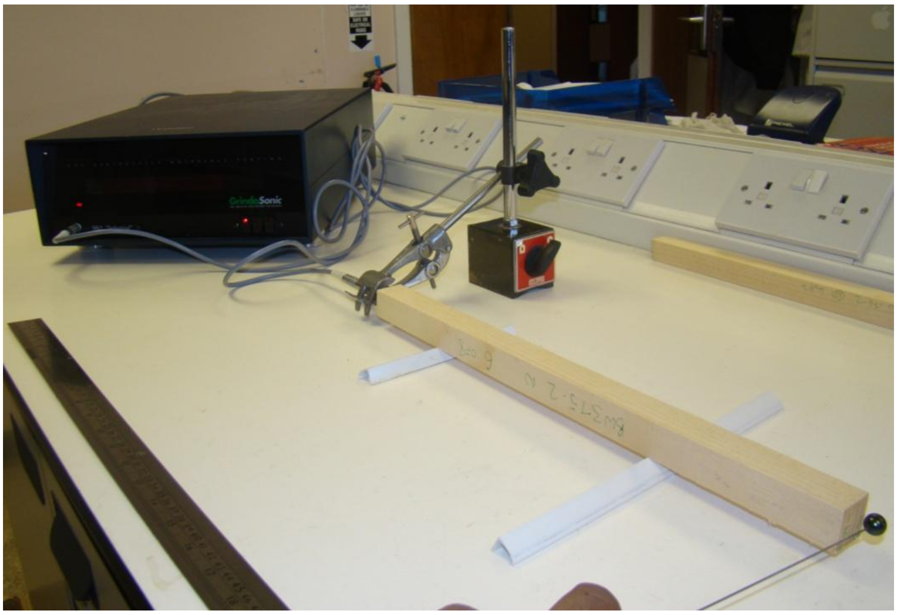


Figure 7-3 Test setup of clear wood specimens to determine the modulus of elasticity

7.2.2 Test Procedure of Shear Modulus

All test joists and clear wood specimens were tested under torsion using torsion tester and measuring relative twist of specimens by inclinometers, as mentioned in Chapter 03 and Chapter 04. Test joists were tested in such that shear modulus of consecutive 600mm sections (G_{600mm}) along the length were obtained by mounting multiple inclinometers on top surface of joists. The measurement of rotations of three consecutive 600mm sections also allowed attaining the shear modulus of 1800mm segment ($G_{1800mmm}$). It should be noted that E_{Local} and G_{600mm} represent the same segments and that E_{Global} and G_{1800mm} represent the same 1800mm sections of the joist. The shear modulus joist span (G_{Span}) and of clear wood (G_{CW}) were obtained by measuring rotations of inclinometers mounted nearer the supports as discussed in Chapters 05 and 06 respectively.

7.3 Results and Discussion

7.3.1 Relationship of Modulus of Elasticity and Shear Modulus

To examine the relationship, a linear regression analysis between modulus of elasticity and shear modulus was conducted for joist span to examine the correlation for a structural size timber member. Then, the regression analysis of 1800mm and 600mm sections of joists and of clear wood was carried out. Figure 7-4 shows the correlation that was developed for spans of all tested Sitka spruce and Norway spruce joists. No correlation (R^2 = 0.07) was found between shear modulus and modulus of elasticity for full size structural timber joists. Although modulus of elasticity values were ranged from 5000 to 16000 MPa and the shear modulus values were ranged from 400 to 800 MPa. As mentioned, E_{Span} was achieved acoustically by testing entire sample but G_{Span} was obtained by testing joist span which did not include the clamp end distances (e.g. 300mm each side of 3.6m joists). This might have some influence on correlation and, therefore, a further correlation between modulus of elasticity and shear modulus was examined for 1800mm and 600mm sections.

In this regard, Figure 7-5 gives the correlation between E_{Global} and G_{1800mm} of 1800mm section. Figure 7-6 provides correlation between E_{Local} and G_{600mm} of 600mm segments. It can be noticed clearly that there is no relationship between modulus of elasticity and shear modulus when both mechanical properties correlated at 1800mm (R^2 = 0.041) and at 600mm (R^2 = 0.042). This demonstrates that both modulus of elasticity and shear modulus are independent of each other regardless the length of structural size timber.

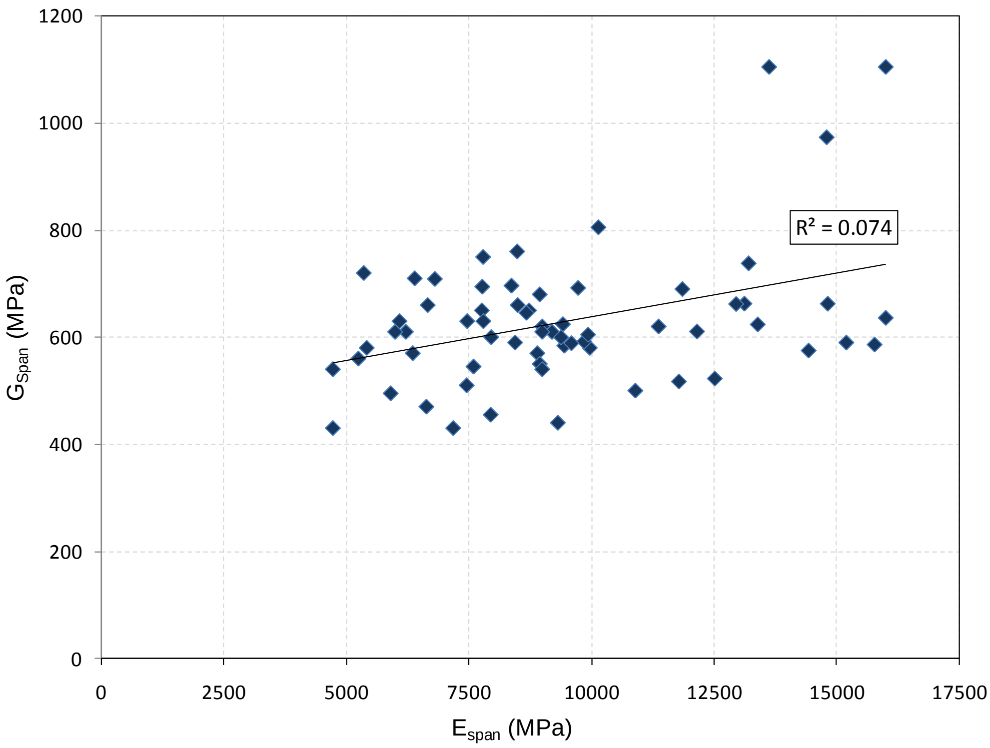


Figure 7-4 Correlation of E_{span} and G_{span} for all tested timber joists.

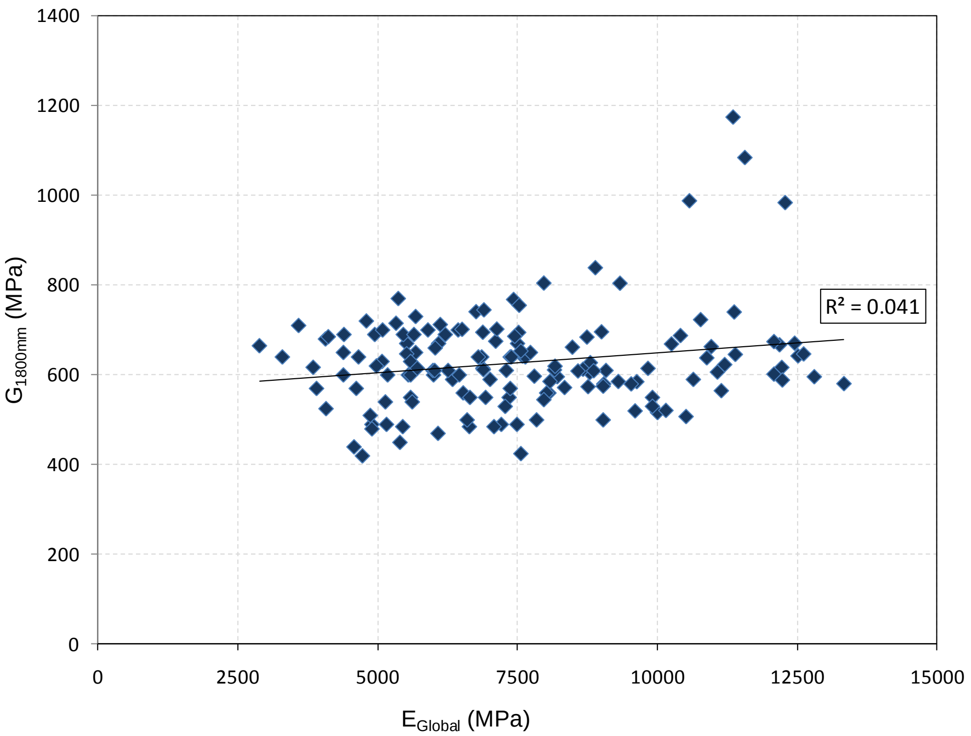


Figure 7-5 Correlation of E_{Global} and G_{1800mm} of all tested timber joists

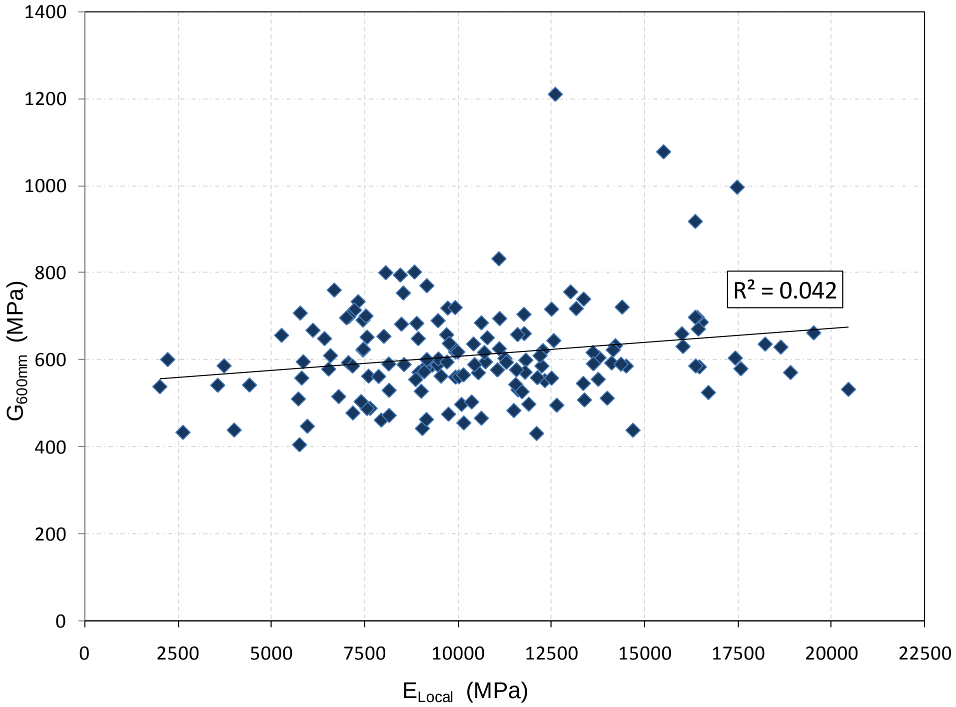


Figure 7-6 Correlation of E_{Local} and G_{600mm} of all tested timber joists

As no correlation was found at structural size timber, a correlation between modulus of elasticity (E_{CW}) and shear modulus (G_{CW}) of clear wood was examined. Structural size timber may have wood defects and it may be possible that wood defects cause some effect on the correlation. Therefore, correlation between E_{CW} and G_{CW}, given in Figure 7-7, may provide information in this regard. However, no evidence of any correlation (R^2 = 0.002) was seen when modulus of elasticity and shear modulus was compared at clear wood level. It can be noticed E_{CW} and G_{CW} values have a wide spectrum as they were ranged from 2000 to 22000MPa and from 400 to 1200MPa, respectively and this indicates that correlation was not conducted at small ranged values. This because small range values may not provide a better relationship as values more scattered within the range. The large spectrum values, in general, represent a better correlation as the values are less scattered within the range.

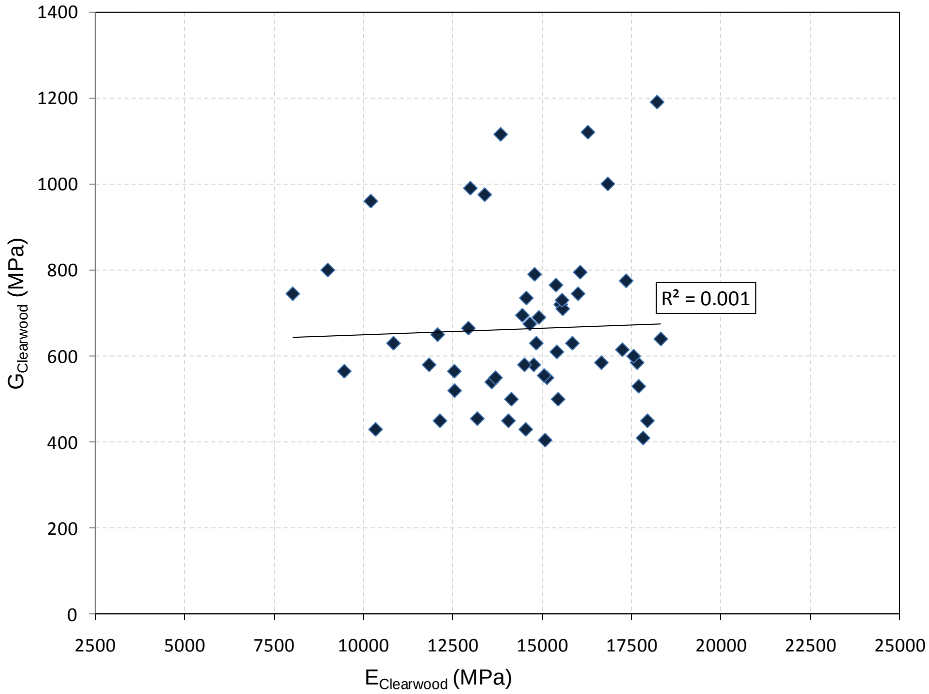


Figure 7-7 Correlation of modulus of elasticity and shear modulus of clear wood

The above investigation showed that modulus of elasticity and shear modulus have no any correlation from structural size joists to small clear wood. However, the correlation was representing the modulus of elasticity and shear modulus of two species which might have altered the actual correlation. Therefore, correlation of modulus of elasticity and shear modulus was examined for individual species which may assist in finding if the relationship has been influenced by merging Sitka spruce and Norway spruce species modulus of elasticity and shear modulus. To this, Figure 7-8 and Figure 7-9 show the correlation between modulus of elasticity and shear modulus for Sitka spruce and Norway spruce joists, respectively. The correlations were conducted for joist spans, 1800mm and 600mm sections. It can be observed that there is no correlation between modulus of elasticity and shear modulus as R^2 values for both species were negligible small. This implies that modulus of elasticity and shear modulus are not correlated with each other when compared for individual species.

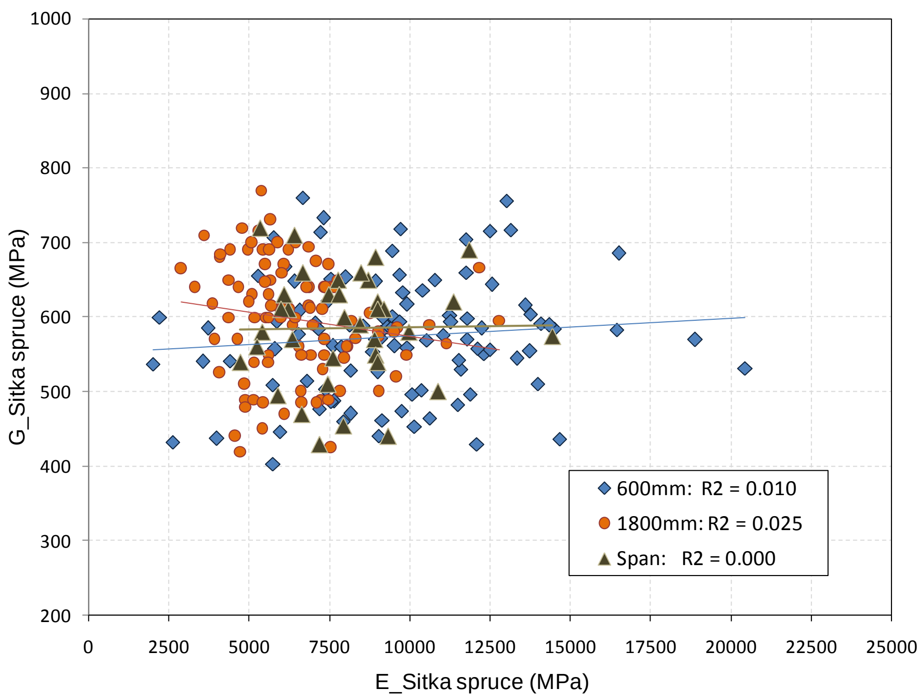


Figure 7-8 Correlation of modulus of elasticity and shear modulus for Sitka spruce joists for span, 1800 and 600mm sections.

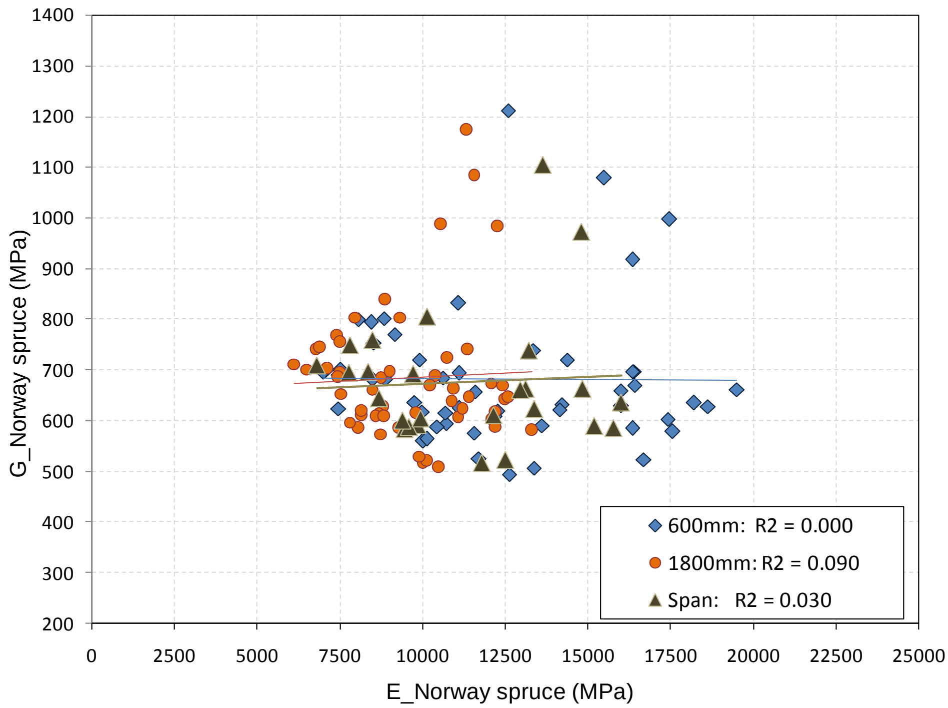


Figure 7-9 Correlation of modulus of elasticity and shear modulus for Norway spruce joists at span, 1800 and 600mm sections.

The correlation between modulus of elasticity and shear modulus was examined from Span of structural joists, sections within joist span and at small clear wood level. The correlations were developed within wide spectrum values of modulus of elasticity and shear modulus of combined two species and on basis of individual species. It was found that both mechanical properties are independent of each other. This indicates of size of timber, presence of knots, shakes, slope of grain and other wood defects do not influence on the correlations. It should be noted that, shear modulus was obtained from torsion test and influence of clockwise and anti clockwise directions, influence of time history and influence of repetitive testing were taken into account to minimize any possible experimental aspects that could have cause affect on correlations.

The main reason of no correlation was found because modulus of elasticity measures from normal stresses that act perpendicular to cross-section of wood and that wood fibres elongate or shortened along longitudinal directions. The shear modulus obtains from shear stresses act tangential to the cross-section of the material and these stresses distort the shape only and that no elongation or shortening takes place in the member. A correlation can be developed between modulus of elasticity and shear modulus based on Poisson ratio if the material itself is an isotropic using Equation (7-5). However wood is an orthotropic material and that have different mechanical properties in its longitudinal, tangential and in radial direction and, therefore, the approach in Equation (7-5) may not be applicable. This can be examined from Figure 7-10 and Figure 7-11.

$$E = G\,2(1 + \vartheta) \qquad (7\text{-}5)$$

From Figure 7-10, it can be seen that if the joists are tested for shear modulus by inducing torsional loads in parallel to longitudinal-radial (LR) plane then the correspondent modulus of elasticity was achieved by applying loads in longitudinal-tangential (LT) plane and vice versa as in Figure 7-11. Sitka spruce and Norway spruce are softwood species and softwood is stiffer in LT plane in relative to LR plane. This is because softwood primarily composed of long-thread like tracheids (cell wall) and these tracheids are fairly uniform in dimension and are in layered form of earlywood and latewood in LT plane and makes wood stiffer in LT plane. In the LR plane, these tracheids have large dimensional variation and are in asymmetrical form of earlywood and latewood which causes lower stiffness of wood in the plane. Therefore, most joists when were tested for shear modulus in LT plane in that case modulus of elasticity was obtained in LR plane and Vice versa. Since both properties were attained from applying loads in two different plane of wood and may be the reason for no correlation.

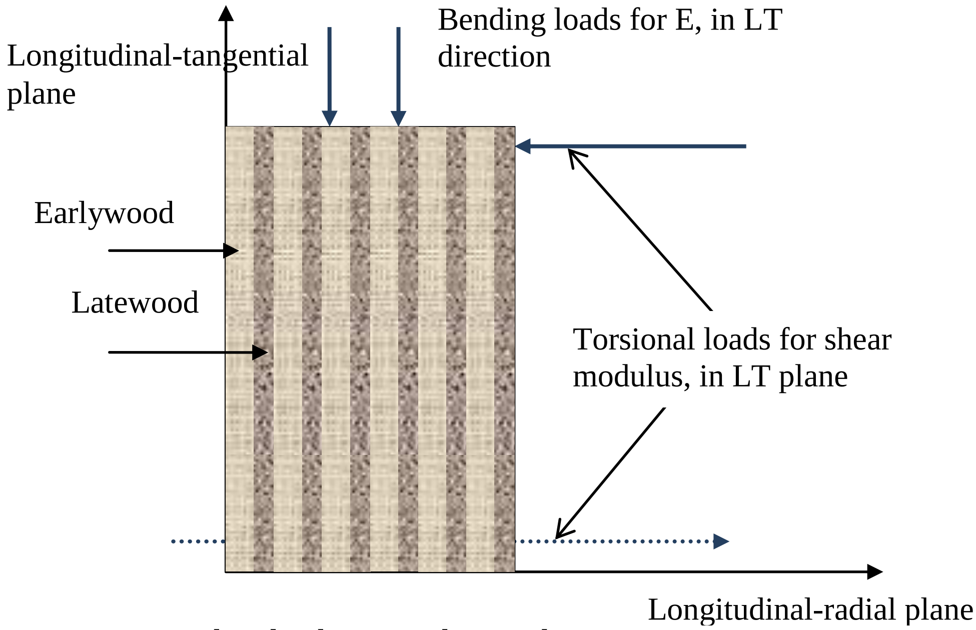


Figure 7-10 Bending loads in LT plane and torsional loads in LR plane

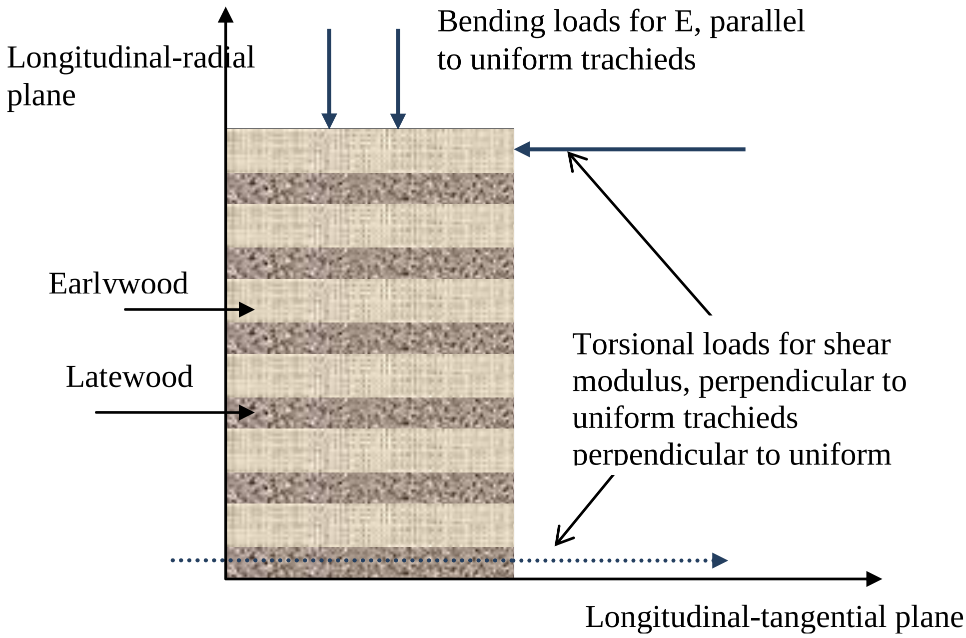


Figure 7-11 Bending loads in LT plane and torsional loads in LR plane

7.3.2 Modulus of Elasticity and Shear Modulus Ratio

In this study it was found that shear modulus and modulus of elasticity do not have any correlation. However, in timber design, shear modulus of timber mainly obtained on basis of E G ratio of 16 to 1, as discussed in Chapter 02. Therefore, this work also modulus of elasticity to shear modulus ratio and compared it E G 16:1 ratio to assess if 16:1 is appropriate to obtain the shear modulus. For this, ratio of modulus of elasticity and shear modulus of joist span, 1800mm (Global) and 600mm (Local) sections were determined. Table 7-1 details the mean, minimum and maximum E to G ratios of local, global and span level and are categorised in tested joists, species, strength grade and clear wood. In Table 7-1, ratios for Global modulus of elasticity (E_{Global}) and shear modulus is highlighted as most often E_{Global} is considered for predicting shear modulus.

Table 7-1: Ratio of modulus of elasticity to shear modulus for tested joists.

Group			Modulus of elasticity to shear modulus ratio								
			Mean			Minimum			Maximum		
			E G ratio	E	G	E G ratio	E	G	E G ratio	E	G
All tested joists		Span	15:1	9420	625	7:1	4730	430	27:1	16000	1100
		Global	**12:1**	7590	630	**4:1**	2890	420	**23:1**	13330	1175
		Local	18:1	10550	610	4:1	2010	400	39:1	20450	1210
Species	SP	Span	14:1	8020	590	7:1	4730	430	25:1	14440	720
		Global	**11:1**	6500	600	**4:1**	2890	420	**21:1**	12800	770

		Local	17:1	9550	575	4:1	2010	400	39:1	20450	760
	NS	Span	17:1	11360	680	10:1	6810	520	27:1	16000	1100
		Global	**15:1**	9660	690	**9:1**	6120	510	**23:1**	13330	1175
		Local	19:1	12410	685	10:1	7010	500	32:1	19520	1210
Strength Grade	C16	Span	16:1	9200	590	7:1	4730	430	27:1	16000	760
		Global	**12:1**	7330	600	**4:1**	2890	420	**23:1**	13330	800
		Local	18:1	10470	585	4:1	2010	400	39:1	20450	800
	C24	Span	14:1	10320	760	10:1	6810	600	20:1	14800	1110
		Global	**12:1**	8940	765	**9:1**	6120	610	**17:1**	12280	1175
		Local	15:1	11000	760	10:1	7010	610	25:1	17470	1210
Clear wood			**23:1**	14500	665	11:1	8020	410	43:1	18320	1190

This research found a lower mean E_{Global} to G_{1800mm} ratio with respect to all tested joists, species and strength grade when were compared to E-G ratio of 16:1. This can be observed in Table 7-1 as joists of both Sitka spruce and Norway spruce species produced E_{Global} to G_{1800mm} ratio of 12:1. Sitka spruce joists produced the lowest E_{Global} to G_{1800mm} ratio of 11:1 and a higher ratio of 15:1 was attained when only Norway spruce joists were taken into account. Also, strength grades of C16 (includes of Sitka spruce and Norway spruce) and C24 gave a lower ratio of 12:1 and 14:1, respectively, for of E_{Global} to G_{1800mm}. This indicates that E to G ratio is not a constant value and varies according to wood species and strength grade. A higher variation was also noticed in E to G ratio when modulus of elasticity and shear modulus was considered for local and span. It was found that E_{Local} to G_{600mm} provided a higher ratio of about 19:1 for all categories compare E G ratio of 16:1. However, E_{Span} to G_{Span} produced ratios closer to E G ratio of 16:1.

It was seen that clear wood produced the highest mean modulus of elasticity to shear modulus ratio of 23:1. This is significantly higher than E G ratio of 16:1 and E_{Global} to G_{1800mm} of 12:1 tested joists. This may suggest that the ratio from clear wood may underestimate the shear modulus value if modulus of elasticity value of clear wood is used to predict. It was noticed that E to G ratio has a high variation as for the most joist groups the minimum to maximum ratio of E to G was from 4:1~10:1 to 21:1 ~ 39:1. A slightly higher variation was seen for clear wood as E G ratio from minimum to maximum of 11:1 to 43:1 was obtained. A higher variation in E to G ratio was also found by various research works. A higher variation of E G ratio from 8:1 to 43:1 was also observed by Chui (1991) when he tested small clear wood. Harrison (2006) found that E to G ratio was varying from 10:1 to 26:1 for structural size joists.

This concludes that predicting shear modulus from modulus of elasticity is not appropriate approach. In particular, E_{Local} may also not be considered to predict shear modulus as E_{Local} and G_{600mm} represents smaller sections of joists and that minimum to maximum ratio was varied from 4:1 to 39:1. E_{Span} seems better approach as E_{Span} and G_{Span} represents the values of actual joist and produced mean ratio of 15:1, closer to E G ratio of 16:1. However, E_{Span} was obtained on basis of acoustic testing method and the method is not often used for obtaining the modulus of elasticity. Therefore, it is more appropriate to consider E_{Global} to G_{1800mm} ratio to compare with the current E G ratio of 16:1. CEN (EN338:2008, 2008) standard test method provided design shear modulus values that were determined from E_{Global} of the joist. It can be seen from Table 7-1 that E_{Global} to

G_{1800mm} ratio was found about 12:1 and is considerably lower than currently used ratio of 16:1. This may indicates that E G ratio of 16:1 approach is not appropriate because this may not provide the actual shear modulus.

The shear modulus is an important factor in general timber design, specifically for design of lateral torsional stability and serviceability limit state design. The advent of engineered wood products allows producing long, thin and deep joists and these joists are more often used as continuous member with no lateral supports. These long continuous joists are more prone to getting horizontal deflection along with vertical deflections under loads and may experience lateral buckling. The lateral buckling becomes more unavoidable if these joists have no bracing and or act as overhanging members. As a result, these joists must be designed to have adequate lateral torsional stability. The shear modulus is one of the key factors to provide satisfactory torsional stability.

The shear modulus is also used in serviceability limit design for timber. The criteria for satisfactory deflection is mainly depend on modulus of elasticity and the shear modulus. Also, shear modulus is very important factor to design for vibrational serviceability of wood based floors. Various finite element models (e.g. Chui, 2002) requires shear modulus values as a one of the input parameters. Therefore it is very essential to obtain shear modulus using appropriate method, such as torsion tests as current approach of using E G ratio of 16:1 may not provide the actual values of the shear modulus. The E G of 16:1 ratio was developed on the basis of testing small clear wood under plate bending and plate twisting by Bodig and Goodman (1973). The 16:1 may be applicable for short conventional solid timber joists but not within the current construction approaches which allow fabricating of long, thin and deep joists. Therefore, 16:1 approach may be not applicable with the modern construction practice.

An earlier version of CEN (EN408:2003, 2003) provided a procedure for shear modulus based on modulus of elasticity that obtained from flexural tests. However, recent draft of CEN (EN408:2009, 2009) has included the torsion test procedure for evaluating the shear modulus. This research also endorsed the recommendation of CEN of using torsion test for obtaining the shear modulus values of timber. Yet the design shear modulus values in CEN (EN338:2008, 2008) are still calculated on the basis of E G 16:1 ratio approach. Therefore, there is a need to obtain general design values of shear modulus by employing torsion test approach and by testing a range of various species. Also, a system can be implicated in Machine Stress-Rated (MSR) system in that addition to modulus of elasticity, the shear modulus of lumber can be measured by inducing small twists in timber and measuring relative rotations. It can be possible to test clear wood to obtain the general shear modulus values of strength grades but in this research it was found that clear wood may provide higher shear modulus values in compare to actual structural lumber.

7.3.3 Relationship of Local and Global Properties

In this section, a correlation between E_{Local} and E_{Global}, as well as, between G_{600mm} and G_{1800mm} was examined. This was conducted because it was found that both shear modulus and modulus of elasticity were not correlated with each other. Therefore, it became more important to find if both properties are correlated within themselves.. For 3.6m Sitka spruce, correlations were developed between three sections (S2, S3 and S4, as described in Chapter 04) of 600mm and relative 1800mm sections. Two segments (S2 and S3) of 600mm and relativev1800mm were considered for 2.8m Sitka spruce and for Norway spruce. In this regard, Figure 7-12 provides the correlation between G_{600mm} and G_{1800mm} for S2, S3 and S4. It should be noted that S4 only represents the 3.6m Sitka spruce joists. It can be seen that there is a strong correlation between G_{600mm} and G_{1800mm} for S2 and S3 (R^2 = 0.92) and slightly lower correlation (R^2 = 0.65) for Segment 4. The good correlation suggest that G_{1800mm} mainly driven from G_{600mm} sections.

Table 7-2 gives the average shear modulus of 600mm and relative 1800mm sections. It can be noticed that s mean G_{600mm} and G_{1800mm} values are the same. This may indicate that shear modulus is uniform along the length of a joist. However, it was found that shear modulus has a considerable variation within segments of tested joists (as described in Chapter 04). The variation in shear modulus was not observed here because when joists were tested for 1800mm the test measurements covered a larger region of the joists which overlapped the measured G_{600mm} of various sections includes of a sections with varied shear modulus. Also the variation was seen within the segments of individual joist, whereas, this correlation was developed on the basis of average G_{600mm} and G_{1800mm} of a segment of all tested joists.

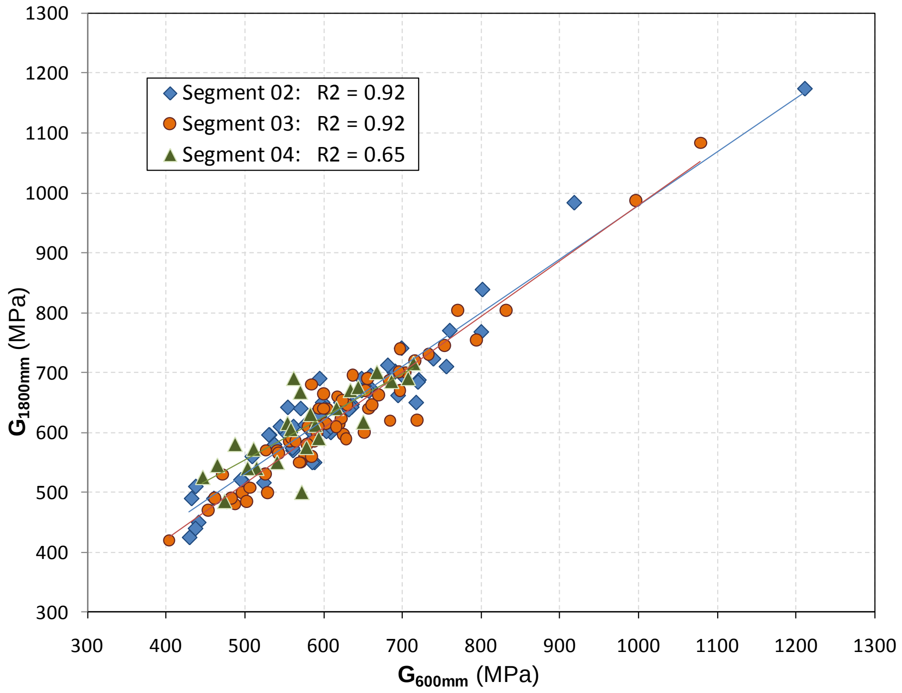


Figure 7-12 Correlation between shear modulus of 600mm and 1800mm sections

A weak correlation was found between local and global modulus of elasticity as shown in Figure 7-13. Although a strong correlation for segment 03 ($R^2 = 0.64$) was attained but a weak ($R^2 = 0.15$) was found for segment 04. This may be because shear deformation was being taken into account when E_{Global} was obtained, whereas, E_{Local} was obtained on the basis of pure bending. The influence of shear deformation can be examined from Table 7-2 as about 40% higher mean E_{Local} value (10550 MPa) was obtained in compare to mean E_{Global} value of 7540 MPa. The higher values for E_{Local} was obtained mainly because E_{Local} was calculated from shorter sections and that may not account the effects of wood defects entirely as was accounted for the E_{Global} as it covers a larger portion of the joists.

Table 7-2: Test values of shear modulus and of modulus of elasticity of joists

<table>
<tr><th>Properties</th><th>Segment 1</th><th>Segment 2</th><th>Segment 3</th><th>Segment 4</th><th>Segment 5</th><th>Mean</th></tr>
<tr><td>G_{600mm}</td><td></td><td>620</td><td>620</td><td>580</td><td></td><td>610</td></tr>
<tr><td rowspan="3">G_{1800mm}</td><td colspan="3">640</td><td colspan="2"></td><td rowspan="3">630</td></tr>
<tr><td></td><td colspan="3">630</td><td></td></tr>
<tr><td></td><td></td><td colspan="3">610</td></tr>
<tr><td>E_{Local}</td><td></td><td>10500</td><td>10750</td><td>10200</td><td></td><td>10550</td></tr>
<tr><td rowspan="3">E_{Global}</td><td colspan="3">7720</td><td colspan="2"></td><td rowspan="3">7540</td></tr>
<tr><td></td><td colspan="3">7750</td><td></td></tr>
<tr><td colspan="2"></td><td colspan="3">6860</td></tr>
</table>

The other reason of weak correlation was may be due to variation within local elasticity as it was found that E_{Local} varies significantly and within the segments. This can be examined from another correlation that was developed between segment 2 and segment 3 for both E_{Local} and E_{Global}, as shown in Figure 7-14. It can be seen that E_{Global} have high correlation ($R^2 = 0.90$) between S2 and S3 but a weak correlation was obtained ($R^2 = 0.30$) for E_{Local}. The result shows that E_{Global} values were more consistence mainly because covers larger sections of the joists which overlap the segments. The E_{Local} have a lower correlation because of the variation in E_{Local} within the segments. Ridley-Ellis et al. (2008) also found that local modulus of elasticity have a higher variation within joist length and have a lower correlation with global modulus of elasticity. This may also raise the question to whether estimating shear modulus from bending tests is valid or not.

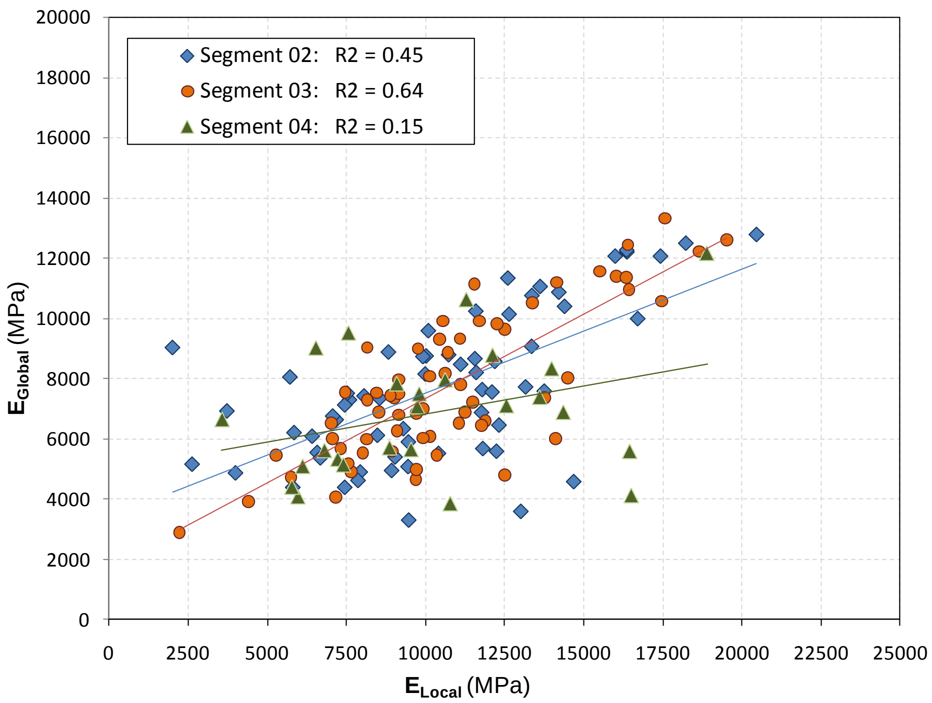


Figure 7-13: Correlation between local modulus of elasticity and global modulus of elasticity

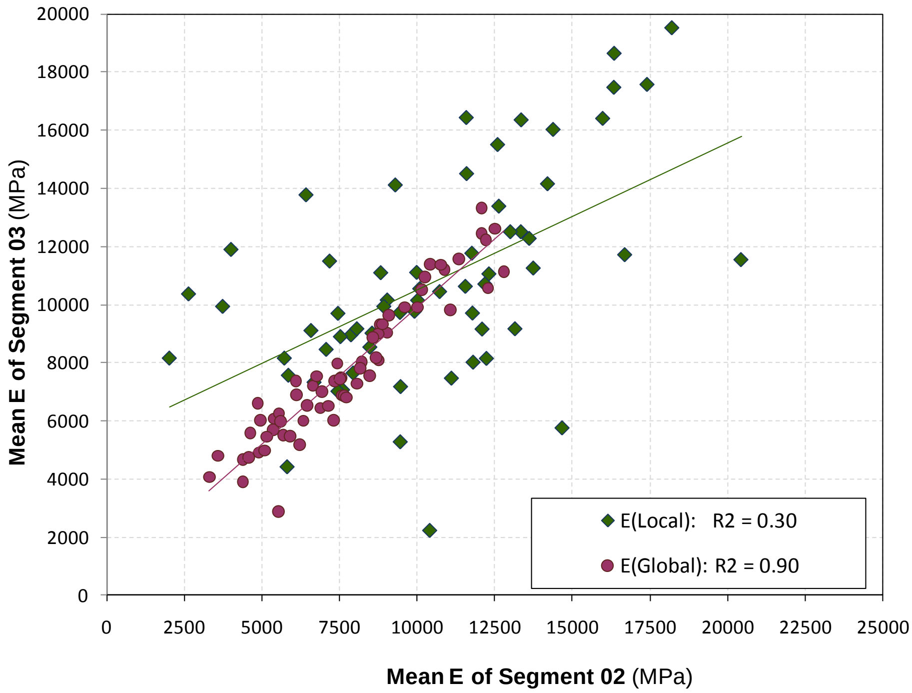


Figure 7-14: A correlation between segment 02 and segment 30 for E_{Local} and E_{Global}

7.4 Summary

In this chapter a relationship of shear modulus and modulus of elasticity was examined. The shear modulus was obtained using torsion test method and modulus of elasticity was achieved using four-point bending tests. The correlation of modulus of elasticity and shear modulus was developed from small clear wood to structural size timber. For this, Structural size Sitka spruce and Norway spruce joists and small clear Sitka spruce specimens were tested. No correlation between modulus of elasticity and shear modulus was obtained when both properties were compared for joist span, within longer and shorter sections of joists and for small clear wood. The results from this study leads an issue to whether or not it is appropriate to obtain shear modulus from flexural tests or from E:G ratio of 16:1.

An E:G ratio from this study was also examined. It was seen that modulus of elasticity and shear modulus produced a lower E:G ratio and that the ratio is not constant and varies within strength grades and species. A correlation between shear modulus values of 1800mm and 600mm joist sections was also observed. A good correlation was found within the shear modulus values, however, correlation between local and global modulus of elasticity has good agreement at some extent. This may lead that if it is appropriate to drive shear modulus from modulus of elasticity based on variable and single span methods as recommended by CEN (EN408:2003, 2003).

8. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

This thesis investigated the use of the torsion test method to evaluate the shear modulus and shear strength of timber joists. The torsion test is a better approach to determine the shear properties because it induces only shear stresses and creates a purer state of shear in the timber joists. However, the torsion test is rarely used due to a lack of information available for proper use of the method. Therefore, this research study was undertaken to provide a more comprehensive understanding of the use of torsion test to evaluate the shear properties of timber.

The research was mainly focused on an experimental study of torsion on solid timber joists and small clear wood beams. Full-size Sitka spruce and Norwegian Spruce joists of structural grade of C16 and C24 were used. A torsion tester was employed to apply torque and relative twists were measured from inclinometers mounted on the topside of the joists. The shear modulus was evaluated by testing joists under torsion within the elastic range. To identify the elasticity limit, preliminary torsion tests were conducted on joists until they fractured.

To understand more about torsion, joists were tested in clockwise and anti-clockwise direction to determine the influence of the direction of torque and of spiral grain on shear modulus. Furthermore, the applicability of torsion test to account of variation in shear modulus was also examined. This was conducted by mounting multiple inclinometers along the length of joists and inducing torque within the elastic range. The influence of knots on shear modulus was also determined using total knot area ratio (TKAR) (BS4978:2007, 2007).

The same test joists were re-tested till they ruptured and shear strength was obtained from maximum applied torque. The relationship between shear strength and shear modulus was examined to assess if the properties are correlated (same test approach). Various fracture types were observed under torsion. The fracture types were categorized into four failure modes based on initiation and location of cracks and the crack path. Fracture location was also taken into account to examine if the shear modulus has substantial variation at the fracture location. Torsion tests were also undertaken on clear wood to obtain the shear properties of defect free wood.

The correlation between the shear modulus and the modulus of elasticity is also another important aspect investigated in this research. In current timber design, the shear modulus is mainly determined from the modulus of elasticity, therefore, this work was conducted to identify any relationship between these two properties. The modulus of elasticity of test joists and small clear wood was obtained using four-point bending and acoustic test methods. The correlation of modulus of elasticity and shear modulus was developed for joist span, various sections within joists and for small clear wood. This research yielded numerous results and findings that are summarized in the following section.

8.2 CONCLUSIONS

The main conclusion of this research is that torsion is an appropriate test method to evaluate the shear modulus and shear strength of timber joists. The above conclusion is made on the basis of the following key results from this study:

1. This study found that torsion produces up to a 15% higher mean shear modulus of joist in comparison to the published values in CEN (EN338:2008, 2008) that were determined from modulus of elasticity.

2. The torsion test permits the measurement of variation of shear modulus within a single joist. It was found that Sitka spruce joists (obtained locally) have a higher variation in shear modulus in compare to Norway spruce (commercial) joists. A variation up to 30% was found in shear

modulus within sections in Sitka spruce joists. Norway spruce joists revealed a variation in up to 20% within the sections along the length.

3. The application of torque in either clockwise or anti-clockwise direction does not influence shear modulus substantially. This may suggest that torque can be applied in either direction.

4. These results suggest, but provide inconclusive evidence, that knot size does not have substantial influence on the shear modulus. It was seen that there was no correlation between shear modulus and TKAR within a joist segments and that variation in shear modulus was independent of the TKAR.

5. The torsion test was found to be a more appropriate approach to determine the shear strength of timber joists. It was noticed that torsion tests produced up to 200% higher characteristic shear strength values for different species in comparison to published values in CEN (EN338:2008, 2008). This raises serious concerns on the adoption of shear strength values on the basis of bending strength of joists. Also, torsion test provided about 13% higher mean shear strength values than those published in the Wood Handbook (USDA, 1999).

6. This research concludes that torsion test yields predominantly shear failure in the joists and subjects the joists in state of pure shear. This is because it was found that the fractures were commonly initiated within clear wood and propagated parallel to the longside of joists where shear stresses were presumed to be maximum under applied torque.

7. Based on the results from this study, fractures were categorized into four failure modes. Support conditions were found to be important as 40% of test joists were fractured at supports by crushing (crushing failure) mainly due to induction of additional stresses by testing clamps. In some joists, it

was seen that cracks started due to large shear and then propagated along long side as if being pull open like a tensile failure. Also, shear stresses initiate cracks due to a knot and propagated diagonally along the long side to rupture the specimen in shear due to another knot (shear failure). In most of Norwegian spruce shear cracks initiated from clear wood and travelled horizontally along long side towards end supports and ruptured the joists (horizontal shear failure)

8. Both shear modulus and shear strength indicated a good correlation to a degree within the grades C16 and C24 and within various failure modes. The correlation also demonstrates that the elastic region of joists lies between 5% and 25% of maximum applied torque was found be proper to evaluate the shear modulus.

9. This investigation reveals a good correlation between the fracture location within a joist section and the shear modulus values. This may lead to the prediction of the location of fracture by examining the variation in shear modulus along the length of joists.

10. No correlation was found between shear modulus and modulus of elasticity. The correlation was examined from a small clear wood to a full-size timber joists. It was also found that E:G ratio is not a constant value but it changes with the type of species and structural grades.

8.3 RECOMMNEDATIONS

1. This investigation found that shear strength and shear modulus can be obtained from torsion tests. It is, therefore, proposed that test standards allow the torsion test to be used to determine shear modulus and shear strength. This work also endorsed the inclusion of torsion test by CEN (EN408:2009, 2009) to evaluate the shear modulus of joists. However, CEN (EN408:2009, 2009) does not adequately address the test procedure

in greater details. This work provides inclusive information on the use of torsion to evaluate the shear properties and this could be included in standard test methods such as CEN (EN408:2009, 2009).

2. The shear modulus design values become significantly important in current timber design practice, more specifically for lateral torsional stability and vibrational serviceability of wood floors. However, the shear modulus have been determined from modulus of elasticity and E:G ratio of 16:1. This research found no evidence that the shear modulus is related to modulus of elasticity and that E:G ratio is not a constant value. Therefore, it is strongly recommended that shear modulus must be attained from torsion test, especially for thin and deep joists when used as continuous beams and with no lateral supports.

3. Previous research work paid less attention on the fracture mechanism of wood and no information was available about the failure modes under torsion. This investigation provides an in-depth detail on the fracture of joists under torsion and suggests four general failure modes. The failure modes can be generalized for design purposes and used as a guideline for future investigations on torsion.

4. Support conditions were found to be important. It was noticed that test clamps induced additional compressive stresses which lead to crushing of the wood at the supports and premature failure for some joists. Therefore, it is important to design such test clamps so that they minimise the localised compressive stresses.

5. This investigation was limited to tests of solid timber joists. Further research needs be conducted to evaluate the shear properties of I-joist and glued laminated timber beams. This research has shown that shear modulus varies along the length and shear cracks more likely occurs at the middle of the longside. In timber construction, very long and deep with

thin web I-joists are often used. The thin and deep web becomes more crucial as it is the only member that most likely takes the shear stresses under the normal loading condition. Assigning of the shear properties on basis of bending of I-joists for design may not be a suitable approach and may lead to a premature shear cracks in the web, especially when I-joists are as a continuous beam with no lateral supports. Therefore, an urgent investigation is required to evaluate the shear properties of I-joists using torsion tests.

6. This study found that shear modulus decreased substantially at the fracture locations. This suggests that shear modulus values should be estimated with caution and that it may be potentially dangerous to assign timber to a grade on the basis of the modulus of elasticity obtained from a strength grading machine as it does not necessarily confirm that shear modulus values are adequate for the grade. A system can be implicated in Machine Stress-Rated (MSR) system, in that in addition to the modulus of elasticity, the shear modulus of lumber can be measured by inducing small torque in timber and measuring relative rotations.

7. Torsion tests on clear wood to obtain the shear properties may not be an appropriate approach as in this research it was found that clear wood tests gave higher shear modulus values in compare to full-size joists.

8. It will be useful to test timber joists with higher spiral direction grains to examine whether torque induced in clockwise and in anti-clockwise direction influence the shear properties.

9. It was found that based on TKAR, knot size seems have no substantial influence on shear properties of timber. TKAR may not be a suitable procedure to measure knot size, therefore, this needs more attention to measure knot size with a more appropriate method.

10. This research work may assist in developing finite element models to for further study of timber joists under torsion at a larger scale. Also, this work may lead to large scale experimental work to determine the shear modulus and shear strength of various wood species.

11. It would be of high interest to further investigate the correlation of modulus of elasticity and shear modulus for various types of timber joists. Also, there is a need to develop an analytical model that can assist to provide a better understanding of the correlation and can also incorporate of wood defects and orthotropy of wood.

REFERENCES

ASTM-D143-94. (1996). *Standards method for testing small clear specimen of timber*. American Society of Testing and Materials, West Conshohcken, PA, USA.

ASTM-D198-94. (1996). *Standard methods of static tests of lumber in structural size.* American Society for Testing and Materials, West Conshohocken, PA, USA.

Biblis, E. J. (1965). Shear deflection of wood beams. *Forest Products Journal , 15* (11), 492-498.

Bodig, J., & Goodman, J. R. (1973). Prediction of elastic parameters for wood. *Wood Science , 05* (04), 249-264.

Boresi, A. P., & Schmidt, R. J. (2003). *Advanced mechanics of materials.* USA: Jon Wiley and Sons Inc.

Brandner, R., Gehri, E., Bogensperger, T., & Schickhofer, G. (2007). Determination of modulus of shear and elasticity of glued lamineted timber and related examination. *International Council for Research and Innovation in Building and Construction, Working Commision W18-Timber Structures (CIB W18)*, (pp. 40-12-2). Bled, Slovenia.

BS373:1957. (1957). *Methods of testing small clear specimens of timber.* British Standard, London, UK.

BS4978:2007. (2007). *Visual strength grading of softwood - Specification.* British Standards, London, UK.

BS4978:2007. (2007). *Visual strength grading of softwood-specification.* British Standard, London, UK.

Burdzik, W. M., & Nkwera, P. D. (2003). The relationship between torsional rigidity and bending strength characteristics of SA pine. *Southern African Forestry Journal* , 17-21.

Chui, Y. H. (2002). Application of ribbed-plate theory of predict vibrational serviceability of timber floor systems. *The Proceeding of the 7th World Conference on Timber Engineering WCTE*, *4*, pp. 87-93. Shah Alam, Malaysia.

Chui, Y. H. (1991). Simultaneous evaluation of bending and shear moduli of wood and influence of knots on these parameters. *Wood Science and Technology* , *25* (2), 125-134.

Cofer, W. F., Proctor, F. D., & Mclean, D. I. (1997). Prediction of the shear strength of wood beams using finite element analysis. *Mechanics of Cellulosic Material ASME* , *AMD-Vol 221/MD-Vol 77*, 69-78.

Divos, F., Tanaka, T., Nagao, H., & Kato, H. (1998). Determination of shear modulus on sonstruction size timber. *Wood Science and Technology* , *32* (06), 393-402.

EN338:2003. (2003). *Structural timber - Strength classes.* European Committee for Standardization, Brussels, Belgium.

EN338:2008. (2008). *Timber structures - strength classes.* European Committee for Standardization, Brussels, Belgium.

EN384:2003. (2003). *Structural timber-determination of characteristic values of mechanical properties and dnesity.* European Committee for Standardization, Brussels, Belgium.

EN384:2008. (2008). *Structural timber - Determination of characteristic values of mechanical properties and density*. European Committee for Standardization, Brussels, Belgium.

EN408:2003. (2003). *Timber structures-structural timber and glued laminated timber-determination of some physical and mechanical properties*. European Committee for Standardization, Brussels, Belgium.

EN408:2009. (2009). *Timber structures - structural timber and glued laminated timber - determination of some physical and mechanical properties perpendicular to the grain*. European Committee for Standardiztion, Brussels, Belgium.

Foschi, R. O., & Barrett, J. D. (1975). Longitudinal shear strength of Douglas-fir. *Canadian Journal of Civil Engineering , 03* (02), 198-208.

Gorlacker, R., & Kurth, J. (1994). Determination of shear modulus. *International Council for Research and Innovation in Building and Construction, Working Commision W18-Timber Structures (CIB W18) Meeting 27*. Sydney, Australia.

Gunnerson, R. A., Goodman, J. R., & Bodig, J. (1973). Plate tests for determination of elastic parameters of wood. *Wood Science , 05* (04), 241-248.

Gupta, R., & Siller, T. (2005a). Shear strength of structural composite lumber using torsion tests. *Journal of Testing and Evaluation , 33* (02), 100-117.

Gupta, R., & Siller, T. (2005b). Stress distribution in structural composite lumber under torsion. *Forest Products Journal , 55* (55), 51-56.

Gupta, R., Heck, L. R., & Miller, T. H. (2002a). Experimental evalutionof the torsion test for determining shear strength of structural lumber. *Journal of Testing and Evaluation , 30* (04), 283-290.

Gupta, R., Heck, L. R., & Miller, T. H. (2002b). Finite-element analysis of the stress distribution in a torsion test of full-size, structural lumber. *Journal of Testing and Evaluation , 30* (04), 291-302.

Harrison, K. S. (2006). Comparison of shear modulus test methods. *MSc Thesis* . Blacksburg, VA, USA.

Hindman, D. J., Manbeck, H. B., & Janowiak, J. J. (2001). Effects of E:G ratios of SCL on structural performance. *ASAE Annual International Meeting, Paper Number:01-4014.* Sacramento, CA, USA.

Hindman, D. P., Manbeck, H. B., & Janowiak, J. J. (2005a). Torsional rigidity of rectangular wood composite materials. *Wood and Fiber Science , 37* (02), 283-291.

Hindman, D. P., Manbeck, H. B., & Janowiak, J. J. (2005b). Torsional rigidity of wood composite I-joists. *Wood and Fiber Science , 37* (02), 292-303.

Keenen, F. J. (1974). Shear strength of wood beams. *Forest Products Journal , 24* (09), 63-70.

Lekhnitskii, S. G. (1963). *Theory of elasticity of an anisotropic body.* Mascow, Russia: MIR Publishers.

Longworth, J. (1977). Longitudinal shear strength of timber beams. *Forest Products Journal , 27* (08), 19-23.

Lyon, A., Moore, J., Mochan, S., & Gardiner, B. (2007). The use of acoustic NDT tools for predicting wood properties of Sitka spruce. *Proceeding of the 15th international symposium of NDT of wood.* Duluth, MN, USA.

Mandery, W. L. (1969). Relationship between perpendicular compressive stress and shear strength of wood. *Wood Science , 01* (03), 177-182.

Meadows, J. C. (1956). Longitudinal shear in wooden beams. *Forest Products Journal , 05* (09), 337-339.

Norris, C. B. (1957). Comparison of standard block-shear test with panel-shear test. *Forest Products Journal , 07* (09), 299-301.

Radcliffe, B. M., & Suddarth, S. K. (1955). The notched beam shear test for wood. *Forest Products Journal , 05* (02), 131-135.

Rammer, D. R., Soltis, L. A., & Lebow, P. K. (1996). *Experimental shear strength of unchecked solid-sawn Douglas-fir.* Madison, WI, USA: FPL-RP-553, USDA Forest Services, Forest Products Laboratory.

Ridley-Ellis, D., Moore, J., & Khokhar, A. M. (2009). Random acts of elasticity: MoE, G and EN408. *Cost Action 53 - Qualitiy Control for Wood and Wood Products.* Bled, Slovenia.

Riyanto, D. S., & Gupta, R. (1998). A comparison of test methods for evaluating shear strength of structural lumber. *Forest Product Journal , 48* (02), 83-92.

Soltis, L. A., & Rammer, D. R. (1994b). *Experimental shear strength of glued-laminated beams.* Madison, WI, USA: FPL-RP-527, Forest Products Laboratory, USDA Forest Services.

Soltis, L. A., & Rammer, D. R. (1994a). Shear strength of unchecked glued-laminated beams. *Forest Products Journal , 44* (01), 51-57.

Timoshenko, S., & Goodier, J. N. (1930). *Theory of elasticity.* New York, NY, USA: McGraw-Hill.

USDA. (1999). *Wood Handbook: Wood as an engineering material.* US Department of Agriculture and Forest Product Laboratory, Madison, WI, USA.

Vafai, A., & pincus, G. (1973). Torsional and bending behaviour of wood beams. *Journal of Structural Engineering ASCE , 99* (ST6), 1205-1221.

Yoshihara, H., & Furushima, T. (2003). Shear strengths of wood measured by various short beam shear test methods. *Wood Science and Technology , 37* (03), 188-197.

Yoshihara, H., & Ohta, M. (1997). Shear stress/shear strain relationship in wood obtinaed by torsion test. *Mokuzai Gakkashi , 43* (6), 457-463.