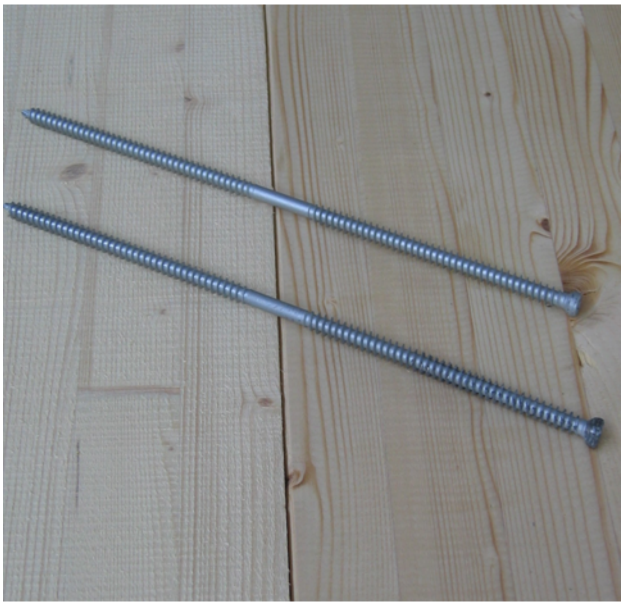
The influence of end- and edge distances on the load-carrying capacity in joints with inclined screws



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Report TVBK - 5125

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The influence of end- and edge distances on the load-carrying capacity in timber joints with inclined screws

Kant- och ändavståndens inverkan på bärförmågan i träförband med vinklade skruvar

Pernilla Malmborg

2004

Abstract

Using an inclined screw brings many advantages compared to traditional methods, e.g. high load-bearing capacity, fast and easy montage and high fire resistance. This report contains a study of the influence end- and edge distances have on the load-carrying capacity in joints with inclined screws. Tests were carried out in order to determine if a few of the end- and edge distances proposed by Kevarinmäki is too conservative. Further, withdrawal tests in slender construction timber were made, in order to determine if the SFS WT-T 8,2x300 screw is suitable in roof truss connections.

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Pernilla Malmborg

Abstract

Using an inclined screw brings many advantages compared to traditional methods, e.g. high load-bearing capacity, fast and easy montage and high fire resistance.
The WT-T screw is applied without pre-drilling. A common application is when a secondary beam is attached to a beam (see Figure 1), using a joint connection with screws exposed only to tension, or a joint with crossed screws- one exposed to tension and the other to compression. Joints with inclined screws haven't been used for long. There is no code saying which end- and edge distances that should be used. Keverinmäki (2002) made a proposal for end- edge distances and spacing concerning inclined screws. These distances are, however, considered to be conservative. In this master thesis tests have been carried out to investigate this statement concerning the parameters $a_{3,t}$, the distance to a loaded end face parallel to the grain, and $a_{4,c}$, the distance to and unloaded edge perpendicular to the grain. Further, withdrawal tests in slender construction timber were made, in order to determine if the SFS WT-T 8,2x300 screw is suitable in roof truss connections.
The screws used in all tests were the SFS WT-T 8.2x300 screw.
The tests were made on inclined tension screws with an angle of 45º between the screws and the grain direction. The average values of the load-carrying capacity received from the tests were compared with calculated average values based on calculation methods proposed by Kevarinmäki (2002) and Bejtka and Blaß (2002).

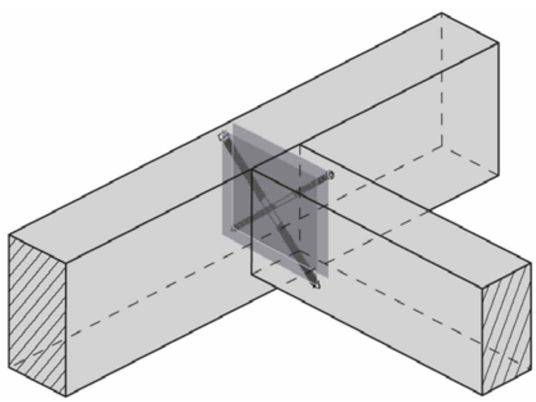


Figure 1: A crossed joint connection with the SFS WT-T screw is used to attach the secondary beam to the primary beam.

The load-carrying capacity according to Bejtka and Blaß is higher then according to Kevarinmäki, because they include the effect of shear action in the inclined screw. The tests indicate that $a_{3,t}$ and $a_{4,c}$ proposed by Kevarinmäki are conservative and can be decreased. Further tests were carried out with $a_{3,t}$ set to a lower value proposed by Kreuzinger and Spengler (2004). The joint reached full load-carrying capacity despite the fact that $a_{3,t}$ now only was 40% of the distance proposed by Kevarinmäki.

The withdrawal tests showed that the SFS WT-T 8,2x300 screw reaches full withdrawal capacity even in slender construction timber. Thus, the withdrawal capacity is no concern when using the screw in roof truss connections.

More test concerning the edge and end distances for inclined screws many tests need to be carried out before general rules can be set. Test with different angles between the screw and the grain direction has to be done in order to determine the influence the angle has on the load-carrying capacity and the splitting tendencies of the joint. The moisture content of the timber and the timber species also have to be varied.

1. Introduction

1.1 Background

SFS WT-T is a double-threaded self-tapping wood screw, which haven't been used practically for a very long time. The WT-T screw is applied without pre-drilling. A common application is when a secondary beam is attached to a beam, using a joint connection with screws exposed only to tension, or a joint with crossed screws- one exposed to tension and the other to compression. Using an inclined screw brings many advantages compared to traditional methods, e.g. high load-bearing capacity, fast and easy montage and high fire resistance. Not much research upon joints with inclined screws has been published. Further are the codes not yet adjusted to inclined screws which mean there are no design formulas given, nor are the edge distances and spacing between screws.

1.2 Purpose

The purpose of the master-thesis is, using available literature, to determine the requirements that should be put upon edge distances and spacing between screws in joint connections with inclined screws. With tests, the effect edge distances have on the load-bearing capacity of oblique angled timber joints, regarding e.g. split failure, is determined. Previous tests performed by Kevarinmäki (2002) resulted in a proposal for which end- and edge distances that should be used. These are considered to be too conservative, and the intention with the tests is to examine this statement for a few variables.

1.3 Method

The first step was to study literature that deal with timber joint connections in general to learn more about e.g. the different failure modes that can occur. These studies also led to the knowledge of which different factors affect the load-bearing capacity in a timber joint connection, and the theoretical reasons.
The next step was to study existing research on joint connections with inclined screws to get knowledge in this field. Based on the gained knowledge tests and test setups were designed. Tests were carried out on laterally loaded screws and inclined tension screws. The average values received from the tests upon laterally loaded screws were compared to the calculated average values according to Eurocode 5 and BKR 2003. The average values from the tests concerning inclined tension screws were compared to the calculated average values according to the methods proposed by Kevarinmäki and Bejtka and Blaß.

1.4 Limitations

Performing all types of tests to cover all aspects is a very large project and doesn't fit in a master-thesis. The tests were limited and carried out with the following assumptions;

- Withdrawal tests were made out of beams, 36 and 45 mm wide, made of Norway Spruce
- Joint connection tests were made with wood members out of glulam
  Angles tested: 90° and 45°
  Distances tested: $a_{3,t}$ and $a_{4,c}$
- The test pieces were stored in a constant climate, 20º C / 60% RH.
- The only screw used is the SFS WT-T 8,2x300.

2. Material properties

2.1 Material properties of timber and glulam

Timber

Wood is a composite material made out of a chemical complex of cellulose, hemicellulose, lignin and extractives. It is divided into two categories, hardwoods and softwoods. In the Nordic countries the use of softwoods is highly dominating. There are not only differences between hardwoods and softwoods and between species, but also within one specimen. These differences depend on if it is sapwood or heartwood, earlywood or latewood, but also on the arrangement of the pores and the appearance of reaction wood. These phenomena are the result of the development and growth of wood tissue (Hoffmeyer, 1995).

Wood is a pronounced anisotropic material, which means it has considerable differences in its physical properties depending on the direction, transversal or longitudinal. There are also differences in the directions radial and tangential to the growth rings, but they can usually be neglected. Wood is not a homogenous material which means that it contains defects such as knots, grain deviation, cracks, decay and large growth ring width. These imperfections have a great effect on the properties of the wood and can lead to a reduction of strength and stiffness. To get a more uniform material with regard to appearance and strength, the timber is sorted into different strength classes depending on the extent of its imperfections (Carling et al, 1992).

Therefore the properties of the timber also depends on the volume of the wood member, the larger the volume the more likely it is that it will contain defects (Isaksson, 2000).

Wood as well as for example concrete experience creeping. When designing a timber member for long-term permanent loads the strength values used are only approximately 60% of the strength values found in a short-term laboratory test (Hoffmeyer, 1995).

Despite its many problems there are also many advantages. Its high thermal resistance makes it suitable in timber-frame house constructions. It is also a light buildingmaterial and has high stiffness. Timber is cheap and if used properly it is ecological, and it is a renewable resource (Dinwoodie, 1989).

Glulam

Glued laminated timber (glulam) is built up of at least four laminations with the same grain direction, glued together under a pressure of between 0,4 and 1,2 N/mm^2. For curved members or hardwood laminations a higher pressure is necessary (Colling, 1995). Glulam is often used when large cross-sections are needed. There are no theoretical limits, only practical, of how long, wide or curved the members can be made (Isaksson, 2000).
According to Colling (1995) the most important advantages of glulam compared to solid timber are:

- *Beam sizes*

Unlimited beam sections and lengths are theoretically possible.

- *Beam shapes*

Unconventional beam shapes are available due to the possibility of curving the single laminations before gluing. This also allows beams to be precambered to accommodate dead load deflection.

- *Higher strength and stiffness*

Thanks to the production process, knots are spread more evenly within the volume of the beam leading to a more homogenous material. The influence of single potential failure areas due to knots is reduced, resulting in a lower variability and a higher mean strength.

- *Combined glulam*

Using laminations makes it possible to match the lamination quality to the level of stress. In case of a bending member for instance, laminations of a higher strength class are positioned in the outer highly stressed regions, whereas in the inner zones laminations of a lower quality may be used. This allows a more economical use of the available wood material.

- *Dry wood*

To provide damage caused by deformations occurring during the drying process in the construction, the laminations are kiln dried to a moisture content of about 12 %, since the equilibrium moisture content of wood used indoors varies between approximately 9 and 12%.

- *Dimensional accuracy*

The drying of the laminations and the production process also allow the production of glulam beams with accurate dimensions. Since small tolerances are important for the use, and combination, of prefabricated members of different materials, the dimensional accuracy can determine the use of glulam even if sawn timber would have been sufficient in terms of strength and stiffness.

2.2 Strength and stiffness in different directions

As mentioned earlier wood is an anisotropic material, which means that its physical properties vary considerably depending on direction. A definition of the different directions is shown in Figure 2.1. If you visualize wood as a bunch of straws glued together, it is not hard to realize that the strength will be different depending on if the load is applied parallel or perpendicular to the straws (Isaksson, 2000).
The strength of the material reaches its highest values in the direction parallel to the grain whereas perpendicular to the grain the lowest values are obtained.

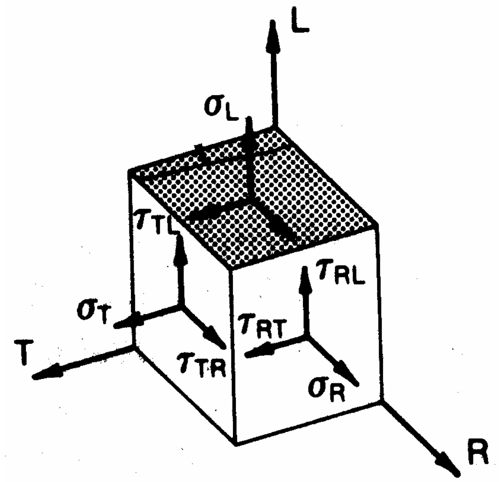


Figure 2.1: Cylindrical orthotropic material with the main directions in the longitudinal direction (L)- called grain direction, radially (R) and tangentially (T) (Carling et al, 1992).

Flawless wood, also known as clear wood, has its maximum strength at tension parallel to the grain. The tensile strength is so high that it is hard to determine experimentally, because it often leads to failure at the connections between the test piece and the test device. Strength as well as stiffness is highly reduced by defects, especially by knots and cross grain. The influence of the knots does not only reduce the effective cross section but also causes eccentricity and stress peaks where the stresses can not be compensated. Furthermore does the fibre aberration around a knot cause stresses perpendicular to the grain (Carling et. al., 1992).

The bending strength of a timber beam depends on the nature of the moment distribution along the beam. It can be expected that a beam with a point load at mid-span should have a strength that is higher than for a beam of the same length loaded with a constant bending moment along the entire length. The various effects of lengthwise variability were studied by for example Isaksson (1999). In the bending strength model seen in figure 2.2 it is assumed that timber is composed of local weak zones connected by segments of clear wood and that failure is primarily initiated in the weak zones. The weak zones correspond to knots or groups of knots randomly distributed along the length of the beam. Simulations based on the Monte Carlo method, with assumed statistical input data, determined in tests, demonstrated how length and load configuration effects are related to the lengthwise variability (Isaksson, 1999).

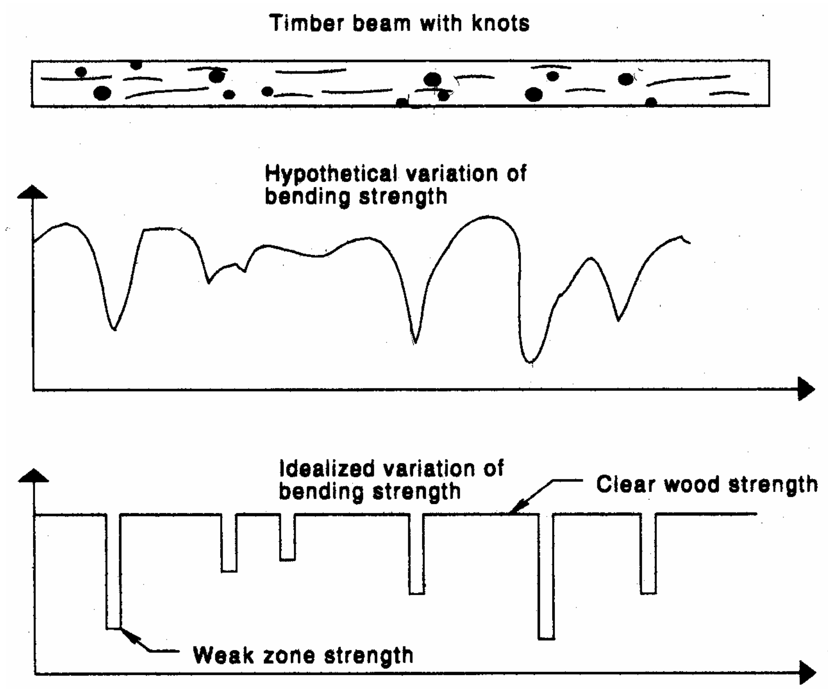


Figure 2.2: Modeling of lengthwise variation of bending strength in timber beams (Isaksson, 1999).

The strength for tension perpendicular to grain is the lowest one that wood possesses. It should only be exploited where secondary effects cause transversal tensile stresses, e.g. in curved glulam constructions. The strength perpendicular to grain is very dependent on the volume and lies approximately around 0,5 MPa (Carling et. al., 1992).

To determine the compression strength the length of the test specimens are chosen so that buckling doesn't arise. Failure occurs when the fibres buckle and push its way out between each other. This phenomenon can be described as stability failure in the fibres, and it is usually called kinking. The differences in strength values are not as large as by tension, but knots and cross grain also here lead to a reduction of the strength (Carling et. al., 1992).

When performing compression test perpendicular to the grain no failure is observed, but the deformations will increase substantially and failure is usually defined when a certain deformation is reached. The strength reaches it maximum value at compression tangentially to the growth rings. A 45° angle between the growth rings and the force direction gives the minimum value. Practically the lowest value has to be assumed. Young's modulus for compression over the whole cross section is about the same as for tension perpendicular to the grain (Carling et. al., 1992).

A compilation of strengths and stifnesses in different directions is found in Table 2.1.

Table 2.1: Strength and Stiffness in different directions.
A compilation based on Carling et. al., (1992) and Isaksson and Mårtensson (1999).

	Clear wood Average values	Construction timber Characteristic values
Bending parallel to grain f_m		12-35 MPa
Tension parallel to grain f_t	100 MPa	10-35 MPa
Tension perpendicular to grain f_{t90}		0,5 MPa
Compression parallel to grain f_c	40-50 MPa	25-40 MPa
Compression perpendicular to grain f_{c90}		≈ 7 MPa
Shear parallel to grain f_v		≈ 3 MPa
Shear perpendicular to grain f_{v90}		$0{,}5 \cdot f_v$
Young's modulus parallel to grain E	15 GPa	10-13 GPa
Young's modulus perpendicular to grain E_{90}		400-500 MPa
Shear modulus G		500-800 MPa

There are three different types of shear failure corresponding to the three different types of shear stresses shown in figure 2.3.

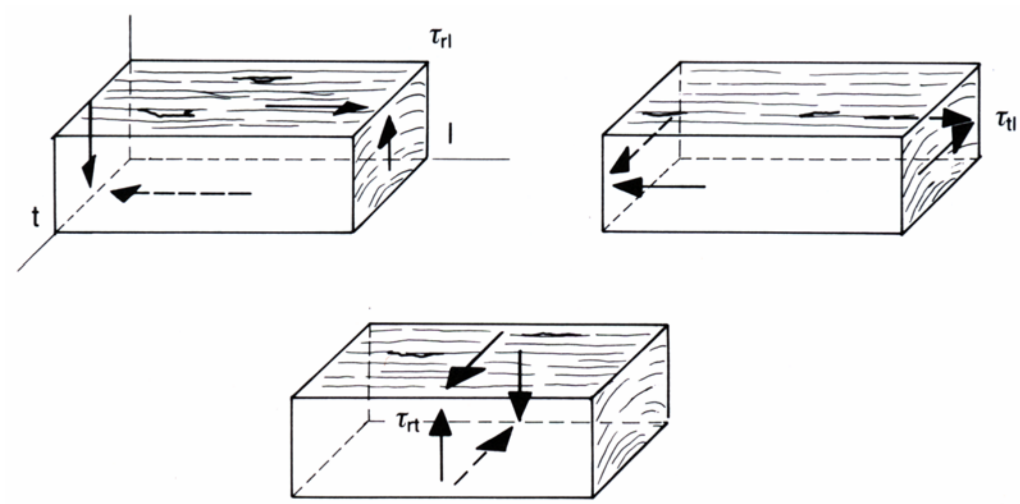


Figure 2.3: Above to the left: longitudinal shear radially to the growth rings.
Above to the right: longitudinal shear tangentially to the growth rings.
Below: Member with both shear components perpendicular to the grain (rolling shear) (Carling et al, 1992)

When designing a structure it is not possible to separate the two different types of shear perpendicular to the grain, so the minimum value is used.
The shear strength is different depending on if the failure is caused by shear force or torque. Cracks have a negative influence on the shearing strength while knots often have a reinforcing effect. (Carling et. al., 1992)

2.3 The influence of the moisture content

Wood is a hygroscopic material, therefore it continually exchanges moisture with its surroundings, that is, for any combination of temperature and humidity in the environment there will be a corresponding moisture content of the wood where the inward diffusion of moisture equals the outward movement. This is referred to as the equilibrium moisture content.

Moisture content is defined as the ratio of the mass of removable water to the dry mass of the wood. The fibre saturation point (FSP) is defined as the moisture content when the cell wall is saturated with moisture, but no free water exists in the cell lumen. The average FSP is about 28%. Below the fibre saturation point most physical and mechanical properties changes dramatically and above the FSP most properties are approximately constant. This means the fibre saturation point is of great importance in engineering. However, wood-members with moisture content as high as 28% are rarely used.

Since the climatic conditions are frequently changing wood is rarely in a state of moisture equilibrium. It is not only the level of moisture content that has an influence on the engineering properties of the wood but also the magnitude and speed of moisture fluctuations (Hoffmeyer, 1995).

Drying timber below about 30% moisture content brings a considerable shrinkage perpendicular to the grain, whereas the shrinkage along the grain is often small enough to be ignored. The shrinkage can amount up to approximately 7% of the cross-sectional dimensions. To avoid problems timber should be installed at a moisture content close to the equilibrium moisture content likely to be achieved in service. Restrained shrinkage deformations in service can, e.g. in connections, cause tension perpendicular to the grain and hence potential failure. The different shrinkage in radial and tangential directions can lead to cracks if large cross-section timber dries to fast. Normally cracks do not reduce the strength of the timber members. The risk of splits can be reduced by kiln drying.

With decreasing moisture content, the strength and modulus of elasticity values generally increase. Timber under load shows an increase of deformation with time. While keeping timber in a constant climate, the creep deformation would only exceed the elastic deformations by about 50% in 20 years. However, if the moisture content of the wood varies, the creep deformations may exceed several hundred percent of the initial deformations. The importance of creep-deformations does not only lie on the possibility of excessive deformations, but also on the fact that they can lead to a reduction in load-carrying capacity due to creep-buckling effects.

As mentioned earlier the duration of load also significantly influences the strength and deformations of timber and timber structures. With increasing load duration, the strength of timber decreases (Steer, 1995).

2.4 General about screws

The definition of a screw is an external threaded cylindrical body, with or without head (Carling et. al., 1992). The main type of screw used for structural application is the coach screw (see figure 2.4). The size varies from 6 to 20 mm in diameter and 25 to 300 mm in length. The coach screw is used in combination with washers (Racher, 1995).
Wood screws have a slightly tapered shank. They are inserted with a screwdriver. The design values for laterally loaded screws are comparable to the value for nails. Due to the gripping action of the threads, wood screws have a significantly larger allowable design values for withdrawal load than nails (Breyer et. al, 1999).
In large connections they are suitable for holding connectors in place. They can also be used to fix joist hangers or framing anchors in combination with nails. Their use is limited as a result of the pre-drilling needed to install the coach screws (Racher, 1995).

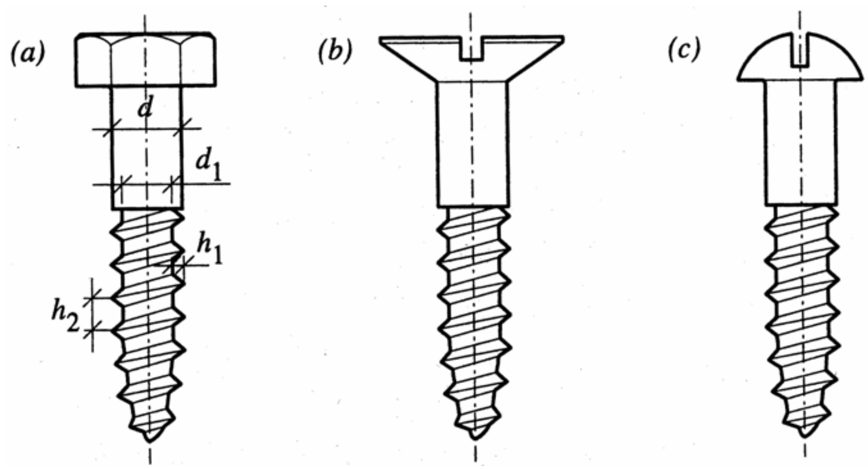


Figure 2.4: Typical wood screws: (a) coach screw (b) countersunk head (c) round head (Ehlbeck and Ehrhardt, 1995).

In elderly timber constructions direct transmission of forces were utilized through contact pressure between the wood members. Dowels and screws just held the members together. The joints that were developed were suitable for transferring compression forces, and could only bear smaller tension forces. Eventually mechanical connector and glued joint connections came to dominate (Carling et. al, 1992).

2.5 Material properties of the SFS WT-T 8.2x300 screw

2.5.1 Dimensions

The SFS WT-T 8.2 x 300 screw has a nominal diameter of 8,2 mm, defined as the external diameter of the threaded part. The total length of the screw is 300 mm. For calculations the root diameter d_k, has been set to 5,5 mm.

Table 2.2: Dimension of the SFS WT-T 8.2x300 screw

d (mm)	8,2
d_k *(mm)*	5,5
L (mm)	300

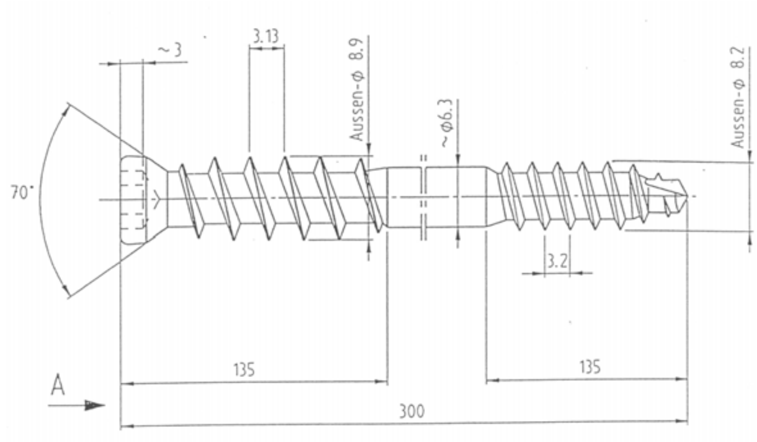


Figure 2.5: Nominal dimensions, WT-T 8.2 x 300

2.5.2 Tensile capacity of individual screws

The SFS WT-T screw is manufactured in 9.8 steel; therefore it has an ultimate stress of 900 N/mm^2 and a yield stress (f_{yk}) of 720 N/mm^2 (Hansson and Thelandersson, 2003).

2.5.3 Yield moment of individual screws

The characteristic value of the yield moment is based on tests and shown in the table below (Hansson and Thelandersson, 2003).

Table 2.3: Characteristic yield moment calculated in accordance to Eurocode 5

Type of screw	Root diameter [mm]	Characteristic yield moment, M_{yk}, [Nm]
WT-T 8.2xL	5.5	22.0

2.5.4 Withdrawal capacity of the WT-T screw

According to test results collected and analyzed by Hansson and Thelandersson (2003), the characteristic withdrawal capacity for the WT-T 8.2 x L screw can be taken from Table 2.4. This value can be applied for angles 30°<α<60°, also for screws in end grain. The table is valid for service classes 0, 1 and 2 only. $f_{a,1,k}$ and $f_{a,2,k}$ are included in the design formula for joints with inclined screws, see chapter 4.2.

Table 2.4: Withdrawal capacity, f_{a1k}, f_{a2k}, characteristic values, based tests. Applicable for angles of $30 \leq \alpha \leq 60$ degrees, also for screws in end grain. Valid for service classes 0, 1 and 2 only (Hansson and Thelandersson, 2003).

d	$f_{a,1,k}$, $f_{a,2,k}$ [N/mm²]
8.2	4.5

2.5.5 Field of application

The SFS WT-T self-tapping screws have a wide range of application. They can be applied in an angle or perpendicular to the grain. Instead of using traditional methods when attaching a secondary beam to a primary beam a crossed joint connection with the SFS WT-T double threaded screws can be used. This method is cost saving due to the fact that few operations enable installation time to be reduced considerably. It can also advantageously be used as reinforcement in notched timber beams. The SFS WT-T screw can also be used as reinforcement in arched beams and for connecting roof batten.

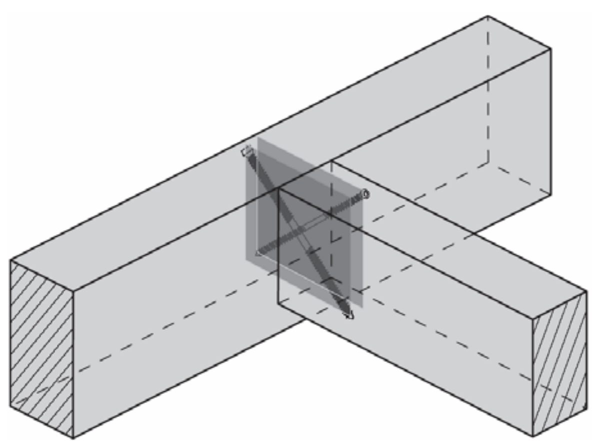


Figure 2.6: A crossed joint connection with the SFS WT-T screw is used to attach the secondary beam to the primary beam. (from Der Zimmerman, 2002)

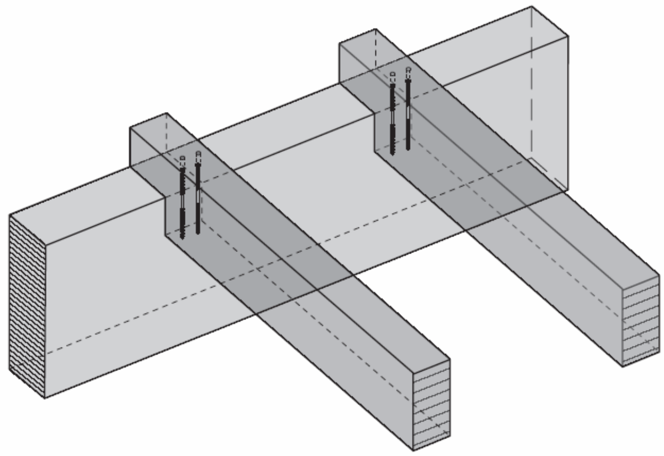


Figure 2.7: The SFS WT-T screw is used as reinforcement in notched timber beams. (from Der Zimmerman, 2002)

The load-bearing capacity is high for inclined screws and they can transfer both shear- and vertical forces (SFS, 2001).

New applications are continuously suggested as a result of communication with experts in the business. One idea is to replace transversal pre-stressed reinforcement in slabs, with screws exposed to tension and compression in timber bridges (Schmid, 2001). The use of unprotected timber in bridges is, however, not considered to be a good idea by all experts.

3. Screw joint connections- shear joint

3.1 General

When structural members are attached with fasteners or some other type of hardware, the joint is said to be a mechanical connection. Generally connections are classified according to the direction of loading. Shear connections have the load applied perpendicular to the length of the screw. The other major type of loading in a wood connection has the applied load parallel to the length of the fastener, and the fastener is loaded in tension. When screws are subjected to this type of loading, the concern is that the screw may pull out of the wood member. This type of connections is also referred to as axially loaded connections (Breyer et. al. 1999).

3.2 Failure modes

In a screwed joint connection there are several possible failure modes (or yield modes), depending on if the failure occurs in the wood member, in the metal fastener or in both. The failure modes normally referred to are the ones below presented by Johansen in 1949 (Carling et. al, 1992).

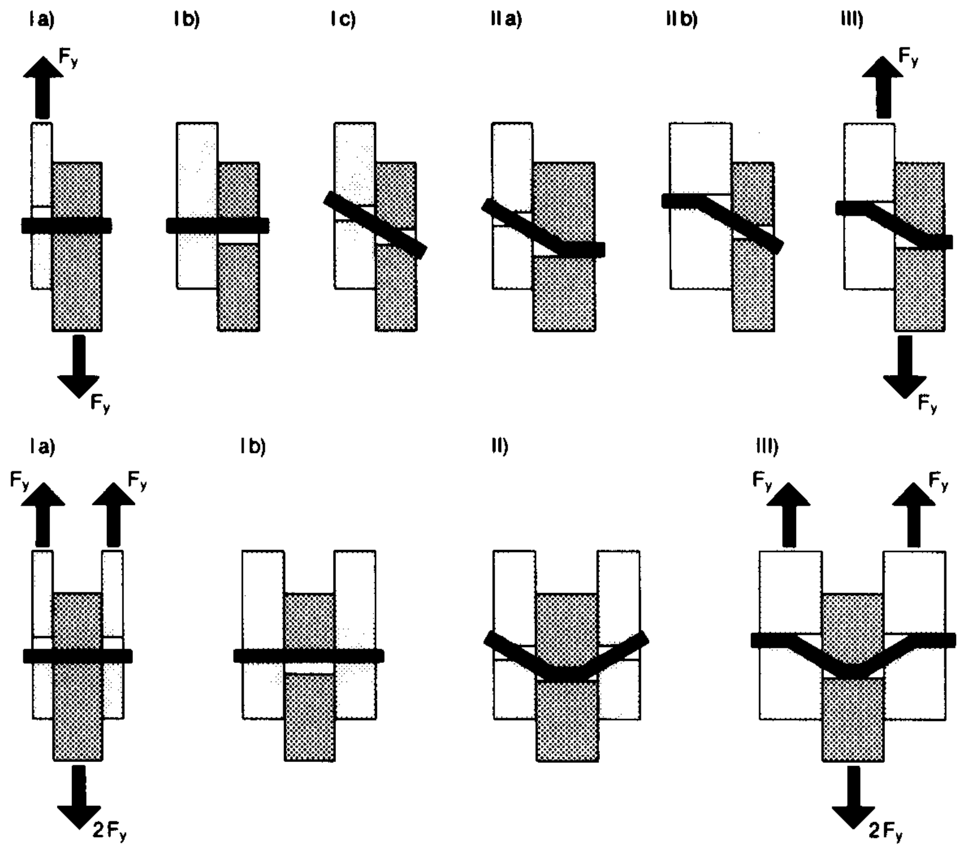


Figure 3.1: Failure modes for timber and panel connections.

Basically there are three phenomena that lead to failure (Larsen and Riberholt, 1999);

- the screw is pushed through the wood without rotating
- the screw rotates but remains intact
- the screw is bent and a plastic hinge occurs

In the yield modes shown in figure 3.1 the cause of failure is described below (Breyer et. al. 1999).

Mode	
Ia	embedding in side member
Ib	embedding in main member
Ic	rotation of fastener
IIa , IIb , II	plastic hinge and crushing in wood member
III	two plastic hinges per shear plane

3.3 Parameters affecting the load-carrying capacity and reasons for failure

There are several different parameters affecting the load-carrying capacity in screwed joint connections. For some of them their influence on the load-carrying capacity is already well-known, whereas for some others it is known that the parameters affect the strength of the connection but not exactly how.

The parameters below have an influence on the load-carrying capacity but will not be further mentioned.

- bending capacity of the fastener
- friction between timber members
- grain direction
- fabrication tolerances
- timber thickness
- moisture content

The following parameters also have an influence on the load-carrying capacity and will be discussed below:

- edge- and end distances
- spacing between fasteners
- number of fasteners in a row
- dowel slenderness ratio
- the withdrawal capacity of the screw
- embedding strength of the timber members

Multiple fastener connections are commonly used in timber structures. The failure is often caused by the timber parts and not by the fastener. According to Mischler and Gehri (1999) there are three possible failure modes in the timber:

- Splitting of the timber in a row of dowels
- Tensile failure of the timber in the reduced net section
- Combination of splitting, block shear and tensile failure

To prevent block shear failure short and wide joints should be used in order to minimize the number of fasteners in line with the load and grain direction (Johnsson, 2003).
For a multiple fastener joint, the load-carrying capacity is often significantly lower than the strength of one fastener times the number of fasteners. Therefore, strength reduction factors for multiple fastener joints had to be introduced. Mischler and Gehri (1999) carried out tests to examine the influence e.g. number of dowels in line, number of dowel rows, spacing and end distances have on the load-carrying capacity.

An important condition in the Johansen theory is that the joints allow large ultimate deformations in order to reach the plastic failure modes. As the timber fails in a brittle way, these deformations are possible if the failure occurs after significant plastic deformation of the dowel. This failure mode is reached when fasteners of effective slenderness ratio bigger than the limit slenderness ratio λ_y, are used. λ is the ratio between timber thickness and diameter of the fastener, see Figure 3.2. Mischler and Gehri (1999) tested two different slenderness ratios:

$\lambda = 3$	The dowel remains straight, failure mode I
$\lambda = 6$	Two plastic hinges are formed in the dowel, failure mode III

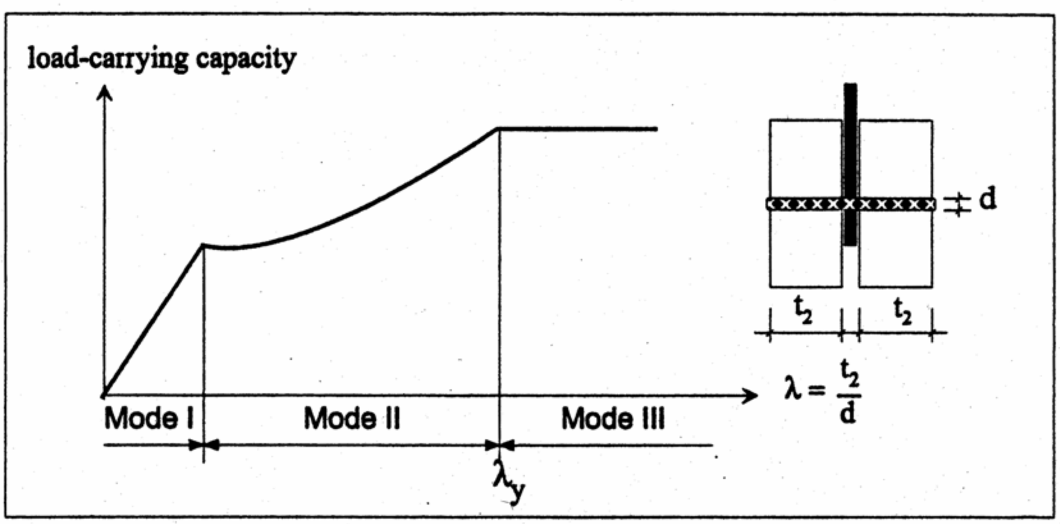


Figure 3.2: The load-carrying behavior of a dowel type connection according to Johansen's theory (Mischler and Gehri, 1999)

For each slenderness ratio test were made with one dowel in line, and three dowels in line. The connections with dowel slenderness ratio $\lambda = 3$ and three dowels in line only show small deformations and fail in splitting. The ultimate load of the connection is only 72% of the sum of the single fastener strengths. The connections with dowel slenderness $\lambda = 6$ show a very ductile failure, and plastic hinges occur in the fasteners. In a connection with three dowels in line the fasteners reach the same ultimate load per dowel as the single fastener connection. Mischler and Gehri show that the dowel slenderness ratio has a significant influence on the strength of the connection. This in difference to Jorrissen (Jorrissen 1998, from Mischler and Gehri 1999) who only found a minor influence of the dowel slenderness, and suggested a simplified design rule excluding the influence of the dowel slenderness ratio.

If end distances and spacing among the dowels are not long enough, the timber fails in splitting before reaching the load-carrying capacity of the fasteners. Mischler and Gehri (1999) also performed tests with three dowels in line, varying the slenderness ratio, end distance and spacing. The results of the tests are presented in the table below.

Table 3.1: Influence of end distance and spacing on the ultimate load of connections with 3 dowels in line compared to the load-carrying capacity of the single fastener (Mischler and Gehri, 1999).

end distance/spacing	7d / 4d	10d / 4d	7d / 7d	10d / 7d	10d / 10d
λ=3	53%	-	-	72%	90%
λ=6 mild steel dowels	91%	98%	98%	100%	100%

The influence of end distance and spacing on the load-carrying capacity is much bigger in connections with dowels of low dowel slenderness ratio than in connections with dowels of high dowel slenderness ratio of mild steel. According to Blass, Frasson and Schmid (2002) the fastener spacing parallel to the grain, a_1, is the major influencing parameter on the splitting tendency of timber in the connection area. For joints with more than one fastener $a_{3,t}$ and $a_{4,c}$ are of minor importance. Their tests also showed that for similar geometry and the same fastener slenderness the absolute diameter has a significant influence as well.

For definitions of a_1, $a_{3,t}$ and $a_{4,c}$ see chapter 4.3.

Embedding strength is the property of wood that affects the design value of the load carrying capacity. It is related to the crushing strength of the wood member under loading from a screw subjected to a shear load (Breyer et. al., 1999). It is defined as the ultimate stress obtained from a special type of joint test called an embedment test. The test arrangement is shown in Figure 3.3. The embedding strength also depends on the angle between force- and grain direction, the cross section and lateral dimension of the dowel (Carling et. al., 1992).

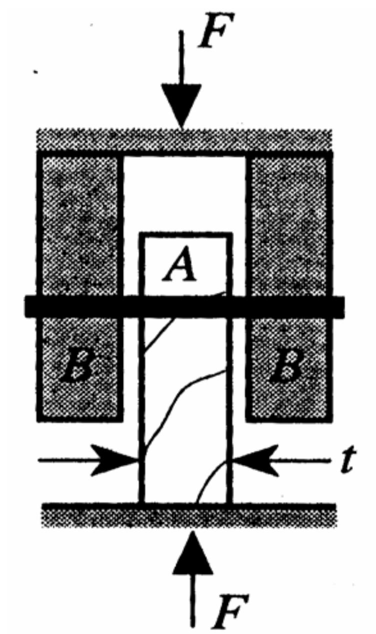


Figure 3.3: Typical embedment test arrangement. A – specimen, B – steel plates rigidly clamping fastener (Hilson, 1995).

To be able to reach full load-carrying capacity and avoid splitting, rules that take the influence of screw spacing and edge distances under consideration have to be followed. In a joint connection there is risk of splitting when a force with a large component perpendicular to the grain is being transferred to the wood member. Normally it is the tension strength perpendicular to grain being exceeded but splitting can also be caused by high shear stresses (Carling et. al., 1992).

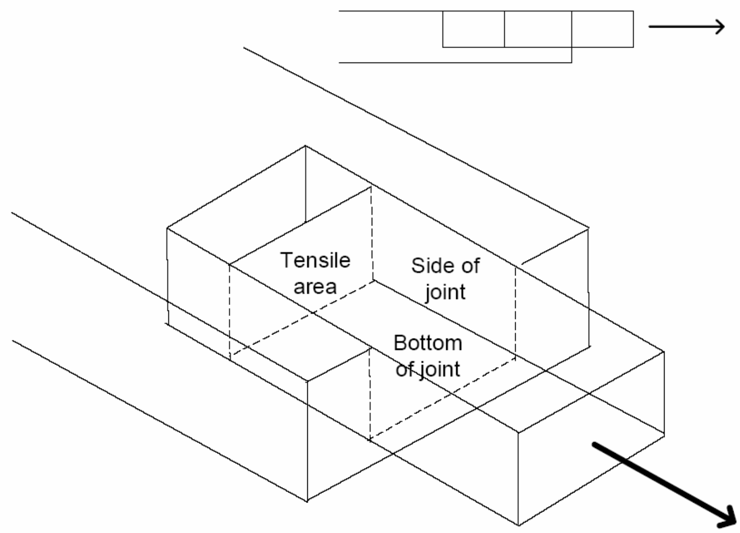


Figure 3.4: Schematic plug shear failure (Johnsson, 2003)

When timber is stressed by a group of fasteners loaded in tension parallel to the grain it results in both tension and shear stresses parallel to the grain, where the bottom and side areas are loaded in shear, see Figure 3.4. The resistance of the joint is the lowest of the fastener embedding and the plug shear resistance. Normally block shear failure is limiting for large doweled connections loaded in tension parallel to the grain. As mentioned earlier a method to avoid this type of failure is the use of short and wide joints, in order to minimize the number of fasteners in line with the load and grain direction (Johnsson, 2003). However, this is not always possible because it leads to high cross-sections which increase the costs.

3.4 Load-carrying capacity

For the determination of the load-carrying capacity of laterally- and axially loaded screws each country has their own code with different design formulas. A compilation of a few different formulas according to, e.g. the Swedish code, the German code and Eurocode 5, is found in Appendix A1-A2.

3.5 Edge- and end distances

In order to reach full load-carrying capacity and avoid failure by splitting certain demands is put on edge- and end distances in a screwed joint connection. Also these vary depending on the country referred to as they are based on the "rule of thumb". A comparison between a few different codes is found in Appendix A3-A4. The definitions of end- and edge distances as well as spacing are shown in Figure 3.5.

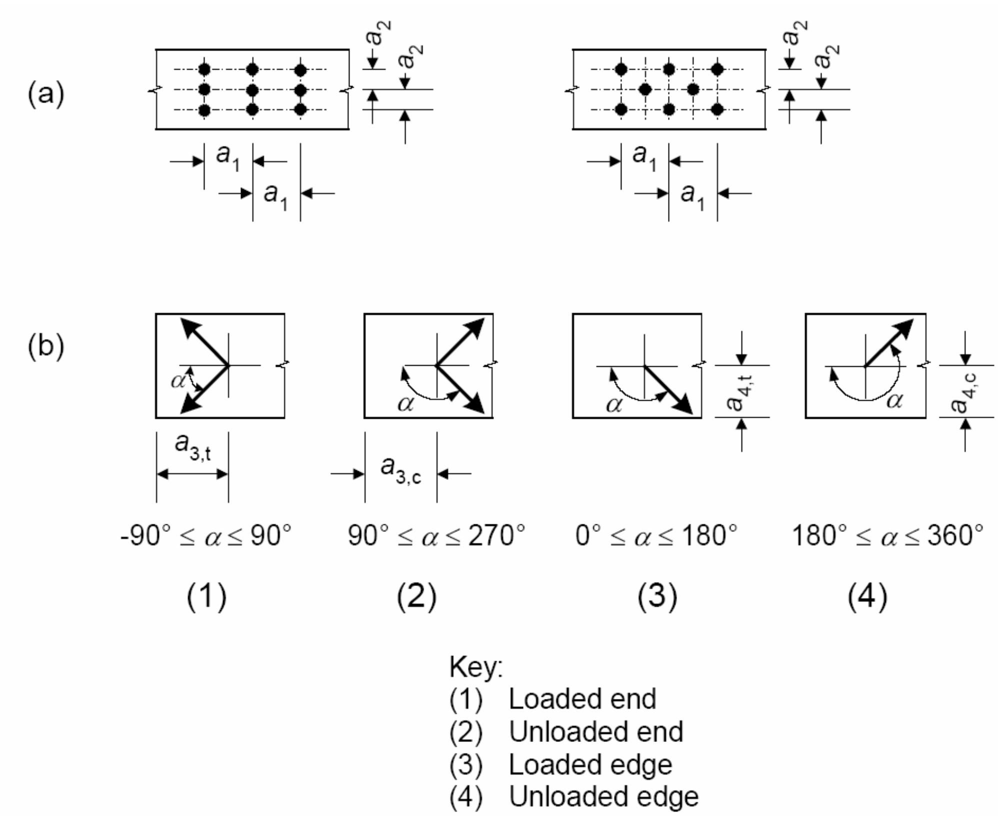


Figure 3.5: Definitions of edge- and end distances and spacing between screws. (a) Spacing parallel and perpendicular to grain, (b) Edge and end distances (α is the angle between the <u>force</u> and the <u>grain</u> direction)

4. Joints with inclined screws

4.1 General

For joints with dowel type fasteners loaded perpendicular to the fastener axis, the ultimate load is limited by the embedding strength of the timber members and the bending capacity of the fasteners. Using long screws in a joint connection gives it higher load-carrying capacity, due to a higher withdrawal capacity. Placing the screws under an angle between 40° and 75° between the screw axis and the grain direction, instead of perpendicular to the interface between the members, increases the load-carrying capacity even more (Bejtka and Blaß, 2002). The inclined self-tapping screws may be used effectively as fasteners in timber-to-timber connections, because of the screws high resistance against withdrawal (Kevarinmäki, 2002). For joints with inclined screws the ultimate load is limited by the embedding strength of the timber members, the bending capacity and the withdrawal capacity of the fasteners as well friction between the timber members. While the screw is loaded in tension the contact surface between the members are under compression (Bejtka and Blaß, 2002). Tests have shown that the best connection stiffness was achieved in loading direction of 45°: about two times higher stiffness than with the angle of 60°, and almost 15 times higher than the stiffness for screws loaded perpendicular to the fastener axis (Kevarinmäki, 2002). According to Bejtka and Blaß (2002) inclined self-tapping screws provide opportunities for rationalization and cost reduction in timber connections, particularly during design and installation.

The inclined screws are often used in one of the following types of joint connection:

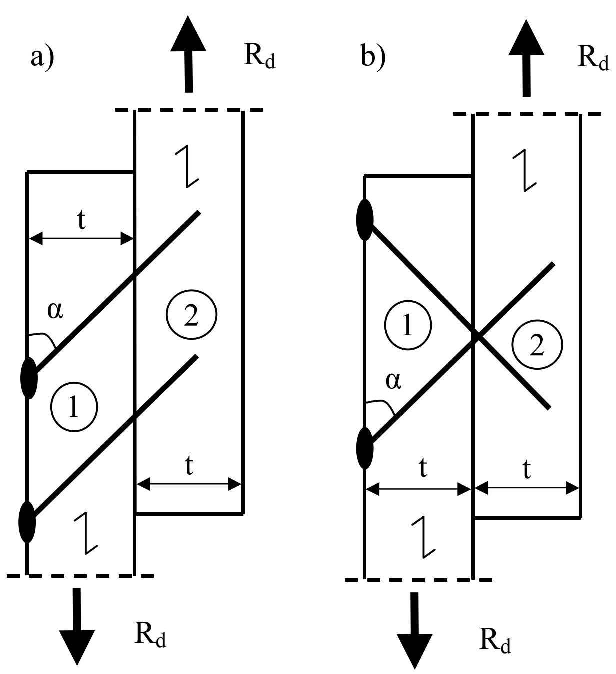


Figure 4.1 a) Joint with inclined tension screws; b) Joint with crossed inclined screws. Inclination defined according to Kevarinmäki (2002).

4.2 Methods for determining the load-carrying capacity.

Not many reports investigating the behavior of inclined screws are available. Bejtka and Blaß (2002) and Kevarinmäki (2002) have suggested different methods for calculating the load-carrying capacity. In Hansson and Thelandersson (2003) the equations from Kevarinmäki (2002) are applied on the SFS WT-T screw, and the Swedish code is used for some calculations, why this will be occasionally referred to.

In the model proposed by Bejtka and Blaß (2002) the load-carrying capacity of the joint depends on the following three interacting mechanisms:

- Withdrawal capacity of the screw
- Friction between the wood members in the joint
- Dowel action of the fastener in shear

Johansen's yield theory assumes an ideal rigid-plastic material behavior of the timber in embedding and of the fastener in bending. In Bejtka and Blaß (2002) model Johansen's yield theory is extended to include joints with inclined screws. The load-displacement behavior of screws loaded in withdrawal is taken into account. Thus, when the connection reaches its ultimate load the screw may or may not have reached its withdrawal capacity, depending on the axial displacement at this point. Further the distribution of the shear stress along the length of the screw consequently is regarded in the model. This is done using reduced withdrawal strength. This also takes the interaction between embedding and withdrawal strength of fasteners loaded parallel and perpendicular to the fastener axis into account. The distribution of the embedding stresses along the length of the screw depends on the failure mode, which means that also the modified withdrawal strength depends on the failure mode.

The basic assumptions for deriving the extended design equations for Johansen's failure mode 3 are:

- Ideal rigid-plastic material behavior for the timber under embedding stresses and of the fastener in bending.

- Averaged modified withdrawal parameters $f_{1,mod,i,j}$ for different failure modes considering the withdrawal behavior depending on the lateral load.
  i timber member (member 1 or 2)
  j Johansen's failure mode (1a,l - 1a,r - 1b - 2a - 2b or 3)

- The angle or the inclination, α, is defined as the angle between the screw axis and the direction perpendicular to the grain. (Kevarinmäki uses the opposite definition of α.)

If the reader has further interest in the derivation itself or the method for calculation of the reduced withdrawal parameter, Bejtka and Blaß' paper "Joints with inclined screws" CIB-W18/35-7-5 (2002) is recommended.

Hansson and Thelandersson (2003), however, recommend that the effect of friction in a joint with tension is taken into account, since under favorable climatic conditions, contact between the wood surfaces can be guaranteed in a joint with screws under tension. In their model the shear action of the inclined screw could be disregarded with the justification it is on the safe side. Further the proposed model by Bejtka and Blaß (2002) for shear action of the dowel when using inclined screws is complicated and not practical. According to Hansson and Thelandersson (2003) additional studies and tests are necessary in order to support even higher level of utilization in joints of this type.

4.2.1 Bejtka and Blaß' model

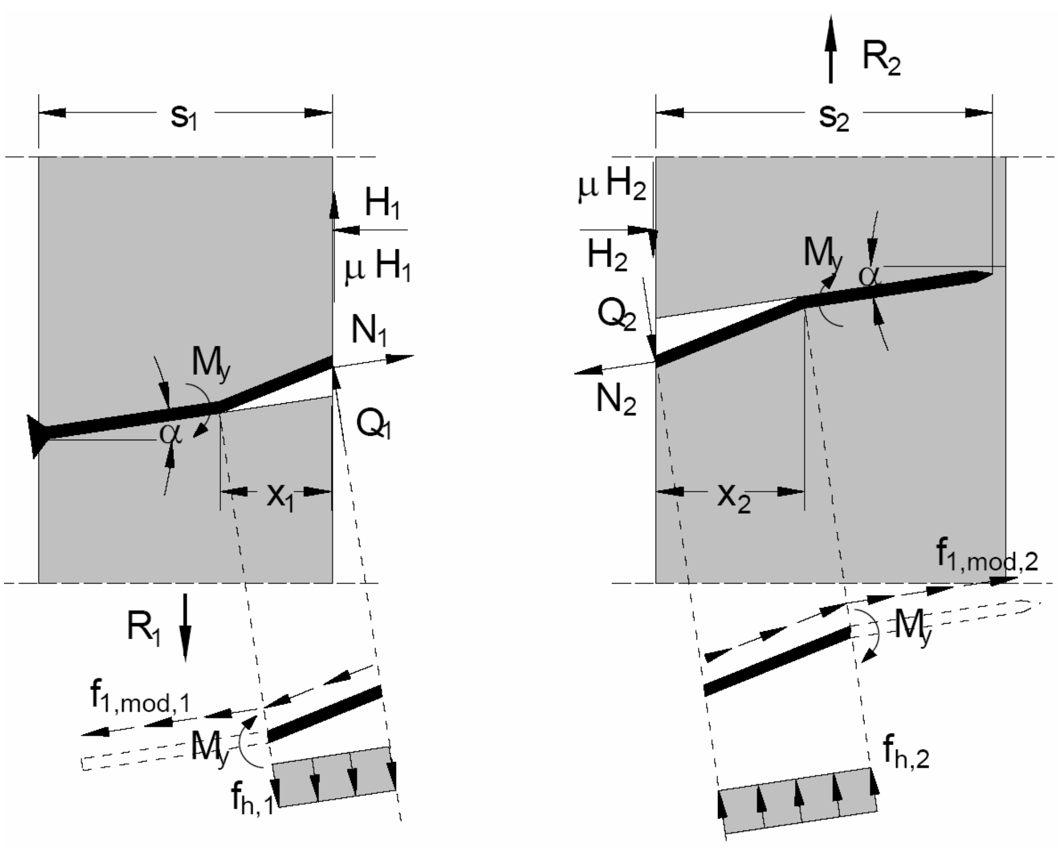


Figure 4.2: Forces and stresses and the definition of α in a timber-to-timber connection with an inclined screw for Johansen's failure mode 3 (Bejtka and Blaß, 2002).

The load carrying capacity for inclined screws in timber-to timber connections using Johansen's extended failure mode 3 is:

$$R_{VM3} = R_{ax,3} \cdot (\mu \cdot \cos\alpha + \sin\alpha) + (1 - \mu \cdot \tan\alpha) \cdot \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{2 \cdot M_y \cdot d \cdot f_{h,1} \cdot \cos^2\alpha} \quad (4.1)$$

with the withdrawal capacity $R_{ax,3}$ for Johansen's failure mode III, see Figure 3.1:

$$R_{ax,3} = \min \begin{cases} f_{1,\mathrm{mod},1,3} \cdot d \cdot \dfrac{s_1}{\cos\alpha} \\ f_{1,\mathrm{mod},2,3} \cdot d \cdot \dfrac{s_2}{\cos\alpha} \end{cases} \quad (4.2)$$

For the other failure modes the following equations are given:

$$R_{VM1a,1} = R_{ax,1a1} \cdot \sin\alpha + f_{h,1} \cdot d \cdot s_1 \cdot \cos\alpha \quad (4.3)$$

$$R_{VM1a,r} = R_{ax,1ar} \cdot \sin\alpha + f_{h,2} \cdot d \cdot s_2 \cdot \cos\alpha \quad (4.4)$$

$$R_{VM1b} = R_{ax,1b} \cdot (\mu \cdot \cos\alpha + \sin\alpha) + \frac{f_{h,1} \cdot d \cdot s_1}{1+\beta} \cdot (1 - \mu \cdot \tan\alpha) \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{s_2}{s_1} + \left(\frac{s_2}{s_1}\right)^2\right] + \beta^3 \cdot \left(\frac{s_2}{s_1}\right)^2} - \beta \cdot \left(1 + \frac{s_2}{s_1}\right) \right] \quad (4.5)$$

$$R_{VM2a} = R_{ax,2a} \cdot (\mu \cdot \cos\alpha + \sin\alpha) + (1 - \mu \cdot \tan\alpha) \cdot \frac{f_{h,1} \cdot s_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_y \cdot \cos^2\alpha}{f_{h,1} \cdot d \cdot s_1^2}} - \beta \right] \quad (4.6)$$

$$R_{VM2b} = R_{ax,2b} \cdot (\mu \cdot \cos\alpha + \sin\alpha) + (1 - \mu \cdot \tan\alpha) \cdot \frac{f_{h,1} \cdot s_2 \cdot d}{1 + 2 \cdot \beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2 \cdot \beta + 1) \cdot M_y \cdot \cos^2\alpha}{f_{h,1} \cdot d \cdot s_2^2}} - \beta \right] \quad (4.7)$$

where

α	is the angle between the screw axis and the direction perpendicular to the grain
μ	is the friction coefficient
$f_{h,1}, f_{h,2}$	are the embedding strength for timber member 1 respectively 2.
β	is the ratio between $f_{h,1}$ and $f_{h,2}$
d	is the screw diameter
s_1, s_2	are the penetration depth of the screw in timber member 1 and 2 perpendicular to the timber surface.
M_y	is the yield moment
$R_{ax,j}$	is the withdrawal capacity for failure mode j

The load-carrying capacity in timber-to-timber connections with inclined screws is now:

$$R = \min\begin{Bmatrix} R_{VM1a,l} \\ R_{VM1a,r} \\ R_{VM1b} \\ R_{VM2a} \\ R_{VM2b} \\ R_{VM3} \end{Bmatrix} \qquad (4.8)$$

$$R_{ax,j} = \min\begin{Bmatrix} f_{1,\mathrm{mod},1,j} \cdot d \cdot \dfrac{s_1}{\cos\alpha} \\ f_{1,\mathrm{mod},2,j} \cdot d \cdot \dfrac{s_2}{\cos\alpha} \end{Bmatrix} \qquad (4.9)$$

With

$f_{1,mod,1,j}$	are the averaged modified withdrawal parameters for different failure modes considering the withdrawal behavior depending on the lateral load.
j	Johansen's failure mode (failure mode 1a,l; 1a,r; 1b; 2a; 2b or 3)

Be aware that the definition of $f_{1,mod}$ differ from Kevarinmäkis with a factor of π.

4.2.2 Kevarinmäki's model

The minimum thickness t in mm for the joined wood elements shown in figure 4.1 is proposed by Kevarinmäki (2002) as:

$$t = \max \begin{cases} 7d_k \\ (13d_k - 30)\dfrac{\rho_{k,i}}{400} \end{cases} \tag{4.10}$$

where

d_k is the root diameter of the screw[mm] and

$\rho_{k,i}$ is the characteristic density of the wood elements [kg/m^3].

Joints with crossed screws

It is proposed that the design load capacity R_d of a joint with crossed screws, see Figure 4.1b, be calculated in accordance with equations (4.11-4.13) below:

$$R_d = n_p\left(R_{C,d} + R_{T,d}\right)\cos\alpha \tag{4.11}$$

$$R_{C,d} = \min \begin{cases} f_{a,1,d}\pi d s_1 \\ f_{a,2,d}\pi d s_2 \\ 0.8F_{u,d} \end{cases} \tag{4.12}$$

$$R_{T,d} = \min \begin{cases} f_{a,1,d}\pi d s_1 + f_{head,d}\cdot d_h^2 \\ f_{a,2,d}\pi d\left(s_2 - d\right) \\ F_{u,d} \end{cases} \tag{4.13}$$

where

n_p is the number of pairs of crossed screws ($n_p \leq 3$),

α is the applied angle (30º ≤ α ≤ 60º), see Figure 4.1

$f_{a,1,d}$ is the design withdrawal strength in the wood closest to the screw head [N/mm^2]

$f_{a,2,d}$ is the design withdrawal strength in the wood closest to the screw tip [N/mm^2]

d is the nominal diameter of the screw [mm]

s_1 is the threaded length in part 1 (closest to the screw head) [mm]

s_2 is the threaded length in part 2 (closest to the screw tip) [mm]

$F_{u,d}$ is the design tensile strength of the screw [N].

d_h is the head diameter of the screw [mm]

$f_{head,d}$ is the design pull trough strength of the screw head determined experimentally according to EN 1383 [N/mm^2]

As there is no pronounced effect on the withdrawal capacity due to the length of the WT-T screw the design withdrawal strength, $f_{a,i,d}$, is calculated in accordance with BKR03 as

$$f_{a,i,d} = \frac{\kappa_r f_{a,i,k}}{\gamma_m \gamma_n} \tag{4.14}$$

where

κ_r is a conversion factor that takes into account moisture and load duration

γ_m is the partial coefficient for the material and

γ_n is the partial coefficient for the safety class.

$f_{a,i,k}$ is the characteristic withdrawal strength of the thread i, where $i = 1$ refers to the thread closest to the head of the screw and $i = 2$ refers to the thread closest to the tip, for SFS WT-T see Table 2.4.

The values of these factors should be chosen in accordance with BKR03 for timber joints.

The characteristic withdrawal capacity F_a can be calculated from the relation:

$$F_a = \pi d \ell_{ef} \cdot f_{a,i,k} \tag{4.15}$$

where

d is the nominal screw diameter (external thread) [mm]

ℓ_{ef} is the effective anchoring length [mm]

The design tensile strength f_{ud} of the screw steel is calculated in accordance with BSK 99

$$f_{ud} = \frac{f_{uk}}{1.2\gamma_n} \tag{4.16}$$

where f_{uk} can be set to 900 N/mm^2.

Joints with tension screws

A typical joint of this type is a secondary beam suspended from a primary beam, see Figure 2.6. In a joint where the screws are loaded in tension, see figure 4.1a, the contact pressure between the joined surfaces will increase with increased loading, which is why it is reasonable to allow for friction in the joint. The design load capacity R_d for this type of joint can be calculated as follows:

$$R_d = nR_{T,d}(\cos\alpha + \mu\sin\alpha) \tag{4.17}$$

where

n	is the number of screws in the joint ($n \leq 6$),
$R_{T,d}$	is the withdrawal capacity of the screws determined in accordance with eq (4.13),
μ	is the kinetic friction coefficient between the wood surfaces, see table 4.2
α	is the angle of the screw to the contact surface between the joined wood elements

Table 4.2: Proposed values for the friction coefficient μ with tension screws where contact between the wood surfaces can be guaranteed (Kevarinmäki, 2002 from Vorreiter, 1949). Valid for service classes 0, 1 and 2, only.

Wood surfaces	μ
Both surfaces parallel to the grain *	0.26
End grain wood against a surface parallel to the grain *	0.26**

* Planed and without a coating of e.g. paint (spruce, pine)

** No values have been found but it is assessed that it can be set at the same level as for the grain direction.

4.3 Minimum end- and edge distances, and spacing between screws

There are no proper codes available giving the end- and edge distances and spacing between screws, when designing connections with inclined screws. Based on tests Kevarinmäki (2002) recommend that the following end- and edge distances as well as spacing are used when designing joint connections with tension- or crossed screws see Table 4.3 and Figure 4.3.

Table 4.3: Proposal for minimum screw spacings, end and edge distances for inclined screw joints based on the spacings and distances used in joint tests. For cross joints the distances for tensioned and compressed screws (a_5 and a_6) are given minimum and maximum values. L is the screw length and t_1 is member thickness of the one towards the screw head. d is the nominal diameter. (Keverinmäki, 2002)

	Cross screw joint	**Tension screw joint**
a_1 in the grain direction $a_{1,p}$ consecutive crossed screws	$12d$-a_6 $2L\cos\alpha \geq 14d$-a_5	$8d$
a_2 perpendicular to the grain direction	$4d$	$4d$
$a_{3,t}$ joint in tension $a_{3,c}$ joint in compression	$12d$-a_6 $10d$-a_6	$8d$ $6d$
a_4 edge distance	$4d$	$4d$
a_5 pairs of crossed screws $a_{5,p}$ consecutive crossed screws	$0 \ldots 3t_1$ $0 \ldots 2d$	

As mentioned earlier some of these distances are, however, considered to be too conservative.

Kreuzinger and Spengler (2004) made a proposal concerning the SFS WT-T screw, saying that the diameter used when determining edge- and end distances and spacing should be set as $d = 0{,}8 \cdot \phi$, where ϕ is the nominal diameter. Further the distances should be taken from the new German code, DIN-BEKS 2004, which however is for predrilled screws loaded in shear. See appendix A4.

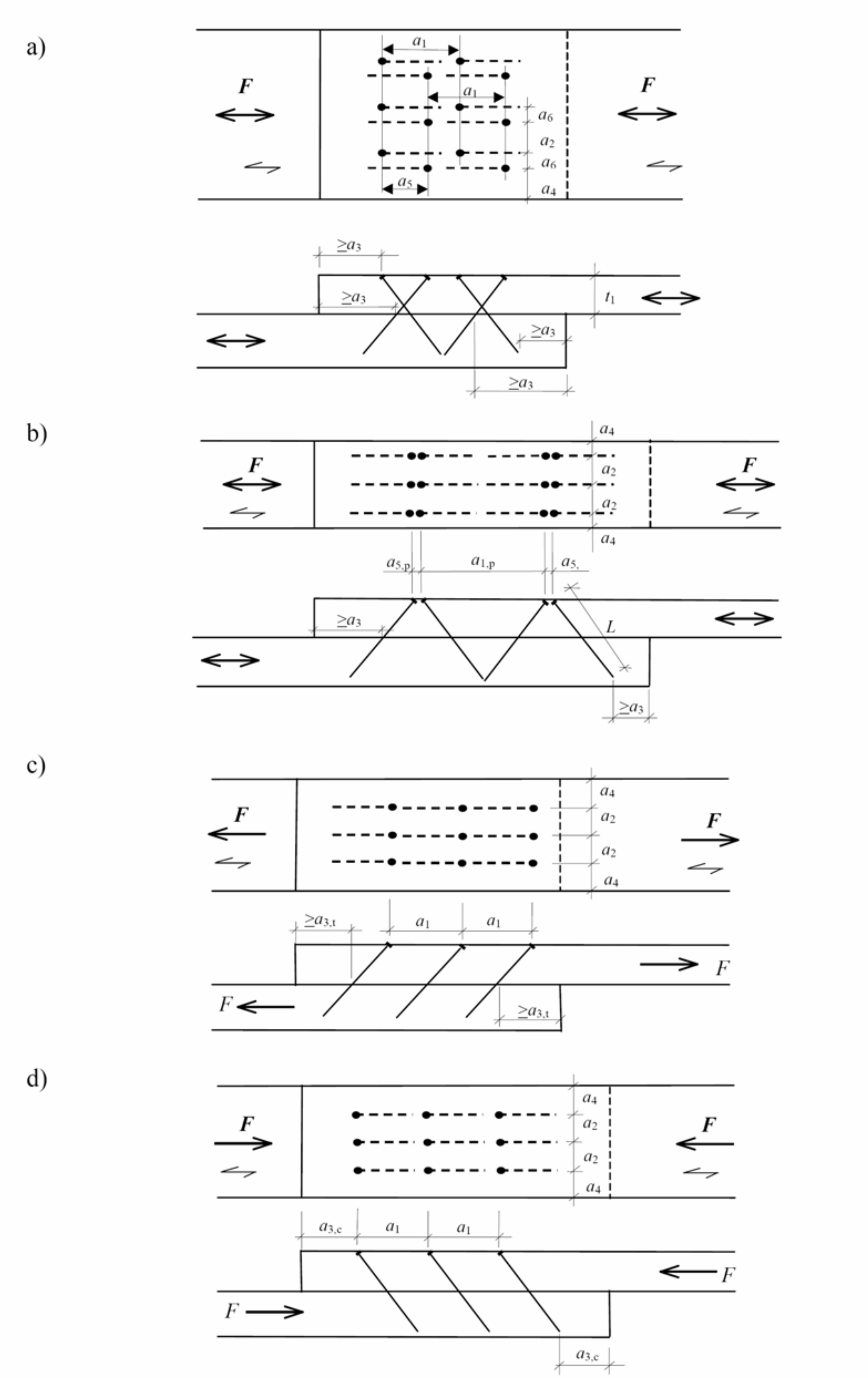


Figure 4.3: Inclined screw joints: a) staggered cross joint, b) consecutive cross joint, c) tension screw joint under a tension force and d) tension screw joint under a compression force. (Kevarinmäki, 2002)

4.4 Determination of edge- and end distances

The rules saying which edge- and end distances that should be used when designing a screwed joint connection, are different depending on country. The distances given in national codes are based on “rule of thumb”. Further Eurocode 5 is a compilation of the codes in the different member states (Larsen, 2004). Some of these proposed distances are conservative, some are not. As a result of lack of knowledge no standard procedure has been proposed to determine these distances (Blaß, 2004). These vague motivations for which distances that should be applied on laterally loaded screws, makes it hard to settle a new code applied on inclined screws. To be able to fully exploit the high load-carrying capacity of the inclined screw, it is of importance that the edge- and end distances and spacing are not too much on the safe side. If the high load-carrying capacity is not utilized, it won’t lead to the cost reduction aimed at.

5. Tests and test results

5.1 General

For all performed tests some general information is given in the list below:

- From a local store for building material 45x145 mm of construction timber K24 was delivered. The 115x115mm and 115x180mm glulam-beams were delivered wrapt in plastic film directly from the glulam producer.
- The glulam was sawn into about 600 mm long test-pieces and the construction timber was sawn into 800 mm long ones.
- All timber was stored in constant climate 20°C / 60% RH. The construction timber was stored in a bit over one month and the glulam for 2,5 months.
- Before testing some of the construction timber was sawn into pieces with a width of 36 mm and some of the glulam was sawn into 49 mm wide pieces before testing.
- In order to apply the screws in a precise angle, special made devices made of aluminium was provided by SFS. See Figure 5.1. The screwdriver used is produced by “Makita”. The model is called DP4700 and has a capacity of 0-550 rpm.
- The tests were carried out directly after the screws were applied.
- The loading was performed in a MTS 322 Test Frame.
- The load was applied in accordance to the loading procedure described in EN 26891 (see Appendix A5). The estimated maximum load, F_{est}, was set to 20 kN. The load was applied up to 0,4 F_{est} and held for 30 s. Thereafter the load was reduced to 0,1 F_{est} and maintained for 30 s. Then the load was increased until reaching the ultimate load. To be able to compare the results $\mathrm{F_{est}}$ was identical for test with both laterally loaded screws and inclined screws.
- For laterally loaded screws and inclined tension screws the slip between the side-members and main-member was recorded.
- For the withdrawal tests the ultimate load and failure type were documented.
- For the tests made upon laterally loaded screws and inclined tension screws the ultimate load, failure type and load at a slip of 15 mm, $\mathrm{F_{15mm}}$, were documented. The load-carrying capacity is defined as the ultimate load or the load corresponding to a 15 mm slip.
- For the withdrawal tests the beams were designed so the bending capacity would not be exceeded and to avoid crushing of the wood at the supports.
- The test setup for laterally loaded screws and inclined tension screws were designed to lead to failure in either one of the side-members. This was however not the case in the majority of the tests.
- After each test a small piece of the beam was sawed up in order to determine the moisture content and density by weighing the test-piece before and after drying it in an oven until all free water evaporated (see chapter 5.2 and 5.3). When necessary the density was adjusted to a 12% moisture content in accordance to the international standard, ISO 3131-1975 (E).
- The screws used in all tests were the SFS WT-T 8,2x300.

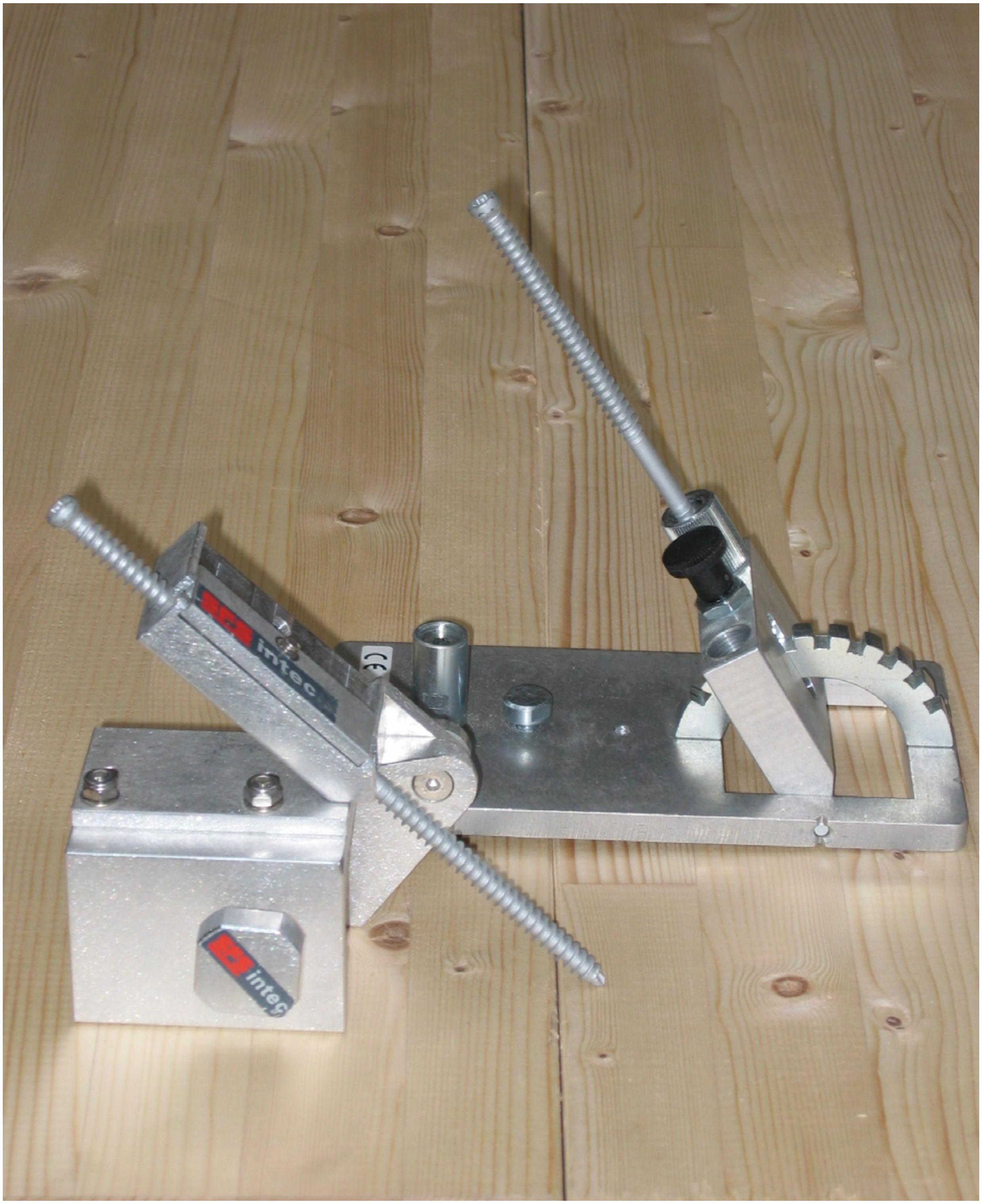


Figure 5.1: Special made devices used to apply the SFS WT-T screw.

5.2 Calculation of characteristic values

The calculations of the characteristic values are based on "Design by testing" (1994). They have been calculated at a confidence level of 75 % assuming that test data are normal distributed. When calculating characteristic values, the coefficient of variation, COV, (COV= standard deviation/mean value) has been set to 10 % in those cases where estimated COV estimated from the experimental data have been lower, unless otherwise specified. Although this rule is not in the Swedish regulations, it is standard practice in e.g. the German DIN standards and in Eurocode. The fractile used is the 5 % fractile. The following formulas were used:

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n} \tag{5.1}$$

$$s = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})}{n-1}} \tag{5.2}$$

$$COV = \frac{s}{\bar{x}} \tag{5.3}$$

$$x_k = \bar{x}\left(1 - k_{pn} \cdot COV\right) \tag{5.4}$$

where

$\bar{x}$	is the arithmetic mean value
x_i	is the test value
n	is the number of values
s	is the standard deviation
COV	is the coefficient of variation
x_k	is the characteristic value
k_{pn}	is a factor depending on the fractile and number of values, see Table 5.1.

Table 5.1: k_{pn} for a few values of n

Number of values, n	5% fractile k_{pn}
3	3,19
4	2,68
5	2,46
6	2,33
7	2,24
8	2,18
9	2,14
10	2,10
100	1,76

5.3 Calculation of the density and moisture content

The dry density and the density at the moisture content W at the time of the test for each test piece were determined. The following equations were used:

$$\rho_{00} = \frac{m_0}{V_0} \tag{5.5}$$

$$\rho_w = \frac{m_w}{V_w} \tag{5.6}$$

$$w = \frac{m_w - m_0}{m_0} \tag{5.7}$$

where

m_0	is the mass after drying [kg]
V_0	is the volume after drying [m^3]
ρ_{00}	is the dry density [kg/m^3]
m_w	is the mass at moisture content W [kg]
V_w	is the volume at moisture content W [m^3]
ρ_w	is the density at moisture content W [kg/m^3]
w	is the moisture content [%]

If the calculated moisture content differs from 12% it has to be adjusted according to the rules in ISO 3131-1975 (E).

$$\rho_{12} = \rho_w \left[1 - \frac{(1-K)(W-12)}{100} \right] \tag{5.8}$$

$$K = 0{,}85 \cdot 10^{-3} \rho_w \tag{5.9}$$

where

ρ_{12}	is the density at moisture content 12% [kg/m^3]

5.4 Withdrawal tests in thin construction timber

5.4.1 Test arrangements

Tests were carried out to examine if it is possible to use the SFS WT-T 8,2x300 screw when connecting roof trusses in thin construction timber without causing splitting. The thickness of the wood-members was 36 mm and 45 mm. The about 800 mm long pieces for the 36 mm and 45 mm thick beams were named A_{36}-I_{36} and A_{45}-F_{45} respectively. Table 5.2 shows how many tests that were performed. The test arrangement is shown in Figure 5.2.

Table 5.2 Withdrawal tests

width, *b*	number of tests, *n*
45 mm	9
36 mm	6

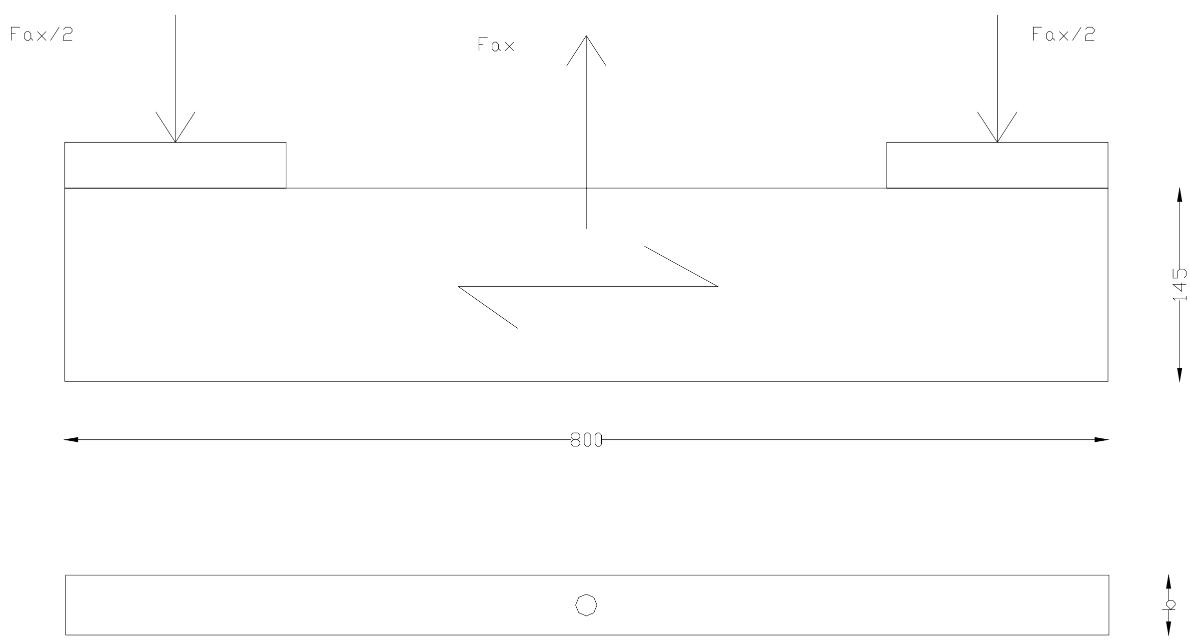


Figure 5.2: Test arrangement for withdrawal tests.

5.4.2 Results of withdrawal tests

The screw used is the SFS WT-T 8,2x300, and the penetration depth is 135 mm in all tests. The test data is presented in Table 5.3. The timber had a moisture content of 14,1 ±1,2%. From the tests made with the width, *b*, set to 36 mm three of them showed screw failure, the rest failed in withdrawal. Test piece D_{36}, E_{36} and F_{36} were taken from the same plank and show a significantly higher density then the others. This is the explanation to why these tests reached such a high load that it led to screw failure. Further, test piece B_{45} also reached screw failure and has a relatively high density. Figure 5.3 shows that there are a few big knots in the penetration-area of the screw and this might have had a reinforcing effect in the timber and led to screw failure in test piece B_{45}.

Table 5.3: Test data received from withdrawal tests. Three tests are made from the same beam, A_{36}-C_{36}, D_{36}-F_{36}, G_{36}-I_{36}, A_{45}-C_{45}, D_{45}-F_{45}.

Test piece	b [mm]	$F_{failure}$ [kN]	ρ_{12} [kg/m^3]	w moisture content [%]	Failure type
A_{36}	36	17,9	*	14,8	withdrawal
B_{36}	36	16,9	*	14,8	withdrawal
C_{36}	36	17,8	*	14,9	withdrawal
D_{36}	36	21,5	504	15,2	screw
E_{36}	36	21,3	556	12,9	screw
F_{36}	36	21,5	545	13,2	screw
G_{36}	36	20,7	392	14,7	withdrawal
H_{36}	36	20,0	396	14,8	withdrawal
I_{36}	36	20,3	396	14,9	withdrawal
A_{45}	45	19,3	382	15,0	withdrawal
B_{45}	45	21,4	397	14,7	screw
C_{45}	45	17,8	377	14,7	withdrawal
D_{45}	45	18,9	379	14,9	withdrawal
E_{45}	45	20,0	387	14,7	withdrawal
F_{45}	45	19,0	375	14,7	withdrawal

* missing data

Figure 5.3: Test piece B_{45}. Screw failure.

In Table 5.4 the calculated average values are presented and compared to theoretical average values according to Eurocode 5 and a proposal made by Hansson and Thelandersson (2003) based on Kevarinmäki (2002). The average values are calculated in two different ways; the first ones only consider the tests that failed in withdrawal and the second ones consider all tests performed for each value of the width *b*. The characteristic values of the withdrawal capacity according to Eurocode 5 depend on the density. The characteristic densities based on Table 5.2 from the tests are used for the calculations and this is why they vary even though the diameter and the penetration depth are the same. It can be seen that the capacity determined from Eurocode 5 is quite conservative and that the thickness of the timber member apparently doesn't affect the withdrawal capacity.

Table 5.4: Results of withdrawal tests. Screws applied 90° to the grain. The bold values are to be compared.

Failure type included in analysis	b [mm]	Average value from test [kN]	Std [kN]	No. of tests	No. of screw failures	Calculated Average value* ****** [kN]	Calculated Average value EC 5 ****** [kN]	Characteristic Value* [kN]	Characteristic Value EC 5 [kN]
withdrawal	36	**18,9**	1,6	6	0	**17,8**	**13,4**	14,7	10,3**
all	36	**19,8**	1,8	9	3	**18,0**	**14,3**	14,7	11,0***
withdrawal	45	**19,0**	0,8	5	0	**17,8**	**14,5**	14,7	11,3****
all	45	**19,4**	1,2	6	1	**17,9**	**14,9**	14,7	11,7*****

* proposed by Hansson and Thelandersson (2003) based on Kevarinmäki (2002)

** the characteristic density used is calculated from the values of ρ_{12} in table 5.2 (G_{36}-I_{36})

*** the characteristic density used is calculated from the values of ρ_{12} in table 5.2 (D_{36}-I_{36})

**** the characteristic density used is calculated from the values of ρ_{12} in table 5.2 (A_{45},C_{45}-F_{45})

***** the characteristic density used is calculated from the values of ρ_{12} in table 5.2 (A_{45}-F_{45})

****** Calculated from: $x_m=x_k+1.65*s$ (where x_m is the average value, x_k is the characteristic value and s is the standard deviation). $s > 0,1 \cdot x_m$ according to Eurocode and DIN.

5.5 Laterally loaded screws

5.5.1 Test arrangements

Figure 5.4 shows how the glulam was sawn and assembled for the tests.

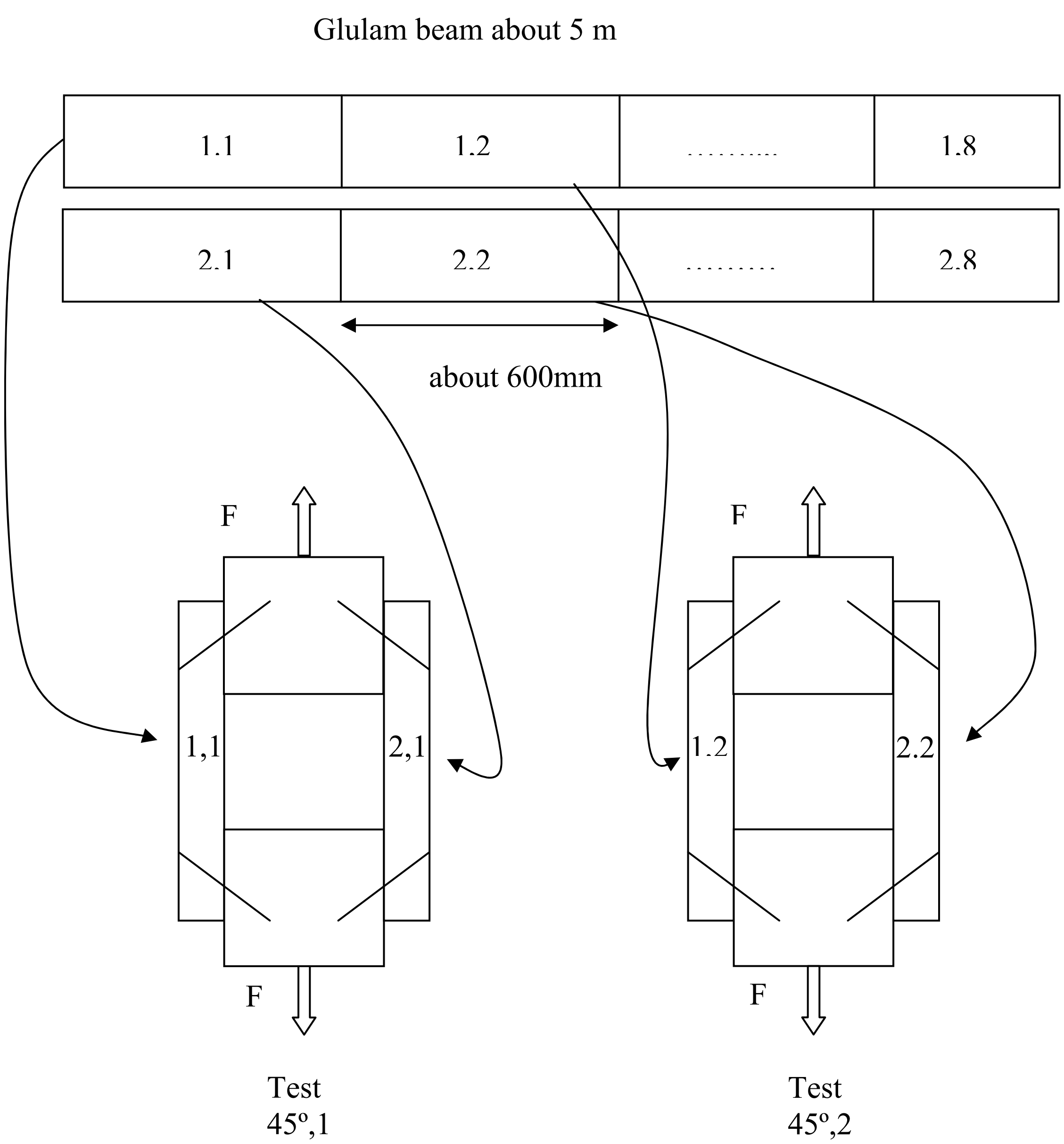


Figure 5.4: The sketch shows how the glulam was sawn and assembled in the tests performed on laterally loaded screws and joints with inclined screws. The test pieces were named after the inclination angle and the test number, as an example 45°,2.

By varying edge- and end distances the influence on the load-carrying capacity was examined. The test arrangement is shown in Figure 5.5.
The distances examined were $a_{3,t}$ and $a_{4,c}$. When performing tests concerning $a_{3,t}$, the distance $a_{4,c}$ was chosen to be very much on the safe side not to influence the results, and vice versa.
The load was transferred to the test piece through a steel device, see Figure 5.6.
Table 5.5 shows which tests were carried out.

Table 5.5 Tests performed on laterally loaded screws.

Parameter examined	width, *b*	$a_{3,t}$	$a_{4,c}$	*n*
$a_{3,t}$	115	**57mm (7d)**	57mm (7d)	5
$a_{4,c}$	49	115mm (14 d)	**25mm(3d)**	3

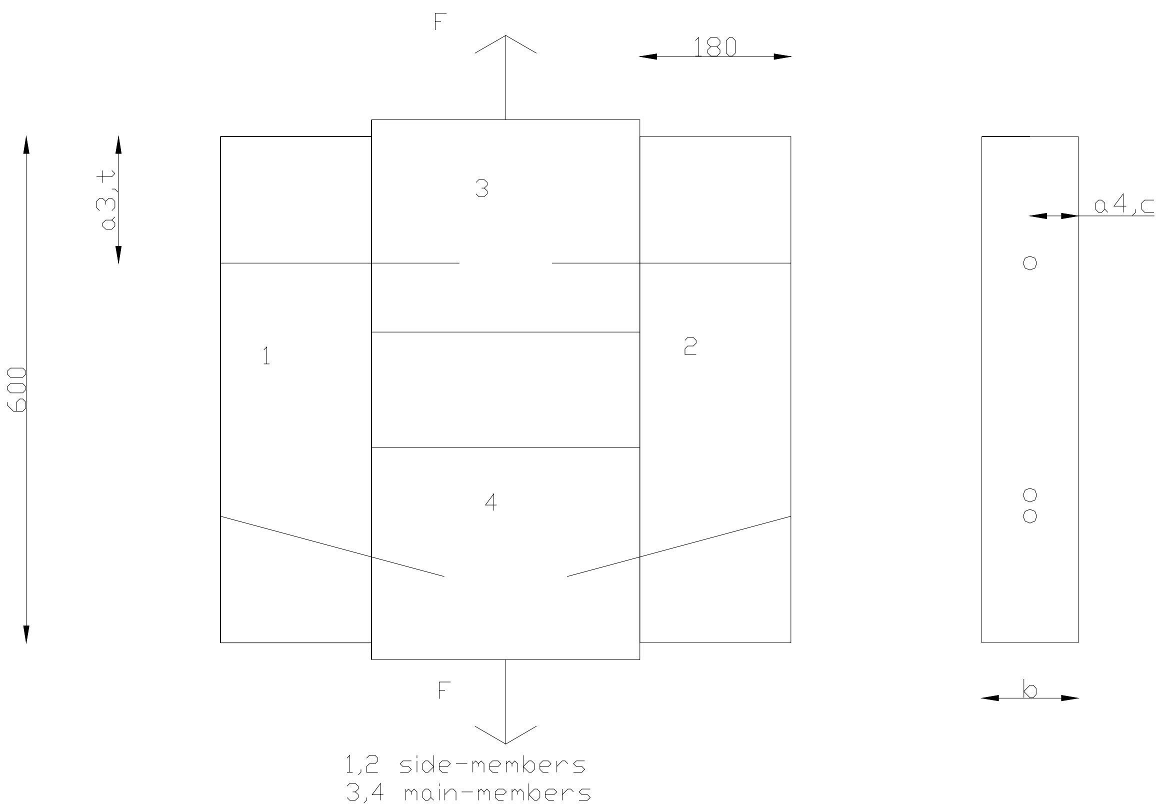


Figure 5.5: Test arrangement for laterally loaded screws.

5.5.2 Results of tests with laterally loaded screws

Joints with laterally loaded screws were tested. The screws used were SFS WT-T 8,2x300. The screw was applied until the middle of the screw was in the intersection point between the main- and side-member. The timber had a moisture content of 12,4 ± 0,3 % for test 90º,1 to 90º,5 and 12,1±0,2 for test 90º,6 to 90º,8 according to Table 5.6. Two different parameter were examined in the tests, $a_{3,t}$ and $a_{4,c}$. The test setup is shown in Figure 5.6. The test data is presented in Table 5.6.

Figure 5.6: Test setup when examining $a_{3,t}$ (left) and $a_{4,c}$ (right) in joints with laterally loaded screws.

When the influence of the distance from the screw to a loaded end, $a_{3,t}$, was examined and the width of the side members were 115 mm, the joints failed exclusively in withdrawal from main- or side member. The connections didn’t show any splitting tendencies. However, when the influence of the distance from the screw to an unloaded edge, $a_{4,c}$, was being examined two splitting failures occurred in the side members but apparently without affecting the strength of the joint. The deformations in all joints were large, and this often leads to withdrawal failures. The relationship between load and density is not known since the density of the main member was not measured.
In Figure 5.8 and 5.9 the relation between the average displacement of the side-members and the load is presented. The displacement temporarily ceases to increase due to the unloading prescribed in EN 26891.

Table 5.6: Test data, laterally loaded screws.

Test	Parameter examined	Test piece	b [mm]	$F_{ultimate}$ [kN]	F_{15mm} [kN]	ρ_{12*} [kg/m^3]	w [%]	Failure type
90°,1	$a_{3,t}$ = 57 mm	12,1	115	21,5	15,8	455	12,6	Withdrawal in main member
		13,1				425	12,4	
90°,2	$a_{3,t}$ = 57 mm	12,2	115	23,5	18,6	458	12,6	Withdrawal in main member
		13,2				436	12,6	
90°,3	$a_{3,t}$ = 57 mm	12,3	115	24,1	18,0	497	12,5	Withdrawal in side member
		13,3				435	12,6	
90°,4	$a_{3,t}$ = 57 mm	12,4	115	22,9	16,6	458	12,6	Withdrawal in main member
		13,4				433	12,7	
90°,5	$a_{3,t}$ = 57 mm	12,5	115	24,9	18,7	450	12,4	Withdrawal in main member
		13,5				414	12,1	
90°,6	$a_{4,c}$ = 25 mm	12,6	49	24,5	18,6	415	11,9	Splitting in side member See fig. 5.7
		13,6				439	12,1	
90°,7	$a_{4,c}$ = 25 mm	12,7	49	22,5	16,6	440	12,3	Withdrawal in main member
		13,7				447	11,9	
90°,8	$a_{4,c}$ = 25 mm	12,8	49	23,3	18,4	436	12,3	Splitting in side member
		13,8				456	12,3	

* Density in side-member regardless if the failure occurred in the main- or side-member.

Figure 5.7: Splitting failure in side-member, test 90°,6.

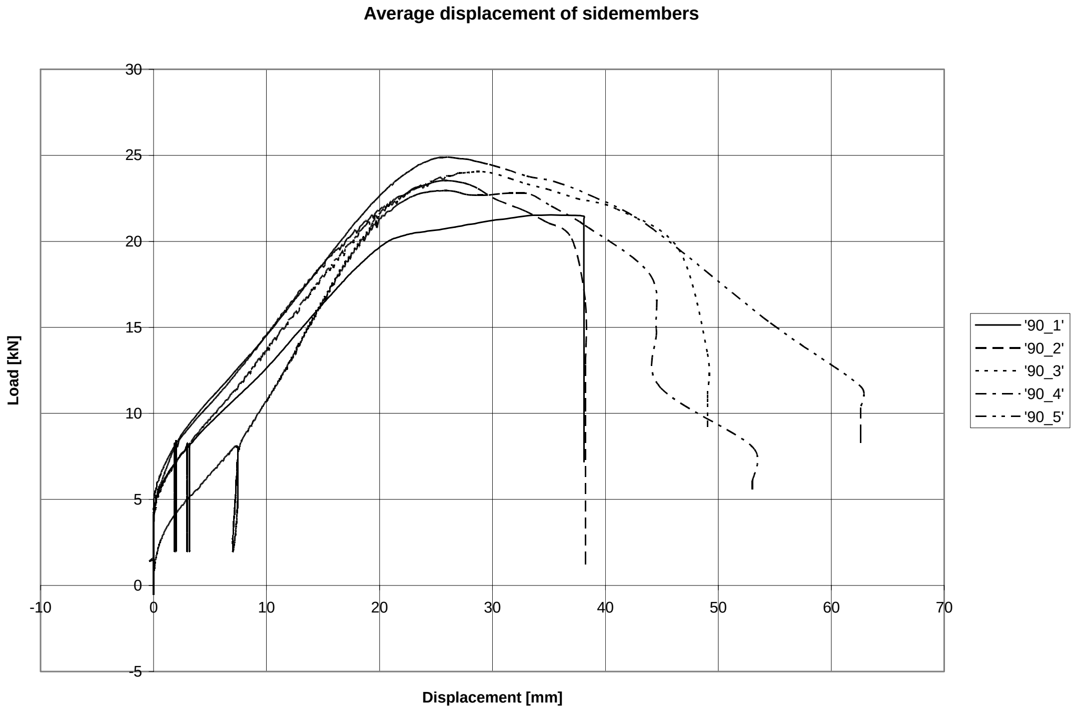


Figure 5.7: The average displacement of side-members for test 90°,1 to 90°,5 (see Table 5.6). The examined distance $a_{3,t}$ = 57 mm.
Width *b*, defined in Figure 5.5 is 115 mm.

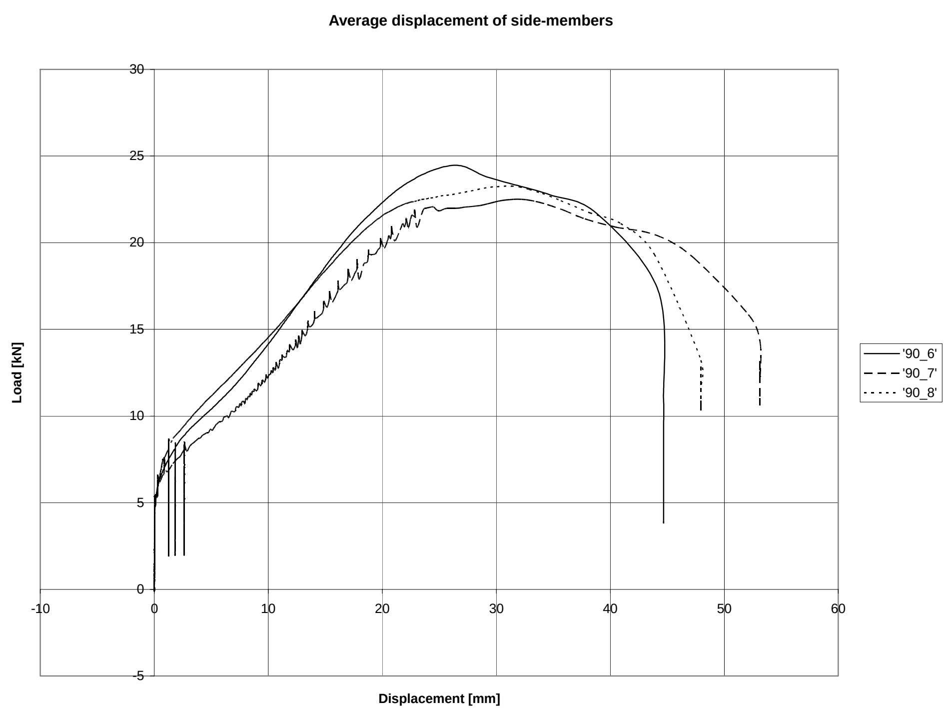


Figure 5.8: The average displacement of side-members for test 90°,6 to 90°,8 (see Table 5.6). The examined distance $a_{4,c}$ = 25mm.
Width *b*, defined in Figure 5.5 is 49 mm.

In Table 5.7 a summary of the tests found in Table 5.5 for laterally loaded screws are presented together with calculated average values according to Eurocode 5 and BKR 2003. The average values are calculated based on the ultimate load F_{ult}, and the load that correspond to a displacement of 15mm, F_{15mm}. The reason is that failure is defined as when the maximum load is reached and the connections fails or when a deformation of 15 mm is reached. The average values are compared with the calculated average values according to Eurocode 5 and BKR 2003. The calculated average values based on BKR 2003 that exclude the rope effect are very conservative. The calculated average values based on Eurocode 5 agree better with the test results but are still somewhat conservative.

Table 5.7: Summary of tests with laterally loaded screws. The test setup is shown in figure 5.6. Two screws in each joint. The bold values are to be compared.

Angle [°]	b [mm]	Edge/ End	Force	Average Value [kN]	Std [kN]	No. of tests	Calculated Average value EC 5** [kN]	Calculated Average value BKR 2003** [kN]	Characteristic value EC 5*** [kN]	Characteristic Value BKR 2003 [kN]
90	115	$a_{3,t}$	$F_{ult.}$	**23,38**	1,29	5	**16,77**	**9,57**	12,92*	5,71
			F_{15mm}	**17,54**	1,28		**15,77**	**8,60**	12,92*	5,71
90	49	$a_{4,c}$	$F_{ult.}$	**23,43**	1,01	3	**16,79**	**9,58**	12,92*	5,71
			F_{15mm}	**17,87**	1,10		**15,87**	**8,66**	12,92*	5,71

* including the rope effect
** Calculated from: $x_m=x_k+1.65*s$ (where x_m is the average value, x_k is the characteristic value and s is the standard deviation). $s > 0,1 \cdot x_m$ according to Eurocode and DIN.
*** ρ_k used for the calculation is 397 kg/m^3.

Mischler & Alabor (2001) performed tests with 8 laterally loaded screws. The distances $a_{3,t}$ and $a_{4,c}$ were not influencing the results as the free end was unloaded. A summary of the test results are presented in Table 5.8.

Table 5.8: Summary for shear joint capacities according to Mischler & Alabor (2001).

Type of joint	No. Of screws	Penetration depth [mm]	No. of tests	Average value [kN]	Std [kN]
”Normal” shear joint	8	135	3	77.3*	5.3

* Calculated from: $x_m=x_k+1.65*s$ (where x_m is the average value, x_k is the characteristic value and s is the standard deviation). $s > 0,1 \cdot x_m$ according to Eurocode and DIN.

Each screw does not reach as high load-carrying capacity as in the joint with 2 laterally loaded screws.

5.6 Joints with inclined tension screws

5.6.1 Test arrangements

The test configurations are shown in Figure 5.10. The loading procedure was the same as for the laterally loaded screws. In the first tests $a_{3,t}$ and $a_{4,c}$ were set to the values Kevarinmäki (2002) have proposed. Kevarinmäki (2002) defined $a_{3,t}$ as the distance from the middle of the screw (intersection point) to the loaded end of the side-member (see Figure 4.3). Table 5.9 shows the distances used in the tests.

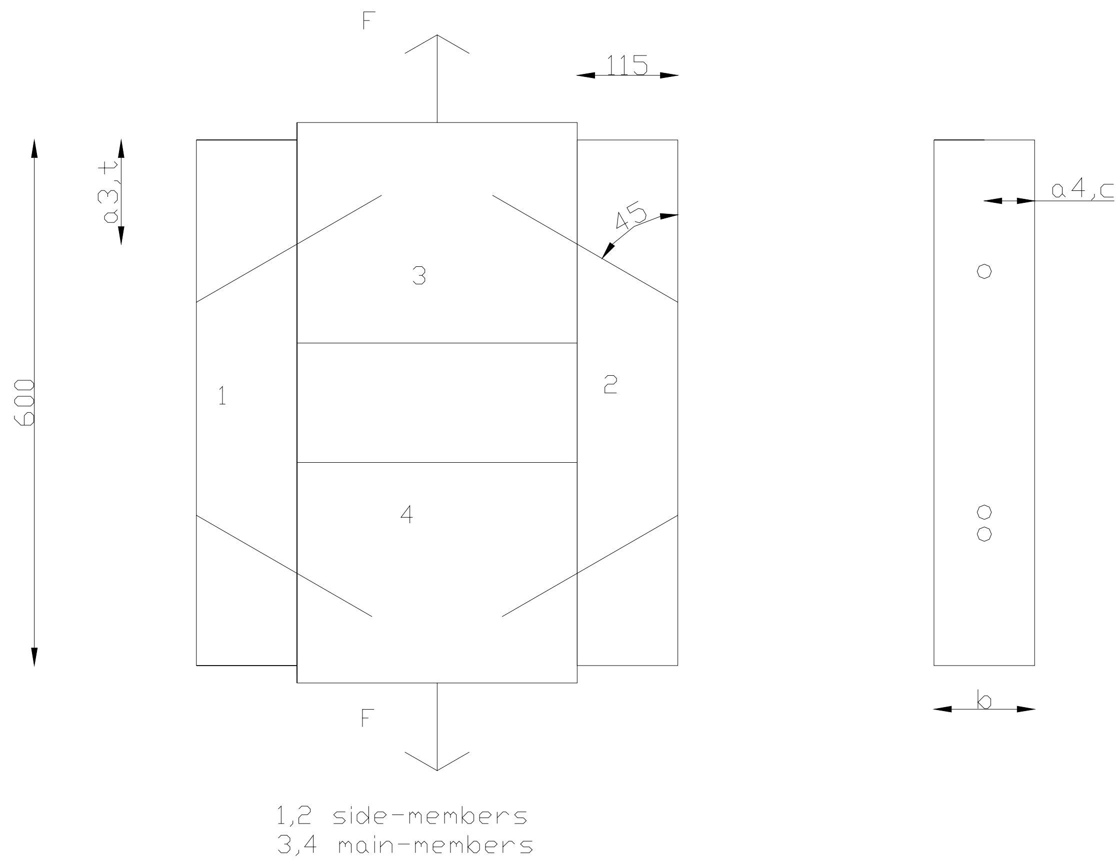


Figure 5.10: Test arrangement for inclined tension screws. $a_{3,t}$ is defined according to Kevarinmäki (2002).

Further tests were carried out with $a_{3,t}$ set to a lower value proposed by Kreuzinger and Spengler (2004), to examine if the one suggested by Kevarinmäki (2002) is too conservative. Kreuzinger and Spengler propose that the end- and edge distances for inclined screws should be taken as the ones applied on laterally loaded screws in the German code, E DIN 1052. In E DIN 1052 $a_{3,t}$ is set to 12d when the load direction is parallel to the grain direction. However, so far, DIN is just applicable on laterally loaded screws and the distances $a_{3,t}$ and a_1 between screws are ambiguous for inclined screws. The proposed definition of $a_{3,t}$ is shown in Figure 5.11. The end distance is measured from the middle of the fastener length per wood piece to the relevant surface. As mentioned earlier Kreuzinger and Spengler also propose that the diameter used in the calculations should be chosen as $d = 0{,}8 \cdot \phi$ where ϕ is the nominal diameter.

The distance from the intersection point of the screw to the loaded end of the side-member will also be displayed to facilitate comparison with the tests made according to Kevarinmäki (2002). The distances are shown in Table 5.10.

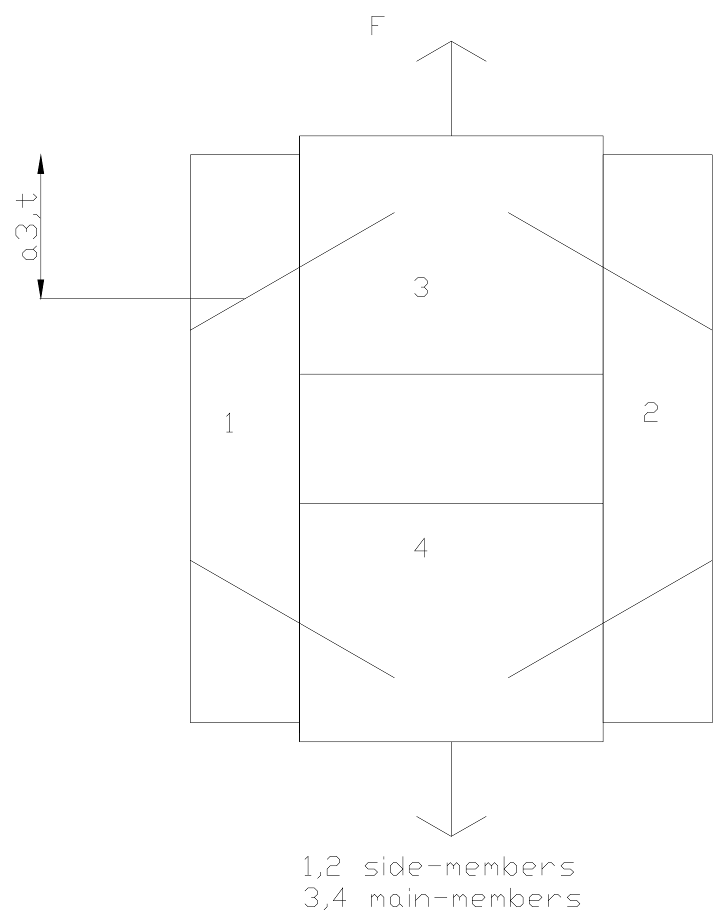


Figure 5.11: The definition of $a_{3,t}$ according to Kreuzinger and Spengler (2004).

Table 5.9: Tests performed on inclined tensions screws with distances according to Kevarinmäki (2002). See Figure 5.10.

Parameter examined	*width b*	*Kevarinmäki*		*Number of tests n*
		$a_{3,t}$	$a_{4,c}$	
$a_{3,t}$	115	**66mm = 8d**	58mm ≈ 7d	4
$a_{4,c}$	49	97mm = 11,8d	**25mm < 3d**	3

Table 5.10: Tests performed on inclined tension screws with $a_{3,t}$ according to Kreuzinger and Spengler (2004) see Figure 5.10 and 5.11.

Parameter examined	***b***	***n***	***Kreuzinger and Spengler***		*Kevarinmäki**	
			$\mathbf{a_{3,t}}$	$\mathbf{a_{4,c}}$	$a_{3,t}$	$a_{4,c}$
$\mathbf{a_{3,t}}$	**115**	**3**	**79mm**	**58mm**	26mm	58mm
a/d d=0,8Φ=0,8·8,2= 6,56mm			**12d**	**8,8d**	3,9d	8,8d
a/d d=8,2mm			**9,6d**	**7d**	3,1d	7d

* The distance proposed by Kreuzinger and Spengler recalculated to match Kevarinmäkis definition to facilitate comparison.

5.6.2 Results of tests with inclined screws

Joints with inclined screws loaded in tension were tested. The screws used were SFS WT-T 8,2x300. The penetration depth was 135 mm. The timber had a moisture content of 11,8 ± 0,3 % for test 45º,1 to 45º,6 and 11,1±0,2 for test 45º,7 to 45º,9 according to Table 5.9. Two different parameter were examined in the tests, $a_{3,t}$ and $a_{4,c}$. The test configuration is shown is Figure 5.12. The test data is presented in Table 5.11.

Figure 5.12: Test set up when examining $a_{3,t}$ (left) and $a_{4,c}$ (right) in joints with inclined tension screws.

When the influence of the distance from the screw to a loaded end,$a_{3,t}$, was examined and the width of the side members were 115 mm, the joints failed exclusively in withdrawal. Neither could any splitting tendencies be observed. Further did the tests performed to study the influence of the distance from the screw to an unloaded edge, $a_{4,c}$, also all fail in withdrawal. This in difference to the previous tests performed on laterally loaded screws. The deformations are not as large as in joints with laterally loaded screws. In none of the tests the displacement of a side-member reached 15 mm. The displacement at the time of failure, $\delta(F_{ultimate})$, is presented in Table 5.11.

In Figure 5.14 and 5.15 the relation between the average displacement of the side-members and the applied load is presented.

Two tests were performed where the screws were screwed in too deep. That is, the mid-point of the smooth shank, at half length of the screws was not at the intersection point of side-members and main-member but approximately 2 cm inside the main-member. With this as the only difference the tests were carried out as tests 45º,1 and 45º,4 to 45º,6. The test data is presented in Table 5.11. The timber used had a moisture content of 11,9±0,3%.

Table 5.11: Test data, inclined tension screws.

Test	Parameter examined	Testpiece	b [mm]	F_{ult} [kN]	$\delta(F_{ult})$ [mm]	ρ_{12***} [kg/m^3]	w [%]	Failure
45°,1	$a_{3,t}$	1,1	115	41,9	4,82	498	11,5	Withdrawal from main-member
		2,1			3,34	509	11,8	
45°,4	$a_{3,t}$	1,4	115	32,3	1,41	511	11,5	Withdrawal from main-member
		2,4			1,93	505	12,1	
45°,5	$a_{3,t}$	1,5	115	37,6	4,84	493	11,5	Withdrawal from side-member
		2,5			2,87	469	11,9	
45°,6	$a_{3,t}$	1,6	115	36,8	7,30	481	11,7	Withdrawal from main-member
		2,6			6,17	478	11,9	
45°,7	$a_{4,c}$	3,1	49	39,7	5,82	456	11,0	Withdrawal from side-member*
		4,1			3,44	475	11,2	
45°,8	$a_{4,c}$	3,2	49	36,9	2,64	460	11,1	Withdrawal from side-member
		4,2			6,26	479	11,2	
45°,9	$a_{4,c}$	3,3	49	37,1	1,94	467	11,3	Withdrawal from side-member
		4,3			4,67	455	11,3	
45°,2**	$a_{3,t}$	1,2	115	29,0	6,76	510	11,6	Withdrawal from side-member
		2,2			3,26	485	12,0	
45°,3**	$a_{3,t}$	1,3	115	27,6	2,24	495	11,7	Withdrawal from side-member
		2,3			4,27	530	12,1	

* see Figure 5.13

** Screws to deeply drawn in.

*** Density in side-member regardless if the failure occurred in the main- or side-member.

Figure 5.13: Withdrawal in side-member, test 45°,7.

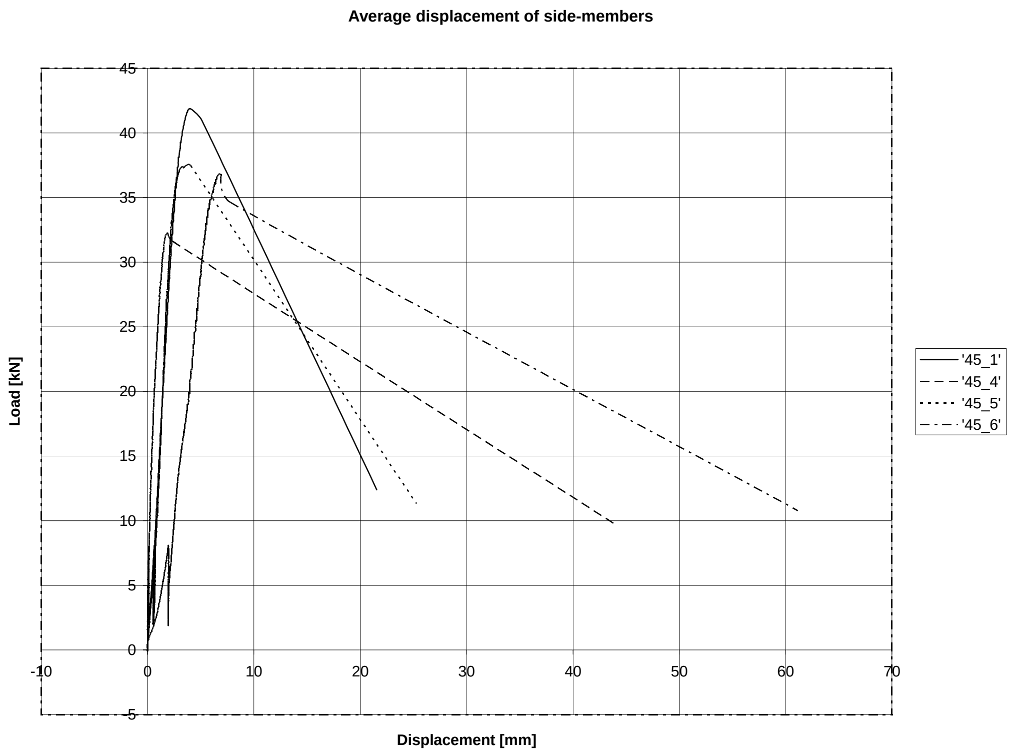


Figure 5.14: Average displacement of side-members for test 45°,1 and 45°,4 to 45º,6. The examined distance is $a_{3,t}$. Width b, defined in Figure 5.10 is 115 mm.

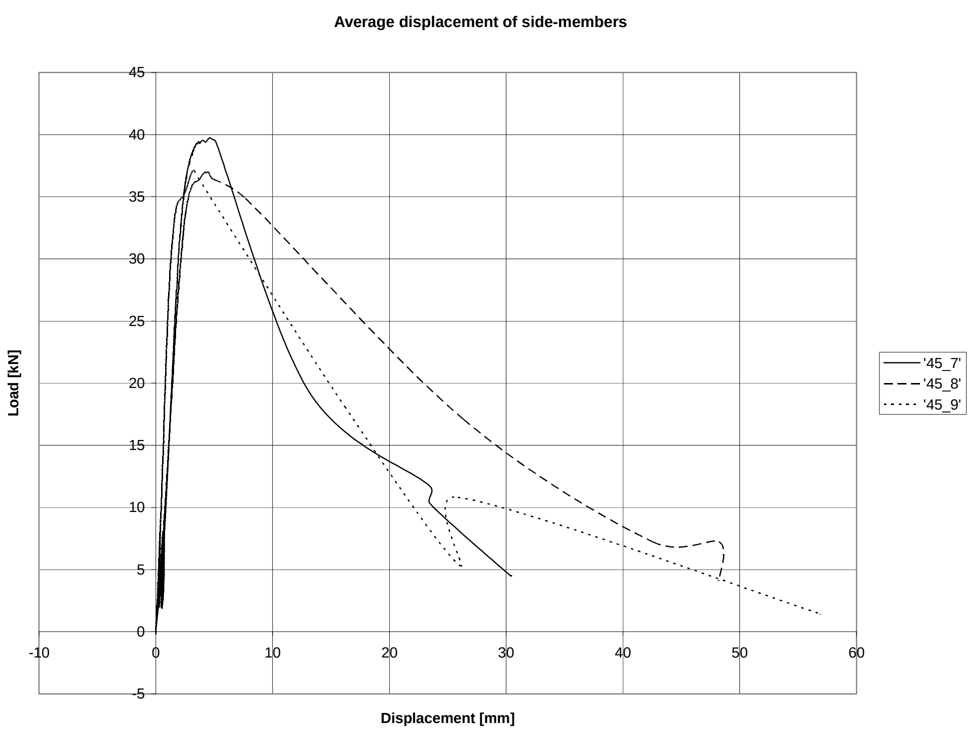


Figure 5.15: average displacement of side-members for test 45°,7 to 45°,9 The examined distance $a_{4,c}$. Width b, defined in Figure 5.10 is 49 mm.

In Table 5.12 a summary of the tests made upon inclined screws loaded in tension are presented. The average values are calculated based on the maximum load, F_{ult}. The calculation of the characteristic value according to Kevarinmäki (see Appendix A6) is based on characteristic values. In the model proposed by Bejtka and Blaß the characteristic embedment strength is included, and is calculated in accordance to Eurocode 5. The value of the density used when calculating the characteristic embedment strength, is the average density for all side-members used in tests with inclined screws. Besides the density only characteristic values are used. The load-carrying capacity according to Bejtka and Blaß consist to 94% of withdrawal strength and the connection would fail in failure mode III. The calculations according to Kevarinmäki and Bejtka and Blaß are found in Appendix A6 and A7.

Table 5.12: Summary of tests with 2 inclined tension screws, 45°. Bold values are to be compared.

b [mm]	Edge/ End	Force	Average value from test [kN]	Std [kN]	No. of tests	Calculated Average value* Kevarinmäki [kN]	Calculated Average value* Bejtka&Blaß [kN]	Characteristic value Kevarinmäki [kN]	Characteristic value Bejtka&Blaß [kN]
115	$a_{3,t}$	$F_{ult.}$	**37,2**	3,93	4	**32,7**	**37,5**	26,2	31**
49	$a_{4,c}$	$F_{ult.}$	**37,9**	1,56	3	**32,5**	**37,3**	26,2	31**
115	$a_{3,t}$	$F_{ult.}$	**28,3*** **	0,7	2	**30,9**	**35,7**	26,2	31**

* Calculated from: $x_m=x_k+1.65*s$ (where x_m is the average value, x_k is the characteristic value and s is the standard deviation). $s > 0,1\cdot x_m$ according to Eurocode and DIN.

** 29,0 kN from withdrawal capacity and 1,93 kN from embedding strength. (see Appendix 7)

*** Average value from tests with too deeply drawn in screws.

The calculated average values according to Kevarinmäki are about 15% lower then the test results. The calculated average values in accordance to the model Bejtka and Blaß proposed are a bit higher partly due to the contribution of the embedment strength. They accord very well with the test results.

As for the too deeply drawn in screws the average value is lower than the calculated average values. The average value of the ultimate load only is about 75% of the average value gotten from the tests with the correct penetration depth. This leads to the conclusion that great precision is required during installation to avoid failure at a load lower than the design load.

Since the edge- and end distances proposed by Kevarinmäki didn't have a negative influence on the load-carrying capacity, further tests with a decreased value on $a_{3,t}$ (see chapter 5.6.1) were carried out. The side members had a moisture content of 11,7±0,2%. The test data is presented in Table 5.13.

Table 5.13: Test data for inclined tension screws with $a_{3,t}$ proposed by Kreuzinger and Spengler. (2004).

Test	Parameter examined	Testpiece	b [mm]	$F_{ultimate}$ [kN]	$\delta(F_{ultimate})$ [mm]	ρ_{12*} [kg/m^3]	w [%]	Failure
45°,10	$a_{3,t}$ = 79mm	3,4	115	30,4	2,91	441	11,6	Withdrawal from main-member
		4,4			3,56	440	11,8	
45°,11	$a_{3,t}$ = 79mm	3,5	115	34,8	3,09	446	11,9	Withdrawal from main-member
		4,5			2,56	432	11,7	
45°,12	$a_{3,t}$ = 79mm	3,6	115	36,8	5,88	451	11,9	Withdrawal from side-member
		4,6			4,16	462	11,9	

* Density in side-member regardless if the failure occurred in the main- or side-member.

In Table 5.14 a summary of the test results are presented. The average value doesn't differ much from the average value gotten from test with $a_{3,t}$ according to Kevarinmäki. The tests show that the joint connection reaches full load-carrying capacity despite reducing $a_{3,t}$.

Table 5.14: Results of tests with inclined tension screws II, 45º.
$a_{3,t}$ was 79 mm in the tests. Bold values are to be compared.

b [mm]	Edge/End	Force	Average value from test [kN]	Std [kN]	No. of tests	Calculated Average value* Kevarinmäki [kN]	Calculated Average value* Bejtka&Blass [kN]	Characteristic value Kevarinmäki [kN]	Characteristic value Bejtka&Blass [kN]
115	$a_{3,t}$ = 79mm	$F_{ult.}$	**34,0**	3,27	3	**31,8**	**36,6**	26,2	31,0**

* Calculated from: x_m=x_k+1.65*s (where x_m is the average value, x_k is the characteristic value and s is the standard deviation). s > 0,1·x_m according to Eurocode and DIN.

** 29,0 kN from withdrawal capacity and 1,93 kN from embedding strength. (see Appendix 7)

Mischler & Alabor (2001) carried out similar tests on joints with 4 SFS WT-T 8,2x300 screws inclined 45°. Compared to this study the end was unloaded in the tests by Mischler and Alabor. A summary of the test results are presented in Table 5.15.

Table 5.15: A summary of test results on inclined tension screws, 45°, according to Mischler & Alabor (2001)

Type of joint	No. Of screws	Penetration depth [mm]	No. of tests	Average value [kN]	Std [kN]
Tension 45°	4	135	5	**76.2**	10.2

* Calculated from: x_m=x_k+1.65*s (where x_m is the average value, x_k is the characteristic value and s is the standard deviation). s > 0,1·x_m according to Eurocode and DIN.

In difference to the laterally loaded screws, here each screw reach as high or higher load-carrying capacity compared to the tests performed on 2 inclined tension screws.

5.7 Splitting test

5.7.1 Test arrangement

In order to examine the splitting tendency and cracks between screws 28 SFS WT-T 8,2x300 screws were applied without pre-drilling in the end grain of a 115x180mm glulam beam in strength class L40. The placement of the screws is shown in Figure 5.16. The test piece was placed in a climatic chamber with a temperature of 35° C and 45% relative humidity, for 5 weeks.

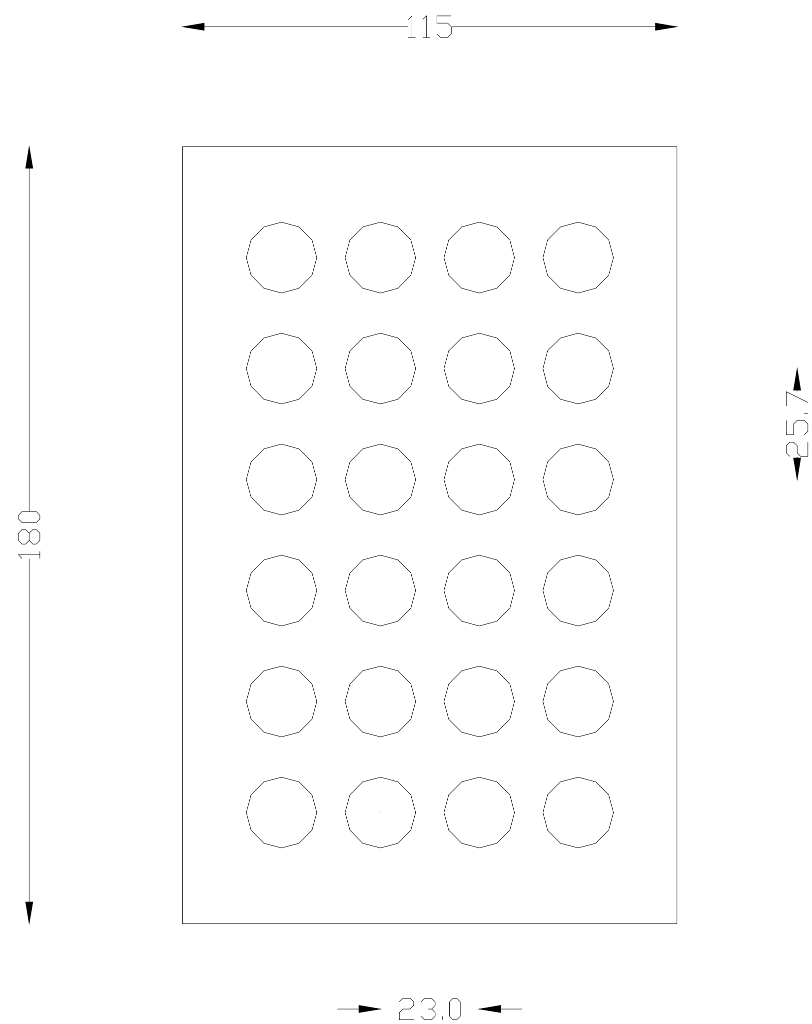


Figure 5.16: The placement of the screws in splitting test.

5.7.2 Results of splitting tests

The density and moisture content of the test piece were determined and are presented in Table 5.16.

Table 5.1 Test data for splitting test.

Test	ρ_{12} [kg/m³]	w [%]
Splitting	442	8,6

Figure 5.17 and 5.18 show the test piece before being placed in a climatic chamber and after 5 weeks in 35ºC and 45% RH.

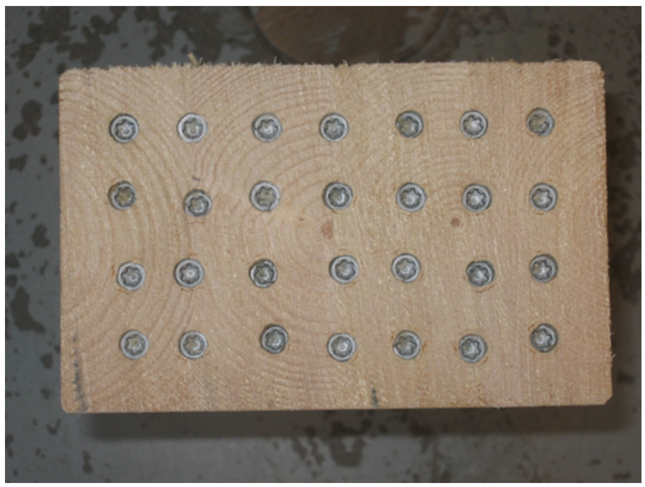


Figure 5.17: The test piece before being placed in a climatic chamber.

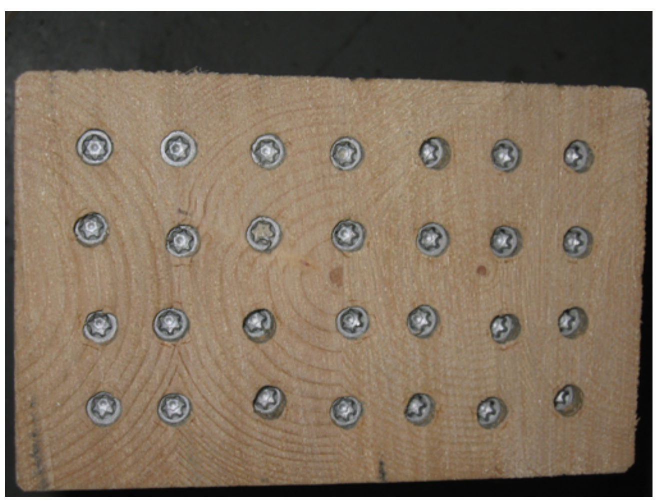


Figure 5.18: The test piece after drying.

The test piece didn’t show any splitting tendencies despite the small distance between the screws. Small cracks around the screws were observed but the extent of them is very small. More tests with screws applied perpendicular to the grain should be carried out before any further conclusions are drawn.

6. Conclusions

The conclusions drawn from this study is listed below.

- The withdrawal capacity of the SFS WT-T 8,2x300 screw in slender construction timber is significantly higher than according to Eurocode 5 but in line with the proposal by Hansson and Thelandersson (2003). Full withdrawal capacity is reached also in slender construction timber so this is not a concern when using the screw in roof truss connections.
- Even though two of the tests made upon laterally loaded screws failed in splitting they still reached full load-carrying capacity.
- The test results show that Kevarinmäki's proposal concerning the distance to a loaded end, $a_{3,t}$, and the distance to an unloaded edge is conservative.
- When decreasing the value of $a_{3,t}$ in accordance to Kreuzinger and Spengler (2004) the connection still reaches full load-carrying capacity and doesn't show any splitting tendencies.
- In difference to the connections with laterally loaded screws, did neither of the connections examining $a_{4,c}$, using inclined screws fail in splitting.
- However, other problems may occur in screwed joints with inclined screws when the distance to the end surface is this short. Using long screws under certain angles can result in the screw coming out of the end surface of the side-member if the distance is not long enough.
  It is important that the definition of $a_{3,t}$ is settled, that is, from where it should be measured. The load-carrying capacities of the screws are known. When the definition is clear more tests can be carried out in order to determine which end- and edge distances that should be used when designing screwed joints with inclined screws. Using different definitions cause problems, for instance when using very long screws, as described above. It is also difficult comparing different test results when the definitions of the end distance are not the same.
- Further are the deformations much smaller in joints with inclined tension screws than in joints with laterally loaded screws.
- The screw is not allowed to be applied further into the timber member than it is designed for in order to reach full load-carrying capacity.
- A comparison of the results from this study made with the test performed by Mischler and Alabor (2002), indicate that the load-carrying capacity is better utilized in joints with inclined tensions screws than in shear joints.
- When no load is applied the screws can be placed very close to one another in the end grain without causing splitting.

7. Discussion

The importance of relevant end, edge distances as well as the distance between screws have been emphasized by the industry. As mentioned before the distances in use today for laterally loaded screws are based on the “rule of thumb” and no comprehensive document is to be found as a result of lack of knowledge in the underlying mechanisms. In the meanwhile the distances have to be based on tests.
The tests should investigate the influence different parameters have on the load-carrying capacity and splitting tendencies. The parameters that can be of importance are principally the same as for laterally loaded screw (see chapter 3.3). However, there are also a few additional parameters, and a few that will be of a different influence compared to screws loaded in shear:

- edge- and end distances
- spacing between screws
- number of screws in a row
- the inclination of the screw
- withdrawal capacity
  because of simultaneous withdrawal and shear

As for the edge and end distances for inclined screws many tests need to be carried out before general rules can be set. Test with different angles between the screw and the grain direction has to be done in order to determine the influence the angle has on the load-carrying capacity and the splitting tendencies of the joint. The moisture content of the timber and the timber species also have to be varied.
Further studies concerning the relationship between number of screws in a row and the load-carrying capacity should be performed.
Although the model Bejtka and Blaß proposed accord well with the test results it is difficult to know which in-data that ought to be used.

A1. Axially Loaded Screws

The withdrawal capacity according to different codes.

DIN 1052 May 2000-German Standard

$$R_{ax,k} = \min\begin{cases} f_{1,k} \cdot d \cdot l_{ef} \\ f_{2,k} \cdot d_k^2 \end{cases}$$

Where:

$R_{ax,k}$ is the characteristic withdrawal capacity [N]

$f_{1,k}$ is the characteristic withdrawal parameter [MPa]

$f_{2,k}$ is the characteristic pull-through parameter strength [MPa]

l_{ef} is the point side penetration length of the threaded part [mm]

d is the nominal diameter [mm]

d_k is the outer diameter of the screw head [mm]

BEKS, DIN (Neu)-German Standard

$$R_{ax,k} = \min\begin{cases} \dfrac{f_{1,k} \cdot d \cdot l_{ef}}{\sin^2\alpha + \frac{4}{3}\cos^2\alpha} \\ f_{2,k} \cdot d_k^2 \end{cases}$$

Where:

$R_{ax,k}$ is the characteristic withdrawal capacity [N]

$f_{1,k}$ is the characteristic withdrawal parameter [MPa]

$f_{2,k}$ is the characteristic pull-through parameter strength [MPa]

l_{ef} is the point side penetration length of the threaded part [mm]

d is the nominal diameter [mm]

d_k is the outer diameter of the screw head [mm]

DIN (Alt)-German Standard

$$zul\ N_Z = 3 \cdot s_g \cdot d_s$$

Where:

N_z	is the maximum withdrawal force allowed [N]
s_g	is the penetration length of the threaded part [mm]
d_s	is the shank diameter measured on the smooth part [mm]

BKR 2003-Swedish Standard

$$R_{tk} = 11(2{,}5 + d)(l_g - d)$$

Where:

R_{tk}	is the characteristic withdrawal capacity [N]
d	is the shank diameter [mm]
l_g	is the length of the threaded part in the point side member [mm]

NS-Norwegian Standard

$$R_k = (20 + 8d)(l_{ef} - d)$$

Where:

R_k	is the characteristic withdrawal capacity [N]
d	is the shank diameter [mm]
l_g	is the length of the threaded part in the point side member [mm]

DS-Danish Standard

$$F_{ax,k} = (30 + 12d)(l_g - d)$$

Where:

$F_{ax,k}$	is the characteristic withdrawal capacity [N]
d	is the shank diameter [mm]
l_g	is the length of the threaded part in the point side member [mm]

$$F_{ax,\alpha,Rk} = n_{ef} \left(\pi d l_{ef} \right)^{0,8} f_{v,ax,\alpha,k}$$

$$f_{ax,k} = 3,6 \cdot 10^{-3} \cdot \rho_k^{1,5}$$

$$f_{v,ax,\alpha,k} = \frac{f_{ax,k}}{\sin^2 \alpha + 1,5 \cos^2 \alpha}$$

Where:

$F_{ax,\alpha,Rk}$	is the characteristic withdrawal strength of the connection at an angle to the grain [N]
n_{ef}	is the effective number of screws [-]
d	is the outer diameter measured on the threaded part [mm]
l_{ef}	is the point side penetration length of the threaded part minus one screw diameter [mm]
$f_{ax,k}$	is the characteristic withdrawal strength perpendicular to the grain [MPa]
ρ_k	is the characteristic density [kg/m^3]
$f_{v,ax,\alpha,k}$	is the characteristic withdrawal strength at an angle α to the grain [MPa]

Effective number of screws

$$n_{ef} = n^{0,9}$$

A2. Laterally loaded screws

DIN 1052 May 2000-German Code

The following formulas are valid for wood screws with a diameter less or equal to 8 mm.

$$M_{y,k} = 100 \cdot d^{2,6}\ Nmm$$

Where:

M_{yk}	is the characteristic yield moment [Nmm]
d	is the nominal diameter [mm]

$$R_k = \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{2 \cdot M_k \cdot f_{h,1,k} \cdot d}$$

The required thickness $t_{1,req}$ for side member 1 is:

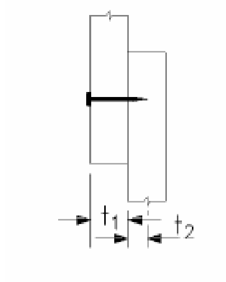
$$t_{1,req} = \left(2 \cdot \sqrt{\frac{\beta}{1+\beta}} + 2 \right) \cdot \sqrt{\frac{M_{y,k}}{f_{h,1,k} \cdot d}}$$

The required thickness $t_{2,req}$ for side member 2 in a single shear connection is:

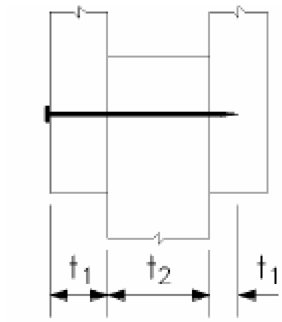
$$t_{2,req} = \left(2 \cdot \frac{1}{\sqrt{1+\beta}} + 2 \right) \cdot \sqrt{\frac{M_{y,k}}{f_{h,2,k} \cdot d}}$$

The required thickness $t_{2,req}$ for the central member in a double shear connection is:

$$t_{2,req} = \left(\frac{4}{\sqrt{1+\beta}} \right) \cdot \sqrt{\frac{M_{y,k}}{f_{h,2,k} \cdot d}}$$



Single shear



Double shear

Where :

R_k	is the characteristic load-carrying capacity per shear plane per fastener
t_1, t_2	are the thicknesses of the timber members or the penetration depth (the lower value is determinant)
$f_{h,1,k}$, $f_{h,2,k}$	are the characteristic embedment strength in timber member 1 or 2
β	$f_{h,1,k} / f_{h,2,k}$
d	is the diameter of the connector
$M_{y,k}$	is the characteristic yield moment

BEKS, DIN (Neu)-German Code

The following formulas are valid for wood screws with a diameter less or equal to 8 mm.

$$M_{y,k} = 0{,}15 \cdot f_{u,k} \cdot d^{2,6}\ Nmm$$

Where:

M_{yk}	is the characteristic yield moment
$f_{u,k}$	is the characteristic tensile strength
d	is the nominal diameter

$$R_k = \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{2 \cdot M_k \cdot f_{h,1,k} \cdot d}$$

The required thickness $t_{1,req}$ for side member 1 is:

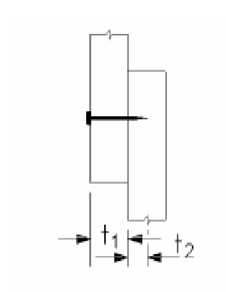
$$t_{1,req} = 1{,}15\left(2 \cdot \sqrt{\frac{\beta}{1+\beta}} + 2\right) \cdot \sqrt{\frac{M_{y,k}}{f_{h,1,k} \cdot d}}$$

The required thickness $t_{2,req}$ for side member 2 in a single shear connection is:

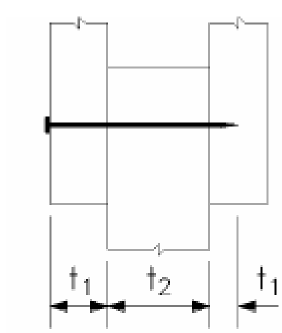
$$t_{2,req} = 1{,}15\left(2 \cdot \frac{1}{\sqrt{1+\beta}} + 2\right) \cdot \sqrt{\frac{M_{y,k}}{f_{h,2,k} \cdot d}}$$

The required thickness $t_{2,req}$ for the central member in a double shear connection is:

$$t_{2,req} = 1{,}15\left(\frac{4}{\sqrt{1+\beta}}\right) \cdot \sqrt{\frac{M_{y,k}}{f_{h,2,k} \cdot d}}$$



Single shear



Double shear

Where :

R_k	is the characteristic load-carrying capacity per shear plane per fastener
t_1,t_2	are the thicknesses of the timber members or the penetration depth (the lower value is determinant)
$f_{h,1,k}$, $f_{h,2,k}$	are the characteristic embedment strength in timber member 1 or 2
β	$f_{h,1,k} / f_{h,2,k}$
d	is the diameter of the connector
$M_{y,k}$	is the characteristic yield moment

DIN (Alt)-German Code

<table>
<tr><td rowspan="3">Angle between force- and grain direction α</td><td>Nominal ϕ</td><td colspan="4">$zul\ N_1^{1)}$</td></tr>
<tr><td rowspan="2">d_s</td><td colspan="2">$s \geq 8d_s$</td><td colspan="2">$4d_s \leq s < 8d_s$</td></tr>
<tr><td>$a_1^{2)} \geq 4{,}25d_s$</td><td>$a_1^{2)} < 4{,}25d_s$</td><td>$a_1^{2)} \geq 4{,}25d_s$</td><td>$a_1^{2)} < 4{,}25d_s$</td></tr>
<tr><td></td><td>mm</td><td>N</td><td>N</td><td>N</td><td>N</td></tr>
<tr><td rowspan="4">arbitrary</td><td>4</td><td>270</td><td rowspan="8">$4\,a_1\,d_s$</td><td rowspan="8">$2{,}125\,d_s\,s$</td><td rowspan="8">$0{,}5\,a_1\,s$</td></tr>
<tr><td>5</td><td>425</td></tr>
<tr><td>6</td><td>610</td></tr>
<tr><td>8</td><td>1090</td></tr>
<tr><td rowspan="4">$0^{\circ\,3)}$</td><td>10</td><td>1700</td></tr>
<tr><td>12</td><td>2450</td></tr>
<tr><td>16</td><td>4350</td></tr>
<tr><td>20</td><td>6800</td></tr>
</table>

1) s, a_1, d_s in mm

2) Minimum thickness of steel plates: FP,HFM: min a_1 = 6mm
HFH: min a_1 = 4mm

3) $\alpha \neq 0°$: reduce zul N with $\eta_s = 1 - \alpha / 360°$

Effective number of screws

for d_s < 10 mm $\quad ef\ n = 10 + (n-10)\cdot 2/3 \qquad n \leq 30$

for $d_s \geq$ 10 mm $\quad ef\ n = 6 + (n-6)\cdot 2/3 \quad \leq 10$

BKR 2003-Swedish Code

The characteristic load-carrying capacity per shear plane per fastener, R_{vk} [N],is the minimum value calculated with the following formulas:

$$R_{vk} \le \begin{cases} 6(\kappa_1 t_1 + \kappa_2 t_2)d & \text{only single shear} \\ 12\kappa_2 t_2 \ d & \text{only double shear} \\ 24\kappa_1 t_1 d & \\ 4\kappa_1 t_1 d + 22 d^2 & \\ 30 d^2 \sqrt{\kappa_1 + \kappa_2}\sqrt{\dfrac{f_{yk}}{240}} & \end{cases}$$

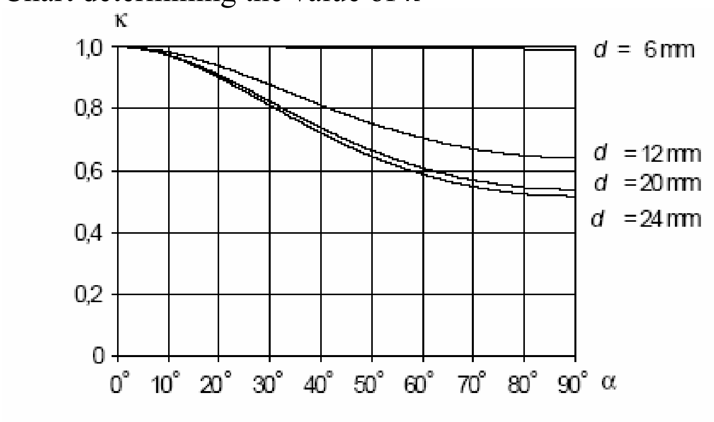
where:

t	is the thickness of the timber member [mm]
d	is the screw diameter [mm]
f_{yk}	is the characteristic tensile strength of the screw material [MPa]
κ	is a factor that takes the angle between force and grain direction into consideration . κ is determined by the chart or equation below
α	is the angle between force- and grain direction [°]

In double shear index 1 stands for side member and index 2 stands for central member. In single shear it is assumed that the index is chosen so that

$$\kappa_1 t_1 \leq \kappa_2 t_2$$

Chart determining the value of κ



Formulas determining the value of κ

$$\kappa = \frac{\kappa_{90}}{\kappa_{90}\cos^2\alpha + \sin^2\alpha}$$

$$\kappa_{90} = 0{,}45 + 8d^{-1{,}5}$$

<u>NS-Norwegian Code</u>

The following condition should be fulfilled:

$$\frac{F_f}{R_d} \leq 1{,}0$$

The characteristic load-carrying capacity, R_k [N], per shear plane per fastener for screws with a minimum yield stress f_y 240 N/mm^2 is calculated as the minimum value from the following equations:

Single shear:

$$R_k = 5\left(k_{v1} \cdot t_1 + k_{v2} \cdot t_2\right) \cdot d$$

Single and double shear:

$$R_k = 25 \cdot k_{v1} \cdot t_1 \cdot d$$

$$R_k = 21 d^2 \cdot \sqrt{\frac{f_y}{240}} + 5\, k_{v1} \cdot t_1 \cdot d$$

$$R_k = 42\, d^2 \cdot \sqrt{\frac{2\, k_{v1} \cdot k_{v2}}{k_{v1} + k_{v2}}} \cdot \sqrt{\frac{f_y}{240}}$$

$$k_{90} = 0{,}32 + 10 \cdot d^{-1{,}5}$$

$$k_{v1} \ or \ k_{v2} = \frac{k_{90}}{k_{90}\ \cos^2 \alpha + \sin^2\ \alpha}$$

where:

t_1	is the thickness of the wood member that is getting attached [mm]
d	is the shank diameter of the screw [mm]
k_{v1}, k_{v2}	are factors depending on the angle between force- and grain direction k_{v1} is the wood member under the screw head k_{v2} is the wood member with the screw point
f_y	is the yield stress of the steel [N/mm^2]

It is assumed that the length, *l*, of the screw in the wood member containing the screw point, is 8d. This length can not be less than 4d. For lengths between 4d and 8d the load-carrying capacity is decreased with a factor *l*/8d.

If there are more than six screws after one another in the force direction the effective number of screws is taken as:

$$n_{ef} = 6 + (n - 6) \cdot \frac{2}{3}$$

DS-Danish Code

The characteristic load-carrying capacity per shear plane per fastener, $F_{tv,k}$ [N] is taken as the minimum value from the following expressions:

$$F_{tv,k} = \min \begin{cases} 25\, k_{1,\alpha}\, t\, d \\ 5\, k_{1,\alpha}\, t\, d + 25\, d^2 \\ 58\, d^2 \sqrt{\dfrac{k_{1,\alpha} + k_{2,\alpha}}{2}} \sqrt{\dfrac{f_y}{240}} \end{cases}$$

where:

t	is the thickness of the wood member [mm]
d	is the screw diameter in mm, measured on the smooth shank [mm]
$k_{1,\alpha}$ and $k_{2,\alpha}$	are factors depending on the angle between force- and grain direction $k_{1,\alpha}$ is the wood member under the screw head $k_{2,\alpha}$ is the wood member with the screw point For beech and oak $k_{1,\alpha}$ and $k_{2,\alpha}$ shall be increased with a factor 1,5.
f_y	is the yield stress of the steel [N/mm^2]

$k_{1,\alpha}$ and $k_{2,\alpha}$ are calculated according to the following formulas:

$$k_{\partial} = \frac{k_{90}}{k_{90} \cos^2 \alpha + \sin^2 \alpha}$$

$$k_{90} = 0{,}45 + 8\, d^{-1{,}5}$$

Eurocode 5 Final Draft (2002-10-09)

The characteristic load-carrying capacity for screws per shear plane per fastener should be taken as the minimum value gotten from the following expressions:

Single shear:

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} t_1 d & (a) \\ f_{h,2,k} t_2 d & (b) \\ \dfrac{f_{h,1,k} t_1 d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1 + \dfrac{t_2}{t_1} + \left(\dfrac{t_2}{t_1}\right)^2\right] + \beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1 + \dfrac{t_2}{t_1}\right)\right] & (c) \\ 1{,}05\dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d\, t_2^2}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} & (d) \\ 1{,}05\dfrac{f_{h,1,k} t_2 d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta) + \dfrac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} d\, t_2^2}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} & (e) \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{ax,Rk}}{4} & (f) \end{cases}$$

Double shear:

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} t_1 d & (g) \\ 0{,}5\, f_{h,2,k} t_2 d & (h) \\ 1{,}05\dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} & (j) \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{ax,Rk}}{4} & (k) \end{cases}$$

With

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}}$$

Where:

$F_{v,Rk}$	is the load carrying capacity per shear plane per screw [N]
t_i	is the timber or board thickness or penetration depth, with i either 1 or 2 [mm]
$f_{h,i,k}$	is the characteristic embedment strength in timber member i [MPa]
d	is the fastener diameter [mm]
$M_{y,Rk}$	is the characteristic fastener yield moment [Nm]
β	is the ratio between the embedment strength of the members [-]
$F_{ax,Rk}$	is the characteristic axially withdrawal capacity of the screw [N]

When plasticity of joints can be assured failure modes (f) and (k) are governing.

In the equations above the first term is the load carrying capacity according to Johansen´s yield theory, and the second term $F_{ax,Rk}/4$ is the contribution by the rope effect. The contribution due to the rope effect should be limited to 100% of the Johansen part.

The effect of the threaded part of the screw shall be taken into account in determining the load carrying capacity, by using and effective diameter d_{ef}.

Smooth shank screws with a diameter d > 6 mm:

The following value for the characteristic yield moment should be used:

$$M_{y,Rk} = 0{,}3 \cdot f_{u,k} \cdot d^{2{,}6}$$

where $f_{u,k}$ is the characteristic tensile strength in N/mm^2.

For screws up to 30 mm in diameter, the following characteristic embedment strength values in timber should be used, at an angle α to the grain:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90}\sin^2\alpha + \cos^2\alpha}$$

$$f_{h,0,k} = 0{,}082\left(1 - 0{,}01d\right)\rho_k$$

where:

$$k_{90} = \begin{cases} 1,35 + 0,015\,d & \text{for softwoods} \\ 0,90 + 0,015\,d & \text{for hardwoods} \end{cases}$$

and:

$f_{h,0,k}$	is the characteristic embedment strength parallel to grain, [MPa]
ρ_k	is the characteristic timber density, [kg/m^3]
d	is the screw diameter, [mm]

For a row of n bolts parallel to the grain direction, for loads parallel to grain direction, the load carrying capacity in that direction should be calculated using the effective number of screws n_{ef} where:

$$n_{ef} = \min \begin{cases} n \\ n^{0,9}\sqrt[4]{\dfrac{a_1}{13\,d}} \end{cases}$$

where:

a_1	is the spacing distance in the grain direction
d	is the fastener diameter

For loads perpendicular to grain, the effective number of screws should be taken as

$$n_{ef} = n$$

A3.

Axially loaded screws			DIN 1052*	BEKS DIN	Blass (2002) Spax S	DIN (1988-04)
			formula	formula**	formula	formula
	Description	Angle				
a_1	‖ to grain	$0^\circ \le \alpha \le 360^\circ$	$(4+3\lvert\cos\alpha\rvert)\,d$	$(4+3\cos\alpha)\,d$		
a_2	∟ to grain	$0^\circ \le \alpha \le 360^\circ$	$(3+\lvert\sin\alpha\rvert)\,d$	3d		
$a_{3,t}$	loaded end	$-90^\circ \le \alpha \le 90^\circ$	$(7+5\lvert\cos\alpha\rvert)\,d$	$(7+5\cos\alpha)\,d$		
$a_{3,c}$	unloaded end	$90^\circ \le \alpha < 150^\circ$ $150^\circ \le \alpha < 210^\circ$ $210^\circ \le \alpha \le 270^\circ$	7d	7d		
$a_{4,t}$	loaded edge	$0^\circ \le \alpha \le 180^\circ$	$(3+4\lvert\sin\alpha\rvert)\,d$	$(3+4\sin\alpha)\,d$		
$a_{4,c}$	unloaded edge	$180^\circ \le \alpha \le 360^\circ$	3d	3d		

Axially loaded screws			BKR 2003	NS***	EC 5		DS****
			formula	formula	formula		formula
	Description	Angle		Same as for screw under shear	Screws driven at right angle to grain	Screws driven in end grain face	
a_1	‖ to grain	$0^\circ \le \alpha \le 360^\circ$			4d	4d	7d
a_2	∟ to grain	$0^\circ \le \alpha \le 360^\circ$			4d	4d	4d
$a_{3,t}$	loaded end	$-90^\circ \le \alpha \le 90^\circ$			4d	2,5d	4d
$a_{3,c}$	unloaded end	$90^\circ \le \alpha < 150^\circ$ $150^\circ \le \alpha < 210^\circ$ $210^\circ \le \alpha \le 270^\circ$			4d	2,5d	4d
$a_{4,t}$	loaded edge	$0^\circ \le \alpha \le 180^\circ$			4d	2,5d	2d
$a_{4,c}$	unloaded edge	$180^\circ \le \alpha \le 360^\circ$			4d	2,5d	2d

* German code
** Absolute value missing?
*** Norwegian code
**** Danish code (from Larsen and Riberholt, 1999)
** Absolute value missing?

A4.

Laterally loaded screws			DIN 1052	BEKS DIN	Blass (2002) Spax S	DIN (1988-04)**
			formula	formula*	formula	formula
	Description	Angle				
a_1	‖ to grain	$0° \le \alpha \le 360°$	(4+3|cos α|) d	(4+3cos α) d	5d	5d
a_2	∟ to grain	$0° \le \alpha \le 360°$	(3+|sin α|) d	3d	5d	5d
$a_{3,t}$	loaded end	$-90° \le \alpha \le 90°$	(7+5|cos α|) d	(7+5cos α) d		10d
$a_{3,c}$	unloaded end	$90° \le \alpha < 150°$ $150° \le \alpha < 210°$ $210° \le \alpha \le 270°$	7d	7d	5d	5d
$a_{4,t}$	loaded edge	$0° \le \alpha \le 180°$	(3+4|sin α|) d	(3+4sin α) d		5d
$a_{4,c}$	unloaded edge	$180° \le \alpha \le 360°$	3d	3d	4d	3d

Laterally loaded screws			BKR 2003***	NS****	EC 5	DS*****
			formula	formula	formula	formula
	Description	Angle				
a_1	‖ to grain	$0° \le \alpha \le 360°$	7d	7d	(4+|cos α|) d	(4+ 3 |cos α|) d
a_2	∟ to grain	$0° \le \alpha \le 360°$	4d	4d	4d	4d
$a_{3,t}$	loaded end	$-90° \le \alpha \le 90°$	7d	7d	max (7d; 80 mm)	max (7d ; 80 mm)
$a_{3,c}$	unloaded end	$90° \le \alpha < 150°$ $150° \le \alpha < 210°$ $210° \le \alpha \le 270°$	7d	4d	max ((1+6 sin α)d ;4d) 4d max ((1+6 sin α)d ;4d)	max ((1+6|sin α|)d; 4d)
$a_{4,t}$	loaded edge	$0° \le \alpha \le 180°$	4d	4d	max((2+2 sin α) d ; 3d)	max ((2+2 sin α) d ;3d)
$a_{4,c}$	unloaded edge	$180° \le \alpha \le 360°$	2d	2d	3d	3d

* Absolute values missing?
** German code (from Schneider, 2002)
*** Swedish code
**** Norwegian code
***** Danish code (from Larsen and Riberholt, 1999)

A5.

Extract from European Standard EN 26891 “Timber structures- Joints made with mechanical fasteners- General principles for the determination of strength and deformation characteristics” (CEN-European Committee for Standardisation, 1991)

Description of the loading procedure.

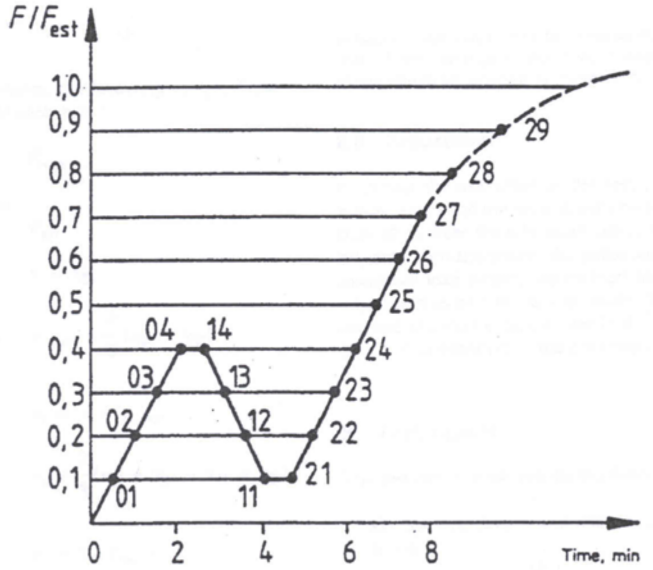


Figure 1 — Loading procedure

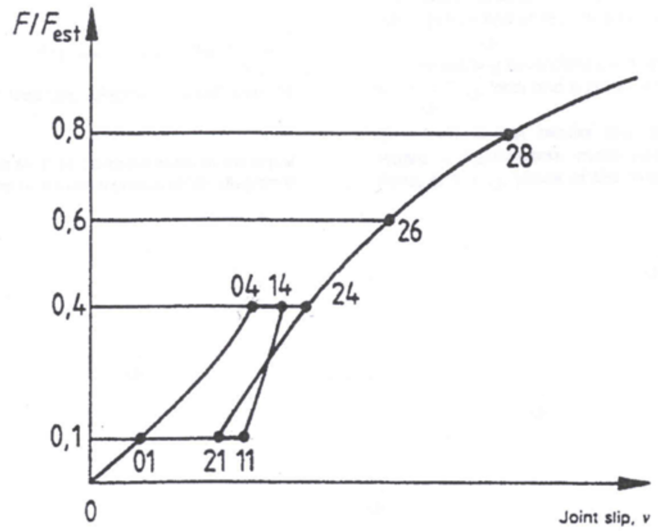


Figure 2 — Idealized load-deformation curve and measurements

A6.

Calculation of the characteristic load-carrying capacity for inclined tension screws according to Kevarinmäki.

$$R_{T,k} = \min\begin{cases} f_{a,1,k}\pi d s_1 + f_{head,k} \cdot d_h^2 \\ f_{a,2,k}\pi d(s_2 - d) \\ F_{u,k} \end{cases}$$

$$R_d = nR_{T,d}(\cos\alpha + \mu\sin\alpha)$$

$f_{a,1,k} = f_{a,2,k} = 4{,}5\ N/mm^2$
$s_1 = s_2 = 135\ mm$
$d = 8{,}2mm$
$f_{head,k} = 0$ for the SFS WT-T screw
$d_k = 0$ for the SFS WT-T screw
$F_{u,k} = 900\ N/mm^2$

$n = 2$
$\alpha = 45°$
$\mu = 0{,}26$

$$R_{T,k} = \min\begin{cases} 4{,}5 \cdot \pi \cdot 8{,}2 \cdot 135 = 15{,}7\ kN \\ 4{,}5 \cdot \pi \cdot 8{,}2 \cdot (135 - 8{,}2) = 14{,}7\ kN \\ 900 \cdot \pi \dfrac{5{,}4^2}{4} = 20{,}6\ kN \end{cases}$$

$$R_d = 2 \cdot 14{,}7(\cos 45 + 0{,}26\sin 45) = 26{,}2\ kN$$

A7.

Calculation of the characteristic load-carrying capacity for inclined tension screws according to Bejtka and Blass.

The load-carrying capacity was calculated using MATLAB 6.5.

```
clear all; close all;clc;
%dbstop in inclined_screw at 53
n=1;
% Blass data

% Serie 1
ah=[45];        % The German definition
alpa=90-ah;             % The Finnish definition
d=8.2e-3;
%l=300*1e-3;
s1=[150*cos(45)]*1e-3;
s2=s1;
My=22;
my=0.25;
fh1=36.7e6;
fh2=fh1;

f1mod=fh1/19.3*[16.5 19.3 19.3 16.5 18.4 18.7 18.3 18.8 19.1 18.4 19.1 18.9];
%see Bejtka and Blass "Joint with inclined screws" CIB-W-18/35-7-5


for i=1

[fail_mode(i),R(i),R_draw(i),R_embed(i)]=inclined_screw_blass(f1mod(i,:),d,s1(i),s2(i),ah(i),my,fh1,f
h2,My);
end
fail_mode
R
R_draw
R_embed
disp('');
disp('Failure load [N]')
R
```

```
function [fail_mode,R,R_draw,R_embed]=inclined_screw_blass(f1mod,d,s1,s2,ah,my,fh1,fh2,M)
% Calculate capacity of inclined screw
% INPUT:
%       f1mod:     vector with all modified embedding parameters
%       d:         nominal diameter of screw
%       s1, s2:    thickness of timber part 1 and 2 respectively
%       ah:        alpha angle. Inserted angle into timber. Degrees
%       my:        friction coefficient. Equal 0.25 in German DIN 1052
%       (May 2000)
%       fh1, fh2:  embedding parameter in timber part 1 and 2 respectively
%       M:         yielding moment
% OUTPUT:
%       fail_mode:  failure mode 1="1a,l";  2="1a,r"  3="1b"  4="2a"  5="2b"  6="3"
%       R:          Load capacity
%       R_draw:     The tension part of the load capacity
%       R_embed:    The embedment part of the load capacity
% joints with inclined screws. CIB W18-35-7-5
% Bejtka I., Blass H. J.

betha=fh2/fh1;
ah=ah*pi/180;       % Radian

% Modified withdrawal parameter f,1,mod,i,j
% i: timber member (member 1 or 2)
% j: Johanssen's failure mode (1a,l; 1a,r; 1b; 2a; 2b; 3)
f1mod11al=f1mod(1);
f1mod21al=f1mod(2);

f1mod11ar=f1mod(3);
f1mod21ar=f1mod(4);

f1mod11b=f1mod(5);
f1mod21b=f1mod(6);

f1mod12a=f1mod(7);
f1mod22a=f1mod(8);

f1mod12b=f1mod(9);
f1mod22b=f1mod(10);

f1mod13=f1mod(11);
f1mod23=f1mod(12);

% eq 14
Rax1ala=f1mod11al*d*s1/cos(ah);
Rax1alb=f1mod21al*d*s2/cos(ah);
Rax1al=min(Rax1ala,Rax1alb);

Rax1ara=f1mod11ar*d*s1/cos(ah);
Rax1arb=f1mod21ar*d*s2/cos(ah);
Rax1ar=min(Rax1ara,Rax1arb);

Rax1ba=f1mod11b*d*s1/cos(ah);
Rax1bb=f1mod21b*d*s2/cos(ah);
Rax1b=min(Rax1ba,Rax1bb);

Rax2aa=f1mod12a*d*s1/cos(ah);
Rax2ab=f1mod22a*d*s2/cos(ah);
Rax2a=min(Rax2aa,Rax2ab);
```

```
Rax2ba=f1mod12b*d*s1/cos(ah);
Rax2bb=f1mod22b*d*s2/cos(ah);
Rax2b=min(Rax2ba,Rax2bb);

Rax3a=f1mod13*d*s1/cos(ah);
Rax3b=f1mod23*d*s2/cos(ah);
Rax3=min(Rax3a,Rax3b);

%eq 12
Rvm1a1_draw=Rax1al*sin(ah); Rvm1a1_embed=fh1*d*s1*cos(ah);
Rvm1a1=Rvm1a1_draw+Rvm1a1_embed;

Rvm1ar_draw=Rax1ar*sin(ah); Rvm1ar_embed=fh2*d*s2*cos(ah);
Rvm1ar=Rvm1ar_draw+Rvm1ar_embed;

Rvm1b_draw=Rax1b*(my*cos(ah)+sin(ah));
Rvm1b_embed=fh1*d*s1/(1+betha)*(1-
my*tan(ah))*(sqrt(betha+2*betha^2*(1+s2/s1+(s2/s1)^2)+betha^3*(s2/s1)^2)-betha*(1+s2/s1));
Rvm1b=Rvm1b_draw+Rvm1b_embed;

Rvm2a_draw=Rax2a*(my*cos(ah)+sin(ah));
Rvm2a_embed=fh1*d*s1/(2+betha)*(1-
my*tan(ah))*(sqrt(2*betha*(1+betha)+(4*betha*(2+betha)*M*(cos(ah))^2)/(fh1*d*s1^2))-betha);
Rvm2a=Rvm2a_draw+Rvm2a_embed;

Rvm2b_draw=Rax2b*(my*cos(ah)+sin(ah));
Rvm2b_embed=fh1*d*s2/(1+2*betha)*(1-
my*tan(ah))*(sqrt(2*betha^2*(1+betha)+(4*betha*(2*betha+1)*M*(cos(ah))^2)/(fh1*d*s2^2))-betha);
Rvm2b=Rvm2b_draw+Rvm2b_embed;

Rvm3_draw=Rax3*(my*cos(ah)+sin(ah));
Rvm3_embed=(1-my*tan(ah))*sqrt((2*betha)/(1+betha))*sqrt(2*M*d*fh1*(cos(ah))^2);
Rvm3=Rvm3_draw+Rvm3_embed;

R=min([Rvm1a1 Rvm1ar Rvm1b Rvm2a Rvm2b Rvm3]);

fail_mode=find([Rvm1a1 Rvm1ar Rvm1b Rvm2a Rvm2b Rvm3]==R);

if fail_mode==1
   R_draw=Rvm1a1_draw;
   R_embed=Rvm1a1_embed;
elseif fail_mode==2
   R_draw=Rvm1ar_draw;
   R_embed=Rvm1ar_embed;
elseif fail_mode==3
   R_draw=Rvm1b_draw;
   R_embed=Rvm1b_embed;
elseif fail_mode==4
   R_draw=Rvm2a_draw;
   R_embed=Rvm2a_embed;
elseif fail_mode==5
   R_draw=Rvm2b_draw;
   R_embed=Rvm2b_embed;
elseif fail_mode==6
   R_draw=Rvm3_draw;
   R_embed=Rvm3_embed;
else
   disp('Error in determining failure mode');
end
```

```
% In some cases the failure modes will give the same answer
if length(fail_mode)>1
    disp('More than one mode have the same strength')
    fail_mode=min(fail_mode);
    R=min(R);
end
```

Result:

```
fail_mode =6

R =3.0957e+004

R_draw =2.9028e+004

R_embed =1.9298e+003

Failure load [N]

R =3.0957e+004
```

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