# The Performance of Structural Screws in Canadian Glulam

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# ABSTRACT

The development of engineered wood products (EWPs), such as glued laminated timber, has led to the production of structural members with increased strength and spanning capabilities compared with solid sawn lumber and timber, and similar to what one could attain with commonly sized steel and reinforced concrete members. An important reason for the underutilisation of wood for heavy construction is the limitation of the CSA O86 Wood Design Standard especially as it applies to the design of fastenings for highly loaded connections. This thesis addresses the connection design of glulam and sawn wood using structural self-tapping/self-drilling screws from Europe.

The CSA–O86 Standard provides formulas for the design of both lag and wood screws in Canada. Recently in Europe structural screws have been developed which combine the advantages of both lag and wood screws. These screws have high load carrying capacities and withdrawal strengths and are also self-tapping/self-drilling, hence in most cases do not need lead holes. However CSA O86 has no specific design provisions for these screws. To assess their viability for use in Canada, two test programs were carried out; the first on the withdrawal resistance of the screws and the second on the performance of inclined screws in joist-to-header connections. Douglas fir Larch(20f-E), Spruce Pine (20f-E) and Nordic Lam (24f-1.9E) glulam in conjunction with a variety of 6, 8, 10 and 12mm diameter European structural screws were used for the withdrawal tests, while the joist to header connections were made of No.2 white pine timber connected by double threaded 8.2mm WT-T screws from SFS intec. In all 1960 withdrawal test were carried out. The test setup and procedure was modelled after ASTM D1761.The joist to header test set-up, involving 14 tests was modelled after ASTM D7147.

The main aim of the withdrawal test program was to recommend a generic equation for use in the design of these screw connections with Canadian glulam. In the process the effects of Canadian wood density, depth of penetration, screw

diameter and lead holes on the withdrawal resistance of the screws were assessed. The aim of the joist to header connection test program was to compare the performance of dry specimen with that of the same connection in wet timber.

The test results demonstrated that the withdrawal strength per unit length increases with denser wood, except 6mm and 8mm in Nordic Lam glulam. The depth of penetration affects the withdrawal strength, where for larger screws an increase of slightly more than double was obtained for 12d penetration compared to 6d.The orientation of the glulam, that is either top or side, was insignificant other than the effect it had on the scatter of the strength results. The use of lead holes was shown for both the 8mm and 10mm not to influence the withdrawal strength. However the lead holes improved the ease of installation for the larger screws, especially in the dense glulam.

The tests results were compared with the predicted characteristic and average withdrawal resistance values, which were calculated using formulas found in timber codes around the world, namely CSA O86 (Canada), NDS (USA), Eurocode 5 (Europe), DIN 1052 (German) and from other researchers including Frese and Blaβ (GER), Pirnbacher and Schickhofer (AUT) and McLain (USA). All methods resulted in a reasonable prediction of the withdrawal resistance except for the CSA O86 formula for lag screws which was very conservative. The McLain formula for lag screws provided the closest prediction of the test result, but bearing in mind the variability of wood, all the other methods could be considered as acceptable except for the CSA O86 lag screw equation. Regarding the joist-to-header cross screw connections, the dry tests were measured to have a 35% increase in resistance compared with the wet specimens; furthermore, the Kevarinmäki formula provided the most accurate prediction of the resistance.

# RÉSUMÉ

Le développement de produits de bois d'ingénierie a entrainé la production d'éléments de structure de la résistance et de portée supérieures, et similaires à ce qu'il pourrait être atteint avec de l'acier de taille commune et des éléments porteurs en béton armé. Une raison importante pour la sous-utilisation de bois pour la construction lourde est la limitation des règles de calcul des charpentes en bois CSA O86, en particulier lorsqu'elles s'appliquent au design d'assemblages soumis à de charges importantes. Cette thèse étudie la conception des assemblages en bois lamellé-collé et bois scié utilisant des vis auto perçantes européennes.

La norme CSA O86 fournit des formules pour la conception de tire fond et des vis à bois au Canada. En Europe, des vis qui présentent à la fois les avantages de tire fond et des vis à bois ont été récemment développées. Ces vis ont de résistance latérale et à l'arrachement élevée, et sont auto-perçantes. Toutefois, la norme CSA O86 ne contient aucune disposition spécifique pour la conception de ces vis. Pour évaluer la viabilité de leur utilisation au Canada, deux séries de test ont été réalisées: la première sur la résistance à l'arrachement des vis et la seconde sur la performance de vis inclinées dans les connexions entre solive et poutre. Du lamellé-collé Douglas-Mélèze (20f-E), Épinette-Pin (20f-E) et Nordic Lam (24f-1.9E) ont été utilisés avec des variétés de vis européennes de 6, 8, 10 et 12 mm de diamètre pour les essais d'arrachement, tandis que les connections solive-poutre étaient faites de pin blanc no 2 relié par des vis WT-T à double filetage 8.2mm. Au total, 1960 tests d'arrachement ont été effectués d'après ASTM D1761 tandis que 14 du test solive-poutre mis en place ont été effectués d'après ASTM D7147.

L'objectif principal du programme de tests d'arrachement était de recommander une équation générique à utiliser dans la conception de ces connexions de vis avec du lamellé-collé canadien. Les effets de la densité du bois, de la profondeur et du diamètre de la vis et des trous pilotes sur la résistance à l'arrachement des vis ont été évalués. Test de connexion solive-poutre était de comparer la performance de l'échantillon sec avec celle de la même connexion en bois humide.

Les résultats montrent que la résistance à l’arrachement augmente avec la densité du bois, à l'exception de vis 6 et 8 mm dans le lamellé-collé Nordic Lam. La résistance à l’arrachement est influence par la profondeur de pénétration et pour les plus grandes vis, il a été un peu plus que doublée pour une pénétration 12d par rapport à 6d. L'orientation du lamellé-collé, qui peut être transversale ou latérale, n’a pas d’importance autre que son effet sur la dispersion des valeurs de résistance. Il a été montré que trous pilotes n’influence pas la résistance à l’arrachement pour les diamètres 8 mm et 10 mm mais elles améliorent la facilité d'installation pour les vis plus grandes, en le bois lamellé-collé dense.

Les résultats des tests ont été comparés avec les valeurs caractéristiques prédites par calculs selon des formules trouvées dans cette codes ; CSA O86 (Canada), NDS(USA), l'Eurocode 5 (Europe),DIN 1052(Allemagne) et de chercheurs, dont Frese et Blaβ, Pirnbacher et Schickhofer, et McLain. Toutes les méthodes ont conduit à une prédiction raisonnable de la résistance à l'arrachement, à l'exception de la formule de CSA O86 pour les vis de compression, qui est très conservatrice. La formule de McLain pour les vis à compression donne la prédiction la plus proche des résultats des essais, mais compte tenu de la variabilité du bois, toutes les autres méthodes peuvent être considérés comme acceptables, à l’exception de celle de la CSA O86 pour les tire fond. En ce qui concerne les connexions solive-poutre par des vis inclinées, les essais à sec montrent une augmentation de 35% de la résistance par rapport aux spécimens humides;en outre la formule de Kevarinmäki donne la prédiction de résistance la plus précise.

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# 1 INTRODUCTION

## 1.1 General Overview

Wood is one of the oldest construction materials, which has been used for centuries. It is a naturally occurring and renewable construction material unlike many other materials used for structural members. The properties of wood are variable due to factors such as rate of growth, growing conditions, species and moisture content. Its variability as well as understated strength and durability have often led to its underutilisation in construction as compared to steel and reinforced concrete. The development of engineered wood products (EWPs), such as glue laminated timber, has led to the production of structural members with increased strength and spanning capabilities compared with solid sawn lumber and timber, and similar to what one could attain with commonly sized steel and reinforced concrete. Even with its advantages the per capita consumption of structural glulam in Europe is four times that of Canada (Williams 2005). Another important reason behind the underutilisation of wood is the limitation of the CSA O86 Wood Design Standard (2009), especially as it applies to the design of fastenings for connections that carry heavy loads.

Connections play a very important role in every structure; they provide integrity and stiffness to a structure and affect its constructability. As a general rule, it is straight forward to design the main members in a timber or EWP structure but it is the connections of the members which present a great challenge to the engineer (Madsen 2000). Connections can be the weakest link in a structure. According to Fruhwald et al. (2007) the majority of structural failures are caused by errors in design or lack of proper design. They suggested that high priority should be given to the training of engineers and to control in the design stage, with emphasis on bracing, risk of perpendicular to grain failure, moisture effects and design of joints.

There are a number of fasteners used for wood construction in Canada. The CSA O86 Standard contains clauses that apply to the following fasteners; nails and spikes, joist hangers, truss plate, shear plate connectors, split rings, bolts, drift pins, timber rivets, wood screws and lag screws. These connectors can be grouped into two categories: the dowel type connector and the skin type connector. With the skin type connector such as a shear plate, the load is transmitted from the member to the connector primarily in bearing. The dowel type connectors are less expensive, commonly available and easy to install. In these types of connectors, the load is transmitted from the wood member to the connector along the length of the connector; this generates both bending stresses and shear stresses along the shank of the connector (Madsen 2000). Slender dowel type connector exhibit ductile behaviour while larger diameters such as stocky bolts often have brittle failure. Dowel connectors such as nails are very efficient in conventional light frame construction where a low load capacity is required of the connector. For high loads, steel bolts of large diameters may be used and sometimes with a skin connector such as shear plates to increase the capacity of the connection. Bolts, lag screws, drift pins, split rings and shear plates require pre-drilling before installation which causes an increase in assembly time, a reduction of the cross-sectional area of the members, difficulty in controlling tolerances which may result in fit-up problems during construction and a possible brittle failure mode due to row shear or group tear-out of the fasteners loaded parallel to the grain of wood as well as net shear or splitting of the wood loaded perpendicular to the grain. Timber rivets which were developed in Canada (Madsen 2000) have high stiffness and load carrying capacity. They also have the advantage of not reducing the wood member cross-sectional area and not requiring pre-drilling. The high capacity comes with the use of large numbers of closely spaced rivets which results in tedious installation. Furthermore the large exposed steel plates may diminish the aesthetic value of the wood structure and may cause splitting as a result of drying shrinkage.

Lag screws or lag bolts are bigger than wood screws. They are similar in behaviour to bolts in that they require pre-drilling before installation, and that they can be used in tandem with both split rings and shear plates. They are used in inaccessible areas where a nut cannot be fixed at the end or where a long bolt is needed to go all the way through a deep member. They are best suited where withdrawal resistance is required. Design values in the CSA O86 Standard are based on the use of lag screws conforming to the requirements of ANSI/ASME Standard B18.2.1- 1996 (R2005). The 2009 version of the CSA O86 Standard also contains newly added provisions on the design of wood screws. The wood screws designed according to the Standard should conform to ASME B18.6.1. As presently indicated in CSA O86, wood screws with diameter greater than gauge 12 (5.48mm) should be designed according to the requirements of lag screws. This approach might not be appropriate for large wood screws as they have high withdrawal strength and low lateral load carrying capacity as compared to lag screws of similar dimensions.

The screws used for testing are self-drilling and self-tapping; they are different from both lag screws and wood screws. They can best be described as a hybrid of the two screw types combining the high lateral load carrying capacity of lag screws with the withdrawal capacity and ease of installation of wood screws. They are also referred to as structural screws and European structural screws in this thesis. There are two types, thread forming and thread cutting. With the thread cutting screws, they remove the cut wood material as they are being installed while the thread forming plastically deform the wood they are driven into hence creating a bond which transfer both tensile and compressive forces along the axis of the screw. They can be fully or partially threaded depending on the application. Diameters of 12 mm and lengths of up to 1000 mm can be obtained; hence these fasteners can be used to connect large wood sections as compared to lag screws which have length limitations. The screws are hardened after the threads have being rolled; as such these large and long screws do not usually require predrilling. The hardening of the screws can also increase the mechanical properties such as tensile and compressive strength, torsional and yield strengths. They are made with a drill tip (Figure 1.1a), coated with lubricants to reduce the drilling torque and also have shank cutters (Figure 1.1b) to reduce friction while drilling the screws. The main advantage of these screws is the high lateral load carrying capacity and withdrawal strength. Variability and flexibility in the shape of the heads of these screws allows for their use for aesthetic purposes. These screws are widely used in Europe; a situation made possible by design provisions in Eurocode 5 (EC5) Cl. 8.7 (2004) and the revised edition (EC5) Cl 8.7.2 (2008). Since CSA-O86 has design provisions for both lag and wood screws; it would be very beneficial to incorporate design provisions for this hybrid structural screw since at present engineers in Canada would have to rely on manufacturers' information to carry out design or use the lag screw provisions in CSA O86 which underestimate their withdrawal strength. The existing design provisions used in Europe would not necessarily apply in Canada because of the difference in wood species and material properties. Furthermore, the design philosophy in Europe is based on the limit states design approach in conjunction with the partial factor method using EN 1990:2002, EN 1991 and EN 1995 while the limit states format is implemented for all building codes and material standards in Canada. The performance of these structural screws when used to connect Canadian wood needs to be investigated in order to develop appropriate design equations.

a)  b) 

Figure 1-1 Self-tapping screws   a) Drill tip b) Shank cutter

## 1.2 Statement of Problem

In Canada there is a lack of effective and efficient fastening systems which are strong, stiff and ductile, have a low cost, are easily installed and allow for versatility in joining high capacity structural timber and Engineered Wood Product (EWP) elements. The structural screws can be used with EWPs such as glulam and other wood based products, and as such attend to these needs for fastening systems. Although advantages exist in the use of these structural screws, at present there are no specific provisions in the CSA-O86 Standard to address their design; hence the conduct of the research described herein as a step towards incorporating a design approach in the Standard.

## 1.3 Objectives

The general objective of this research is to develop a design equation for the withdrawal resistance of structural screws perpendicular to grain ($90^0$) for adoption in the CSA O86 Standard. As part of this development process, the following specific objectives were set:

- Investigate the effect of the depth of penetration;
- Investigate the effect of screw diameter;
- Investigate the effect of wood density;
- Investigate the effect of pre-drilled lead holes.
- Investigate the effects of screws from different manufacturers of the same diameter.

The second part of the research which is not directly linked to the main objectives of this research involves the testing of a joist to header assemblage which has been allowed to dry after installation of the screws. A similar research project was conducted by another student at McGill using wet wood. This component of the research project has the following objectives:

- Determine the capacity and stiffness of a typical connection assembly composed of one pair of crossed self-drilling screws.
- Compare test results with strength predictions obtained following the European design provisions so that recommendations regarding the use of these fasteners in Canada may be made;

- Compare the test results with the results of an earlier research project conducted at McGill University using saturated (high moisture content) joist to header assemblages.

## 1.4 Scope of Research

Given the objective of developing a generic withdrawal design method for the structural screws, fasteners from three different manufacturers were included in the study; SFS-intec Inc., Schmid Schrauben hainfeld and WürthGmBH & Co.KG. In order to examine the effect of screw diameter on withdrawal strength, screws of 6, 8, 10 and 12 mm diameters were incorporated in the scope of study. Three different species of wood were used to examine the effects of the wood density; Douglas-fir-Larch, Spruce-Pine and Nordic Lam glulam. Two different installation depths of the screws were used, that is 6d and 12d, where d is the screw diameter. To examine the effects of pre-drilling a lead hole, a set of the 8 and 10mm diameter screws were installed without predrilling, followed by a second set with pre-drilling. Lead holes were not used for the 6mm diameter screws because of their small size while for the 12mm diameter screws lead holes were always drilled in order to avoid splitting of the wood for these relatively large fasteners. In all, 1960 withdrawal tests (including 480 from Laval University and FPInnovations) were conducted. Ancillary tests to measure the moisture content and density of the wood, as well as the tensile strengths of the screws were also carried out.

The second part of the research involved a joist to header set up with one pair of crossed WT-T intec screws. Three different sets of screws were used (WT-T-8.2-160, WT-T-8.2-220 and WT-T-8.2-245). No. 2 grade solid white pine timbers were used and the installed assemblages were stacked in the laboratory for 5 months such that they would be allowed to dry prior to testing. Both static and cyclic tests were performed on the joist-to-header connection specimens. In all 14 assemblages were tested. Ancillary tests to measure the moisture content and density of the wood, as well as the tensile strengths of the screws were also carried out.

The results of the withdrawal and cross screw tests were then compared with the predicted characteristic values calculated using formulas found in timber codes around the world, namely CSA O86 (Canada), NDS (USA), Eurocode 5 (Europe) and DIN 1052 (Germany) and from researchers including Frese and Blaβ (2009), Pirnbacher and Schickhofer (2010) and McLain (1997a) .

# 2 LITERATURE REVIEW

Under this section a review of related literature on the withdrawal resistance of screws and joist to header connections is presented. This review is in two main parts; the first part is about the perpendicular screw withdrawal resistance in glulam and the second part is about the cross screw joist to header connection in white pine timber. The first part also looks at material factors and selected code provisions for withdrawal strength estimation.

## 2.1 Glulam

Glued-laminated timber (glulam) is an engineered wood product fabricated from small sections of timber boards (called laminates) bonded together with water proof adhesives and laid up in such a way that all grains of laminates are parallel to the longitudinal axis (Porteous & Kermani 2007). In Canada glulam is manufactured to meet CSA O122-06 (2011) Structural Glued-laminated Timber. Glulam members are glued under high pressure using a phenol or resorcinol formaldehyde adhesive which is waterproof. The laminates are usually dried to a moisture content of 7-15% before lamination to maximize adhesion and minimize shrinkage in service. The laminates are visually and mechanically sorted for strength and stiffness into lamstock grades. Douglas fir-Larch, Spruce-Pine and occasionally Hem-Fir are the main species groups from which glulam are produced in Canada. There are two grading approaches of glulam:

- Stress grade, and
- Appearance grade.

The stress grade refers to the strength of the material. Glulam members are produced by placing higher quality lamstock in high stress regions. High strength laminations placed at the top and bottom of the cross section results in high flexural strength and stiffness .With columns and tension members the laminations are evenly distributed across the cross section. A 20f-E grade for example means the member has an allowable bending stress of 2000psi (no longer used in design), f means flexural member whiles *c* and *t* refers to compression and tension grades respectively, E indicates mechanical sorting of the laminations. For instance f-E means high laminations are placed at the bottom and f-EX means that high laminations are positioned at both the top and bottom of the member. (CWC 2005)

## 2.2 Material Factors

### 2.2.1 Wood Density

Wood is made up of cells and has different values for the ratio of its cell wall thickness to total cell diameter; its strength, density and stiffness increases as this ratio increases (Dinwoode 2000).The density of wood is variable, varying by a factor of 10 from the lowest average value at 176kg/m$^3$ for Balsa to about 1230kg/m$^3$ for the densest hardwood as noted by Dinwoode. The density and consequently the strength and stiffness of wood vary due to its hygroscopic nature. The strength and stiffness are directly linked to wood density (Bindzi & Sampson, 1995); studies into the behaviour of joints using both empirical and elastic theory approaches have shown that joint stiffness and strength are linear functions of wood density (Brock 1957, Wilkinson 1972).

### 2.2.2 Embedment strength

The embedment strength, which is a direct material property, has an influence on the load carrying capacity of screw joints. The embedment strength is the ultimate pressure per unit length of screw divided by the screw diameter. According to Noren (1968) who conducted research on Nordic pine and spruce connections, the embedment strength is a function of wood density and moisture content. The embedment strength decreases with increasing moisture content. This fact is independent of wood species and fastener diameter (Rammer & Winistorfer, 2001). In addition, studies have shown that at high moisture content (e.g. >21%) the embedment strength remains constant with any further increase in moisture content. This point is close to the wood saturation point however no clear conclusion has being made about the relationship between the two parameters (Guillaume 2010).Bejkta and Blaß (2002) also stated that the ultimate load of joints with dowel type fasteners loaded perpendicular to the fastener axis is limited by the embedment strength of the timber members. Several formulae have been developed for the estimation of embedment strength and according to Porteous (2009) the differences in results is due to the lack of a common standard for testing to determine the embedment function.

### 2.2.3 Screw diameter and length

The diameter and depth of penetration has an effect on the withdrawal resistance of screws. Johnson (1967) conducted direct withdrawal tests of various size screws on commercial particle board and plywood and found that the load generally varied in proportion to the screw size. Pirnbacher and Schickhofer (2010) found that the screw diameter can be described independently from density, material and angle to the grain, meaning the effect of screw diameter does not depend on these three factors. For most screws the tapered end is deducted in the empirical estimation of resistance since it is assumed to not contribute to the withdrawal resistance. According to Newlin and Gahagan (1938) the withdrawal strength of self-tapping screws increases linearly with the depth of penetration. Wilkinson and Laatch (1970) in a later research project also agreed with that conclusion. Some design standards, however, have limitations on the depth of penetration in order to take full advantage of ductility in the screws.

### 2.2.4 Moisture Content

The strength and stiffness properties of wood are affected by the level of moisture. An increase in moisture content decreases the strength and elastic properties of wood up to the fibre saturation point, after which there are no effects on these properties (Bodig and Jayne 1993). The relationship between moisture content and timber properties has been assumed to vary linearly when the moisture content is between 8% and 20% (STEP 1 1995) even though the overall relationship has not been found to be linear. Pearson et al. (1962) conducted research with Australian wood species and suggested a strength factor of 1.25 between green and dry wood. This is lower than the 1.39 strength factor suggested by Mack (1966) when he developed a joint strength equation from green and comparative dry wood. For short lateral loadings Mack also suggested there was no difference between the joint strength behaviour of green wood compared to that of wood at 12% moisture content. Morris (1970) developed a relationship between joint strength and moisture content where $P=a(\delta)^{b}$, P being the joint load at deformation δ, a and b are constants of fit, where b is also a function of the moisture content. For joints where all members are solid $b=0.778m_p^{-0.092}$, where $m_p$ is the percentage moisture content of the timber. The equation is only valid up to a slip of 0.25 mm and the value of b only changes marginally between a moisture content of 12 and the fibre saturation point. Kuiper et al (1965) investigated the effects of moisture content on embedment strength in Norwegian timber and developed the moisture content function $f_{ew} = f_e(\frac{26}{w+14})$ where $f_{ew}$ is the embedment strength at *w%* moisture content. The higher the moisture the more pronounced the duration of load effect would be (Foschi 1991). This is accounted for in the strength modification factors for service classes and load duration classes given in Eurocode 5. Pirnbacher and Schickhofer also observed in perpendicular to grain screw withdrawal test in solid wood conducted at 0% , 9%, 14% and 19% moisture content that moisture content below 10% sharply increases brittle failure, thus splitting of the wood and at moisture content greater than 10% the shear strength steadily deceased up to 5% at 20% moisture content.

### 2.2.5 Grain direction

Grain is the alignment of wood cells which run vertical in a tree. Wood is an anisotropic material hence its strength depends on the direction of grain as stated by Bodig. For example the modulus of elasticity of Beech when loaded along the grain is over 12 times that obtained when loaded tangentially at right angles to the grain as stated by Dinwoodie. According to Hankinson (1921) the load at an angle to the grain is given by

$$n = \frac{pq}{psin^{s}\alpha + qcos^{s}\alpha} \qquad 2.1$$

Where n is the unit strength at an angle α to the grain direction, p is the unit strength parallel to the grain and q is the unit strength perpendicular to the grain, while experimental results have shown s =2 as stated by Bodig.

### 2.2.6 Pre-drilling of Lead Holes for Screws

The pre-drilling of lead holes is required for certain types of wood and large diameter screws to prevent splitting. It also allows screw spacing to be reduced while still allowing for the wood to behave in a ductile nature. Joints in which the screws are installed without lead holes are more ductile since the fasteners are able to rotate in the initial cracks whereas if lead holes were used the screws would bend (Dalon & Ramskill, 2004). According to CSA O86 the lead holes for lag screws should be 65-85% of the shank of the screw for dense wood, 60-75% of shank diameter for Douglas fir and 40-60% for less dense wood. Note, the shank diameter and the outer diameter of the threaded section of the lag screw are the same. The length of the lead hole should be at least equal to the threaded length of the screw. For less dense wood of relative density less than 0.5, lead holes are not required for wood screws according to CSA O86. For withdrawal loading the lead hole diameter is 0.7 times the root diameter for relative densities less than or equal to 0.6 and 0.9 the root diameter for relative densities greater than 0.6. According to ASTM D1761 (2006) the lead holes for screws should be 70% of the root diameter. In Eurocode 5 (EC5) (2004) the lead holes for the threaded part of the screws should be 70% of the shank diameter and for self-drilling screws it should not be more than the inner thread diameter. Also pre-drilling is not required in softwoods for screw diameters ≤ 6mm but required in screws of diameters greater than 6mm and in all hardwoods. For densities greater than 500kg/m$^3$ testing is required to determine the lead hole diameter. According to Dalon and Ramskill, the larger the diameter of the screw, the bigger the effect of the pilot hole. They found that shear capacity was reduced by 45% and 67% in tests with screw diameters of 9.5 mm and 12.7 mm respectively that were installed without lead holes. They explained that this was due to cracking as a result of wedging action resulting in tension perpendicular to grain stresses. They also concluded that 6.2 mm and lesser diameter screws should not have predrilled holes unless the wood is prone to splitting and for bigger diameters with lead holes less than 70% of the screw diameter, the capacities were negatively affected. Pirnbacher and Schickhofer working on self-tapping screws found that there was no difference in withdrawal resistance in both pre-drilled and self-drilled 8mm diameter screws.

## 2.3 Design Standards

### 2.3.1 CSA O86-2009

The CSA O86 Standard (2009) has no specific provisions for the design of the structural screws. It rather contains design equations for both lag and wood screws. The lag screw equation was developed from the report of Burgess and Huggins (1982). They used data from unpublished data based on five species (redwood, white pine, Douglas-fir, southern pine and white oak) and seven different sizes of lag screws. The data included approximately six replications for each combination of species and screw size. Adjustments have being made to the lag screw provisions in the CSA O86 editions over the years, for instance in the 1984 edition resistance ($\phi$) values for limit states design withdrawal calculations were 0.7 but in the 2009 edition $\phi$ was set equal to 0.6 (CSA O86-2009). According to Cl.10.6.5 of CSA O86 the withdrawal resistance of lag screws is calculated as follows;

$$P_{rw} = \phi Y_W L_T n_F J_E \qquad 2.2$$

where

$\phi$ = 0.6

$$Y_W = y_W(K_D K_{SF} K_T) \qquad 2.3$$

$y_W$ = basic withdrawal resistance per millimeter of penetration N/mm

$L_T$ = length of penetration of threaded portion of lag screw in main member, mm

$J_E$ = end grain factor for lag screws

= 0.75 in end grain, or

= 1.00 for all other cases

$n_F$ = number of screws

$K_D$ = load duration factor

$K_{SF}$ = service factor

$K_T$ = treatment factor

Wood screws are a recent introduction to the CSA O86 Standard. This as noted in the O86 Standard was a result of the harmonization of the lag screw withdrawal values with the withdrawal values of wood screws which led to the adoption of the withdrawal capacity models by McLain (1997a) for wood screws for predicting ultimate withdrawal strength. Clause 10.11.5 deals with the withdrawal strength of wood screws. The factored withdrawal resistance is equal to the lesser of the factored screw withdrawal resistance of the main member(Cl 10.11.5.2) or the factored head pull-through resistance of the side member(Cl 10.11.5.3)

The withdrawal resistance $P_{rw}$ of the main member is taken as

$$P_{rw} = \phi Y_W L_{Pt} n_F \qquad 2.4$$

where

$\phi$ = 0.6

$$Y_W = y_W (K_{SF} K_T) \qquad 2.5$$

$y_w$ =basic withdrawal resistance per millimeter of threaded shank penetration of main member

$$= 68 d_F^{k0.82} G^{1.77}, N/mm \qquad 2.6$$

G = mean relative density of main member (Table A.10.1 (CSA O86-2009))

$d_F$ = nominal wood screw diameter, mm (Table 10.11.1(CSA O86-2009))

$L_{Pt}$ = threaded length penetration in main member

$n_F$ = number of wood screws in the connection

The head pull-through resistance for joints with light gauge steel side plates, *Ppt*, is taken as

$$P_{Pt} = 1.5\phi t_1 d_w f_u n_F \qquad 2.7$$

For joints with lumber or structural panel side plates, the factored head pull-through resistance is

$$P_{Pt} = 75\phi t_1 n_F \qquad 2.8$$

where,

$\phi =$ 0.4

$t_1 =$ side plate thickness, mm

$d_w =$ diameter of screw head, mm

$f_u$ =ultimate tensile strength of steel, MPa

$n_F$ = number of wood screws in the connection

### 2.3.2 National Design Specification

The National Design Specification (NDS 2005) for Wood Construction (AWC 2005) published by the American Wood Council (AWC) is used for the design and construction of wood structures and connections in the United States. The NDS covers both the design of lag and wood screws. The formula for lag screws is as a result of the research conducted by Newlin and Gahagan (1938) on a range of screw diameters in five wood species (northern white pine, redwood, Douglas-fir, southern pine and white oak). They conducted tests on 234 test configurations, with each configuration having 5 to 7 replicas. Their test limitations and recommendations are reflected in the specific provisions of the NDS. For instance, pilot holes are to be 40 to 70% of the shank diameter for less dense wood and 65 to 85% for more dense wood as noted, by McLain. According to Cl. 11.2.1, the reference withdrawal design value in lb/in of penetration of lag screw is;

$$W = 1800\, G^{3/2}\, D^{3/4} \quad (2.9)$$

The equation for the reference withdrawal strength for wood screws was derived from the data of the research conducted by Fairchild (1926). He conducted over 10,000 separate tests on cut thread wood screws in seven wood species (Yellow-poplar, cypress, southern pine, sycamore, hard maple and white oak). According to Cl.11.2.2 which deals with wood screws the reference withdrawal design value in lb/in of penetration for a single wood screw is;

$$W = 2850 G^2\, D \quad (2.10)$$

where

$G$ = specific gravity of wood

$D$ = unthreaded shank diameter of screw

$W$ = reference withdrawal design value in lb/in

The constants in Equation 2.10 have changed since the 1935 edition of the Wood Handbook (1991) because of the re-indexing of the basis for the withdrawal strength from strength per inch of the total screw length to strength per inch of the engaged thread as noted, by McLain. For both lag and wood screws

- Wood should not be loaded in withdrawal through the end grain.
- When wood screws are loaded in withdrawal, the adjusted tensile strength of the wood screw at the net (root) section shall not be exceeded

- The characteristic withdrawal strength is obtained by multiplying the reference withdrawal design value by the depth of penetration and appropriate service factors.

### 2.3.3 McLain Equations

McLain (1997a) came up with equations for lag and wood screws as a revision to those in the NDS as part of the development of a new load and resistance factor design standard for wood. McLain postulated that his new equations better described the available test data than those in the Wood Handbook (1987). The equations were derived from data gathered from research conducted into both wood and lag screws by researchers including Fairchild (1926), Cockrell (1933), Johnson (1959), Cizek and Richardson (1957), Stern (1951), Newlin and Gahagan, and McLain and Caroll(1990). McLain's formulas are very similar to those in the NDS except that the coefficients and constants in both equations are different. According to McLain the reference withdrawal design value in lb/in of penetration of lag screws is;

$$W = 1620\, G^{1.35} D^{0.61} \qquad 2.11$$

For wood screws the reference withdrawal design value in Ib/in of penetration for a single wood screw is;

$$W = 1810 G^{1.77} D^{0.82} \qquad 2.12$$

where

G = specific gravity of wood

D = unthreaded shank diameter of screw

W = reference withdrawal design value in lb/in

### 2.3.4 Eurocode5

The EC5 (2004) provides specific details as to the design of structural screws. It does not separate lag and wood screws but rather has a design equation which deals with screws in general.

#### 2.3.4.1 Modification to Johansson's Equations (Rope effect)

As noted by Porteous & Kermani (2007) the EC5 2004 has adopted modifications of the Johansson (1949) equations to account for frictional effects and axial withdrawal of the fastener in the load carrying capacity of screws. There are two types of friction

- When the members are in contact on assembly and this effect is neglected by Eurocode 5 due to the effects of shrinkage on joints
- The other is when the fastener yields and pulls the members together when it deforms under lateral loads.

Taking these effects into account the characteristic lateral load carrying capacity of a fastener, $F_{V,Rk}$ in the Eurocode 5 is written as

$F_{V,Rk}$ = friction factor x Johansen yield load + (withdrawal capacity/4)

The latter part of the above equation is referred to as the rope effect forces.

#### 2.3.4.2 Axially loaded Screws

According to Cl 8.7.2 these are the possible failure modes of axially loaded screws

- Withdrawal of the threaded part of the screw
- When used with steel plates, there is the risk of tearing off the screw head
- Failure by the screw head pulling through the timber or wood product
- The screw failing in tension
- When used in conjunction with steel plates there is the risk of a block shear or plug shear failure.

To ensure the minimum withdrawal resistance is met and control block failures, minimum screw penetrations and spacing requirements are specified. The minimum penetration of the threaded part is 6d.

The characteristic withdrawal capacity of a screw perpendicular to the grain $F_{ax,Rk}$ can be calculated

$$F_{ax,Rk} = n_{ef}(\pi d l_{ef})^{0.8} f_{ax,a,Rk} \qquad 2.13$$

where

$n_{ef}$ = the effective number of screws

d = is the outer diameter of the screw measured on the threaded part

$l_{ef}$ = is the pointside penetration of the threaded part of the screw minus one screw diameter (to account for pointed end of screw)

$F_{ax,a,Rk}$= characteristic withdrawal strength at an angle α to the grain

$$F_{ax,a,Rk} = \frac{f_{ax,k}}{1.5cos^2\alpha + sin^2\alpha} \qquad 2.14$$

$$F_{ax,k} = 3.6 \times 10^{-3} \rho_k^{1.5} N/mm^2 \qquad 2.15$$

where;

$F_{ax,k}$ = the characteristic withdrawal strength perpendicular to the grain

$\rho_k$ = the characteristic density of the timber or wood product.

α = $90^o$ for this thesis

Blaß and Bejtka (2004) suggested changing $3.6 \times 10^{-3}$ in Equation 2.15 to $2.85 \times 10^{-3}$ to reflect the results they had and hence to avoid over prediction of the withdrawal resistance. Blaß et al (2006) after analysing 800 withdrawal test results suggested that the characteristic withdrawal resistance is

$$F_{ax,a,Rk} = \frac{0.52d^{0.5} {l_{ef}}^{0.9} \rho_k^{0.8}}{1.2cos^2\alpha + sin^2\alpha} , \qquad 0^o \leq \alpha \leq 90^o \qquad 2.16$$

Equation 2.16 formed the basis for the regulations in EN 1995-1-1 (2007) and hence the new formula in Equation 2.17. The EC5 revised edition defines $F_{ax,k}$ as the characteristic withdrawal capacity of a screw perpendicular to grain in terms of the screw diameter, depth of penetration, density of wood and a constant as compared to the 2004 edition which defines it in terms of only density and a constant (Eq. 2.15).The characteristic withdrawal values are the 5th percentile values. The characteristic withdrawal capacity of a screw perpendicular to the grain $F_{ax,a,Rk}$ can be calculated

$$F_{ax,a,Rk} = \frac{n_{ef} f_{ax,k} d l_{ef} k_d}{1.2cos^2\theta + \ sin^2\theta}, \quad \alpha \geq 30^{o} \qquad 2.17$$

$$f_{ax,k} = 0.52 d^{-0.5} l_{ef}^{-0.1} \rho_k^{0.8} \qquad 2.18$$

$$k_d = min \begin{cases} d/8 \\ 1 \end{cases} \qquad 2.19$$

$k_d$ = non-dimensional factor

where the remaining terms are as defined before.

The characteristic tensile resistance of the connection, based on the tensile capacity of the shank is;

$$F_{t,Rk} = \ n_{ef} f_{tens,k} \qquad 2.20$$

Where

$f_{tens,k}$ = is the characteristic tensile capacity of screw determined in accordance with EN 14592(2008)

### 2.3.5 DIN 1052 (German code)

According to DIN 1052 (2008), the first part of Equation 2.21 is to calculate the characteristic withdrawal strength at an angle. It is defined in terms of the characteristic withdrawal strength, screw diameter, depth of penetration and the angle of inclination. The screws are classified into three strength groups (1-3) according to their characteristic axial strength as shown in Table 2.1.Group 1 is any other screw apart from 2 and 3. Group 2 is screws with threads in accordance with DIN 7998 which do not require further approval. Group 3 are hardened screws that are proven to withstand a certain threshold capacity and require general construction approval. The structural/self-tapping screws fall under this category unless otherwise stated in the approval of the particular screw. The second part of the equation is the head pull-through resistance of the screw. It is defined in terms of $f_{2.k}$ which is the characteristic head pull through parameter and $d_k$ which is the outer diameter of the screw head or washer. The head pull-through resistance is not considered in fully threaded screws since the load is transferred through the thread and shaft and not the head. The equation is valid for densities up to 500 kg/m$^3$ and $45^o \leq \alpha \leq 90^o$. Blaβ and Bejtka have shown that the equation holds true for $\alpha = 30^o$. Blaß et al contend that the Equation 2.21 is very conservative and propose changing the values in Table 2.1 for screw type 3 from 80 x $10^{-6}$ to 113 x $10^{-6}$ for $\alpha = 90^o$ connections and 109 x $10^{-6}$ for $\alpha < 90^o$ connections.

$$R_{axK} = min\left(\frac{f_{1.k}.d.l_{ef}}{sin^2\alpha + \frac{4}{3}.cos^2\alpha}; f_{2.k}.d_k^2\right)[N] \qquad 2.21$$

where

$f_{1,k}$ =characteristic withdrawal strength perpendicular to grain

d = diameter of screw

$l_{ef}$ effective depth of penetration

$\rho_k$ = the characteristic density of the timber or wood product

$\alpha$ = angle of inclination of screw

$R_{axK}$ = characteristic withdrawal strength

Table 2.1 Characteristic withdrawal strength formulas for DIN 1052

| Screw type | $f_{1,k}$ | Screw type | $f_{2,k}$ |
|---|---|---|---|
| 1 | $60 \cdot 10^{-6} \cdot \rho_k^2$ | A | $60 \cdot 10^{-6} \cdot \rho_k^2$ |
| 2 | $70 \cdot 10^{-6} \cdot \rho_k^2$ | B | $80 \cdot 10^{-6} \cdot \rho_k^2$ |
| 3 | $80 \cdot 10^{-6} \cdot \rho_k^2$ | C | $100 \cdot 10^{-6} \cdot \rho_k^2$ |
| Characteristic density $\rho_k$ in kg/m³, but not more than 500 kg/m³ | | | |

### 2.3.6 Frese and Blaß

Frese and Blaß (2009) developed a formula for the characteristic withdrawal capacity for self-tapping screws using softwood. In all a total of 1850 withdrawal tests was used in the analysis with screws from different manufacturers. Frese and Blaß claim that the equation provides a more accurate estimate of the withdrawal resistance of wood screws with geometrical properties similar to those used for the tests. The withdrawal resistance is:

$$ln(R_{ax,k}) = 6.54 + l_{ef} \cdot (0.03265 - 1.173 \cdot 10^{-4} \cdot l_{ef}) + 2.35 \cdot 10^{-4} \cdot d \cdot \rho_k \qquad 2.22$$

where

$l_{ef}$ = effective length

d = screw diameter

$\rho_k$ = density of wood

### 2.3.7 Pirnbacher and Schickhofer

Pirnbacher and Schickhofer (2010) conducted 5500 single tests using self-tapping wood screw to investigate the effects of moisture content, temperature at screw-in and screw-out, screw diameter, slenderness ratio, embedment of the screw thread, influence of angle between the screw axial and the grain and predrilling in solid and glulam sections. They developed an equation factoring in the parameters they investigated if they had an effect .The characteristic withdrawal resistance perpendicular to grain, $f_{ax}$ is:

$$f_{ax} = 0.0116. \boldsymbol{\rho_{test}} - 0.272. (2.41d^{0.572}) + 1.97 \qquad 2.23$$

For 8 mm and 10 mm diameter screws Equation 2.22 can be simplified as

$$f_{ax} = 0.01. \rho_{test} \qquad 2.24$$

where

d = diameter of screw

$\rho_{test}$ = density of test samples

## 2.4 Inclined screws

The EC5 editions of 2004 and 2007 make provisions for the design of axially loaded screws at an angle to the grain where α= angle between the screw axis and the grain direction, with $\alpha \geq 30$.

In SFS intec's implementation of Eurocode5 (MSGC-CUST, 2004), the resistance is dependent on a combined single shear and the axial tension or compression interaction. However Blaß and Bejtka (2002) recommend the omission of the shear action of the inclined screws since the shear effects are complicated and are not understood well enough.

### 2.4.1 Kevarinmäki approach for inclined screws

Kevarinmäki (2002) considered the test results of Blaß and Bejtka and decided to formulate design equations compatible with Eurocode5 equations for both tension and cross screw configurations at 45°. Tomasi et al (2005) found that the greatest initial stiffness was achieved at an angle of $45^0$ , failure was more brittle at 0°/90° and the greatest strength at 60°. Kevarinmaki suggests a cross-screw joint design resistance as;

$$R_d = n_p(R_{C,d} + R_{T,d})\cos\alpha \qquad (2.25)$$

where,

$n_p$ = number of screw pairs in the joint,

α = 45°, angle of screw to load,

$R_{C,d}$ = screw compression resistance, and

$R_{T,d}$ = screw withdrawal resistance.

The screw compression resistance is:

$$R_{C,d} = \min\begin{Bmatrix} f_{a,1,d}\ \pi\ d\ s_1 \\ f_{a,2,d}\ \pi\ d\ s_2 \\ 0.8F_{u,d} \end{Bmatrix} \tag{2.26}$$

And the screw withdrawal resistance is :

$$R_{T,d} = \min\begin{Bmatrix} f_{a,1,d}\ \pi\ d\ s_1 + f_{head,d}\ d_h^2 \\ f_{a,2,d}\ \pi\ d\ (s_2 - d) \\ F_{u,d} \end{Bmatrix} \tag{2.27}$$

where,

d = outer diameter of the thread (screw nominal size),

$s_1$ = threaded length of the screw in the member (1) which is towards the screw head,

$s_2$ = threaded length of the screw in the member (2) which is towards the screw tip,

$F_{u,d}$ = design value of the screw tension capacity,

$d_h$ = head diameter of the screw, and

$f_{head,d}$ = design pull through strength of the screw head determined experimentally according to EN 1383 (1999b).

The factored screw withdrawal resistance $f_{a,i,d}$ is defined as :

$$f_{a,i,d} = \frac{k_{mod}}{\gamma_M} f_{ax,45,k} \left(\frac{8d}{s_i}\right)^{0.2} \tag{2.28}$$

where,

$k_{mod}$ = load duration and moisture influence factor from EC 5 for member i,

$\gamma_M$ = safety factor of glued-laminated timber for member i ( =1.3 for connections),

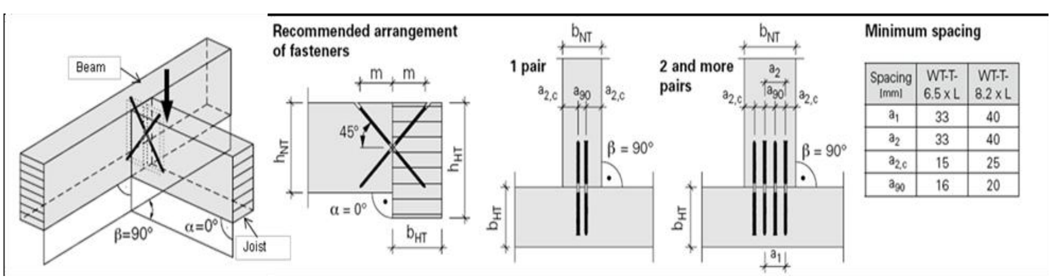
$s_i$ = length of the threaded part of the screw in member i, and

$f_{ax,45,k}$ = characteristic withdrawal strength of the screw determined in grain direction angle of α = 45° and penetration length of $s_2$ = 8d according to test methods in EN 1382 (1999a).

### 2.4.2 Connection resistance predictions of WT fasteners

The WT screw application test involves the joining of secondary glulam or timber purlins to the main support framing of a wood structure. The joint configuration is composed of one or more WT screws in tension at 45° inserted in the main member and one or more matching screws in compression inserted in the secondary member also at 45° but slightly offset from the tension screw(s). The fasteners include an unthreaded midsection referred to as the neck which separates the two differently pitched threads. The recommended arrangement and minimum spacing set by SFS intec based on Eurocode5 can be seen in Figure 2.1. Depending on the width of the joist, two or three pairs of the fasteners may be required at the joint (SFS intec, 2005).

Because the WT fasteners are unique, have a proven performance history in Europe but not in North America and can be used in at least a dozen different configurations, predicting their failure mode and performance is a challenge made all the more difficult by a lack of previous documented research involving Canadian wood species (Prat-Vincent, 2011).




| Spacing (mm) | WT-T-6.5 x L | WT-T-8.2 x L |
|---|---|---|
| $a_1$ | 33 | 40 |
| $a_2$ | 33 | 40 |
| $a_{2,c}$ | 15 | 25 |
| $a_{90}$ | 16 | 20 |

Figure 2-1 Recommended WT screw arrangement and minimum spacing (SFS intec, 2005)

The predicted capacities are calculated using Equations 2.2, 2.13, 2.17 and 2.28.These are the O86 lag screw equation used in Canada, the EC5 formulas and Kevarinmäki formula for inclined screws respectively. Since CSA O86 does not account for the angle of inclination, an assumption is made where Eq 2.2 is divided by $1.2cos^2\theta + \quad sin^2\theta$ from the Eurcode 5 formula.

Kevarinmäki's proposed *fa,i,d* (Eq. 2.28) limits the characteristic embedment length to eight times the diameter instead of using $l_{ef}$, and hence penalizes the withdrawal of longer screws as noted by Prat-Vincent. It is for this reason that in Prat-Vincent's prediction calculations, the entire characteristic withdrawal capacity (Eq.2.13) is calculated according to the new EC 5 rules (only for one screw in tension) and then divided by the penetration area $\pi dl_{ef}$ to obtain $f_{a,i,k}$. By using this method, all properties are accounted for in the calculations while still remaining relevant to EC 5.

### 2.4.3 Prat-Vincent Observations and Recommendations of Connection Resistance Predictions for Cross Screws

Prat-Vincent (2011) conducted 80 (20 Nordic Lam glulam and 60 white pine timber) cross screw connection test at McGill University. The Nordic glulam was dry (10 conditioned and the other half unconditioned) whereas the white pine specimens were wet. He made the following observations;

1) The CSA O86 design rules for lag screws provide the lowest predicted characteristic values. Furthermore, since longer screws are also limited by the maximum embedment length, the predicted resistance does not increase.

2) As screw diameter increases, the two Kevarinmäki methods, which involves using tested strengths diverge from the alternate method which uses EC 5 based strengths. Even though it is more conservative, the method based on EC 5 strengths still overestimates the capacity by as much as 37 % in white pine.

3) The standards did not predict a tensile failure of the fastener in the Nordic Lam which has a very high density. The Kevarinmäki method assumes a compressive failure before tensile fracture, which may not necessarily occur, as had been seen from testing, where most failures were tensile fracture. The EC 5 calculations showed all withdrawal capacities to be lower than the tensile resistance of the screw, even with the high density of Nordic Lam.

4) Prat-Vincent suggested the adoption of the Kevarinmäki's method for the design of cross screw connections, either using an initial Eurocode 5 derived axial withdrawal thread strength or through pre-existing testing of representative fasteners, wood species and EWPs in tabulated form available to designers.

# 3 MATERIALS AND TEST PROGRAM

## 3.1 Overview

The tests carried out were in two forms and conducted over a period of six months in 2011.The first part involved the perpendicular screw withdrawal tests and the second cross screw joint connection tests. The setups for both tests were inspired by similar studies conducted at FPInnovations, Laval University, McGill University and other institutions as described in the literature. Three glulam species were used in addition to No. 2 white pine timber. Structural screws from different European manufacturers with varying diameters and lengths were included in the scope of testing. The cross screw joint tests used special screws with two threaded sections separated by a smooth shank. A detailed description of the two laboratory components of this research is provided in the following Sections.

## 3.2 Glulam wood

Three different species of glulam were used in the testing in order to obtain results that would be representative of the products commonly found in construction in Canada.; namely Douglas-fir, Spruce Pine and Nordic Lam glulam. The first two species are very common in Canada but the latter is a new engineered wood product being produced in Quebec.

### 3.2.1 Douglas fir Properties

Douglas-fir Larch glulam originates from western Canada and they are made from medium to large trees (tree sizes range from 45-85m).They are actually not firs but from trees called False Hemlock (*Pseudotsuga*). The glulam sections used for testing were 20f-E from Western Archrib Structural Wood Systems. Table 3.1 shows the strength properties in comparison with other species. Two different glulam sectional dimensions were used. They were cut to the required lengths for the test by the manufacturer. Twenty of beam size 215 x 228 x 880 mm and ten of 175 x 304 x 800 was used. Figure 3.1(a) shows a typical section.

Table 3.1 : Specified strengths and modulus of elasticity of various glulams (dry service and `standard load) and No. 2 Northern timber

| Material Property | CSA O86 20f-E S-P | CSA O86 20f-E D.fir-L | Nordic 24f-1.9E | EC5 GL 24c | CSA O86 Northern No. 2 |
|---|---|---|---|---|---|
| $f_{bx}$ (MPa) | 25.6 | 25.6 | 30.7 | 19.2 | 3.9 |
| $f_{vx}$ (MPa) | 1.75 | 2.0 | 2.2 | 1.8 | 1.0 |
| $f_c$ (MPa) | 25.2 | 30.2 | 16.5 | 16.8 | 4.1 |
| $f_{cpx}$ (MPa) | 5.8 | 7.0 | 7.0 | 2.4 | 3.5 |
| $E_x$ (MPa) | 10300 | 12400 | 13100 | 11600 | 6000 |
| Mean Relative Density | 0.44 | 0.49 | 0.47 | | 0.37 |
| Mean density 12%MC(kg/m$^3$) | 470 | 520 | 500 | 350 | |

$f_{bx}$= bending at extreme fibre, $f_{vx}$ = longitudinal shear, $f_c$ = compression parallel to grain ,

$f_{cpx}$ = compression perpendicular to grain, $E_x$ = Shear free modulus of elasticity

### 3.2.2 Spruce Pine Fir

The Spruce-Lodgepole Pine-Jack Pine glulam are predominantly made from white spruce (*Piceaglauca*), black spruce (*Picea Mariana*), jack pine (*Pinubanksiana*), balsam fir (*Abies alba*) and lodgepole pine (*Pinus Contorta*).They are usually small and medium size trees. The members used for testing were 20f-E from Western Archrib Structural Wood Systems. Twenty members of size 215 x 228 x 880mm and ten of 175 x 304 x 800mm were used. They were cut to the lengths required for testing by the manufacturer. Table 3.1 shows the strength properties in comparison with other glulam species in Canada and Europe. Figure 3.1(b) shows a typical section.

### 3.2.3 Nordic Lam Properties

This glulam is made of wood from small trees in the north of Quebec which experience long and harsh winter. Their growth rings are very close to each other and they are very dense. The tests using Nordic Lam were conducted at FPInnovations, Université Laval and McGill University. The glulam was Nordic 24f-1.9E commercial appearance grade from Chantiers Chibougamau Ltd, manufactured using the Enviro-Lam$_{TM}$ proprietary layup process. Nordic Lam consists of 95% Black spruce (*Piceamariana*) and 5% Spruce Pine fir (S-P-F) species produced near Chibougamau in central Quebec (Nordic 2012). The moisture content at the time of fabrication ranged from 11-15%. Table 3.1 shows the strength properties of the Nordic Lam in comparison with other glulam species from Canada and Europe. Figure 3.2 shows a typical section.

## 3.3 White Pine Properties

Eastern white pine (*Pinusstrobus*) timber was provided by Les Produits Forestiers D.G. in St-Come, Quebec via Portbec Ltée, a lumber brokerage firm based in Quebec City, for the second part of the research project. The wood was visually graded No. 2 as per the National Lumber Grades Authority (NLGA) grading rules. It was stacked; air dried for two to six months and was sawn to sizes of 200 x 250 x 4800 mm. The material properties of the Eastern white pine used which is classified as post and timbers and belonging to the Northern species group are listed in Table 3.1. Most of the white pine sections which were delivered to the Université Laval's Department of Wood and Forest Sciences were used in the study by Prat-Vincent (2011) while the remaining two sections (Figure 3.3) were incorporated in the research program described herein. The timbers were further sawn into smaller sections of 100 x 200 x 450 mm and then shipped to the Structures Laboratory at McGill University for test specimen assembly. Figure 3.3 shows a typical section.

a) b)

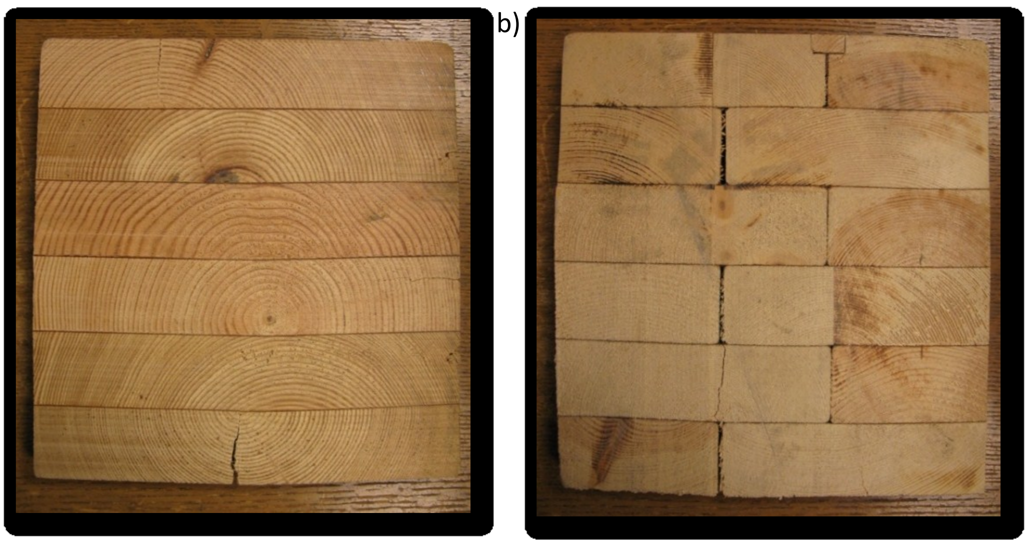


Figure 3-1 a) 20f-E Douglas fir glulam and b) 20f-E Spruce Pine


Figure 3-2 Nordic 24f-1.9E glulam

Figure 3-3 White pine timbers at Université Laval

## 3.4 Screws

Screws from three different European manufacturers were used in the test program. Namely Adolf WürthGmBH & Co. KG, Germany, SFS intec Inc., Switzerland and Schmid Schrauben Hainfeld, Austria. Table 3.2 lists the screws while Table 3.3 provides their properties. Figure 3-4 shows the different screws as well as the two Canadian glulam used for the withdrawal testing. The screws used in the tests program are discussed in the following Subsections and more details including photographs are in Appendix C.

Table 3.2 Screw types used for withdrawal and cross screw connection tests

| **Diameter/Type** | **A** | **B** | **C** |
|---|---|---|---|
| 6mm | | Würth Assy plus VG | Würth Assy 3.0Schr SK |
| 8mm | SFS WFD | Würth Assy VG | Würth Assy SK |
| 10mm | Sonderfertigung | Würth Assy | Schmid |
| 12mm | SFS WFD | Würth Assy | |
| WT-T- 8.2mm, SFS Intec screws used for the cross screw joints tests | | | |

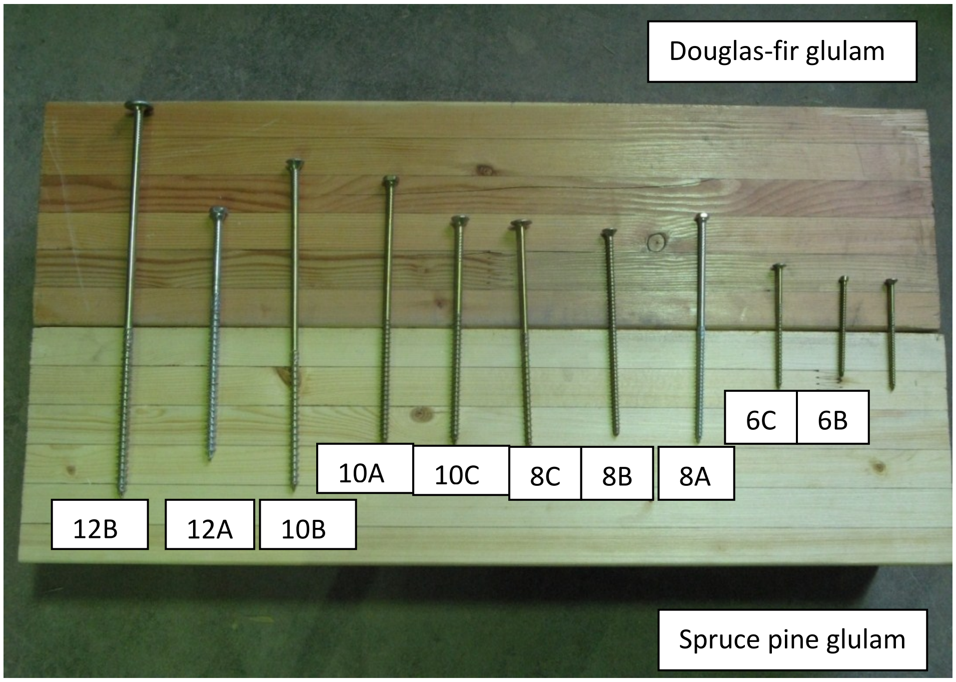



Figure 3-4 Screw types used for withdrawal testing and CSA O86 glulam members

### 3.4.1 Würth Assy plus VG

These are fully threaded wood fasteners made from galvanized carbon steel (Deutschen Institutfür Bautechnik, 2006). They are fabricated from special quality cold-upset wire according to the standards set by German Institute of Construction Engineering. These 6mm and 8mm screws, with lengths ranging from 70-600mm are used to connect solid and laminated wood parts or to connect steel parts to them. They also increase the load carrying capacity of wood parts perpendicular to the direction of the grain. They are also used primarily for static loads. Figure 3.5 shows schematic details of the screw.

### 3.4.2 Würth Ecotast Assy3.0 and Würth Assy 3.0 SK Schrauben

These screws are made of special carbon or stainless steel. Those made from carbon steel are hardened and have organic antifriction coating. They are also electrogalvanised with a yellow or blue chromate or a zinc-nickel coating. They are used to connect solid and laminated wood members or to connect steel plates to them. They have outer thread diameters ranging from 3-14 mm and lengths from 18-1500 mm. Screws with diameters greater than 8mm are recommended for use only in wood products made from any of the following species; spruce, pine and fir. They are used for connections subject to static or quasi static loading. Screws 12 mm in diameter are to be used with washers in wooden materials when subjected to pull-out stress. Figures 3.6-8 show schematic sketches of the screws.

### 3.4.3 SFS WFD

The SFS WFD screws are composed of heat treated steel with surface corrosion protection coating. They are used to fasten timber components, timber materials and steel parts to timber and plywood panels. Its internal drive allows countersinking and removal. It requires no pre-drilling and the hexagonal drive allows for secure gripping during insertion.It has diameters ranging from 6-12mm (SFS intec). Figure 3.8 and Appendix C shows a typical photograph of this screw type.

### 3.4.4 SFS WT-T

These are special screws with threaded ends separated by a smooth shank. They are made of stainless or carbon steel. Those made of stainless steel are waxed while those made of carbon steel are coated with Durocoat. They are available in diameters of 6.5mm and 8.2 mm and of varying lengths. They are fire resistant, easily installed and attractive as the fastener is hidden (SFS intec website).This screw is shown in Figure 3.9.

### 3.4.5 Schmid Schrauben hainfeld

The star drive screw can either be partially threaded or fully threaded. They are used for timber construction and carpentry, and for prefabricated elements. They are made of steel with sharp edge rolled threaded and a tip of $30^{o}$. They have brittle fracture and are specially hardened. They have a diameter range of 3-6mm and a length of 16-300mm. Picture of the screw is shown in Figure C of Appendix C.

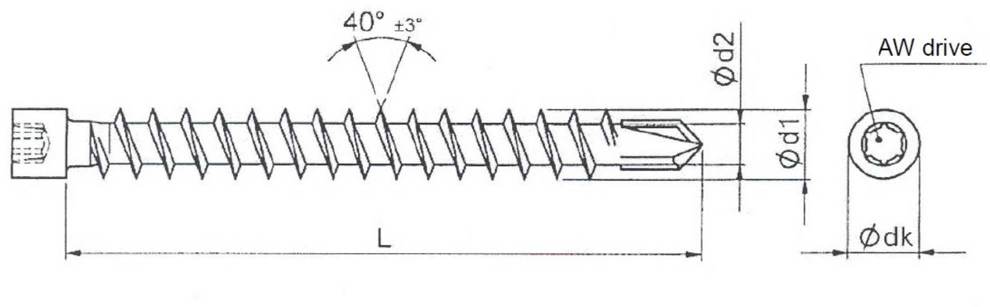



Figure 3-5 Wurth Assy VG Cylindrical head, fully threaded (ETA-11/0190, 2011

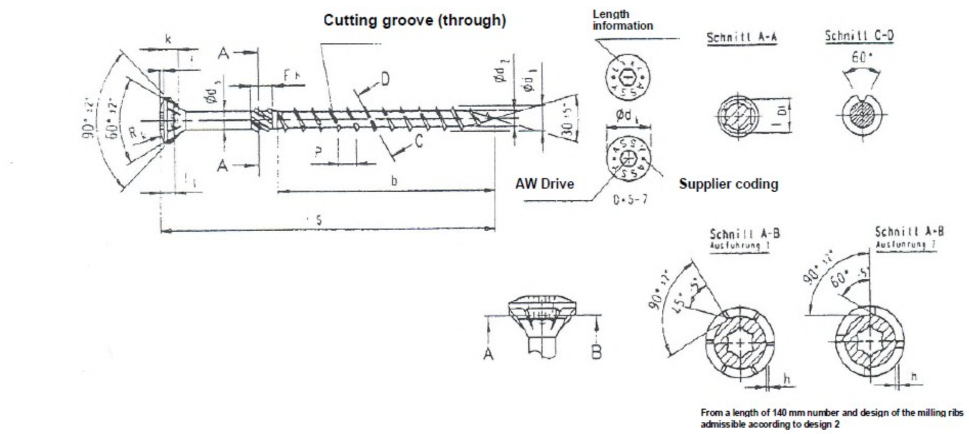



Figure 3-6 Wurth Assy SK oval head, partially threaded (ETA-11/0190)

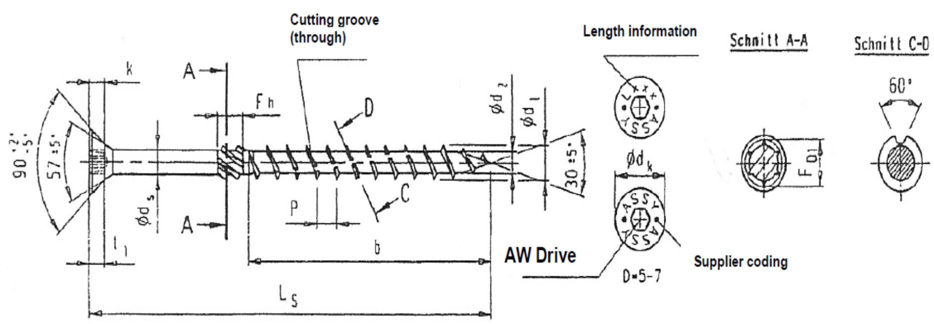



Figure 3-7 Wurth Assy countersunk head, partially threaded (ETA-11/0190)

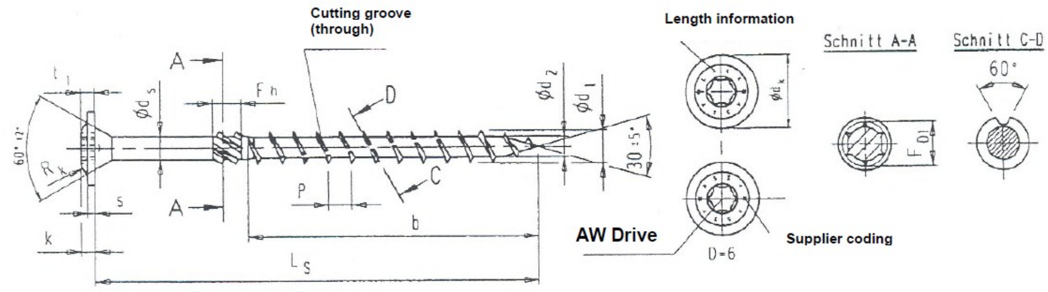



Figure 3-8 Wurth Assy washer head, partially threaded (ETA-11/0190)

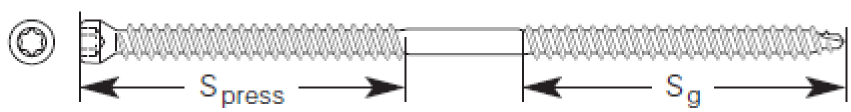



Figure 3-9 WT-T SFS (SFS intec, 2005)

Table 3.3 Properties of screws (Manufacturers' information)

| Type | Total length, (mm) | Length of threaded part,(mm) | Diameter of thread part,(mm) | Diameter of root part,(mm)* | Diameter of Shank part,(mm)* | Tensile strength, kN |
|---|---|---|---|---|---|---|
| Wurth Assy VG | 100 | 100 | 6 | 3.8 | 4.31 | 11 |
| Wurth Assy VG | 200 | 200 | 8 | 5.3 | 5.90 | 20 |
| Wurth Assy SK | 120 | 70 | 6 | 3.9 | 4.20 | |
| WurthAssy Ecofast | 220 | 100 | 8 | 5.1 | 5.61 | 15 |
| WurthAssy Ecofast | 320 | 125 | 10 | 6.4 | 6.87 | 24 |
| WurthAssy SK | 380 | 145 | 12 | 7.10 | 8.16 | 34 |
| SFS WFD | 220 | 100 | 8 | 4.87 | 5.54 | |
| SFS WFD | 260 | 120 | 10 | 6.03 | 6.71 | |
| SFS WFD | 240 | 140 | 12 | 6.72 | 7.82 | |
| SFS WT-T | 160 | *2/57.45 | 8.2 | 5.30 | 6.14 | |
| SFS WT-T | 220 | *2/87.5 | 8.2 | 5.30 | 6.14 | |
| SFS WT-T | 245 | *2/100.50 | 8.2 | 5.30 | 6.14 | |
| Schmidt | 200 | 125 | 10 | 5.90 | 6.77 | |

*measured

## 3.5 Structural screw withdrawal resistance from glulam: test program

### 3.5.1 Test Overview

A total of 1960 withdrawal tests were carried out involving ten different screws and three different glulam types. The responsibility for testing was divided between the author of this thesis and Maxime Coté, a graduate student at

Université Laval. Note, the Douglas fir, Spruce-Pine and 200 specimens of the Nordic Lam glulam specimens were tested at McGill University, while 480 Nordic Lam specimens were tested at the FPInnovations facility in Quebec as well as in the structures laboratory of the Department of Wood and Forest Sciences at Université Laval. The CSA O86 glulam members which were obtained directly from the suppliers, Western Archrib Structural Wood Systems via Timber Systems Ltd were not pre conditioned prior to the installation of the screws and subsequent testing. Depths of 6d and 12d (d = screw diameter) which represents the minimum and maximum depths of penetration in the O86 Standard were used for the threaded portion of the screw. The shank diameters of these structural screws were smaller than the outer thread diameters. The 8mm and 10mm diameters screws were tested both with and without pre drilled lead holes. Lead holes were predrilled for all the 12mm screws to facilitate installation and to avoid the possibility of splitting of the wood. The lead holes were drilled to the full depth of penetration of the screw in question. A drill bit diameter of 5/32" (3.97 mm), 11/64" (4.37 mm) and 13/64" (5.16 mm), respectively, was used for the lead holes for the 8, 10 and 12 mm diameter screws. Imperial sized drill bits are commonly used in construction in Canada, even though the screws were sized in SI units. This resulted in lead hole to screw shank (unthreaded section) diameter ratios between 0.62 and 0.68, which are within the guidelines set for lag screws by CSA O86, and lead hole to outside thread diameter ratios between 0.43 and 0.49 for the three screw sizes. Note, the shank diameter and outer thread diameter of lag screws are of similar dimension, whereas the European structural screws have smaller diameter shanks compared with the threaded section (Table 3.3). Table 3.4 shows the test configurations.

### 3.5.2 Test Specimen Preparation

Each glulam specimen was placed on a horizontal surface and the positions of the various screws were marked and labeled on the top and side of each glulam specimen. The positions were marked to meet minimum spacing requirements set out in EN 1382 (1999) as shown in Figure 3.10.

Table 3.4 Test matrix for 1960 withdrawal specimen

| Screw Type | Depth of Pen. (mm) | Pre-Drill | No Pre-Drill | Number of tests per glulam type | | | Total |
|---|---|---|---|---|---|---|---|
| | | | | Nordic | S-P | D.fir-L | |
| 6B | 36 | - | * | 10/10 | 10/10 | 10/10 | 140 |
| | 72 | - | * | 10/10 | 10/10 | 10/10 | 20** |
| 6C | 36 | - | * | 10/10 | 10/10 | 10/10 | 140 |
| | 72 | - | * | 10/10 | 10/10 | 10/10 | 20** |
| 8A | 48 | * | * | 10/10 | 10/10 | 10/10 | 240 |
| | 96 | * | * | 10/10 | 10/10 | 10/10 | 20** |
| 8B | 48 | * | * | 10/10 | 10/10 | 10/10 | 240 |
| | 96 | * | * | 10/10 | 10/10 | 10/10 | 20** |
| 8C | 48 | * | * | 10/10 | 10/10 | 10/10 | 240 |
| | 96 | * | * | 10/10 | 10/10 | 10/10 | 20** |
| 10A | 60 | * | * | 10/0 | 10/10 | 10/10 | 160 |
| | 120 | * | * | 10/0 | 10/10 | 10/10 | 20** |
| 10B | 60 | * | * | 10/0 | 10/10 | 10/10 | 160 |
| | 120 | * | * | 10/0 | 10/10 | 10/10 | 20** |
| 10C | 60 | * | * | 10/10 | 10/10 | 10/10 | 240 |
| | 120 | * | * | 10/10 | 10/10 | 10/10 | 20** |
| 12A | 72 | * | - | 10//10 | 10/10 | 10/10 | 140 |
| | 144 | * | - | 10/10 | 10/10 | 10/10 | 20** |
| 12B | 72 | * | - | 10/10 | 10/10 | 10/10 | 140 |
| | 144 | * | - | 10/10 | 10/10 | 10/10 | 20** |

Note: 10/10 = 10 tests with screws installed through top of specimen & 10 tests with screws installed in side of specimen. 10/0 = 10 tests with screws installed through top of specimen & 0 tests with screws installed in side of specimen. (for additional information see Section 3.5.1), - = not tested,* = tested,** = additional Nordic Lam test(200 in total)

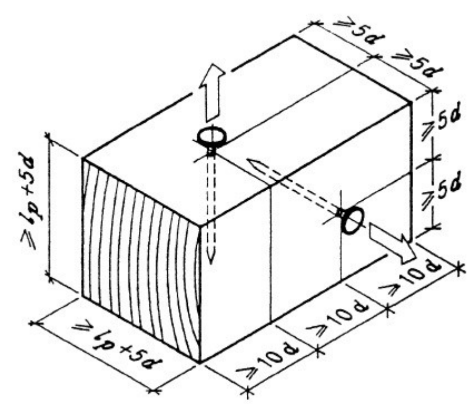



Figure 3-10 Test specimen spacing requirement

Each specimen was assigned an identification label which specify the following

**Specimen identification**: $\underbrace{\#\#}_{1}\underbrace{X}_{2}-\underbrace{\#\#}_{3}-\underbrace{XX}_{4}-\underbrace{\#}_{5}-\underbrace{P}_{6}$

1: Screw diameter

2: Index of screw model as shown in Table 3.2

3: Depth of penetration: 12 for 12d, 06 for 6d

4: Product in which the screw is tested (DF for Douglas-Fir glulam, SP for Spruce-Pine glulam, NDGL for Nordic glulam)

5: Replication number (usually corresponds to a glulam specimen number as each wood block in a series is extracted from a different glulam section)

6: Indicates if the hole is pre-drilled .

**Example:**

**12A-12-T-DF-01-P**: 12mm SFS WFD screw installed with a thread penetration of 12d, (144mm), in the top of glulam block coming from section #1 of Douglas-Fir glulam and installed with a pre-drilled lead hole.

The required depth of penetration as indicated in Table 3.4 was marked on each screw for accurate installation. The glulam sections were marked with the specific identification label of each withdrawal test as illustrated in Figure 3-11. The screws were installed with an SFS intec BO900 tool at right angles in the wood sections with the help of a vertical stand as pictured in Figure 3.12. The same stand and tool were used for the drilling of the lead holes which extended to the full depth of penetration of the screw. For most of the screws, installation with the BO900 was successful; except for some of the 12mm screws from SFS intec, for which it was necessary to use pliers to turn the screws to their full depth due to stripping of the screw head. In all 12 screws were installed on both the top and orthogonal (side) faces of each glulam specimen (Fig 3-11), although not every screw was pre-placed to allow for anchoring of the specimen to the testing pedestal. Once testing of the initially placed screw was complete, they were removed from the wood and the remaining screws were installed and tested.

Figure 3-11 Glulam member with markings and installed screws prior to testing

a)

b)

Figure 3-12 SFS Intec BO900 drill used in-conjunction with vertical stand; a) Predrilling of hole b) Installation of screw

### 3.5.3 Test setup

A MTS 500 kN actuator was installed in a Baldwin-Tate-Emery frame which was secured to the strong floor in the structures laboratory at McGill University. The screw withdrawal test was then incorporated underneath the actuator as illustrated in Figure 3.13(a). This was similar to the test set up at Université Laval as shown in Figure 3.13(b) and at FPInnovations. The direct vertical pullout test was designed based on ASTM D 1761 (2006).Two holes were drilled at the centre in the top and bottom flange of a 150mm Hollow Structural Steel (HSS) member. This was in turn attached to the MTS load cell by means of a threaded bolt and aluminium connector. Steel washers were custom built from steel plates to suit the various fastener diameters and one side beveled to allow full bearing on the sloped underside of the screw heads. Each screw installed glulam specimen was secured to the steel pedestal and the head of the particular screws to be tested were passed through the bottom hole of the HSS. The washer was then slotted under the head of the screw to prevent it from slipping through. The glulam section was attached to the pedestal by steel angles and threaded rods.

a)

b)

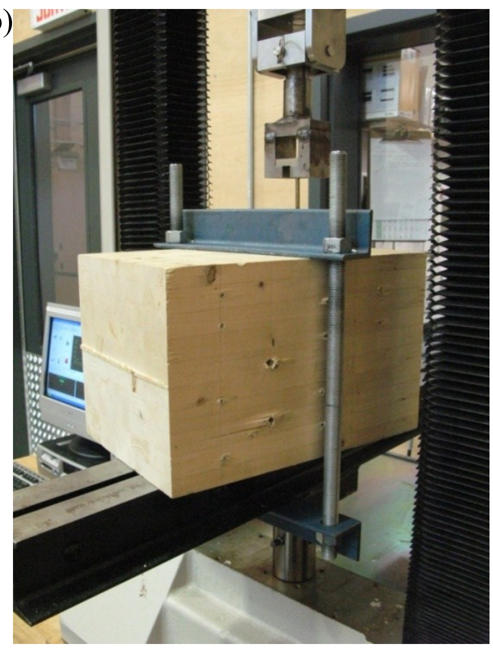


Figure 3-13 Test setup at McGill University and Université de Laval respectively

### 3.5.4 Testing Procedure

The monotonic tests (0.5 mm/min) involved the measurement of load and displacement. This rate was slower than the rate (2.5mm/minute) in ASTM D1761 but according to Johnson (1959) the effects of these slightly different rates is insignificant. Johnson found that test extraction speed greatly affected strength if the rate considerably exceeded 12.7mm/min. A load cell, an LVDT internal to the actuator and two external LVDTs were attached to a data acquisition system composed of Vishay Model 5100B scanners running on Vishay System 5000 StrainSmart software. Similar instruments were used at Université Laval and at FPInnovations. Data were recorded every half second. The maximum load was achieved generally in 6±2 min. The test was stopped when approximately half of the ultimate resistance was reached in the post peak range. The specimen was then disassembled and the damage to the screws and wood recorded.

## 3.6 Cross screw connection test of white pine members

### 3.6.1 Test specimen preparation and setup

In order to evaluate the resistance of the joist-to-header connections a short spanning beam was attached between two headers (acting as girders) as per the ASTM D7147 (2005) standard for the testing of joist hangers. The configuration of the test specimens is illustrated in Figure 3-14 (all dimensions are in mm unless otherwise stated). This assembly thus represented a typical roof joist connection where self-drilling / tapping screw fasteners are often utilized. The wood members used for testing were No 2 white pine timber. The wood members were relatively wet at the time of connection assembly. Sizing of the members was selected based on resistance, which had to be more than the estimated capacity of the connections. Three connection types were tested; a single pair of SFS WT-T-8.2-245, WT-T-8.2-220 and WT-T-8.2-160 screws for joists of size 125mm x 200mmas shown in Table 3.7. The joists were 500mm long and the headers were 250mm.The headers were made as short as possible since they were not going to be re-used for another set of tests.

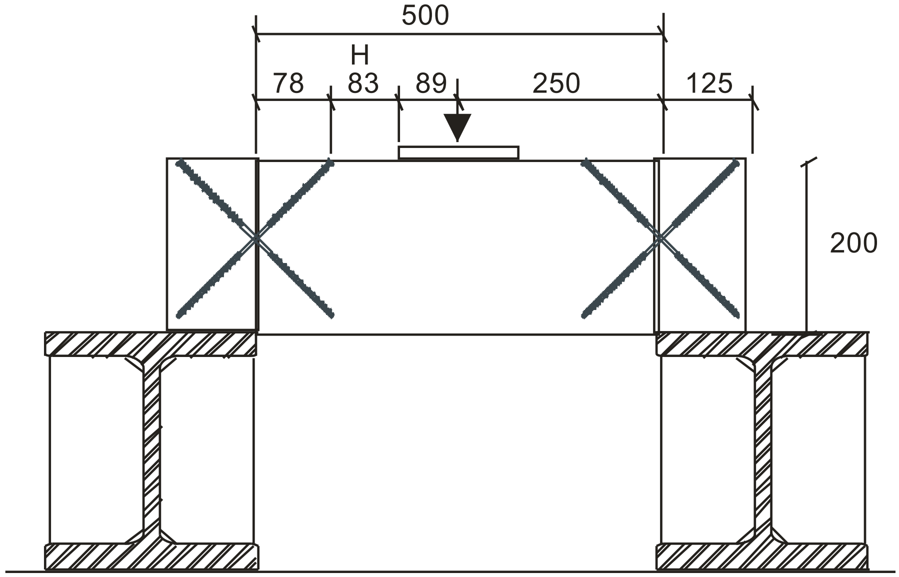



Figure 3-14 Side elevation cross screw connection test

Table 3.5 Joist and Header dimensions for cross screw connection test

| Test Configuration | No of Tests | Joist(mm) | Header(mm) | Fastener | Fastener per joint |
|---|---|---|---|---|---|
| 1 | 6 | 125 x 200 | 125 x 200 | 8.2-160 | 2 |
| 2 | 6 | 125 x 200 | 125 x 200 | 8.2-220 | 2 |
| 3 | 2 | 125 x 200 | 125 x 200 | 8.2-245 | 2 |

The cross screw WT-T fasteners were installed using an SFS intec ZL-WT drill and ZL-WT/U system (Figure 3.15). This ZL WT/U system is comprised of a plate with a spring loaded screw that is fixed to the joist or header, the plate contains notches on all sides, indicating where placement should be relative to the centerline of the joist and the desired length of the screw (this must be calculated beforehand and marked on the members, with a ±10 mm tolerance accounted for). Attached by a hinge to the plate are the guide tubes (interchangeable for different diameter screws). The desired angle ($45^{o}$) was selected by means of an arch with notches at various angles where the hinge can be locked in. Each screw was installed such that its head laid flush with the surface of the wood. The assembled connection specimens were stored in the structural engineering laboratory of McGill University for five months prior to testing to allow for air drying. A similar experimental program was earlier conducted at McGill University by Prat-Vincent (2011) in which he tested white pine cross screw connections specimens in the wet state whilst in this current program the specimens were allowed to dry prior to testing. During the course of this drying period each specimen was weighed once a week and a record of the change in moisture content was kept. Once the moisture content reached equilibrium, testing was carried out. The headers rested on steel supports and were shimmed as needed so the joist would be level (Fig 3-16).

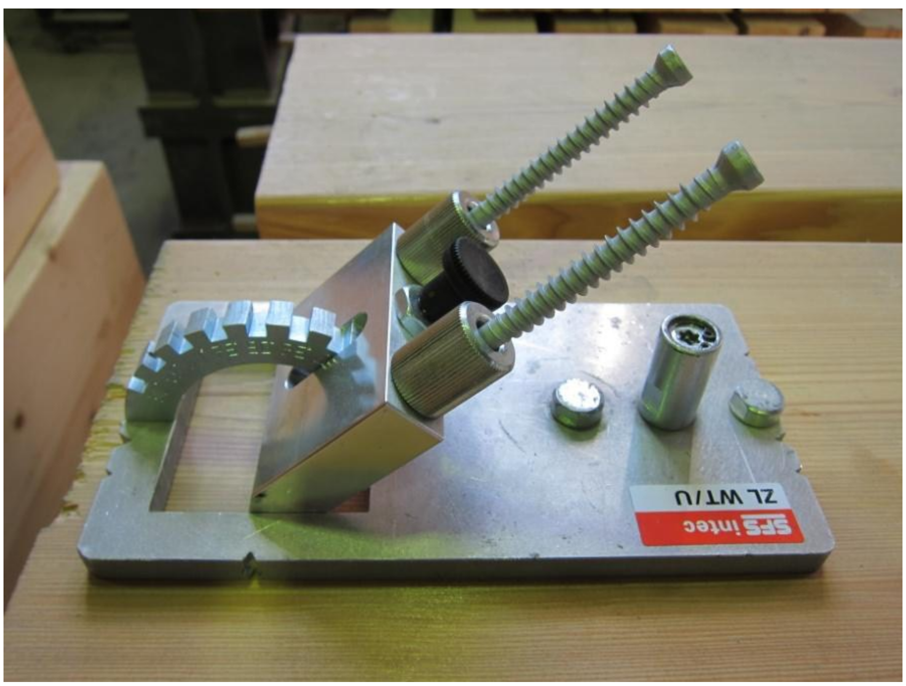



Figure 3-15 Screws installation using ZL WT/U

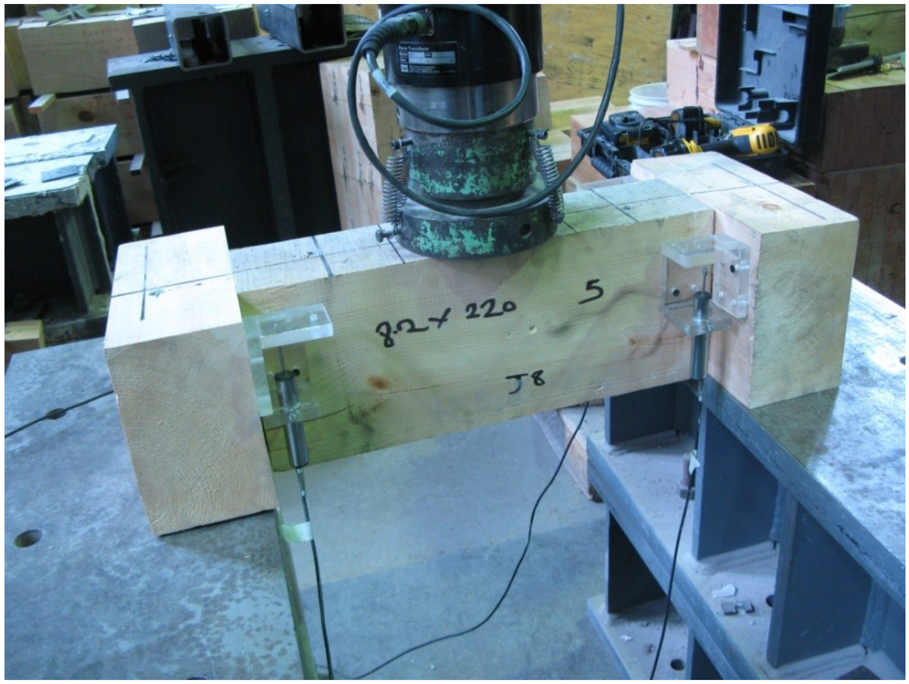



Figure 3-16 Typical cross screw connection test set up ready for testing

### 3.6.2 Connection Test Procedure

The loading procedure was modeled after the ASTM D7147 Standard (2005). It requires the application of an initial preload of not more than 20% of the ultimate load (based on predicted values) which is then removed before reapplying load using one of two protocol described below until failure. The load was applied at the centre of the joist. A computer was used to operate the actuator and another was used to acquire and save the data. The test involved the measurement of load and displacement .The load cell, internal actuator LVDT and four external LVDTs (Fig 3-16) which were used to measure connection resistance and displacement respectively were attached to a data acquisition system composed of Vishay Model 5100B scanners running on Vishay System 5000 StrainSmart software. Displacement and load data was recorded every half second. Each test was stopped when approximately half of the ultimate resistance was reached in the post peak range.

After reviewing the cyclic and monotonic pull-out tests of the prior research program conducted by Prat-Vincent, it was decided to split each sample set in half. Three specimens were tested monotonically until failure at a displacement rate of 0.8mm/minute. The average ultimate loads of these three test was then used to define the protocol for the three remaining specimens. These connections were cyclically loaded up to 15 %, 30 %, and 45 % of the average ultimate monotonic loads. Thirty cycles were run from the low load to 15%, 30 cycles from the low load to 30% and 30 cycles from the low load to 45%. This was followed by monotonic loading until failure. By testing all sample sets under this program it was be possible to identify whether the repeated loads would negatively affect the connection resistance. Each whole test took an average of 15 minutes. Once the ultimate load was reached there would be a sound followed by rapid increase in deflection. After each test, the sample was disassembled and the details of the state of the wood and screws were recorded.

## 3.7 Moisture Content Determination for Glulam and White Pine Wood Specimens

ASTM Standards D4442 (2007) (Method B) and D2395 (2007) (Method A) were used for the determination of MC and density of wood respectively. After the testing of all screws in a glulam specimen had been completed it was disassembled and a 25 mm thick cross sectional slice was cut from the member. Once labelled, weighed and measured for dimensions the slice was dried in an oven for 48 hours at a temperature of 100 +/- 50C. Usually, a constant mass was attained after 48 hours of drying; if not, the wood slice was left in the oven for another 24 hours. The samples were again weighed and measured and the resulting MC and specific gravity were calculated. The density at the time of test was calculated by dividing the mass at the time of testing by the volume at the time of testing.

# 4 RESULTS AND DISCUSSION

## 4.1 General Observation of Screw Withdrawal Test Results

In the majority of cases failure was governed by the withdrawal of the embedded screws from the glulam due to the local failure of the wood shown in Figures 4.1-2. In only a few cases did the screw fail in tension. This was not expected because the tensile strength of the screws was higher than the anticipated holding capacity of the glulam, according to the manufacturer and as measured in screw tests also completed for this study. The data from tests in which screw failure occurred were not included in the statistical evaluation presented herein because the intent was to measure the withdrawal failure resistance under the influence of different parameters including screw diameter, depth of penetration, lead holes, density of wood and the orientation of the wood section. Hence 1740 test results were used for the analysis. An additional 200 test on Nordic glulam was conducted later and added to the final analysis.

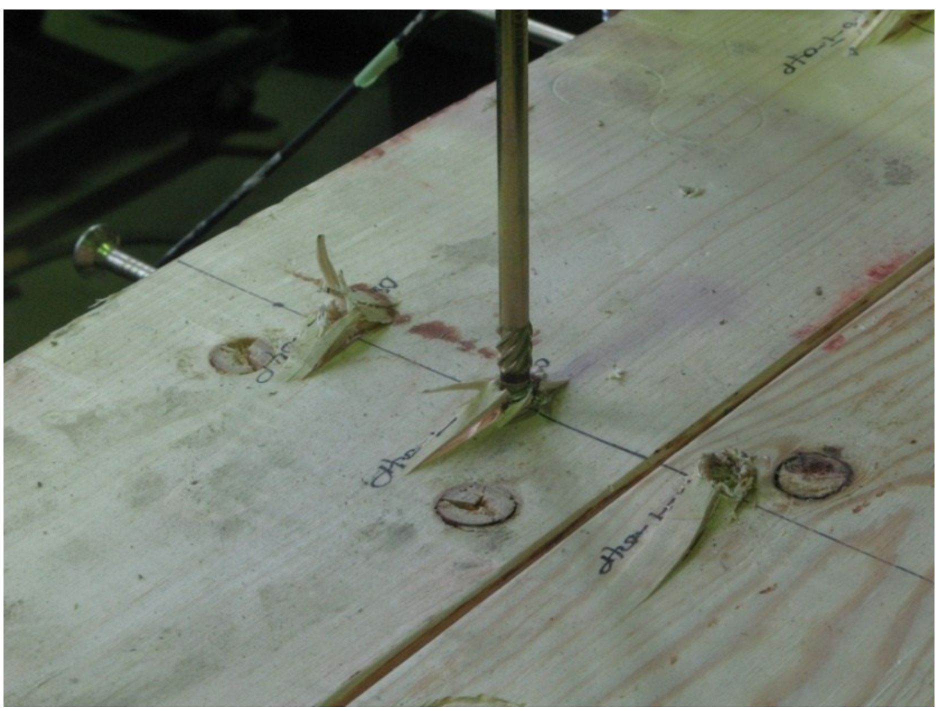


Figure 4-1 Withdrawal failure


Figure 4-2 Detail showing Withdrawal failure

Table 4.1 shows the summary of the test results of all withdrawal tests that were carried out. There was no appreciable difference between screws of the same diameters from different manufacturers as discussed in Section 4.2, hence they were grouped according to screw diameters. The $5^{th}$ percentile values listed in this thesis were calculated using ASTM D2915 (2003) guidelines accounting for the sample size and a confidence level of 75% assuming a normal distribution. From Table 4.1 it can be seen that the average withdrawal strength increases with increase in screw diameter with all other variables being held constant. For instance a 6 mm diameter screw has withdrawal strength of 0.153 kN/mm while an 8mm diameter screw has withdrawal strength of 0.197 kN/mm. Details of screw diameter effects is discussed in Section 4.6. The average withdrawal strengths measured for the Douglas-fir specimens were higher than those for spruce-pine, all other variables being held constant (Table 4.1). The Nordic Lam glulam was expected to provide intermediate screw withdrawal strength between the Douglas-fir and spruce-pine specimens given the mean relative density for connection design (Table 3.4). However, the 10 and 12 mm screws were measured with a higher strength than in the Douglas-fir tests, while the 6 and 8 mm screws, the measured strength was lower than the Douglas-fir but higher than that of the Spruce-pine specimens. An additional 200 test on Nordic glulam showed the same trend except the 8mm diameter screws which were almost equal to that of the Douglas fir. Details of the effects of the glulam types are discussed in Section 4.5. The coefficient of variation ranged from 10-24% (Table 4.1). Appendix A shows the test results of all the configurations tested, which are averages of ten replicas.

Table 4.1 Summary of Withdrawal Strengths of Test Results

| Screw size (mm) | Count | Stats Values | All species kN/mm | Douglas fir kN/mm | Spruce pine kN/mm | Nordic Lam kN/mm |
|---|---|---|---|---|---|---|
| **6** | **277** | **Avg** | 0.153 | 0.175 | 0.137 | 0.148 |
| | | **SD** | 0.027 | 0.028 | 0.024 | 0.018 |
| | | **CoV** | 0.177 | 0.157 | 0.173 | 0.121 |
| | | **5th %** | 0.106 | 0.126 | 0.095 | 0.116 |
| **8** | **774** | **Avg** | 0.197 | 0.215 | 0.171 | 0.203 |
| | | **SD** | 0.030 | 0.026 | 0.024 | 0.022 |
| | | **CoV** | 0.151 | 0.122 | 0.139 | 0.109 |
| | | **5th %** | 0.147 | 0.170 | 0.130 | 0.165 |
| **10** | **610** | **Avg** | 0.225 | 0.235 | 0.196 | 0.255 |
| | | **SD** | 0.039 | 0.035 | 0.028 | 0.028 |
| | | **CoV** | 0.174 | 0.151 | 0.143 | 0.109 |
| | | **5th %** | 0.159 | 0.174 | 0.148 | 0.206 |
| **12** | **279** | **Avg** | 0.263 | 0.265 | 0.230 | 0.285 |
| | | **SD** | 0.042 | 0.043 | 0.033 | 0.029 |
| | | **CoV** | 0.158 | 0.164 | 0.143 | 0.102 |
| | | **5th %** | 0.192 | 0.188 | 0.171 | 0.233 |

Figures 4.3-6 show typical load vs. displacement diagrams for representative configurations (10 tests were carried out per configuration). Only the results for the Douglas-fir glulam specimens are provided with one example for each screw size; all graphs for the remaining test configurations can be found in Appendix B. In Figure 4.3 which is a screw type 6C at a depth of 6d on the top of the Douglas fir section, the average load is about 6.5 kN with an average displacement of about 1.8 mm at the ultimate load. There was no significant difference of test

results on either top or side of the glulam sections as discussed in much detail in Section 4.3. In Figures 4.4 and 4.5 the ultimate average withdrawal loads were about 8.25 kN and 12.5 kN for screw types 8A and 10C at depths of 6d. The average displacements at the ultimate loads were 2.25 mm and 2.50 mm. In addition, the effects of lead holes on the 8 and 10 mm diameter screws were insignificant as explained in Section 4.4. Figure 4.6 displays a type 12A screw at a depth of 6d. The average ultimate load is 17 kN with a corresponding average displacement of 3.0 mm. In almost all the graphs there was consistency in all the 10 replicas plotted in term of ultimate loads and the corresponding displacements. The was a significant effect of depth of penetration and glulam type on withdrawal loads in the graphs and these effects are discussed in details in Section 4.7 and 4.5 respectively.

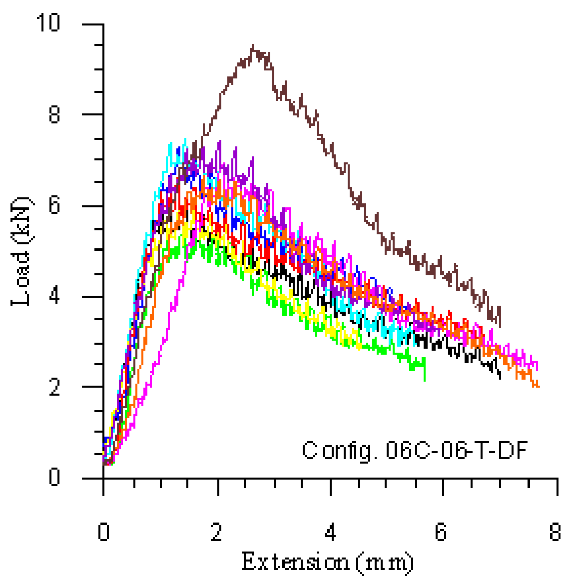



Figure 4-3 Load vs. displacement, 6 mm screw, top, Douglas- fir

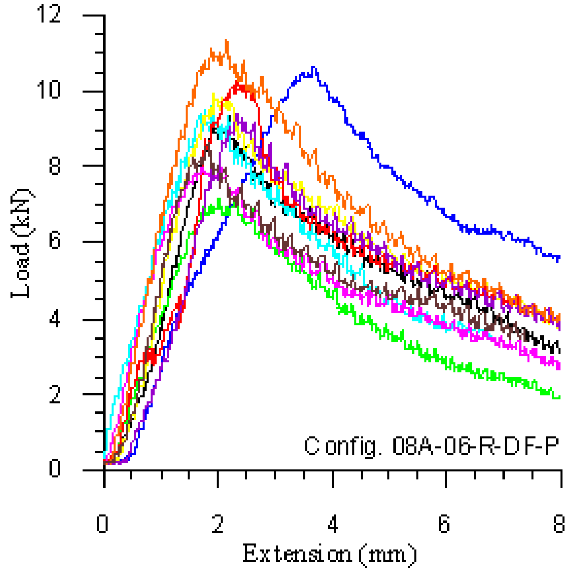



Figure 4-4 Load vs. displacement, 8 mm screw, side, pre-drilled, Douglas-fir

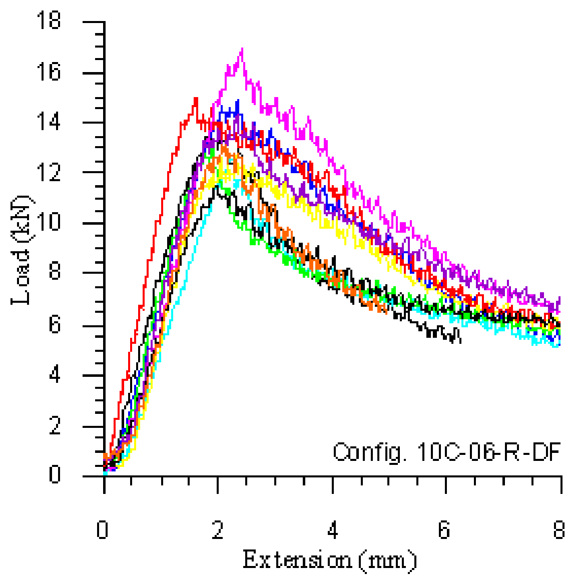



Figure 4-5 Load vs. displacement, 10 mm screw, side, Douglas-fir

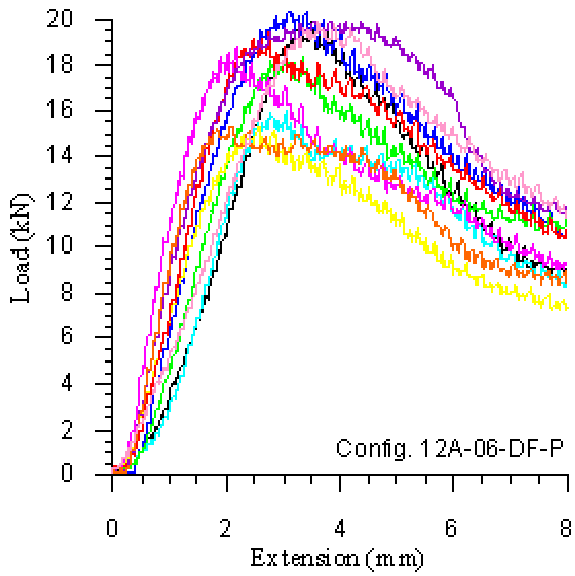



Figure 4-6 Load vs. displacement, 12mm screw, top, Douglas-fir

Table 4.2 shows the average densities and specific gravities of the glulam specimen used at the time of testing. Thus the data includes the Douglas-fir, Spruce pine and as well as the Nordic Lam. Each group is an average of ten glulam sections for the ten replica tests of each screw.The MC of the Douglas-fir, spruce-pine and the Nordic Lam glulam were between 5 and 12%. Figure 4-7 shows a graph of the specific gravities of the glulam specimens used for the testing in which the data for Douglas-fir, Spruce pine and Nordic Lam glulam specimen were combined. A skewed distribution exists with the specific gravity of 0.50 having the greatest frequency of occurrence. For the estimation of the predicted withdrawal values using the formulas from Europe discussed in Section 4.10, the specific gravity values in Table 3.4 are converted to density at 12% moisture content. Hence values of 470kg/m$^3$, 500kg/m$^3$ and 520kg/m$^3$ were used for Spruce pine, Nordic Lam and Douglas fir glulams respectively.

Table 4.2 Density and Specific Gravity of glulam members

| Glulam type | All density | | All Spec Gravity | | Group 1 | | Group 2 | | Group 3 | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Density (kg/m3) | Stand. Dev | Spec Gravity | Stand. Dev | Density (kg/m3) | Spec. Gravity | Density (kg/m3) | Spec. Gravity | Density (kg/m3) | Spec. Gravity |
| Doug. Fir | 521 | 31.8 | 0.515 | 0.0323 | 517 | 0.528 | 551 | 0.53 | 495 | 0.491 |
| Spruce p | 455 | 17.7 | 0.445 | 0.015 | 457 | 0.446 | 447 | 0.436 | 462 | 0.45 |
| Nordic L | 538 | 14.9 | 0.518 | 0.0121 | | | | | | |

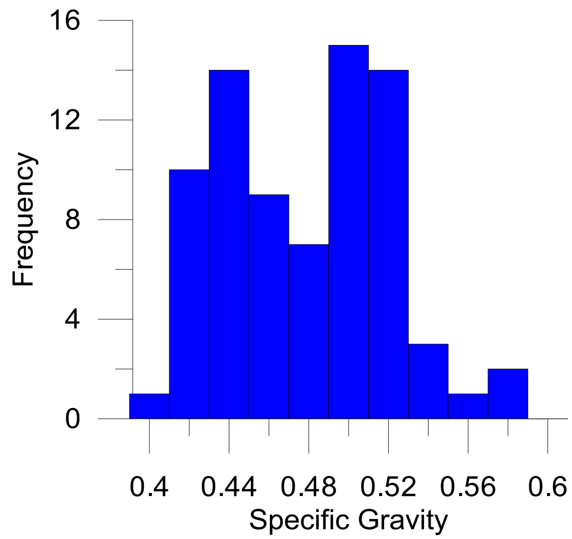



Figure 4-7 Measured Specific gravity of tested glulam section

## 4.2 Effects of Screw Type

Screws of the same diameter from different manufacturers were used in the withdrawal tests (Table 4.3). The aim was to find out if they had similar withdrawal strengths. For 6mm and 12mm diameters screws, two types were used while for 8mm and 10mm diameter screws, three types were used except for Nordic Lam specimens for which new tests of ten screws each were later carried out on screws A and B. These screws were quite different from each other in terms of thread spacing and drilling tips as discussed in Sections 1.1 and 3.4. Tables 3.3-4 and Appendix C give details of the screws used for the withdrawal testing. The withdrawal strength results of the different screws used are shown in Table 4.3. For Douglas fir there was not much difference in the withdrawal test values for the different screws in the screw diameter category. The standard deviations were quite close except for 12B which had 0.06 as compared to 0.03 for 12A. The characteristic ($5^{th}$ percentile) withdrawal strength values were also close to each other. The same observations were made for spruce pine and the Nordic Lam glulam. In the case of the Nordic glulam, the standard deviations of the screws 12A and 12B were close to each other as compared to the other tested glulam species. The withdrawal strength difference for all glulam species between the different types of screws for diameters of 6, 8 , 10 and 12 mm were 4.6%, 5.78%, 9.3% and 5.47% respectively while their coefficient of variations ranged from 10-21%. Hence statistically there was no difference in withdrawal strength between the different screws used and it can be concluded from the data obtained that screws of the same diameter from different manufacturers, within the range of screws tested have similar withdrawal strengths. Since these screws are proprietary products they are not standardised as compared to wood screws. However these screws must meets the standards set by the German Institute for Construction Engineering for approval for general construction and hence it is the responsibility of the manufacturers to ensure that the screws are of that quality.

Table 4.3 Effects of screw type on withdrawal strength

| Glulam type | Statist Values | 6mm | | 8mm | | | 10mm | | | 12mm | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | B | C | A | B | C | A | B | C | A | B |
| | | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm |
| **DF** | Avg | **0.181** | **0.170** | **0.217** | **0.221** | **0.206** | **0.238** | **0.222** | **0.246** | **0.265** | **0.264** |
| | SD | 0.022 | 0.031 | 0.029 | 0.024 | 0.024 | 0.037 | 0.034 | 0.031 | 0.026 | 0.056 |
| | CoV | 0.124 | 0.182 | 0.134 | 0.110 | 0.114 | 0.156 | 0.152 | 0.128 | 0.099 | 0.213 |
| | 5th % | 0.140 | 0.113 | 0.164 | 0.176 | 0.163 | 0.170 | 0.160 | 0.188 | 0.217 | 0.161 |
| | Count | 38 | 40 | 80 | 80 | 80 | 77 | 80 | 78 | 40 | 39 |
| **SP** | Avg | **0.141** | **0.133** | **0.177** | **0.174** | **0.160** | **0.201** | **0.187** | **0.202** | **0.226** | **0.264** |
| | SD | 0.017 | 0.029 | 0.024 | 0.023 | 0.029 | 0.029 | 0.027 | 0.026 | 0.027 | 0.056 |
| | CoV | 0.121 | 0.216 | 0.134 | 0.132 | 0.181 | 0.146 | 0.142 | 0.127 | 0.120 | 0.213 |
| | 5th % | 0.110 | 0.080 | 0.134 | 0.132 | 0.107 | 0.147 | 0.138 | 0.155 | 0.176 | 0.161 |
| | Count | 40 | 39 | 80 | 79 | 76 | 79 | 79 | 77 | 40 | 40 |
| **NL** | Avg | **0.149** | **0.147** | **0.208** | **0.197** | **0.204** | **0.239** | **0.235** | **0.258** | **0.276** | **0.283** |
| | SD | 0.020 | 0.016 | 0.024 | 0.020 | 0.021 | 0.022 | 0.026 | 0.028 | 0.026 | 0.023 |
| | CoV | 0.133 | 0.109 | 0.113 | 0.101 | 0.104 | 0.093 | 0.112 | 0.107 | 0.095 | 0.081 |
| | 5th % | 0.112 | 0.118 | 0.165 | 0.161 | 0.165 | 0.198 | 0.187 | 0.207 | 0.228 | 0.241 |
| | Count | 60 | 60 | 100 | 100 | 99 | 20 | 20 | 100 | 60 | 60 |
| **All Spec.** | Avg | **0.157** | **0.150** | **0.201** | **0.197** | **0.190** | **0.226** | **0.215** | **0.235** | **0.256** | **0.270** |
| | SD | 0.020 | 0.025 | 0.026 | 0.022 | 0.025 | 0.030 | 0.029 | 0.028 | 0.026 | 0.045 |
| | CoV | 0.126 | 0.169 | 0.127 | 0.114 | 0.133 | 0.132 | 0.135 | 0.121 | 0.104 | 0.169 |
| | 5th % | 0.121 | 0.104 | 0.154 | 0.156 | 0.145 | 0.172 | 0.162 | 0.183 | 0.207 | 0.188 |
| | Count | 138 | 139 | 260 | 259 | 255 | 176 | 179 | 255 | 140 | 139 |

Spec – species, Avg – average, SD – standard deviation, CoV – coefficient of variation, $5^{th}$ – $5^{th}$ percentile

## 4.3 Effects of Glulam Section Orientation on Withdrawal Strength

Screws were installed in both the top and the side of the glulam sections. Since the CSA O86 (2009) glulam sections are made of layers of lamina, it means a drilled screw through the top would penetrate through at least one 38mm thick lamination, except for the 6mm, 6d screws. Those screws that were installed on the side would be contained in one whole lamina. The average ultimate loads for the top tests were in a majority cases for Douglas fir and spruce pine slightly higher than those on the lateral side. For the screws in the top it goes through a high quality layer and then lower quality laminations whereas on the side many of the tests were at the mid height of the beam, which means they are in lower quality laminations. The Nordic Lam sections are composed of 38×25 mm laminations across the width and over the height of the section (Figure 3.2); the screws extended through multiple laminations whether placed in the top or side face of the member.Table 4.4 shows the data obtained for both top and side test. As indicated in Table 4.4, the ratio of withdrawal strength of top to side is 1+/-0.06 except for 6mm Douglas fir which was +0.12.The average overall top/side ratio for all the configurations was 1.028. This shows the closeness of the withdrawal values for test carried out on both the top and side. In the D-fir and S-P configurations the side face screws resulted in a higher variation in the withdrawal strength as indicated in the standard deviations and hence co-coefficient of variation. In these products, the exterior laminations are comprised of higher-grade lumber than the interior ones. Therefore, the top face screws driven into one or more exterior laminations of higher quality would generally lead to a higher withdrawal strength and less variation, whereas the strength of side face screws would depend on the quality of the individual laminations and their location along the width of the product (Figure 4-8).The likely strengths of the various laminations in the Douglas fir and spruce pine sections are shown (Figure 4-8). For 20f-E glulam, which was used in the test program, the top and bottom laminations have minimum E values of 13100 and 11000 MPa for Douglas fir and spruce pine respectively. As shown in Figure 4-8 the E values decrease from the outside of the glulam section towards the interior.

a) b)

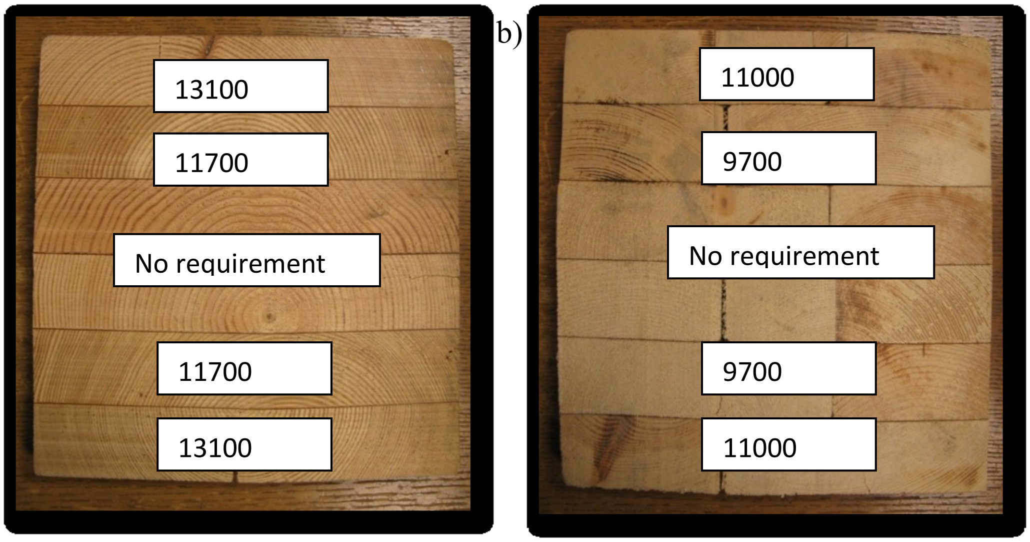



Figure 4-8 Minimum strengths of the various laminas in the CSA O86 a) Douglas fir b) spruce pine

The middle laminas have no specific strength requirements but for 20f-E glulam members they are always lower than the other laminas. The Nordic Lam variation in strength did not exhibit a trend favouring either the top or side face screw locations, likely due to the uniform distribution of laminations of equal quality across the section.

Table 4.4 Effects of Glulam Section Orientation

| Screw diameter | Species | Withdrawal Strength Top (kN) | Side(kN) | Top/side ratio | Standard deviation Top(kN) | Side(kN) | CoV Top | Side |
|---|---|---|---|---|---|---|---|---|
| 6 | D-Fir | 0.185 | 0.165 | 1.125 | 0.023 | 0.028 | 0.169 | 0.169 |
| | S-P | 0.135 | 0.140 | 0.962 | 0.018 | 0.029 | 0.133 | 0.205 |
| | NL | 0.151 | 0.142 | 1.059 | 0.018 | 0.016 | 0.120 | 0.116 |
| 8 | D-Fir | 0.220 | 0.210 | 1.046 | 0.024 | 0.028 | 0.107 | 0.133 |
| | S-P | 0.170 | 0.173 | 0.981 | 0.020 | 0.027 | 0.116 | 0.158 |
| | NL | 0.208 | 0.197 | 1.056 | 0.022 | 0.021 | 0.104 | 0.107 |
| 10 | D-Fir | 0.239 | 0.231 | 1.037 | 0.031 | 0.039 | 0.131 | 0.168 |
| | S-P | 0.200 | 0.192 | 1.042 | 0.024 | 0.031 | 0.120 | 0.162 |
| | NL | 0.252 | 0.262 | 0.964 | 0.029 | 0.024 | 0.114 | 0.093 |
| 12 | D-Fir | 0.261 | 0.269 | 0.971 | 0.036 | 0.050 | 0.137 | 0.188 |
| | S-P | 0.235 | 0.224 | 1.050 | 0.028 | 0.036 | 0.119 | 0.163 |
| | NL | 0.289 | 0.277 | 1.041 | 0.030 | 0.026 | 0.103 | 0.094 |

## 4.4 Effects of lead holes on Withdrawal Strength

The 8 mm and 10 mm screws were installed both with and without lead holes. All the 12 mm screws were predrilled to prevent splitting of wood while the 6 mm screws were not. For this reason, only the 8 mm and 10 mm screws can be used to assess the effects of pre-drilled lead holes. The general finding was that the use of lead-holes did not affect the withdrawal strength of the screw connections (Table 4.5); however it did facilitate installation of the larger screws especially in the higher density glulam. This confirms the finding of Pirnbacher and Schickhofer (2010) who concluded that lead holes have no effects on withdrawal strengths using self-tapping screws. The difference between screws that were self drilled and those with lead holes in the present study was quite low ranging from 0.8% to 8%. A 6% difference was found for the 10mm diameter screws in Douglas fir where the average withdrawal values for the configurations with lead holes were higher. For the 8 mm diameter screws in Douglas fir the difference was 2.3%.The comparison of the average withdrawal values for the self-drilled and predrilled configurations were quite balanced as to which was higher. For instance in Douglas fir the self-drilled withdrawal values in 8mm diameter screws were higher than the pre-drilled ones and vice versa for 10mm diameter screws in the same glulam group. In the Spruce Pine glulam, the average withdrawal strength difference between the predrilled and self-drilled configurations was 0.82%. For the 10mm diameter screws, the average withdrawal strength of the self-drilled configurations was 2.9% higher than the predrilled ones. In the Nordic Lam the difference between self-drilled and pre-drilled was 5.5% for 8mm diameter screws. For 10mm diameter screws the predrilled values were higher and the difference was 8.0%. Generally the lower difference in average withdrawal strength values for this type of structural screw as compared to lag screws, for which lead holes are required in all cases, is to be expected since these structural screws are designed to be self-drilling. They reduce the cracks in wood materials hence tension perpendicular to grain is reduced. It can therefore be concluded that pilot holes have no significant effect on capacity but it makes screw installations much easier. Another issue of important note was the standard deviation. For the

Douglas fir, in all the screws the standard deviations were lower in the pre-drilled test as compared to those without lead holes. This means that the test results were more consistent and closer to each other in the pre-drilled ones as compared to those with lead holes. For the Spruce pine the predrilled configurations were lower or had equal standard deviation as the self-drilled while for the Nordic glulam it was quite mixed.

Table 4.5 Effects of lead hole on Withdrawal Strength

| Screw size | Statistical cal | All species | | Douglas fir | | Spruce Pine | | Nordic Glulam | |
|---|---|---|---|---|---|---|---|---|---|
| | | Self drill | Pre-drill | Self drill | Pre-drill | Self drill | Pre-drill | Self drill | Pre-drill |
| | | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm | kN/mm |
| 8 | Avg | **0.200** | **0.194** | **0.217** | **0.212** | **0.169** | **0.172** | **0.208** | **0.197** |
| | SD | 0.033 | 0.028 | 0.028 | 0.024 | 0.029 | 0.024 | 0.023 | 0.018 |
| | CoV | 0.163 | 0.143 | 0.128 | 0.115 | 0.170 | 0.137 | 0.111 | 0.094 |
| | 5th % | 0.136 | 0.139 | 0.163 | 0.164 | 0.113 | 0.126 | 0.162 | 0.160 |
| | Count | 417 | 357 | 120 | 120 | 117 | 118 | 180 | 119 |
| 10 | Avg | **0.224** | **0.225** | **0.228** | **0.243** | **0.198** | **0.194** | **0.249** | **0.269** |
| | SD | 0.040 | 0.041 | 0.038 | 0.031 | 0.035 | 0.026 | 0.025 | 0.030 |
| | CoV | 0.177 | 0.182 | 0.169 | 0.126 | 0.178 | 0.132 | 0.100 | 0.113 |
| | 5th % | 0.146 | 0.145 | 0.152 | 0.183 | 0.128 | 0.143 | 0.199 | 0.208 |
| | Count | 334 | 276 | 118 | 119 | 116 | 117 | 100 | 40 |

## 4.5 Effects of Density on Withdrawal

The three glulam species used for the testing had different densities. According CSA O86 Douglas fir has a higher density than Spruce pine and this was confirmed by the measured values. The density of the Nordic Lam glulam provided by the manufacturers was 550 $kg/m^3$ but the mean relative density which is used for connection design was 0.47 as recommended by the manufacturers (Nordic, 2012). Densities of 520$kg/m^3$, 455$kg/m^3$ and 538$kg/m^3$ were calculated for the tested specimens of Douglas fir ,Spruce Pine and Nordic Lam glulam respectively. From the test results, density has an effect on withdrawal strength of screws. For instance a 14% increase in density from 440$kg/m^3$ ( Spruce Pine, measured 455 $kg/m^3$) to 490$kg/m^3$ ( Douglas Fir, measured 520$kg/m^3$) resulted in 28%, 25%, 20% and 15% increase in the withdrawal strength per mm of 6mm, 8mm, 10mm and 12mm respectively. For the Douglas fir and Spruce Pine, there was a consistent increase in withdrawal strength per unit length with density increase as shown in Table 4.6 and Figure 4-9. The values for the Nordic were quite inconsistent as compared to the other glulam species. The 6 mm and 8 mm diameter screws had withdrawal strength per unit length lower than that of the Douglas fir while for the 10 mm and 12 mm, the measured values were higher. An additional 200 screw withdrawal tests of the Nordic Lam glulam were carried out; the results are listed in Table 4.6 as New Nordic Lam. The new tests showed similar results to the initial tests for Nordic Lam from Laval University and FPInnovations except a different trend in the 8mm diameter screw. In the new tests the withdrawal strength of the 8mm diameter screw was the same as that of the Douglas fir and in the 6mm screw the difference had reduced from 18 to 12%. The combined results of both the old and additional tesst is shown in Table 4.6.

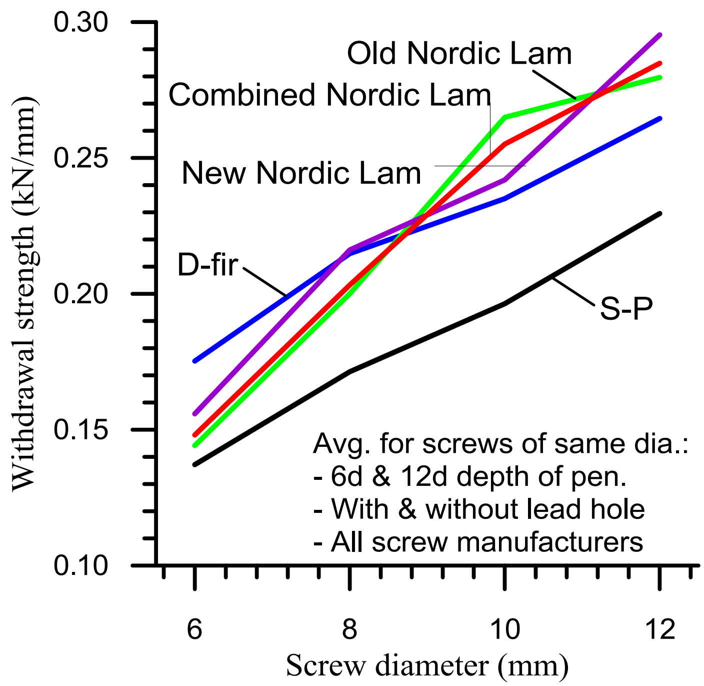



Figure 4-9 Effects of Glulam types (Density) on withdrawal strength

Table 4.6 Effects of Glulam types (Density) on withdrawal Strength

| **Wood Species** | **Calculated Density ($kg/m^3$)** | **Screw Diameter(mm) Withdrawal Strength per mm(kN/mm)** | | | |
|---|---|---|---|---|---|
| | | 6 | 8 | 10 | 12 |
| Spruce P | 455 | 0.137 | 0.171 | 0.196 | 0.230 |
| Douglas Fir | 520 | 0.175 | 0.215 | 0.235 | 0.265 |
| **Old Nordic L** | **538** | **0.144** | **0.200** | **0.265** | **0.280** |
| **New Nordic L** | **538** | **0.156** | **0.216** | **0.242** | **0.295** |
| **Combined Nordic L** | **538** | **0.148** | **0.203** | **0.255** | **0.285** |

Nordic L = Nordic Lam glulam

## 4.6 Effects of Screw Diameter on Withdrawal Resistance

During the course of testing, four different screw diameters were used; namely 6mm, 8mm, 10mm and 12 mm. This was to investigate the effect of screw size on withdrawal strength. As shown in Table 4.7, the average withdrawal strength per unit length was calculated for each screw diameter. Also Figure 4-10 shows a plot of the withdrawal strength vs. screw diameter. As clearly illustrated in the diagram, the withdrawal strength per unit length increases with an increase in the screw diameter. The increase in withdrawal strength is approximately linearly proportional to an increase in screw diameter. For instance between 6 mm and 8mm the withdrawal strength increment is 29%, 14% for between 8 and 10mm, and 17% for between 10mm and 12mm. Also evident is the fact that doubling the screw diameter from 6mm to 12 mm does not result in twice the withdrawal strength. From the test results the increment was 73%. From the graph 4-10 it can be seen that the standard deviations for the various screw diameters are low. This results in coefficients of variation ranging from 15 to 19%.

Table 4.7 Effects of diameter on withdrawal strength

| Diameter(mm) | Count (no) | Strength per mm(kN/mm) | % Strength Increment from 6mm | % Strength Increment from 8mm | % Strength Increment from 10mm |
|---|---|---|---|---|---|
| 6 | 277 | 0.153 | 0 | | |
| 8 | 774 | 0.197 | 29 | 0 | |
| 10 | 610 | 0.225 | 47 | 14 | 0 |
| 12 | 279 | 0.263 | 73 | 34 | 17 |

*Strength per mm of particular screw diameter tested including all glulam species

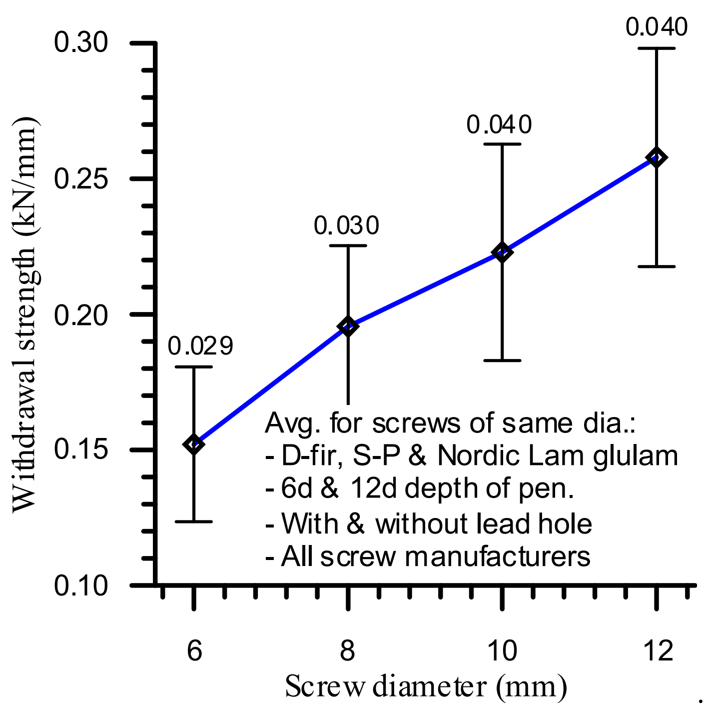



Figure 4-10 Effects of Screw Diameter on Withdrawal strength

## 4.7 Effects of Depth of Penetration on Strength

Two different depths of penetration were used, i.e. 6d (6 x diameter) and 12d (12 x diameter).The average withdrawal strength test values are shown in Table 4.8 and Figure 4-11. The withdrawal strength values increased with an increase in depth of penetration. Doubling the length of penetration results in approximately twice the initial withdrawal strength except for the 6mm diameter screws, for which an increase of only 90% was measured. Thus it can be concluded that the withdrawal strength is directly proportional to the length of penetration of the screw. From Figure 4-11 the plots for both depths of 6d and 12d were almost a straight line. From the results it was noticed that at the same depths of penetration, the larger diameter screws show increased withdrawal strengths. For instance in Table 4.7 at a depth of 72mm, a 6mm screw(i.e. slenderness ratio of 12) has a strength of 10.63kN while a 12mm screw (slenderness ratio of 6) has a strength of 18.21kN.The withdrawal strength difference of the two screws is about 71%. An investigation of the effects of slenderness ratio on withdrawal strength was not conducted; hence, further tests would have to be carried out to make a definite conclusion.

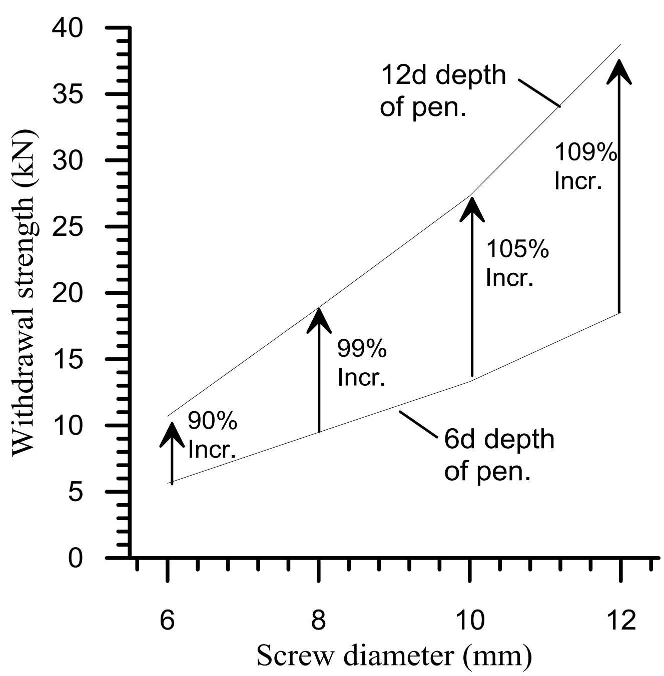



Figure 4-11 Effect of penetration length (6d and 12d) withdrawal strength (kN)

Table 4.8 Effects of depth of Penetration on withdrawal strength

| **Diameter (mm)** | **6** | **6** | **8** | **8** | **10** | **10** | **12** | **12** |
|---|---|---|---|---|---|---|---|---|
| **Length(mm)** | 36 | 72 | 48 | 96 | 60 | 120 | 72 | 144 |
| **Av. Strength(KN)** | 5.636 | 10.696 | 9.481 | 18.885 | 13.305 | 27.319 | 18.521 | 38.767 |
| **Count** | 140 | 137 | 389 | 385 | 302 | 308 | 139 | 140 |
| **% strength Incre. Due to Length** | | 90 | | 99 | | 105 | | 109 |

*Lengths measured to beginning of first thread of screw

** the average strength of all screws in that particular diameter, depth of penetration and involving all glulam types

## 4.8 Comparison of Measured Test Result to Predicted Values

### 4.8.1 General overview of predicted withdrawal strength to measured test results.

Several methods as discussed in the literature review were used to calculate predicted withdrawal resistances. Formulas from North America and Europe were considered. Current European and American design equations for screw withdrawal resistances are based on empirical models of the following form:

$$P_w = A\, d^B L_t^C\, G^D \qquad 4.1$$

where

$d$ is the fastener diameter,

$L_t$ is the depth of penetration of the threaded portion of the fastener,

$G$ is the wood density (in Europe) or specific gravity (in North America),

$A$ is a numerical parameter and the exponents B, C and D are listed in Table 4.9. These models have been developed through curve fitting of experimental data, and the numerical parameter $A$ can be used to adjust the average test values to the characteristic levels and duration of load.

Table 4.9 Model Parameters in Equation 4.1

| Reference | *B* | *C* | *D* |
|---|---|---|---|
| Eurocode 5 [2008] | 0.5 | 0.9 | 0.8 |
| DIN 1052 [2008] | 1.0 | 1.0 | 2.0 |
| NDS LS [2005] | 0.75 | 1.0 | 1.5 |
| NDS WS [2005] | 1.0 | 1.0 | 2.0 |
| McLain LS [1997a] | 0.61 | 1.0 | 1.35 |
| McLain WS [1997] | 0.82 | 1.0 | 1.77 |
| Frese& Blaβ [2009] | 1.0 | 1.0 | 1.0 |
| Pirnbacher & Schickhofer [2010] | 0.572 | 1.0 | 1.0 |

Currently in Canada there are proposals for the adoption of the approach for wood screws as a replacement for the existing lag screw provisions in the O86 Standard, where;

- The coefficient of variation of the data is established
- The equations are adjusted based on the coefficient of variation of the data and the load duration.
- Validation testing maybe required.

Using an assumption of a one sided tolerance limit for normal distribution, a k factor of 0.671 was chosen from Table 3 in ASTM D2915.The k-factor and the coefficient of variation was used to convert the measured average ultimate strength of the test specimens to a 5$^{th}$ percentile strength by this equation;

5$^{th}$ percentile factor = 1- (1.671 x COV)     4.2

Based on CSA O86 duration factors, conversion from short term to standard term load is by a factor of 0.92 x 1/1.15 = 0.8. The 0.92 factor which is used for nails and wood screws adjusts test duration to short term duration of load while 1/1.15 adjusts the short term duration of load to standard-term duration of load. These principles are applied to the predictions of the North American and European formulas used in this thesis. Table 4.10 shows a summary of the average ratios of the prediction to test value results. Pre/M is the average ratio of the predicted strength to the measured strength for all the tested specimens. The predicted strengths are calculated using the measured specific gravity (North American methods include CSA O86, NDS and McLain) or density (European methods include EC5, DIN1052, Frese & Blaß, Pirnbacher and Schickhofer) of the glulam. The specific gravities and densities are indicated in Table 4.2. The calculation of the predicted strength for the various methods is discussed in detail in the subsequent subsections. Pre/M COV is the coefficient of variation of the ratio of the predicted to measured results. From Table 4.10 the average ratio of the predicted to measured results of the McLain lag screw formula was the closest to 1.0 with a COV of 15.1%. The predicted values calculated using all the prediction formulas were adjusted with a factor of M/Pre, which is the average ratio of the measured to the predicted values to reflect the measured results. This ratio, M/Pre is the difference between the measured and the predicted values hence the total average ratio of these adjusted predicted values to the measured results is 1.0.The

5th percentile values are then calculated from each of the adjusted predicted values. The 5th percentile values were obtained by multiplying the adjusted predicted values by a 5th percentile factor obtained using Equation 4.2. For example applying Equation 4.2 to NDS lag screw the 5th percentile factor = 1-(1.647 x 0.147) = 0.754 as shown in Table 4.10. This factor is then multiplied to each of the adjusted predicted values and hence the ratio of the 5th percentile of the adjusted predicted value to the measured results is obtained as indicated in Table 4.10. The specified predicted values are also calculated and these are calculated using specific gravities ( D.fir 0.49, SP 0.44, and NL 0.47) and densities ( D.fir 520 kg/m$^3$, SP 470kg/m$^3$, NL 500 kg/m$^3$) provided in CSA O86. These specified predicted values were then adjusted to the test results and then 5th percentiles calculated as for the measured predicted values which used measured tested sample densities and specific gravities. From Table 4.10 involving NDS lag screws, the adjustment factor was 1.117 x 0.754 = 0.843. This value was multiplied to each of the specified predicted values and then the ratio of the 5th percentile specified predicted values to the measured test results is calculated.

Table 4.10 Summary of Average ratios of Predicted and Test results including correction factors

| Formula | Ratios | | | | Adjustment Factors | | |
|---|---|---|---|---|---|---|---|
| | Pre/M | Pre/M COV | 5 th PTL/M | 5th Spec/M | M/Pre | 5th Factor | 5th Spec Factor |
| CSA O86 LS | 0.404 | | | | | | 0.350 |
| NDS LS | 0.895 | 0.147 | 0.754 | 0.692 | 1.117 | 0.754 | 0.843 |
| NDS WS | 1.048 | 0.159 | 0.734 | 0.654 | 0.954 | 0.734 | 0.700 |
| McLain LS | 1.042 | 0.151 | 0.748 | 0.693 | 0.960 | 0.748 | 0.718 |
| McLain WS | 0.947 | 0.148 | 0.752 | 0.679 | 1.056 | 0.752 | 0.794 |
| EC5 | 0.700 | 0.159 | 0.735 | 0.726 | 1.428 | 0.735 | 1.050 |
| DIN 1054 | 0.893 | 0.168 | 0.720 | 0.694 | 1.120 | 0.720 | 0.806 |
| Frese & Blaß | 0.867 | 0.156 | 0.739 | 0.727 | 1.153 | 0.739 | 0.852 |
| Pinba* | 0.877 | 0.165 | 0.724 | 0.706 | 1.140 | 0.724 | 0.825 |

*Pirnbacher and Schickhofer

### 4.8.2 Predicted screw withdrawal strength using CSA O86

As stated earlier, CSA O86 has no provisions for structural screws but rather has separate withdrawal equations for lag screws and wood screws. In the calculation of the withdrawal resistance, since the glulam sections were dry, a $K_{SF}$ =1.0, for no treatment $K_T$ =1.0 and $J_E$ = 1.0 for side grain factor. For short term loading $K_D$ = 1.15. For lag screws changes are made to the loading condition .The material resistance, yw (Table 10.6.5.1 (CSA O86, 2009), is provided as a standard loading term value. Hence a $K_D$ =1.0 is associated with the predicted resistance, which must be divided by 0.8 (standard load term adjustment factor) to obtain a short term loading prediction resistance. Hence a factor of 1.25 was used. The values in Table 10.6.5.1 (CSA O86) are provided for various wood species (essentially specific gravity classification) and lag screw diameters. These dimensions are slightly different from the fasteners used for the test program on European structural screws. Hence the $y_w$ values were interpolated from the tabulated values to obtain an approximate withdrawal values for the various screws and glulam species. For Nordic Lam glulam (density = 560kg/m$^3$and for connection design = 470kg/m$^3$) is not included in the Table 10.6.5.1 (CSA O86), its $y_w$ values were assumed tobe the same as that of the Douglas fir glulam because they have similar specific gravity values. CSA O86 also has limitations on the depth of penetration that can be used for calculation purposes. For Douglas fir it is nine times the shank diameter and 11 times the shank diameter for Spruce pine and Northern species. As an example, for a 6mm screw in Douglas fir at a depth of 6d, the characteristic resistance is calculated using Equation 2.2.

$$P_{rw} \quad = \phi Y_W L_T n_F J_E$$

Characteristic resistance = 1.25 x 68 x (36-6) x 1 x 1/1000 = 2.550kN

The predicted characteristic resistance values for the various screw diameters, depth of penetration and density of wood were calculated. The ratio of the calculated predicted values to the test values were found. For lag screws in the O86 Standard a very conservative ratio of 0.4 with a COV of 35% (Table 4.10) were determined. The test results indicate that there is a need to replace the tabulated withdrawal values for lag screws in CSA O86 with an equation that provides improved accuracy. Figure 4.11 shows a plot of the predicted values using the CSA O86 lag screw method vs. the test values.

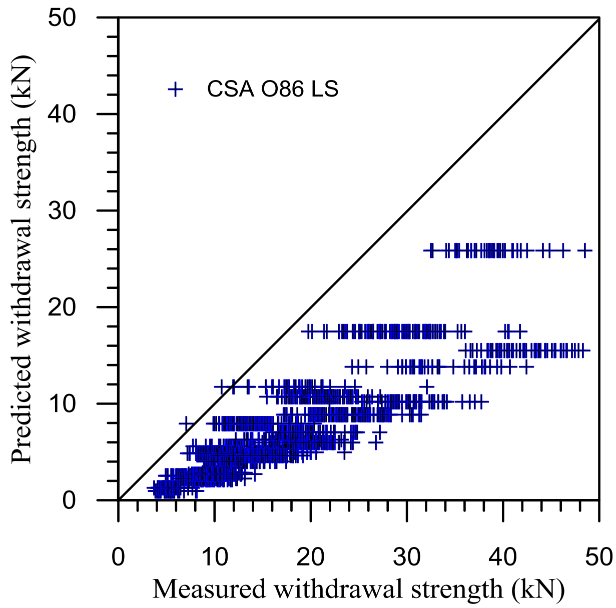



Figure 4 11 CSA O86 Prediction for lag screws vs. Test values

CSA-O86 also contains an equation for lag screws; the basic withdrawal resistance per unit length is calculated from a formula which is based on the density and diameter of the screw. The equation provides a short term loading resistance hence a $K_D$=1.15 is included. The loading factor is modified by multiplying it by 0.8 to obtain an equivalent standard duration load estimate of the resistance. The reciprocal of this, which is equal to 1.087, can then be used to modify the predicted resistance such that it can be compared with the measured test strengths. As an example, for 6mm diameter screw in Douglas fir of depth 6d, the characteristic resistance is calculated using Equation 2.6.

$68d_F^{k0.82}G^{1.77}, N/mm$

Characteristic resistance = 1.087 x 68 x $6^{0.82}$ x $0.49^{1.77}$ x 36/1000 = 3.271KN.

The average ratio of the predicted to the measured values was 0.6. Figure 4-12 shows the plot of the predicted to the measured values. The formula for wood screws in the CSA O86 was adopted from the McLain formula for wood screws (CSA O86 Meeting Handbook, 2012) hence more details is given under the Subsection involving McLain's formula.

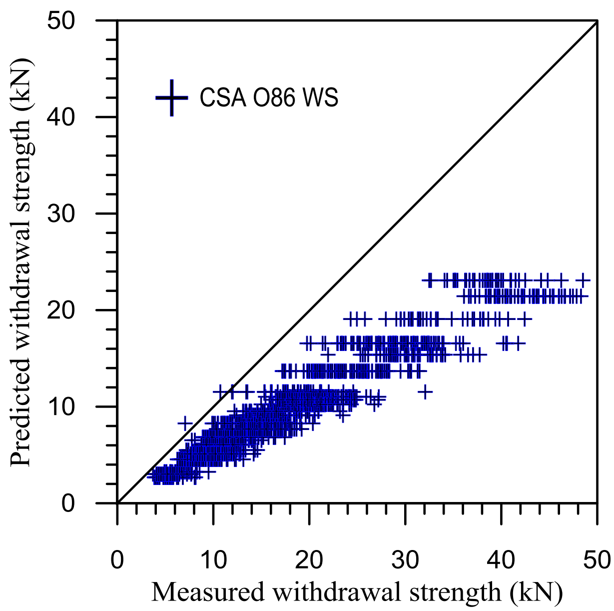



Figure 4-12 CSA O86 Prediction for wood screws vs. Test values

### 4.8.3 Predicted screw withdrawal strength using NDS

The NDS formulas for calculating the predicted withdrawal strength of lag screws are shown in equation 2.9.The constant A in the generic equation (Eq 4.1) is adjusted to account for load duration and also to calculate the predicted ultimate load. As an example a 6mm diameter lag screw in Douglas fir at the depth of 6d the predicted strength is;

$W \quad = 1800\ G^{3/2}\ D^{3/4}$

$W \text{ (in SI)} = 7500 \times 528^{1.5} 6^{0.75}((36\text{-}6)/25.4)^{1.75} \times (4.4482/1000) = 5.129\text{kN}.$

Figure 4.13 shows a graph of the NDS predicted values against measured test results. The average ratio of the predicted to the measured test results is 0.895 which is close to 1.0 considering the variability of wood. The coefficient of variation was 14.7% as indicated in Table 4.10. The co-efficient in front of the NDS LS equation in SI is

$A = 7500/25.4^{1.75} \times 4.4482 = 116$,

Then the general adjustment was made to the NDS LS coefficient by a factor of 1.117 (average test/predicted ratio) (Table 4.10) and an adjusted NDS LS predicted strength is obtained which is plotted in Figure 4-14. The adjusted predicted value for the 6mm example is;

$= 5.129 \times 1.117 = 5.731$.

$A = 1.117 \times 116 = 130$

As shown in the graph 4-14, the plotted points are evenly distributed over the boundary line. Figure 4.15 shows the plot of the 5$^{th}$ percentile values of the predicted strength values against the test results. The adjusted NDS LS coefficient was multiplied by Equation 4.2 to obtain 5$^{th}$ percentile strength values. Using the 6mm diameter lag screw example,

5$^{th}$ percentile predicted strength = $5.731 \times 0.754 = 4.324$KN.

$A = 0.754 \times 130 = 98$

Most of the plotted points fall just under the boundary line. Figure 4.16 shows the plot of the specified 5$^{th}$ percentile predicted strength vs. the test results. Most of the plotted points fall just under the boundary line. The values were calculated using the specified specific gravities in CSA O86 while the plot for the 5$^{th}$ percentile values in Figure 4.15 uses the measured mean specific gravity of the tested glulam. For a 6mm diameter 5$^{th}$ percentile predicted strength is;

$= 7500 \times 490^{1.5} 6^{0.75}((36\text{-}6)/25.4)^{1.75} \times (4.4482/1000) \times (1.117 \times 0.754) =$ 3.861kN.

5$^{th}$ percentile, $A = 7500/25.4^{1.75} \times 4.4482 \times (1.117 \times 0.754) = 98$.

From Table 4.10 the difference between the ratios of the 5$^{th}$ percentile predicted values to the test results and the ratio of specified predicted 5$^{th}$ percentile values is 8.95%.The difference between a predicted strength using the measured specific gravity and the specified CSA O86 specific was quite small, hence the specified specific gravities provides a true reflection of the glulam that in this case was used for the testing.

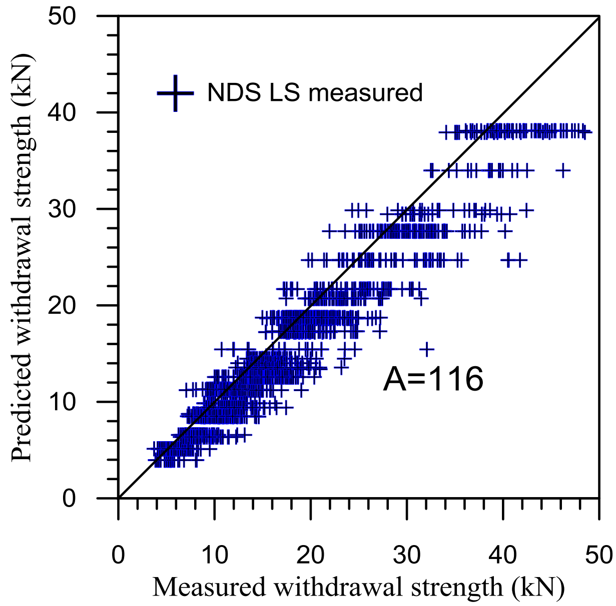



Figure 4-13 NDS LS Predicted withdrawal strength of lag screws vs. Test values

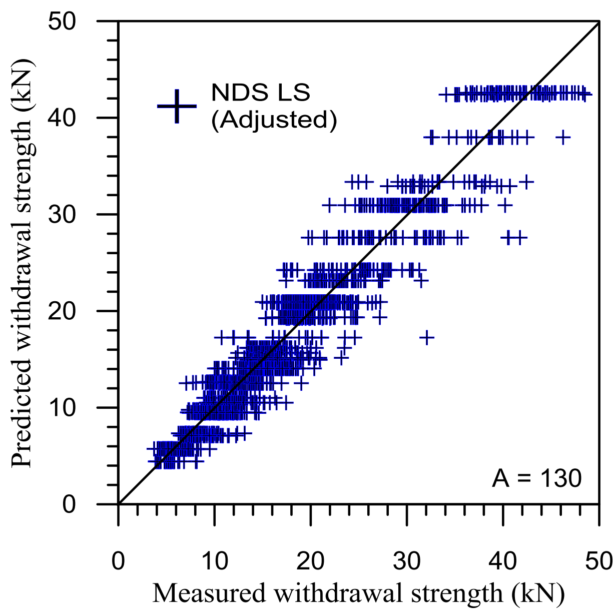



Figure 4-14 NDS LS Adjusted Predicted withdrawal strength of lag screws vs. Test values

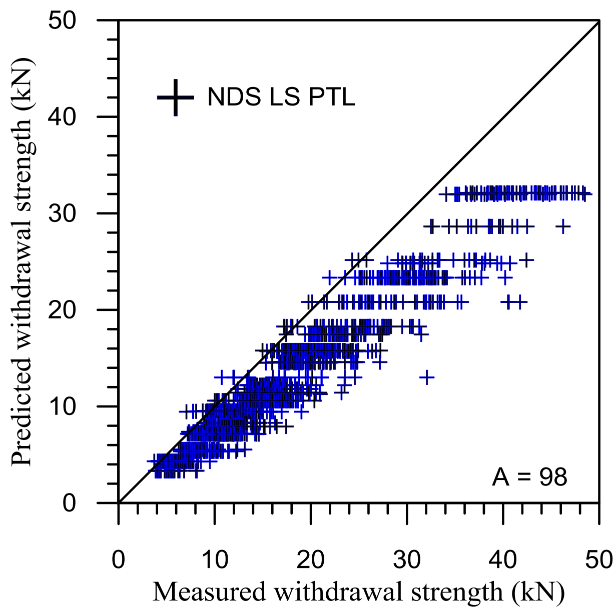



Figure 4-15 NDS LS 5th Percentile Predicted withdrawal strength of lag screws vs. Test values

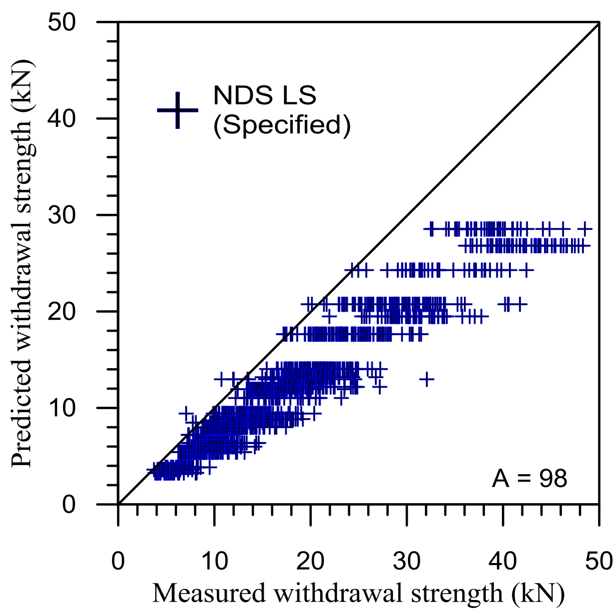



Figure 4-16 NDS LS Specified 5th Percentile Predicted withdrawal strength of lag screws vs. Test values

A similar approach was applied to the data analysis using the prediction of wood screws in the NDS. For a 6mm diameter wood screw in Douglas fir with a depth of 6d the predicted strength is calculated using Equation 2.10.

$$W = 2850G^2 D$$

$W$ (in SI) = 14250 x $528^2$ x 6 x $(36/25.4)^2$ x (4.4482/1000) = 5.926kN.

The average ratio of the predicted ultimate load to the observed test results was 1.048 with a coefficient of variation of 15.9%; Figure 4.17 shows a graph of the predicted values against the observed test results. The coefficient A in front of the NDS WS equation is;

$A = 14250/25.4^2 x 4.4482 = 98.$

Figure 4.18 shows the plot involving the adjusted NDS wood screws predicted strength values against the measured test values which was calculated in a similar fashion to that of lag screws. The plotted points are almost evenly distributed over the boundary line. For the 6mm diameter screw example adjusted predicted strength of the NDS wood screws is ;

W = 5.926 x 0.954 = 5.653kN

A = 98 x 0.954 = 94

Figure 4.19 shows the $5^{th}$ percentile of the predicted strength values plotted against the measured test results. Almost all the plotted points fell just below the ideal boundary line which has a ratio of 1.00. The average ratio was 0.734 with a COV of 15.9% which indicates reasonably conservative results. Using the 6mm diameter calculation example

W ($5^{th}$plt) = 5.653 x 0.734 = 4.15kN

A = 94 x 0.734 = 69

The specified $5^{th}$ percentile values (Figure 4.20) are quite similar to the plot of the $5^{th}$ percentile of the predicted strength. They had an average ratio of 0.654 and COV of 15.7%.The difference between the two $5^{th}$ percentile values was 12.2% which reasonably shows the variability of wood Using the 6mm screw diameter calculation example (in SI);

$W$ (5th ptl spec) = 14250 x 490$^2$ x 6 x (36/25.4)$^2$ x (4.4482/1000) x (0.954 x 0.734)
= 3.568kN

$A$ = 14250/25.4$^2$x 4.4482 x (0.954 x 0.734)

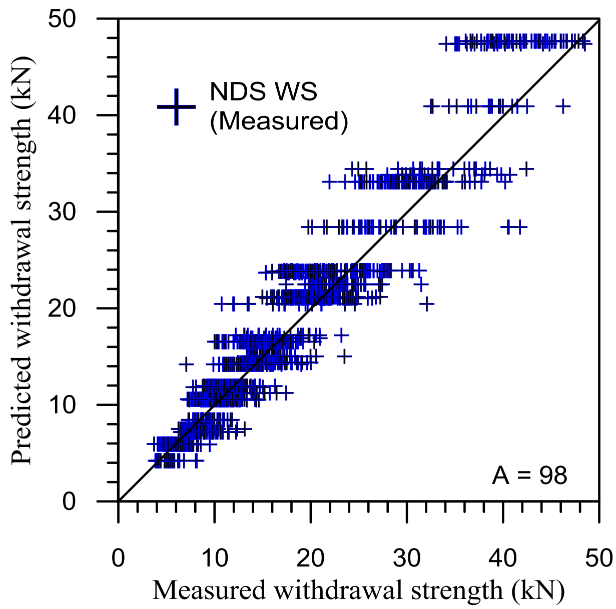



Figure 4-17 NDS WS Predicted withdrawal strength of wood screws vs. Test values

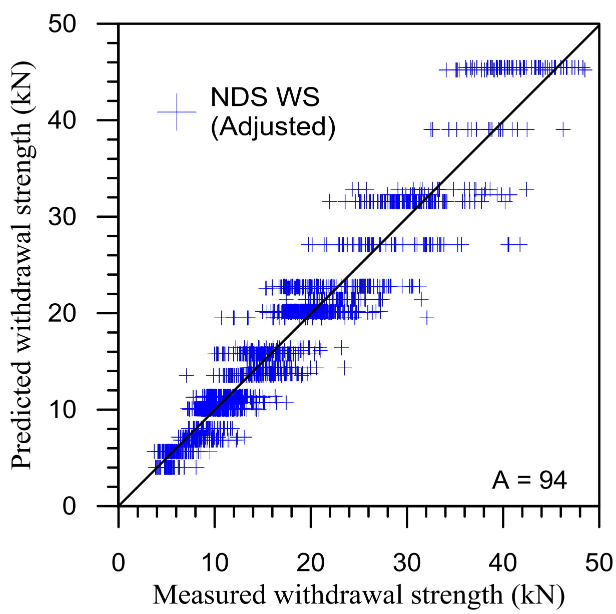



Figure 4-18 NDS WS Adjusted Predicted withdrawal strength of wood screws vs. Test values

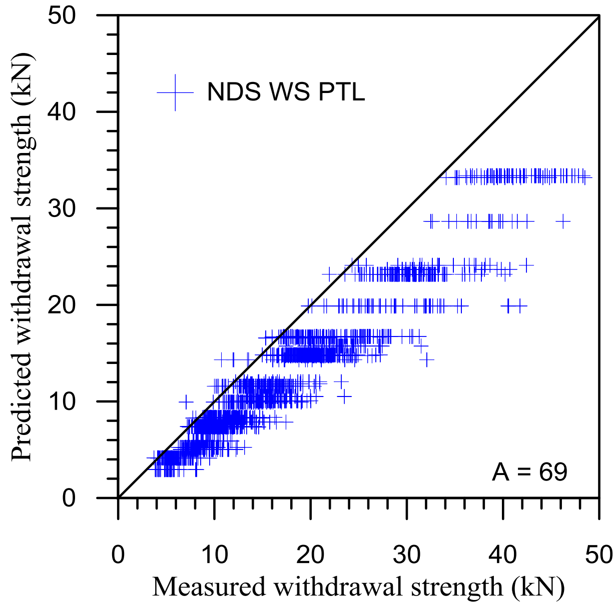



Figure 4-19 NDS WS 5th Percentile Predicted withdrawal strength of wood screws vs. Test values

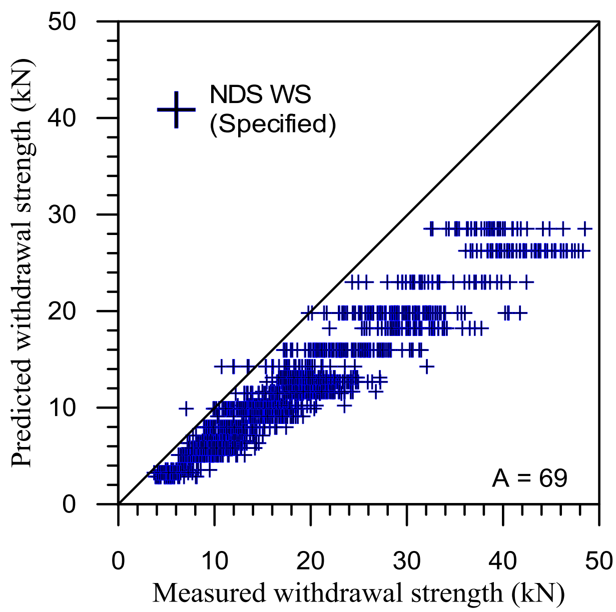



Figure 4-20 NDS WS Specified 5th Percentile Predicted withdrawal strength of wood screws vs. Test values

### 4.8.4 Predicted screw withdrawal strength using McLain

The McLain formula is similar to the NDS formulas since it is a modification of the latter. Lag screws strength predictions are calculated using Equation 2.11.

$W \quad = 1620\ G^{1.35} D^{0.61}$

As an example, using a 6mm diameter screw Douglas fir at a depth of 6d, the predicted strength is;

W (in SI) = $6759 \times 528^{1.35} \times 6^{0.61} \times ((36\text{-}6)/25.4)^{1.61} \times (4.4482/1000) = 6.225$kN.

The coefficient A = $6759/25.4^{1.61} \times 4.4482 = 165$

The average ratio of the predicted strength to the test results was 1.042. It had a coefficient of variation of 15.1% and shows the plotted points were quite close to the ideal boundary line (Figure 4.21). The plotted strength values are almost evenly distributed equally on both sides of the ideal boundary line for the adjusted predicted strengths (Figure 4.22). Using the 6mm diameter screw calculation example, the adjusted strength is;

W (Adjusted) = 6.225 x 0.96 = 5.976kN; A = 165 x 0.96 = 158

Figures 4.23-24 shows the $5^{th}$ percentile predicted values for both the predicted strengths and the specified predicted strengths. Most of the plotted points in these graphs fell just underneath the ideal line hence making them reasonably conservative. The average ratio of the $5^{th}$ percentile predicted strength values to the test results was 0.748 with a COV of 15.1%. Hence,

W ($5^{th}$ ptl) = 5.976 x 0.748 = 4.291kN

A = 158 x 0.748 = 118

For the specified $5^{th}$ percentile predicted values, the ratio with the test results was 0.693 with a COV of 15.4%.The difference between the two percentile values calculated was 7.9% which is lower than the other formulas used in the predictions. Specified predicted strength is;

W (in SI) = $6759 \times 490^{1.35} \times 6^{0.61} \times ((36-6)/25.4)^{1.61} \times (4.4482/1000) \times (0.96 \times 0.748) = 6.225$kN

A = 165 x (0.96 x 0.748) = 118.

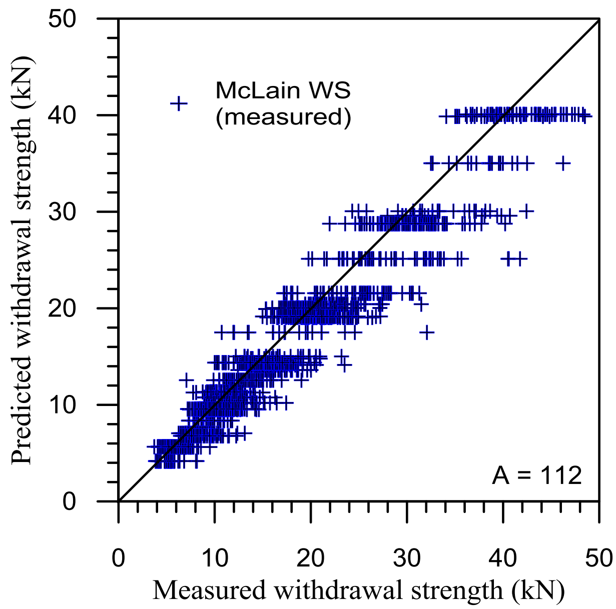



Figure 4-21 McLain LS Predicted withdrawal strength of lag screws vs. Test values

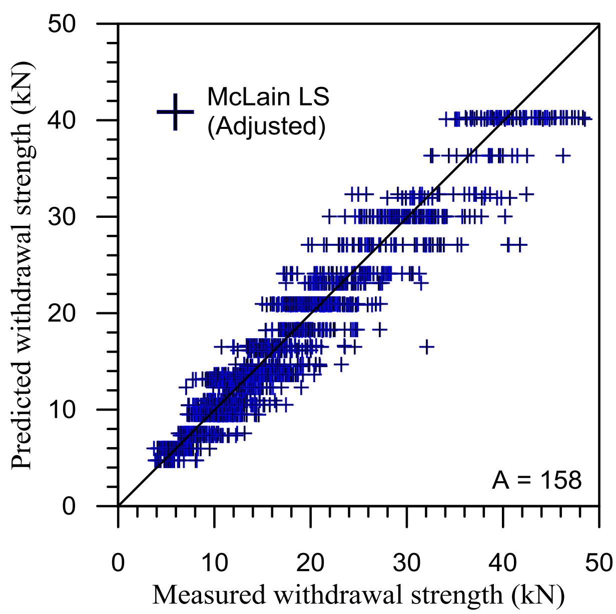



Figure 4-22 McLain LS Adjusted Predicted withdrawal strength of lag screws vs. Test values

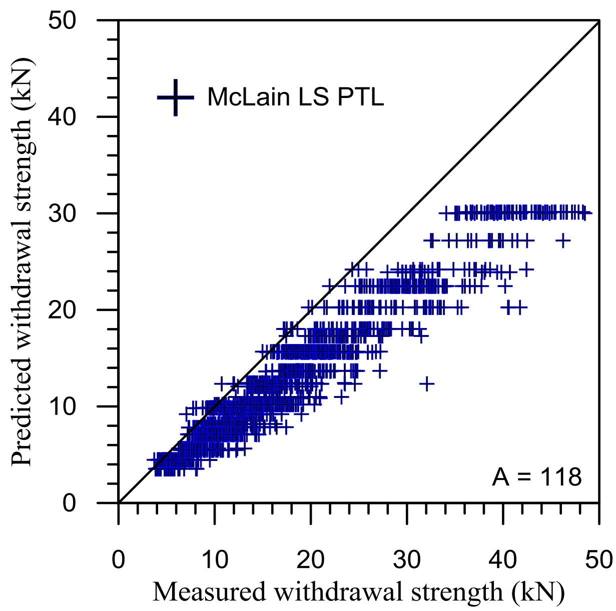



Figure 4-23 McLain LS 5th Percentile Predicted withdrawal strength of lag screws vs. Test values

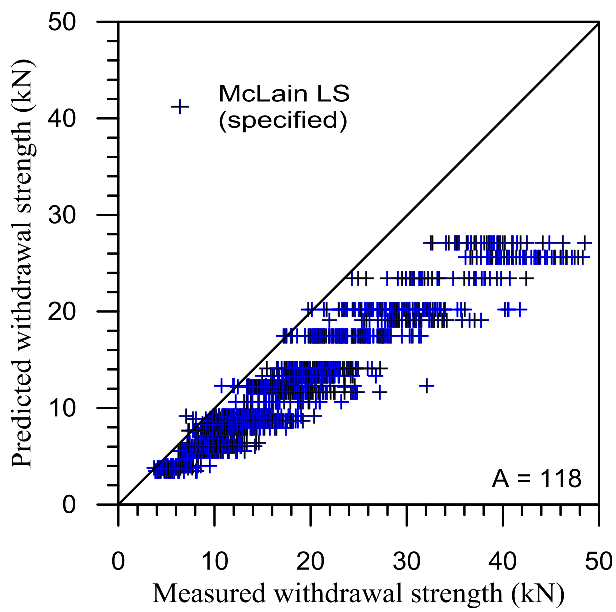



Figure 4-24 McLain Specified 5th Percentile Predicted withdrawal strength of lag screws vs. Test values

McLain's formula for the prediction of withdrawal strength of wood screws is the same as that in CSA O86. The calculation of the predicted strengths is similar to that of lag screws using Equation 2.12.

$$W = 1810G^{1.77}D^{0.82}$$

Using a 6mm diameter screw in Douglas fir with a depth of 6d, the predicted strength is;

W (in SI) = $9048 \times 528^{1.77} \times 6^{0.82} \times (36/25.4)^{1.82} \times (4.4482/1000) = 5.650$kN

A = $9048/25.4^{1.82} \times 4.4482 = 112$

The strength values obtained are plotted against the measured results (Figure 4.25). The majority of the plotted points are close to the boundary line. The average ratio of the predicted strength values to the test results was 0.947 with a COV of 14.8%. The adjusted predicted strength values are plotted in Figure 4.26. For the adjusted predicted strength using the 6mm diameter screw example and adjustment factor of 1.056 (Table 4.10);

W (adjusted) = 5.650 x 1.056 = 5.966kN

A = 112 x 1.056 = 118

The 5th percentile values were calculated and plotted against the test results (Figures 4.27-28). The average ratio of the predicted 5$^{th}$ percentile strength values against the observed test results was 0.752 with a COV of 14.8%. Using the 6mm diameter screw calculation example and adjustment factor of 0.752 (Table 4.10) the predicted 5$^{th}$ strength is;

W (5$^{th}$ ptl) = 5.966 x 0.752 = 4.489kN

Hence A = 118 x 0.752 = 89

The specified predicted 5$^{th}$ percentile strength values ratio to the test results is 0.679 with a coefficient of variation of 14.7%. The difference between the two 5$^{th}$ percentile strength values was 10.75%.The specified 5$^{th}$ percentile value is

W (in SI) = $9048 \times 490^{1.77} \times 6^{0.82} \times (36/25.4)^{1.82} \times (4.4482/1000) \times (1.056 \times 0.752) = 3.926$kN

A= 112 x (1.056 x 0.752)   = 89.

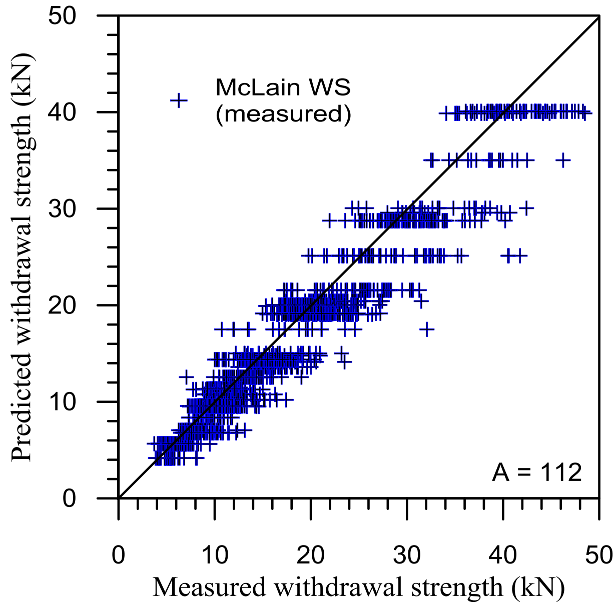



Figure 4-25  McLain WS Predicted withdrawal strengths of wood screws vs. Test values

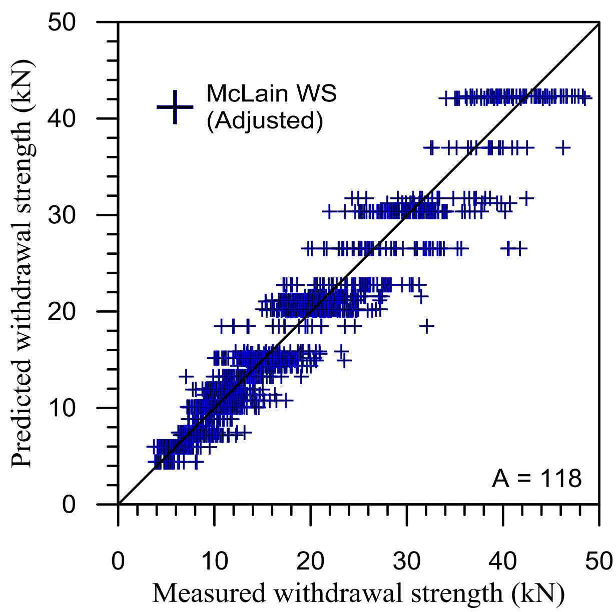



Figure 4-26  McLain WS Adjusted Predicted withdrawal strengths of wood screws vs. Test values

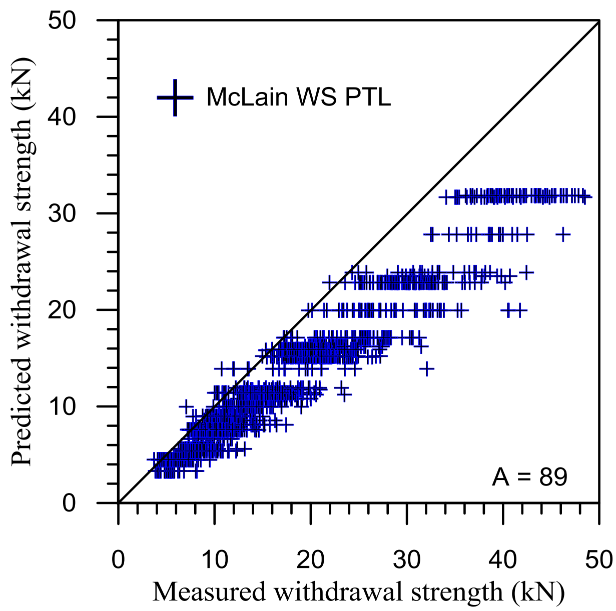



Figure 4-27 McLain WS 5th Percentile Predicted withdrawal strength of wood screws vs. Test values

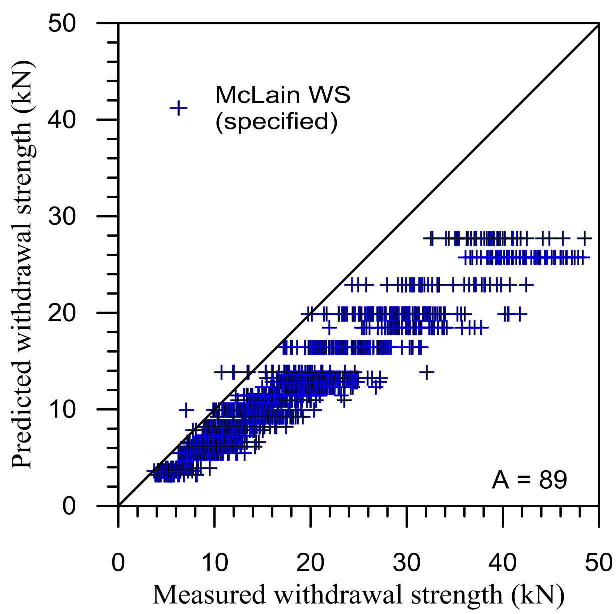



Figure 4-28 McLain WS Specified 5th Percentile Predicted withdrawal strength of wood screws vs. Test values

### 4.8.5 Predicted screw withdrawal strength using Eurocode 5

The Eurocode 5 2007 version is used in these predictions since it superseded the formulas in the 2004 edition. The strengths are predicted using Equation 2.18.

$$f_{ax,k} \quad = 0.52 d^{-0.5} l_{ef}^{-0.1} \rho_k^{0.8}$$

Hence for a 6mm diameter screw in Douglas fir at a depth of 6d, the predicted strength is;

$f_{ax,k}$ = 0.52 x $6^{0.5}$ x $36^{0.9}$ x $517^{0.8}$ x min (6/8, 1)/1000 = 3.559 kN.

The density used is the mean average density of the test samples of 517kg/m$^3$. All the other predicted strengths are calculated involving all the different parameters that is different; screw diameter, depth and density and a graph plotted against the test results (Fig 4.29). Almost all the plotted points were below the boundary with the higher loads obtained from the 12mm diameter screws at depths of 12d further away as compared to the other plotted load values (Fig 4.29). The average ratio of the predicted strength to the tested loads was 0.70 and the COV was 15.9%. Figure 4.30 shows the plot of adjusted values and the plotted points are very close to the boundary line and almost evenly divided on both sides .Using the 6mm diameter screw example and an adjustment factor of 1.428 (Table 4.10), the predicted strength is

$f_{ax,k}$(Adjusted) = 3.559 x 1.428 = 5.08 kN

A = 0.52 x 1.428 = 0.742

Figure 4.31 shows the plot of the 5$^{th}$ percentile strength values against the test results. The average ratio was 0.735 with a COV of 15.9%. Using the 6mm diameter screw example and an adjustment factor of 0.735 (Table 4.10), the 5$^{th}$ percentile predicted strength;

$f_{ax,k}$(5$^{th}$ ptl) = 5.08 x 0.735 = 3.734 kN

A = 0.742 X 0.735 = 0.545

The average ratio of the specified predicted 5$^{th}$ percentile strength to the test results was 0.726 with a COV of 16.7%. Figure 4.32 shows the plot of the specified 5$^{th}$ percentile values against the test results and in this case the density used was 520kg/m3.The difference between the two 5th percentile values was 1.24%.

$f_{ax,k}$(5th ptl spec) = 0.52 x $6^{0.5}$ x $36^{0.9}$ x $517^{0.8}$ x min (6/8, 1)/1000 x (1.428 x 0.735) = 3.735 kN; A= 0.545

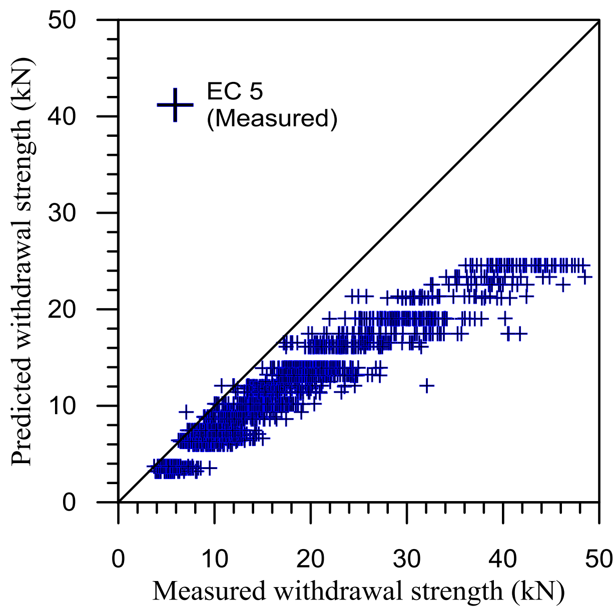



Figure 4-29 Eurocode 5 Predicted withdrawal strength of screws vs. Test values

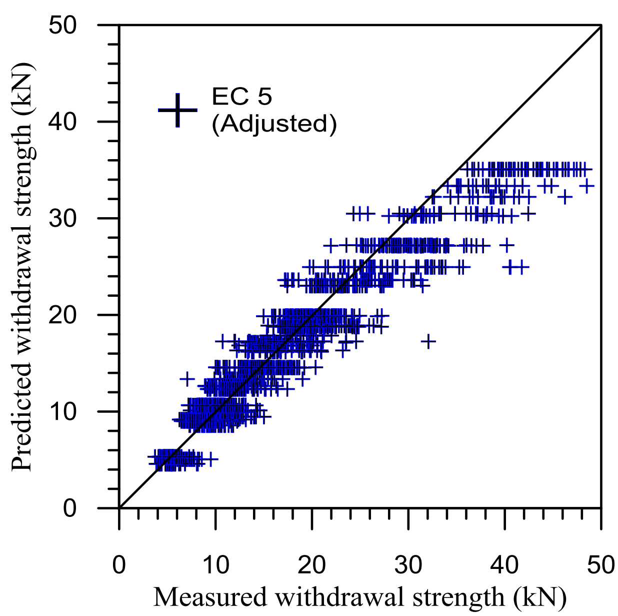



Figure 4-30 Eurocode 5 Adjusted Predicted withdrawal strength of screws vs. Test values

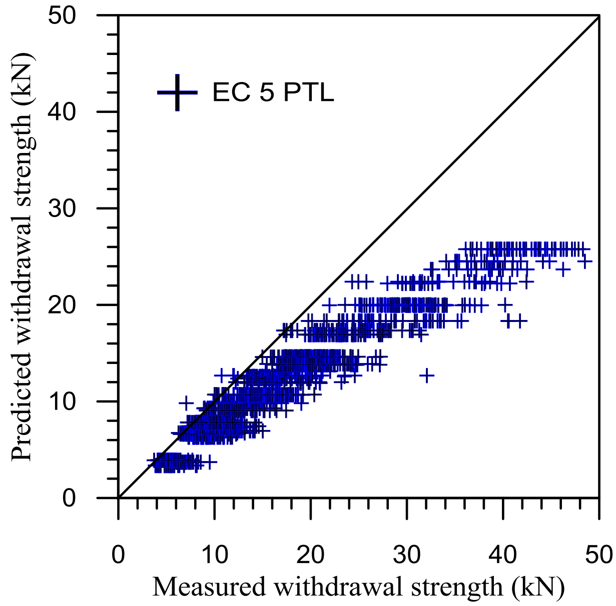



Figure 4-31 Eurocode 5 5th Percentile Predicted withdrawal strength of screws vs. Test values

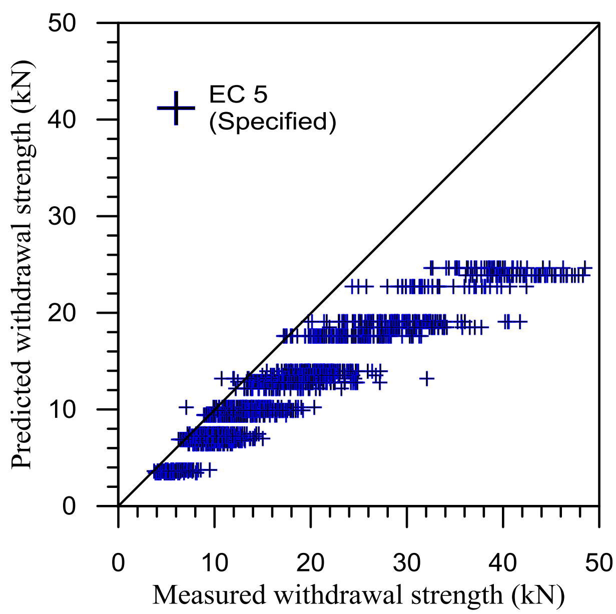



Figure 4-32 Eurocode 5 Specified 5th Percentile Prediction for screws vs. Test values

### 4.8.6 Predicted screw withdrawal strength using DIN 1052

The DIN 1052 formula for the calculation of the prediction strength is (Equation 2.21);

$$R_{axK} = min\left(\frac{f_{1.k}\,.\,d.\,l_{ef}}{sin^2\alpha + \frac{4}{3}.\,cos^2\,\alpha}; f_{2.k.}d_k^2\right)[N]$$

As an example, for a 6mm diameter screw in Douglas fir at a depth of 6d, the predicted strength is;

$R_{axK}$= 80 x $10^{-6}$ x $517^2$ x 6 x 36/1000 = 4.612kN.

A = 80 x $10^{-6}$

Figure 4.33 shows a plot of the predicted withdrawal strength against the test withdrawal strength values. Most of the plotted points were close to the boundary line. The average ratio of the predicted ultimate load to the test results was 0.893 with a COV of 16.8%. The predicted strengths are adjusted by a factor of 1.120 (Table 4.10) and a graph plotted against test results (Figure 4.34). Using the 6mm diameter screw example with adjustment factor of 1.120 (Table 4.10), the adjusted strength is;

$R_{axK}$= 4.612 x 1.120 = 5.165kN.

A = 80 X $10^{-6}$ x 1.120 = 89.6 x $10^{-6}$

The average ratio of the adjusted $5^{th}$ percentile values to the test results is 0.720 with a coefficient of variation of 16.9% (Figure 4.35). Almost all the points fell below the ideal line making them reasonable in the estimation of the $5^{th}$ percentile strength values. Using the 6mm diameter screw example with adjustment factor of 0.720 (Table 4.10), the $5^{th}$ percentile adjusted strength is; $R_{axK}$= 5.165 x 0.720 = 3.719 kN

A = 89.6 x $10^{-6}$x 0.720 = 65 x $10^{-6}$

The average ratio of the specified predicted $5^{th}$ percentile values to the test results was 0.694 and the COV was 15.7% (Figure 5.36). The density used was 520kg/$m^3$. The values fell below the ideal boundary line. The difference between the two $5^{th}$ percentile predictions was 3.75 %.

A = 65 x $10^{-6}$

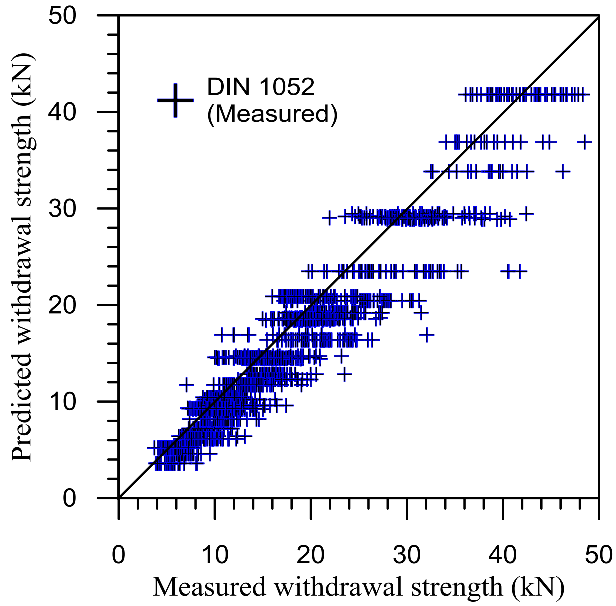



Figure 4-33 DIN 1052 Predicted withdrawal strength of screws vs. Test values

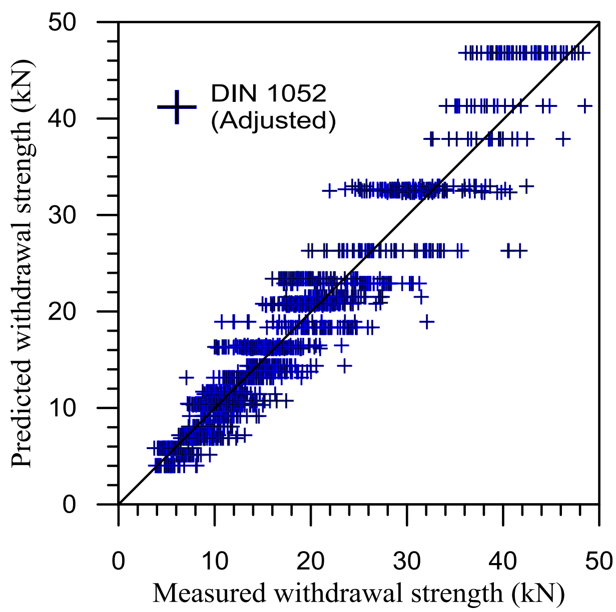



Figure 4-34 DIN 1052 Adjusted Predicted withdrawal strength of screws vs. Test values

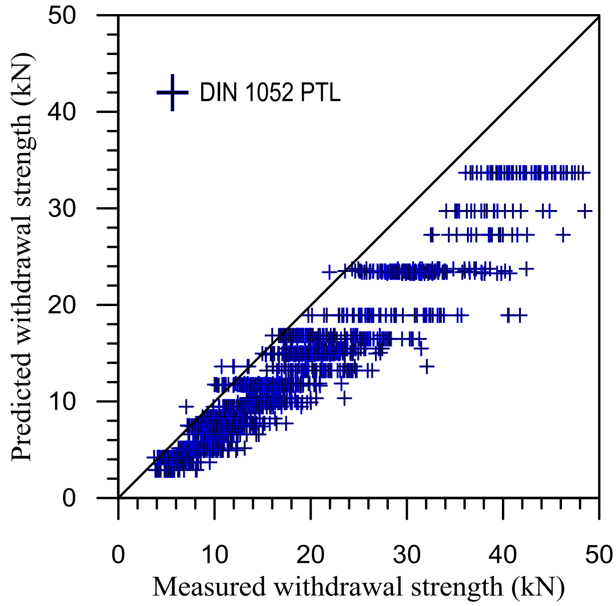



Figure 4-35DIN 1052 5th Percentile Predicted withdrawal strength of screws vs. Test values

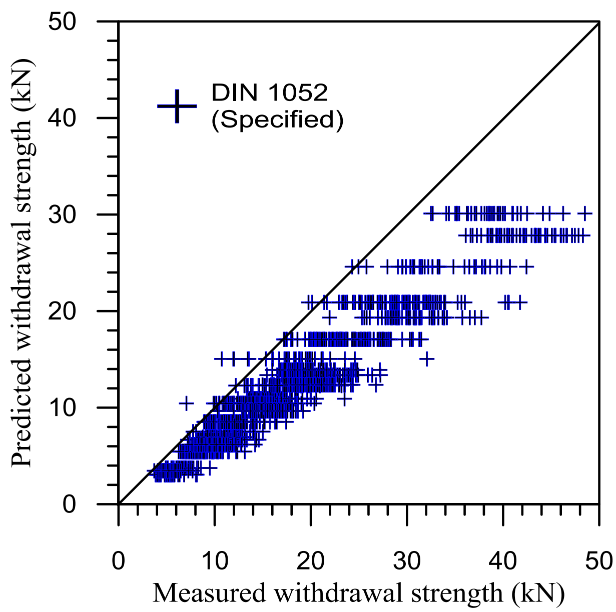



Figure 4-36DIN 1052 Specified 5th Percentile Predicted withdrawal strength of screws vs. Test values

### 4.8.7 Predicted screw withdrawal strength using Frese and Blaß formula

The formula proposed by Frese and Blaß (Equation 2.21) for predicting withdrawal strength is;

$$ln(R_{ax,k}) = 6.54 + l_{ef} . (0.03265 - 1.173 \cdot 10^{-4} \cdot l_{ef}) + 2.35 \cdot 10^{-4} \cdot d \cdot \rho_k$$

As an example, for a 6mm diameter screw in Douglas fir at a depth of 6d, Equation 2.22 is simplified hence the predicted strength is;

$R_{ax,k}$= 0.0857 x 517 x $6^{-0.3423}$ x 6 x 36 x (6/8)/1000 = 3.884 kN

The average density of 517 is used for the calculation. The predicted strengths against the test results are in Figure 4.37. The average ratio of the predicted ultimate loads to the tested results was 0.867 with a COV of 15.6%. The adjusted strength values are shown in Figure 4.38.The predicted values were adjusted using a factor of 1.153 (Table 4.10), hence the adjusted predicted strength for the 6mm diameter example is

$R_{ax,k}$(adjusted) = 3.884 x 1.153 = 4.478kN

The 5$^{th}$ percentile values of the adjusted predicted strengths were then calculated and plotted with the test results (Figure 4.39) .The average ratio of the predicted adjusted 5$^{th}$ percentile values to the test results was 0.739 with a COV of 15.6%. The plotted values all fell below the boundary line (Figure 4.39). Using the 6mm diameter screw in the Douglas fir calculation example, the 5$^{th}$ percentile of the adjusted strength is;

$R_{ax,k}$ (5$^{th}$ ptl) = 4.478 x 0.738 = 3.30kN

The specified 5$^{th}$ percentile strengths are then calculated using Equation 2.21 and a density of 520kg/m$^{3.}$ The obtained values are plotted against the test results (Figure 4.40). The average ratio of the specified predicted 5$^{th}$ percentile values to the test results was 0.727 with a COV of 16.6%. Using the 6mm diameter screw in Douglas fir calculation example, the specified 5th percentile strength is;

$R_{ax,k}$ = 0.0857 x 520 x $6^{-0.3423}$ x 6 x 36 x (6/8)/1000 x (1.153 x 0.738) = 3.330 kN

The difference between the two 5th percentile predicted values was 1.67%.

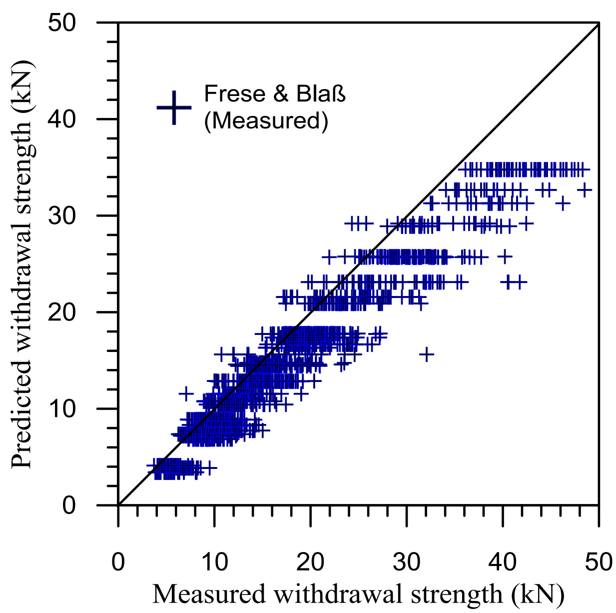



Figure 4-37 Frese & Blaß Predicted withdrawal strength of screws vs. Test values

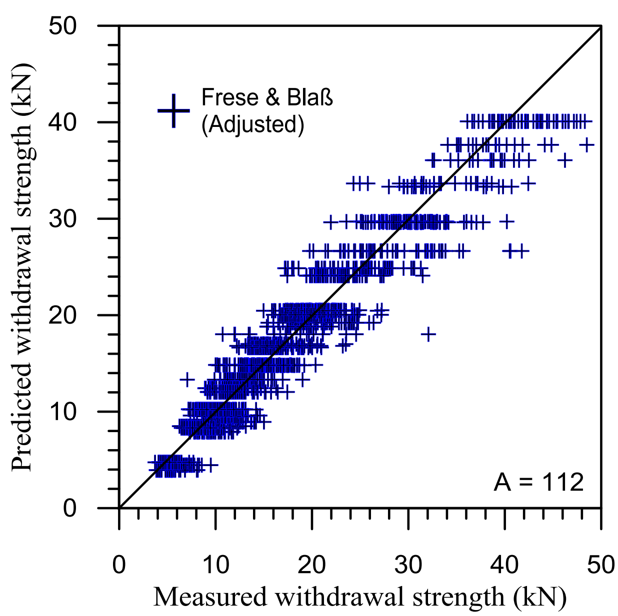



Figure 4-38 Frese & Blaß Adjusted Predicted withdrawal strength of screws vs. Test values

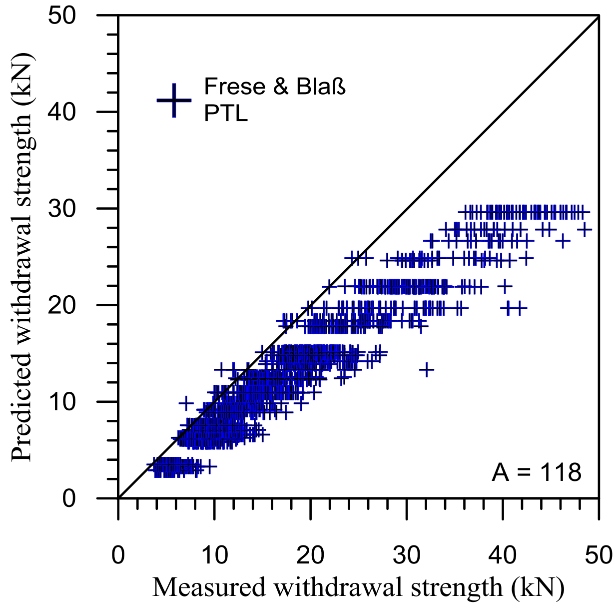



Figure 4-39 Frese & Blaß 5th Percentile Predicted withdrawal strength of screws vs. Test values

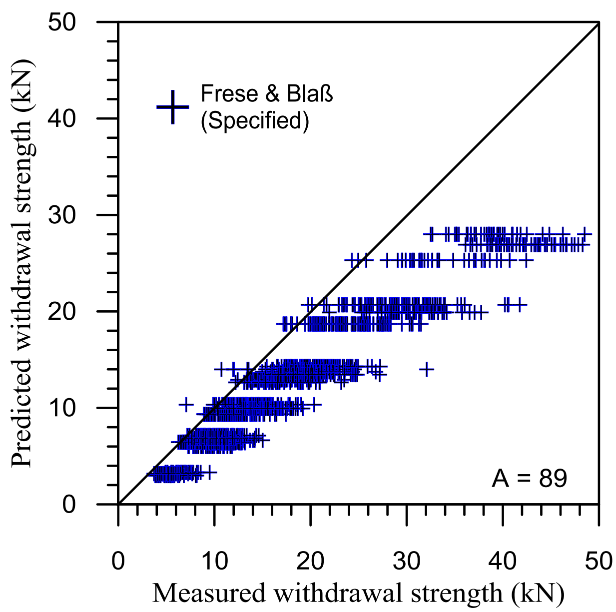



Figure 4-40 Frese & Blaß Specified 5th Percentile Predicted withdrawal strength of screws vs. Test values

### 4.8.8 Predicted screw withdrawal strength using Pirnbacher & Schickhofer formula

The Pirnbacher & Schickhofer formula for predicting withdrawal strength (Equation 2.23) is;

$f_{ax} = 0.0116.\rho_{test} - 0.272.(2.41d^{0.572}) + 1.97$

As an example for a 6mm diameter screw in Douglas fir at a depth of 6d, the predicted strength is ;

$f_{ax}$ = π x (0.0116 x 517 – 0.272 x 2.44 x $6^{0.572}$ + 1.97) x 6 x 36 x (6/8)/1000 = 3.111kN.

The average ratio of the predicted strengths to the tested results was 0.729 with a COV of 16.3% (Figure 4.41). The predicted strength values were adjusted with a factor of 1.140 (Table 4.10) to obtained the adjusted predicted strength values which were then plotted against the test results.(Figure 4.42).Using the 6mm diameter screw in Douglas fir calculation example, the adjusted strength is

$f_{ax}$ (adjusted) = 3.111 x 1.140 = 3.549.87 kN

The adjusted 5$^{th}$ percentile strength values were then calculated and plotted (Figure 4.43) .The average ratio of the predicted adjusted 5$^{th}$ percentile values to the test results is 0.728 with a COV of 16.3%. The plotted values all fell below the ideal line. Using the 6mm diameter screw in Douglas fir calculation example, the 5$^{th}$ percentile adjusted strength is;

$f_{ax}$ (5$^{th}$ ptl) = 3.550 x 0.724 = 2.57kN

Figure 4.44 shows a plot of the specified 5$^{th}$ percentile withdrawal values against the test results. A density of 520kg/m$^3$ is used and the average ratio of the specified predicted 5$^{th}$ percentile values to the test results was 0.716 with a COV of 17.3%. Using the 6mm diameter screw in Douglas fir calculation example, the specified 5$^{th}$ percentile adjusted strength is;

$f_{ax}$ (5$^{th}$ ptl spec.)= π x (0.0116 x 517 – 0.272 x 2.44 x $6^{0.572}$ + 1.97) x 6 x 36 x (6/8)/1000 x (1.140 x 0.724) = 3.124 kN

The difference between the two 5$^{th}$ percentile predicted values was 1.67%.

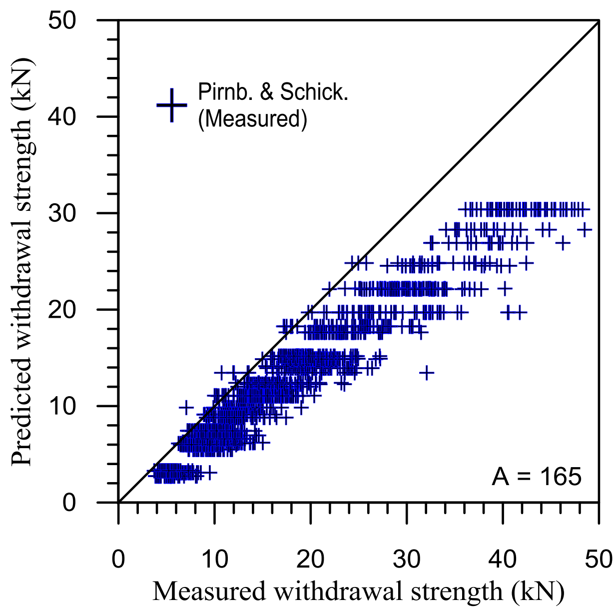



Figure 4-41 Pirnbacher & Schickhofer Predicted withdrawal strength of screws vs. Test values

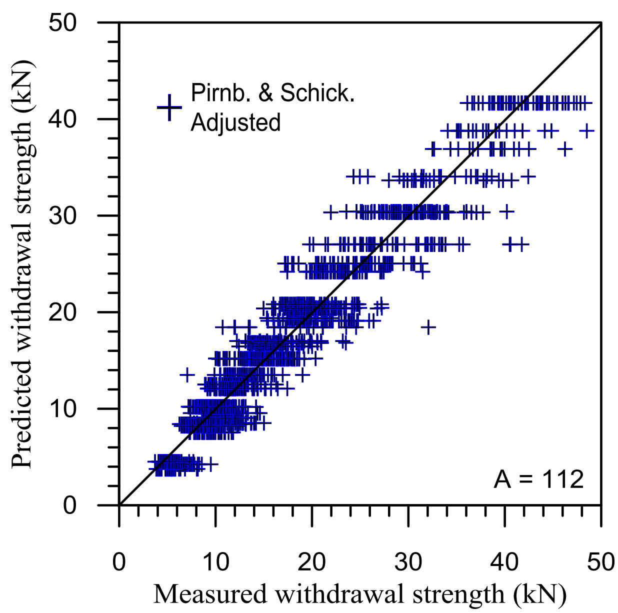



Figure 4-42 Pirnbacher & Schickhofer Adjusted Predicted withdrawal strength of screws vs. Test values

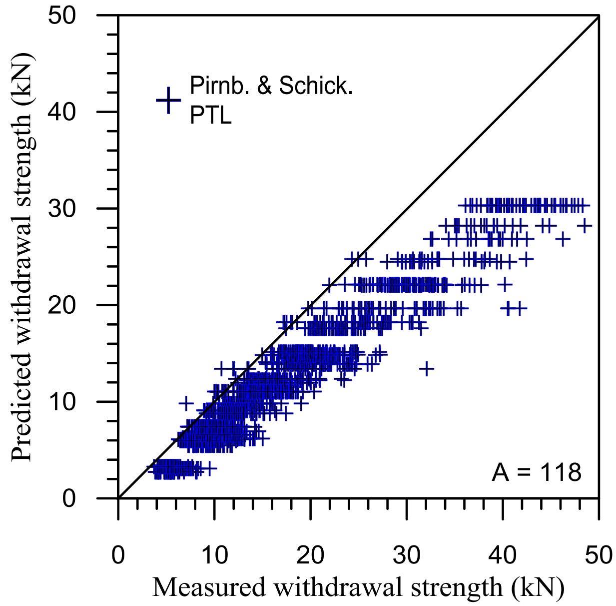



Figure 4-43 Pirnbacher & Schickhofer 5th Percentile Predicted withdrawal strength of screws vs. Test values

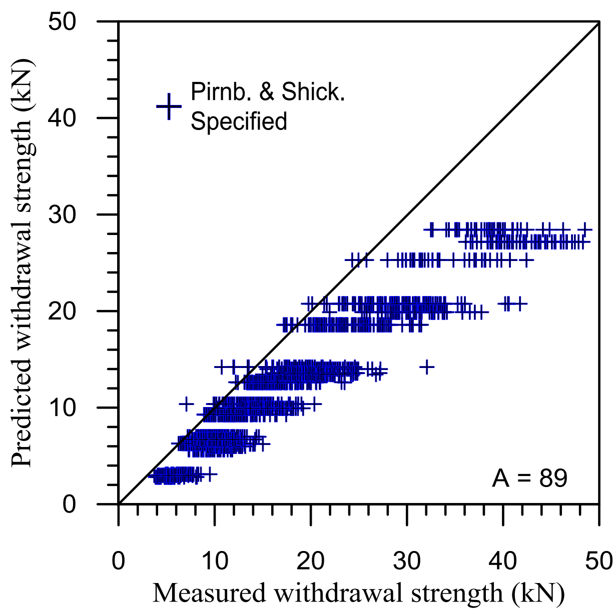



Figure 4-44 Pirnbacher & Schickhofer Spec5th Percentile Predicted withdrawal strength of  screws vs. Test values

## 4.9 Tensile Testing of Screws

All the screws types used for the withdrawal and cross screw connection test program were tested to determine their tensile resistance .This ancillary test was carried out to help understand the performance of the screws and also identify whether they were representative of what is available on the market for construction. A sample of three screws of each of the thirteen different screws was tested. The screws were randomly selected and tested in tension until failure. An MTS Sintech/30G universal testing machine with a load cell of 150kN, which was connected to a computer running MTS Testworks software was used to run the tensile tests. Since there is no standard tensile test procedure available, a simple test was used where the ends of the screws were held in wedge clamps. The head of the screw were cut off to allow for easier gripping. Figure 4-44 shows a picture of the test setup.

Figure 4-45 Picture of screw tensile test setup (Prat-Vincent 2011)

In all the screws tested, the average tensile loads (Table 4.11) were greater than the manufacturers listed screw strength. For instance for screw type 6B the manufacturer's tensile load is 11kN while a 14.4kN was obtained from the test program. In most cases the COV was less than 10% except for screw 8.2 x 220 where the COV was 13%.

Table 4.11 Ultimate Tensile strength of Screws

| Screw size (mm) | Screw type | Length (mm) | Ultimate Tensile load(kN) | | | COV |
|---|---|---|---|---|---|---|
| | | | Min | Max | Mean | |
| 6 | B | 100 | 13.7 | 15.3 | 14.4 | 5.8 |
| 6 | C | 120 | 13.6 | 14.2 | 13.9 | 2.4 |
| 8 | A | 220 | 30.9 | 32.4 | 31.5 | 2.4 |
| 8 | B | 200 | 27.9 | 29.4 | 28.7 | 2.6 |
| 8 | C | 220 | 23.1 | 23.7 | 23.5 | 1.2 |
| 10 | A | 260 | 38.5 | 40.6 | 39.7 | 2.8 |
| 10 | B | 320 | 34.2 | 36.2 | 35.4 | 3.0 |
| 10 | C | 200 | 39.0 | 39.3 | 39.2 | 0.4 |
| 12 | A | 240 | 48.1 | 50.2 | 49.5 | 2.4 |
| 12 | B | 380 | 51.5 | 53.1 | 52.3 | 2.2 |
| 8.2 | | 160 | 24.5 | 26.9 | 25.3 | 5.2 |
| 8.2 | | 220 | 25.1 | 32.7 | 29.1 | 13.0 |
| 8.2 | | 245 | 28.1 | 33.3 | 31.1 | 8.7 |

# 5 RESULTS AND DISCUSSIONS OF CROSS SCREW CONNECTION TESTS

## 5.1 Cross Screw Connections in White Pine Timber

Under this test program 14 test results were obtained, six of which were from configurations involving WT-T- 8.2 x 160 and WT-T-8.2 x 220 screws while two test results were completed for connections constructed with WT-T- 8.2 x 245mm screws. The different configurations had similar failure modes; the white pine cross screw connections failed due to the withdrawal of the screw ,which is characteristic of low density woods with shorter embedment length. The results of the test program are discussed in the subsequent subsections.

### 5.1.1 Inclined Screws

The assembled cross screw connection specimens were stored in the laboratory for five months after assembly before the commencement of testing to allow for the wood to dry. Weight measurements for the assemblages were taken for 12 weeks. Figure 5.1 shows a plot of the average weight against the time in weeks. As shown in the diagram the weight decreased with time indicating that the screwed assemblages were drying. On the day of assembly the average weight was 11.64kg and in about 4 months it decreased to 10.06kg.In the last two weeks prior to testing , the weight started to increase as seen in the graph due changes in the weather hence a decision was taken to begin testing. There was no drastic drop in the weight of the assemblages over the 5 month period because the measured moisture content of the white pine timber was about 15% at the time of assembly. All of the white pine timber had a moisture content of less than 10% (Table 5.1) at the time of testing. The average oven dry density was 351kg/m$^3$ and the characteristic value was 294kg/m$^3$, both with a corresponding coefficient of variation of 8%.These values are close to those stated in EN338:2003 for C14 class softwood (Porteous and Kermani 2007).During the course of drying, gaps developed at the joints between the joist and headers due to drying shrinkage as shown in Fig 5.2. In few cases cracks developed on the top surface of the joint along the embedded screws in the headers.

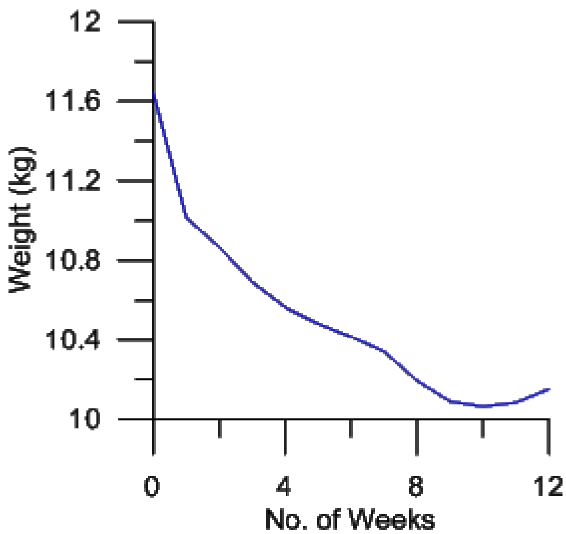



Figure 5-1 Drying Shrinkage of cross screw assemblages over 5 months

All the white pine test configurations failed in a similar manner, which is screw withdrawal. The capacity at each joint of the headers and joist is due to both the tension and compression screws. Each joint was made up of one tension and compression screw; however most of the connection resistance can be attributed to the tension screw (Prat-Vincent 2011) which eventually failed by withdrawal. The compression screw on the other hand resists shearing at the interface of the header and joist, which results in local wood failure as the screw is pushed upwards while the joist is pushed downwards. With increased loading, this leads to the bending of the screw at the connection shear plane as shown in Figure 5.3.

There was not much difference between the resistance measured for the test loaded using cyclic and monotonic protocols. Since only a small number of tests were conducted it was decided to combine the values for the two loading schemes. Earlier Prat-Vincent had also made the same conclusion for the monotonic and cyclic test data. Hence the average connection resistance values in Table 5.1 are a combination of the monotonic and cyclic loading tests. The force measured for each cross screw connection test was divided by two to determine the strength of an individual connection. For the WT-T-8.2mm x 160 the average strength per joint was 11.44 kN with a coefficient of variation of 21.60%. The strength increased to 19.05 kN per joint for connection tests with WT-T-8.2 x 145 mm screws with a coefficient of variation of 1.69. Note that, only two of WT-T-8.2 x 145 cross screw connection test was done. It can be seen (Table 5.1) that the withdrawal strength per connection increases with an increase in the depth of penetration. The 5$^{th}$ percentile values were still calculated for the cross screw connections involving the various screws even though the sample size was very small. The 5$^{th}$ percentiles were calculated using ASTM D 2915, Table 3 at a 75% confidence interval.

The test results were then compared with those of Prat-Vincent (2011) as shown in Table 5.2. Prat-Vincent's white pine timber was saturated (moisture content at 58%) at the time of testing while the white pine timber for this thesis was dry at the time of testing (moisture content < 10%). Two screw types, WT-T-8.2 x 160 and WT-T-8.2 x 220 were common to both the test programmes of Prat-Vincent and this thesis; hence there was a basis for comparison. Using these two aforementioned screws, the difference in the average strength per connection joint was 37% and 34% respectively. This difference in average strength illustrates the effect of moisture content on connection resistance.

Figure 5-2 Gaps at joints of the dried cross screw connection assemblage

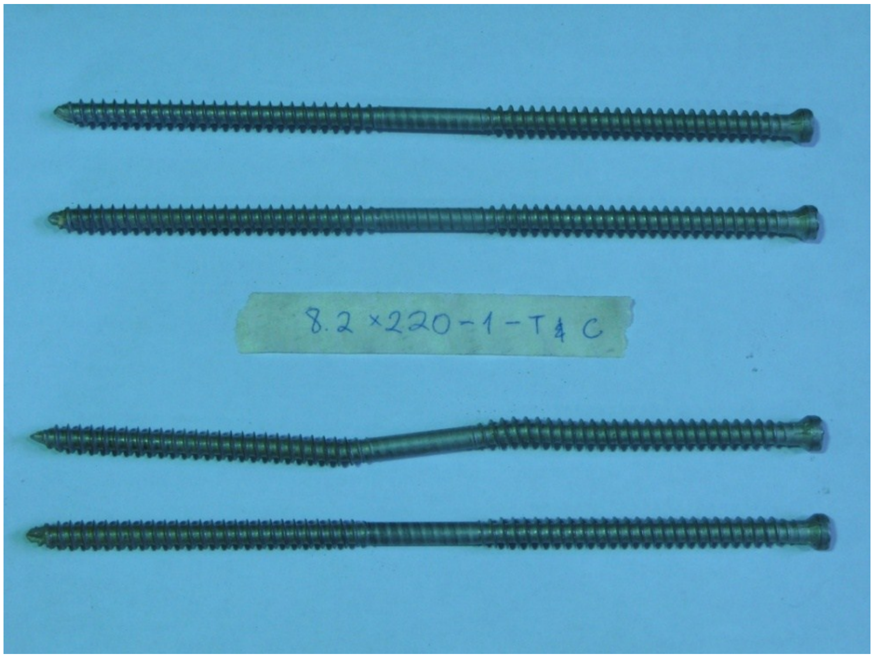



Figure 5-3Connection screws after testing a) Tension screws b) Compression screws

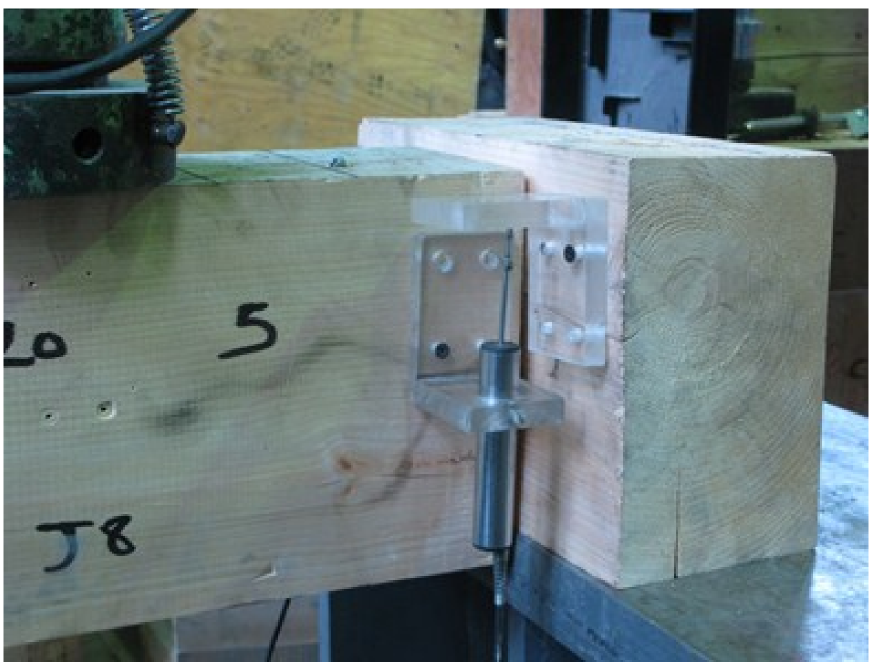



Figure 5-4 Cross screw connection Assemblage after testing

Table 5.1 Cross screw connection test results

| Config. | Fastener x pairs | No of specimen | Average MC % | Average Density (Kg/m$^3$) | Connection Resistance per Joint | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Min (kN) | Max(kN) | Mean(kN) | Standard dev (kN) | CoV % | Charact. (kN) |
| 1 | WT-T 8.2 x 160 x 1 | 6 | 9 | 349 | 9.18 | 14.86 | 11.44 | 4.94 | 21.60 | 5.67 |
| 2 | WT-T 8.2 x 220 x 1 | 6 | 8 | 356 | 15.02 | 19.14 | 16.85 | 3.19 | 9.48 | 13.12 |
| 3 | WT-T 8.2 x 245 x 1 | 2 | 6 | 347 | 18.82 | 19.27 | 19.05 | 0.65 | 1.69 | 13.94 |

## 5.2 Comparison with Predicted values

The methods used for the calculation of predicted connection strengths were presented in Section 2.4.The calculated characteristic strength values are shown in Table 5.2. Figure 5.5 shows the plotted test strengths with the calculated characteristic values (excluding Eurocode 5, 2004 formula).

In the CSA O86 (2009), the white pine belongs to the northern group. The loading condition is considered short term, hence $K_D$=1.15, $K_T$=1.0 and $J_E$=1.0 (screws were installed at an angle in both end and side grains). Finally, since the wood was seasoned by air drying $K_{SF}$=1.0. A standard term load adjustment factor of 0.8 was used to adjust the predicted strengths, hence the predicted strength was multiplied by 1.25(1/0.8). CSA O86 provides the basic withdrawal resistance $y_w$ for various screw diameters with their corresponding wood groupings. Since a diameter of 8.2mm was not available, yw value was obtained by interpolation. The predicted characteristic values were then calculated using Equation 2.2. The CSA O86 formula does not account for the angle of penetration of the screw; hence this was catered for in the thesis by dividing Equation 2.2 by $1.2\cos^2\alpha + \sin^2\alpha$ which is used for calculating Fax, α, Rk in Eurocode 5 as suggested by Prat-Vincent. Another limitation of the CSA O86 is the depth of penetration which is eleven times the screw diameter; the intent is to avoid non ductile screw failure.

The calculation of the characteristic withdrawal resistance using Eurocode 5 (2008) is as discussed in Subsection 2.3.4.

In addition to CSA O86 and Eurocode 5, the Kevarinmäki (2002) approach was used to predict the resistance of the cross screw connection. Prat-Vincent considered two approaches using the Kevarinmäki method. The first approach involved determining the average characteristic withdrawal resistance from tests based on EN 1382 (1999a).The WT-T-8.2-160 unpaired test was used since this was very close to EN 1382. The average resistance obtained was then divided by $\pi d l_{ef}$ which results in $f_{ax,45,k}$=3.55N/mm$^2$ which was used as the $f_{aik}$ for the Kevarinmäki method. The second approach involved calculating the characteristic withdrawal resistance using EC5 2008 and dividing it by $\pi d l_{ef}$ to obtain $f_{aik}$ for the

Kevarinmäki method. The basic difference between the two approaches is that the first approach was based on characteristic test results while the second approach was based on calculated predicted resistance using the Eurocode 5 2008 formula. The characteristic values using the Kevarinmäki method were calculated using Equation 2.25 provided in Section 2.4.2. The following observations were made from the test results and Figure 5.5;

- The CSA O86 formula underestimated the capacity of the cross screw connections.
- The Kevarinmäki method was close to the test values as seen in figure 5.5.The method using tested values in the calculations was much closer to the tested withdrawal strength results and also a little higher than those calculated using EC5.
- The connection strength of the dry assemblages was higher than the wet ones that were tested earlier by Prat-Vincent. The difference was about 35%, that is a factor of 1.35 which is similar to the value of 1.25 stated in literature (Pearson et al, 1962) and 1.39 (Mack, 1966).This shows the significant effects of moisture on wood connections.
- There was a slight difference in connection behaviour as there was much twisting of the dry white pine cross screw connections during testing as compared to the wet ones.
- As observed from the comparison between the test and predicted results the Kevarinmäki method comes very close in the estimation of the capacity of the cross screw joint, which also confirms the conclusions made by Prat-Vincent.

Table 5.2 Cross screw test results (Wet and Dry) with Predicted Values

| Fastener | Tested Resistance | | | | | | % Mean diff | Kevarinmaki | | Eurocode 5 | | CSA O86(kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Wet* | | | Dry | | | | EC5 Stren. (kN) | Test Stren. (kN) | 2006 (kN) | 2008 (kN) | |
| | Mean(kN) | Charac.(kN) | MC(%) | Mean(kN) | Charac(kN) | MC(%) | | | | | | |
| WT-T-8.2-160 | 8.35 | 6.97 | 58.00 | 11.44 | 5.67 | 9 | 37 | 8.40 | 8.80 | 4.80 | 5.50 | 4.50 |
| WT-T-8.2-220 | 12.60 | 10.50 | 58.00 | 16.85 | 13.12 | 8 | 34 | 12.00 | 13.20 | 6.80 | 8.10 | 7.00 |
| WT-T-8.2-245 | | | | 19.05 | 13.94 | 6 | | 13.40 | 14.90 | 7.60 | 9.10 | 7.25 |

*Prat-Vincent(2011)

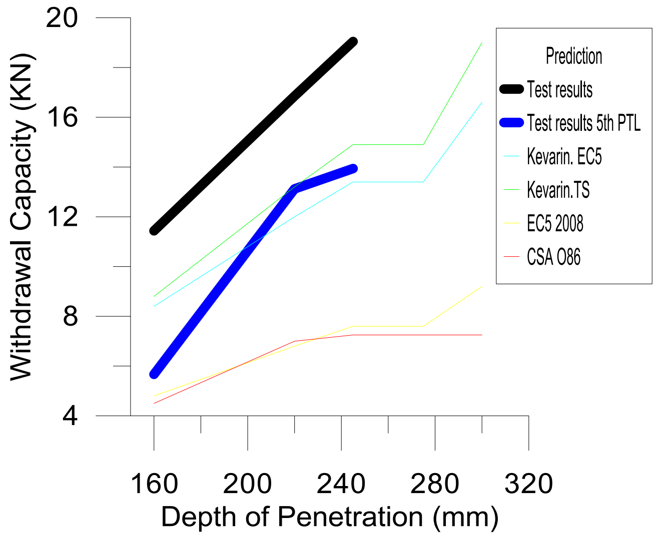



Figure 5-5 Plot of cross screw test results and Predicted resistances

# 6 CONCLUSION AND RECOMMENDATION

## 6.1 Conclusion

A series of 1960 withdrawal tests of European structural screw fasteners in Canadian glulam was carried out under the auspices of this research project. Even though the CSA O86 Standard (2009) can be used in the prediction of the characteristic values of these structural European screws, it has being shown from the test results that the lag screw provisions provided an especially conservative estimate of the withdrawal strength. This indicates there is a need to replace the existing lag screw method for the prediction of strength with a more rigorous model that predicts the withdrawal strength with reasonably accuracy. The formula for the withdrawal strength of wood screws, which is an adaptation of McLain's formulation (1997a), was introduced in the 2009 edition of CSA O86.

All the prediction methods discussed in this thesis: NDS lag and wood screw (2005), McLain lag and wood screw (1997a), Eurocode 5 (2008), DIN 1052 (2008), Frese and Blaß (2009) and Pirnbacher and Schickhofer (2010) were close to varying degrees to the measured withdrawal strengths of the test specimens. In contrast, the CSA O86 formulation for lag screws was very conservative. The accuracy of the predictions using these formulas is reasonable considering the variable nature of wood; hence any of the prediction formulas except the CSA O86 lag screw formulation can be used to make a reasonable estimate of the withdrawal strengths of these European structural screws in Canadian glulam.

In the analysis of the test data, the adjusted McLain formula for lag screws was the closest in the prediction of the measured test results. As stated earlier the derivation of these formulas is based on the general Equation 4.1. where the coefficient A is adjusted to fit the test data. Using the calculated A = 118 obtained from the specified 5$^{th}$ percentile withdrawal strength using McLain's lag screw formulation, the modified original formula ( Equation 2.11) becomes

$$P_w = 118\ d^{0.61} L_t G^{1.35} \quad \text{(in SI)} \qquad 6.1$$

From the test results the following observations were also made;

- Screws of the same diameter from different manufactures did not vary much in withdrawal strength
- The withdrawal strength increased with increase in screw diameter and this was approximately linearly proportional
- The withdrawal strength was linearly proportion to the depth of penetration(6d and 12d were tested)
- Doubling the depth of penetration almost resulted in twice the withdrawal strength
- The withdrawal strength generally increased with an increase in glulam density except the Nordic Lam glulam
- The orientation of the glulam sections had no effects on the withdrawal strength except the scatter of the withdrawal strength values.
- The use of lead holes (8 and 10mm diameter screws) did not have an effect on withdrawal strength. It did however improve the ease of installation for the larger screws.

In relation to the cross screw connection tests, the CSA O86 lag screw formula was conservative in the estimation of the capacity. The O86 formula is even handicapped as it does not account for the angle of penetration of the screw. The Kevarinmäki method either using the EC5 calculated axial withdrawal thread strength or from verified tabulated axial withdrawal test results using different fasteners and wood species was the closest in predicting the capacity of the tested cross screw connections. There was a difference in connection strength of 35% between the test results obtained using wet samples, tested earlier by Prat-Vincent (2011) compared to the dry samples tested described in this thesis. Additional research should be conducted on the cross screw connections involving different wood species to further verify the use of the Kevarinmäki formula.

The structural screws used are 'referred to as self-drilling and self-tapping' screws, meaning they do not require predrilling in most cases when being installed. The intent is for the screws to easily drill into the wood and in most cases to not require lead holes as compared to lag screws. As shown by the test data, self-drilling these European structural screws did not reduce the withdrawal strength of the screws as compared to traditional lag screws which have reduced capacities with larger diameters and denser wood. All the 6mm and 8mm diameter screws were installed with ease with or without lead holes. The 10mm diameter screws proved difficult to install when lead holes were not used, although all the self-drilled were successfully installed. The 12mm diameter screws could not be installed without lead holes; for a select number of specimens the screw head stripped even though a lead hole had been drilled to the full depth of penetration. This information could be very important when selecting these screws for use in construction as they have the potential to facilitate the efficient construction of heavy wood structures. However the EC5 limits the diameter of non-predrilled screws to 6mm and less for wood densities higher than 500kg/m$^3$.An evaluation of the influence of lead hole diameter in the dense glulams would be beneficial to provide guidance as to best practice construction

## 6.2 Recommendations for Future Research

The following are recommended for future research

- The test matrix was comprised of only three types of glulam (Douglas-fir Larch, Spruce pine and Nordic Lam); hence if a revised general withdrawal formulation were to be included in CSA O86 additional tests would have to be carried out using other glulam, engineered wood products and solid sawn wood to verify the applicability of the recommended equation.

- Other depths of penetration would have to be tested especially depths exceeding 12d, given that longer screws are available; to better understand the performance of these screws.

- Tests with screws placed at an angle to the grain would have to be carried out since all fasteners were installed perpendicular to the grain.
- It was found that the 12 mm diameter screws at a depth of 12d in dense wood had higher withdrawal strength than that predicted by any of the models. Additional study of the use of high density woods is suggested.
- Tests of the 6 mm through 12 mm screws in the Nordic Lam glulam proved to be inconsistent in terms of the increase in withdrawal strength with screw size even after an additional 200 tests had been carried out, whereas the Douglas fir and Spruce Pine glulam exhibited a near linear relationship between strength and fastener size. Additional testing of the European screws in Nordic Lam is advised to better understand this finding.

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# Appendix A Summary of Test Results

A1 Withdrawal Test Result of Douglas fir

| Specimen | Wood type | Moisture content | Cal. Av Density (kg/m3) | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|---|---|
| 06B-06-R | D-F | 5-10% | 517 | 6.40 | 0.98 | 0.15 |
| 06B-06-T | D-F | 5-10% | 517 | 7.12 | 0.74 | 0.10 |
| 06B-12-R | D-F | 5-10% | 517 | 11.98 | 1.12 | 0.09 |
| 06B-12-T | D-F | 5-10% | 517 | 12.86 | 1.03 | 0.08 |
| 06C-06-R | D-F | 5-10% | 517 | 6.16 | 1.07 | 0.17 |
| 06C-06-T | D-F | 5-10% | 517 | 6.97 | 1.10 | 0.16 |
| 06C-12-R | D-F | 5-10% | 517 | 10.35 | 1.84 | 0.18 |
| 06C-12-T | D-F | 5-10% | 517 | 12.32 | 1.22 | 0.10 |
| 08A-06-R | D-F | 5-10% | 517 | 10.00 | 1.55 | 0.16 |
| 08A-06-T | D-F | 5-10% | 517 | 10.84 | 0.96 | 0.09 |
| 08A-12-R | D-F | 5-10% | 517 | 21.45 | 3.66 | 0.17 |
| 08A-12-T | D-F | 5-10% | 517 | 21.50 | 2.73 | 0.13 |
| 08B-06-R | D-F | 5-10% | 517 | 14.60 | 1.60 | 0.11 |
| 08B-06-T | D-F | 5-10% | 517 | 14.34 | 1.18 | 0.10 |
| 08B-12-R | D-F | 5-10% | 517 | 19.57 | 1.37 | 0.07 |
| 08B-12-T | D-F | 5-10% | 517 | 21.00 | 1.64 | 0.08 |
| 08C-06-R | D-F | 5-10% | 517 | 9.35 | 1.19 | 0.13 |
| 08C-06-T | D-F | 5-10% | 517 | 10.26 | 0.93 | 0.09 |
| 08C-12-R | D-F | 5-10% | 517 | 20.35 | 2.41 | 0.12 |
| 08C-12-T | D-F | 5-10% | 517 | 20.25 | 2.41 | 0.12 |
| 12A-06-R-P | D-F | 5-10% | 517 | 18.34 | 2.02 | 0.11 |
| 12A-06-T-P | D-F | 5-10% | 517 | 19.12 | 1.74 | 0.09 |
| 12A-12-R-P | D-F | 5-10% | 517 | 38.32 | 4.31 | 0.11 |
| 12A-12-T-P | D-F | 5-10% | 517 | 39.39 | 3.22 | 0.08 |

Table A1 (*Continued*) Withdrawal Test Result of Douglas fir

| Specimen | Wood type | Moisture content | Cal. Av Density (kg/m3) | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|---|---|
| 08A-06-R-P | D-F | 5-10% | 551 | 9.44 | 1.23 | 0.13 |
| 08A-06-T-P | D-F | 5-10% | 551 | 10.57 | 1.56 | 0.15 |
| 08A-12-R-P | D-F | 5-10% | 551 | 21.37 | 1.48 | 0.67 |
| 08A-12-T-P | D-F | 5-10% | 551 | 20.82 | 1.32 | 0.06 |
| 08B-06-R-P | D-F | 5-10% | 551 | 10.07 | 1.08 | 0.11 |
| 08B-06-T-P | D-F | 5-10% | 551 | 10.95 | 1.21 | 0.11 |
| 08B-12-R-P | D-F | 5-10% | 551 | 20.32 | 1.86 | 0.09 |
| 08B-12-T-P | D-F | 5-10% | 551 | 21.01 | 1.28 | 0.06 |
| 08C-06-R-P | D-F | 5-10% | 551 | 9.15 | 1.03 | 0.11 |
| 08C-06-T-P | D-F | 5-10% | 551 | 9.70 | 1.07 | 0.11 |
| 08C-12-R-P | D-F | 5-10% | 551 | 20.40 | 2.65 | 0.13 |
| 08C-12-T-P | D-F | 5-10% | 551 | 20.12 | 1.62 | 0.08 |
| 10A-06-R-P | D-F | 5-10% | 551 | 12.54 | 1.12 | 0.09 |
| 10A-06-T-P | D-F | 5-10% | 551 | 15.23 | 1.28 | 0.08 |
| 10A-12-R-P | D-F | 5-10% | 551 | 28.26 | 2.37 | 0.08 |
| 10A-12-T-P | D-F | 5-10% | 551 | 30.07 | 1.60 | 0.05 |
| 10B-06-R-P | D-F | 5-10% | 551 | 11.83 | 1.26 | 0.11 |
| 10B-06-T-P | D-F | 5-10% | 551 | 14.22 | 1.58 | 0.11 |
| 10B-12-R-P | D-F | 5-10% | 551 | 28.74 | 3.10 | 0.11 |
| 10B-12-T-P | D-F | 5-10% | 551 | 29.13 | 2.25 | 0.08 |
| 10C-06-R-P | D-F | 5-10% | 551 | 14.15 | 1.37 | 0.10 |
| 10C-06-T-P | D-F | 5-10% | 551 | 15.02 | 0.59 | 0.04 |
| 10C-12-R-P | D-F | 5-10% | 551 | 29.69 | 2.19 | 0.07 |
| 10C-12-T-P | D-F | 5-10% | 551 | 31.14 | 2.39 | 0.08 |

Table A1 *(Continued)* Withdrawal Test Result of Douglas fir

| Specimen | Wood type | Moisture content | Av Density (kg/m3) | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|---|---|
| 10A-06-R | D-F | 5-10% | 495 | 12.96 | 2.58 | 0.20 |
| 10A-06-T | D-F | 5-10% | 495 | 12.87 | 1.55 | 0.12 |
| 10A-12-R | D-F | 5-10% | 495 | 28.86 | 5.12 | 0.18 |
| 10A-12-T | D-F | 5-10% | 495 | 29.80 | 3.97 | 0.13 |
| 10B-06-R | D-F | 5-10% | 495 | 12.90 | 1.98 | 0.15 |
| 10B-06-T | D-F | 5-10% | 495 | 12.67 | 2.36 | 0.19 |
| 10B-12-R | D-F | 5-10% | 495 | 24.91 | 5.11 | 0.21 |
| 10B-12-T | D-F | 5-10% | 495 | 26.66 | 4.06 | 0.15 |
| 10C-06-R | D-F | 5-10% | 495 | 13.78 | 1.56 | 0.11 |
| 10C-06-T | D-F | 5-10% | 495 | 13.03 | 1.74 | 0.20 |
| 10C-12-R | D-F | 5-10% | 495 | 31.04 | 6.09 | 0.10 |
| 10C-12-T | D-F | 5-10% | 495 | 29.68 | 2.96 | 0.33 |
| 12B-06-R-P | D-F | 5-10% | 495 | 18.05 | 6.51 | 0.20 |
| 12B-06-T-P | D-F | 5-10% | 495 | 16.91 | 3.40 | 0.13 |
| 12B-12-R-P | D-F | 5-10% | 495 | 40.49 | 5.29 | 0.13 |
| 12B-12-T-P | D-F | 5-10% | 495 | 38.69 | 4.97 | 0.05 |

Table A2 Test results of Spruce Pine

| Specimen | Wood type | Moisture content | Cal. Av Density (kg/m3) | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|---|---|
| 10A-06-R | S-P | 5-10% | 462 | 12.03 | 2.48 | 0.21 |
| 10A-06-T | S-P | 5-10% | 462 | 12.44 | 0.98 | 0.08 |
| 10A-12-R | S-P | 5-10% | 462 | 22.45 | 3.62 | 0.16 |
| 10A-12-T | S-P | 5-10% | 462 | 26.41 | 1.58 | 0.06 |
| 10B-06-R | S-P | 5-10% | 462 | 10.66 | 1.44 | 0.14 |
| 10B-06-T | S-P | 5-10% | 462 | 11.42 | 1.52 | 0.13 |
| 10B-12-R | S-P | 5-10% | 462 | 21.16 | 4.12 | 0.19 |
| 10B-12-T | S-P | 5-10% | 462 | 23.72 | 2.90 | 0.12 |
| 10C-06-R | S-P | 5-10% | 462 | 10.91 | 1.85 | 0.17 |
| 10C-06-T | S-P | 5-10% | 462 | 12.81 | 0.95 | 0.07 |
| 10C-12-R | S-P | 5-10% | 462 | 25.95 | 3.14 | 0.12 |
| 10C-12-T | S-P | 5-10% | 462 | 27.12 | 2.40 | 0.09 |
| 12B-06-R-P | S-P | 5-10% | 462 | 16.70 | 3.78 | 0.23 |
| 12B-06-T-P | S-P | 5-10% | 462 | 17.32 | 2.32 | 0.13 |
| 12B-12-R-P | S-P | 5-10% | 462 | 31.15 | 4.93 | 0.16 |
| 12B-12-T-P | S-P | 5-10% | 462 | 35.27 | 3.64 | 0.10 |

Table A2 *(Continued)* Test results of Spruce Pine

| Specimen | Wood type | Moisture content | Cal. Av Density (kg/m3) | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|---|---|
| 06B-06-R | S-P | 5-10% | 457 | 5.52 | 0.52 | 0.09 |
| 06B-06-T | S-P | 5-10% | 457 | 5.19 | 0.56 | 0.11 |
| 06B-12-R | S-P | 5-10% | 457 | 9.56 | 1.41 | 0.15 |
| 06B-12-T | S-P | 5-10% | 457 | 9.62 | 0.77 | 0.08 |
| 06C-06-R | S-P | 5-10% | 457 | 5.11 | 1.29 | 0.25 |
| 06C-06-T | S-P | 5-10% | 457 | 4.86 | 0.89 | 0.18 |
| 06C-12-R | S-P | 5-10% | 457 | 8.72 | 1.39 | 0.16 |
| 06C-12-T | S-P | 5-10% | 457 | 9.03 | 1.09 | 0.12 |
| 08A-06-R | S-P | 5-10% | 457 | 8.23 | 0.98 | 0.12 |
| 08A-06-T | S-P | 5-10% | 457 | 8.73 | 1.01 | 0.12 |
| 08A-12-R | S-P | 5-10% | 457 | 16.61 | 1.01 | 0.06 |
| 08A-12-T | S-P | 5-10% | 457 | 16.92 | 1.62 | 0.10 |
| 08B-06-R | S-P | 5-10% | 457 | 8.27 | 0.84 | 0.10 |
| 08B-06-T | S-P | 5-10% | 457 | 7.93 | 1.06 | 0.13 |
| 08B-12-R | S-P | 5-10% | 457 | 16.09 | 1.83 | 0.11 |
| 08B-12-T | S-P | 5-10% | 457 | 15.51 | 0.92 | 0.06 |
| 08C-06-R | S-P | 5-10% | 457 | 8.09 | 1.02 | 0.13 |
| 08C-06-T | S-P | 5-10% | 457 | 8.02 | 1.34 | 0.17 |
| 08C-12-R | S-P | 5-10% | 457 | 14.79 | 1.18 | 0.08 |
| 08C-12-T | S-P | 5-10% | 457 | 15.11 | 2.03 | 0.13 |
| 12A-06-R-P | S-P | 5-10% | 457 | 16.70 | 2.35 | 0.14 |
| 12A-06-T-P | S-P | 5-10% | 457 | 15.55 | 1.29 | 0.08 |
| 12A-12-R-P | S-P | 5-10% | 457 | 30.12 | 1.24 | 0.04 |
| 12A-12-T-P | S-P | 5-10% | 457 | 34.43 | 4.15 | 0.12 |

Table A2 *(Continued)* Test results of Spruce Pine

| Specimen | Wood type | Moisture content | Av Density (kg/m3) | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|---|---|
| 08A-06-R-P | S-P | 5-10% | 447 | 8.37 | 1.17 | 0.14 |
| 08A-06-T-P | S-P | 5-10% | 447 | 8.20 | 0.86 | 0.10 |
| 08A-12-R-P | S-P | 5-10% | 447 | 16.14 | 1.54 | 0.10 |
| 08A-12-T-P | S-P | 5-10% | 447 | 17.07 | 1.35 | 0.08 |
| 08B-06-R-P | S-P | 5-10% | 447 | 9.96 | 1.58 | 0.16 |
| 08B-06-T-P | S-P | 5-10% | 447 | 8.36 | 0.64 | 0.08 |
| 08B-12-R-P | S-P | 5-10% | 447 | 15.44 | 0.99 | 0.06 |
| 08B-12-T-P | S-P | 5-10% | 447 | 17.34 | 1.26 | 0.07 |
| 08C-06-R-P | S-P | 5-10% | 447 | 8.09 | 1.50 | 0.19 |
| 08C-06-T-P | S-P | 5-10% | 447 | 7.98 | 1.16 | 0.15 |
| 08C-12-R-P | S-P | 5-10% | 447 | 15.28 | 2.03 | 0.13 |
| 08C-12-T-P | S-P | 5-10% | 447 | 15.07 | 1.46 | 0.10 |
| 10A-06-R-P | S-P | 5-10% | 447 | 12.36 | 2.57 | 0.21 |
| 10A-06-T-P | S-P | 5-10% | 447 | 11.55 | 1.73 | 0.15 |
| 10A-12-R-P | S-P | 5-10% | 447 | 22.62 | 1.63 | 0.07 |
| 10A-12-T-P | S-P | 5-10% | 447 | 24.57 | 2.92 | 0.12 |
| 10B-06-R-P | S-P | 5-10% | 447 | 12.80 | 1.73 | 0.14 |
| 10B-06-T-P | S-P | 5-10% | 447 | 10.84 | 0.83 | 0.08 |
| 10B-12-R-P | S-P | 5-10% | 447 | 21.92 | 2.58 | 0.12 |
| 10B-12-T-P | S-P | 5-10% | 447 | 20.85 | 2.48 | 0.12 |
| 10C-06-R-P | S-P | 5-10% | 447 | 11.58 | 1.99 | 0.17 |
| 10C-06-T-P | S-P | 5-10% | 447 | 11.97 | 0.90 | 0.08 |
| 10C-12-R-P | S-P | 5-10% | 447 | 22.51 | 1.68 | 0.07 |
| 10C-12-T-P | S-P | 5-10% | 447 | 24.01 | 1.38 | 0.06 |

Table A3 Test result of Nordic Glulam

| Specimen | Wood type | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|
| 08A-06-R-P | NDGL | 9.64 | 1.03 | 0.11 |
| 08A-06-T-P | NDGL | 9.48 | 0.93 | 0.10 |
| 08A-12-R-P | NDGL | 18.19 | 0.61 | 0.03 |
| 08A-12-T-P | NDGL | 19.16 | 1.22 | 0.06 |
| 08B-06-R-P | NDGL | 9.20 | 0.56 | 0.06 |
| 08B-06-T-P | NDGL | 8.79 | 1.05 | 0.12 |
| 08B-12-R-P | NDGL | 18.28 | 1.76 | 0.10 |
| 08B-12-T-P | NDGL | 19.34 | 1.40 | 0.07 |
| 08C-06-R-P | NDGL | 9.06 | 0.62 | 0.07 |
| 08C-06-T-P | NDGL | 9.16 | 0.80 | 0.09 |
| 08C-12-R-P | NDGL | 19.87 | 2.04 | 0.10 |
| 08C-12-T-P | NDGL | 21.15 | 1.42 | 0.07 |
| 10A-06-R-P | NDGL | 16.23 | 1.45 | 0.09 |
| 10A-06-T-P | NDGL | 16.71 | 2.51 | 0.15 |
| 10A-12-R-P | NDGL | 31.66 | 2.70 | 0.09 |
| 10A-12-T-P | NDGL | 31.64 | 3.78 | 0.12 |
| 12A-06-R-P | NDGL | 19.43 | 2.54 | 0.13 |
| 12A-06-T-P | NDGL | 19.12 | 1.24 | 0.07 |
| 12A-12-R-P | NDGL | 40.16 | 2.83 | 0.07 |
| 12A-12-T-P | NDGL | 41.70 | 3.70 | 0.09 |
| 12B-06-R-P | NDGL | 19.33 | 1.46 | 0.08 |
| 12B-06-T-P | NDGL | 20.46 | 1.60 | 0.10 |
| 12B-12-R-P | NDGL | 42.02 | 3.33 | 0.08 |
| 12B-12-T-P | NDGL | 41.58 | 2.17 | 0.05 |

Table A3 *(Continued)* Test result of Nordic Glulam

| Specimen | Wood type | Tested Av Ult Load (10 test) | Standard Deviation | CoV |
|---|---|---|---|---|
| 06B-06-R | NLGL | 4.68 | 0.59 | 0.13 |
| 06B-06-T | NLGL | 5.03 | 1.05 | 0.21 |
| 06B-12-R | NLGL | 10.53 | 0.88 | 0.08 |
| 06B-12-T | NLGL | 10.87 | 0.99 | 0.09 |
| 06C-06-R | NLGL | 5.18 | 0.57 | 0.11 |
| 06C-06-T | NLGL | 5.05 | 0.59 | 0.12 |
| 06C-12-R | NLGL | 10.78 | 1.15 | 0.11 |
| 06C-12-T | NLGL | 10.97 | 0.92 | 0.08 |
| 08A-06-R | NLGL | 10.22 | 1.22 | 0.12 |
| 08A-06-T | NLGL | 9.66 | 1.05 | 0.11 |
| 08A-12-R | NLGL | 19.83 | 2.61 | 0.13 |
| 08A-12-T | NLGL | 21.21 | 1.86 | 0.09 |
| 08B-06-R | NLGL | 9.61 | 1.38 | 0.14 |
| 08B-06-T | NLGL | 9.92 | 0.79 | 0.08 |
| 08B-12-R | NLGL | 17.83 | 2.33 | 0.13 |
| 08B-12-T | NLGL | 19.38 | 1.29 | 0.07 |
| 08C-06-R | NLGL | 9.10 | 0.88 | 0.10 |
| 08C-06-T | NLGL | 10.00 | 1.20 | 0.12 |
| 08C-12-R | NLGL | 18.85 | 1.29 | 0.07 |
| 08C-12-T | NLGL | 20.28 | 1.32 | 0.07 |
| 10A-06-R | NLGL | 14.93 | 1.92 | 0.13 |
| 10A-06-T | NLGL | 16.18 | 1.08 | 0.07 |
| 10A-12-R | NLGL | 31.73 | 1.68 | 0.05 |
| 10A-12-T | NLGL | 31.21 | 2.09 | 0.07 |

Table A3 *(Continued)* Additional Test results (200) of Nordic Glulam

| Specimen | Wood Type | Tested Av Ult Load(10 test) | Standard Deviation | CoV |
|---|---|---|---|---|
| 06B-06-T | NDGL | 5.80 | 0.38 | 0.06 |
| 06C-06-T | NDGL | 5.54 | 0.72 | 0.13 |
| 06B-12-T | NDGL | 11.82 | 0.71 | 0.06 |
| 06C-12-T | NDGL | 10.38 | 0.95 | 0.09 |
| 08A-06-T | NDGL | 10.94 | 1.55 | 0.14 |
| 08B-06-T | NDGL | 9.80 | 1.04 | 0.11 |
| 08C-06-T | NDGL | 11.02 | 1.11 | 0.10 |
| 08A-12-T | NDGL | 21.58 | 1.20 | 0.06 |
| 08B-12-T | NDGL | 19.98 | 0.88 | 0.04 |
| 08C-12-T | NDGL | 19.42 | 1.06 | 0.05 |
| 10A-06-T | NDGL | 14.45 | 1.34 | 0.09 |
| 10B-06-T | NDGL | 14.19 | 2.03 | 0.14 |
| 10C-06-T | NDGL | 14.73 | 1.40 | 0.10 |
| 10A-12-T | NDGL | 28.45 | 2.80 | 0.10 |
| 10B-12-T | NDGL | 28.09 | 2.11 | 0.08 |
| 10C-12-T | NDGL | 30.91 | 1.96 | 0.06 |
| 12A-06-T | NDGL | 20.99 | 2.64 | 0.13 |
| 12B-06-T | NDGL | 19.82 | 3.30 | 0.17 |
| 12A-12-T | NDGL | 43.98 | 2.63 | 0.06 |
| 12B-12-T | NDGL | 44.51 | 3.07 | 0.07 |

# Appendix B Graphs of Withdrawal Test Results

**Douglas-fir**

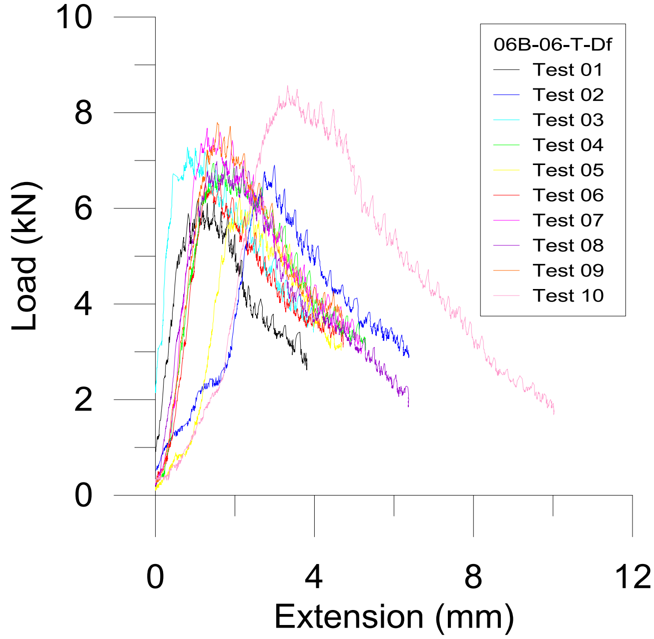


Figure B1 06B-06d-Df load vs. extension for top screws

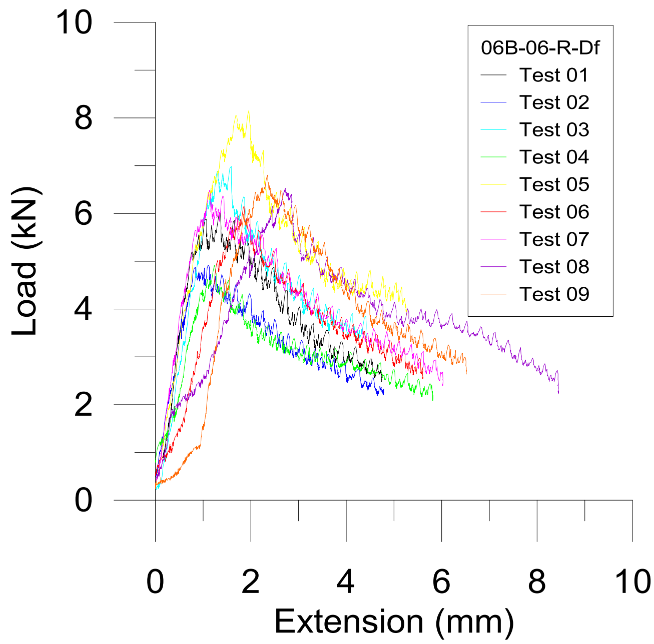


Figure B2 06B-06d-Df load vs. extension for side screws

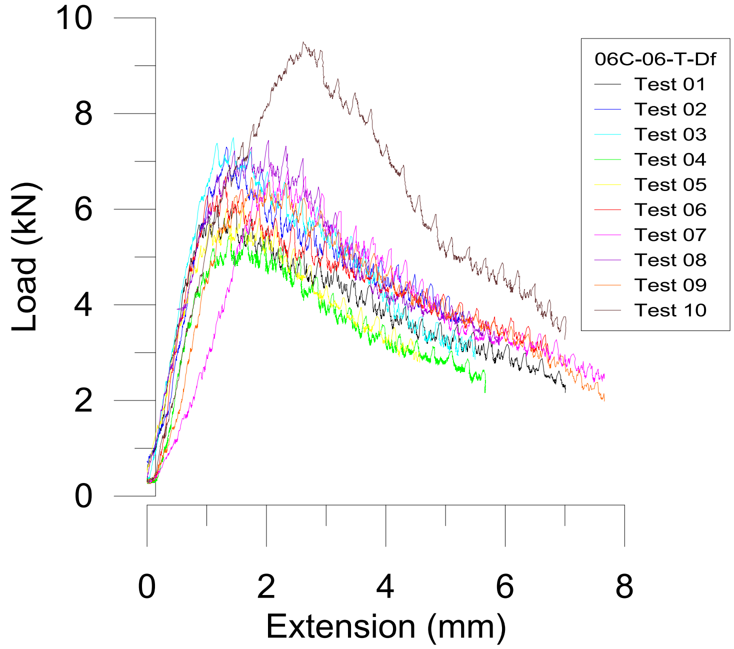


Figure B3 06C-06d-Df load vs. extension for top screws

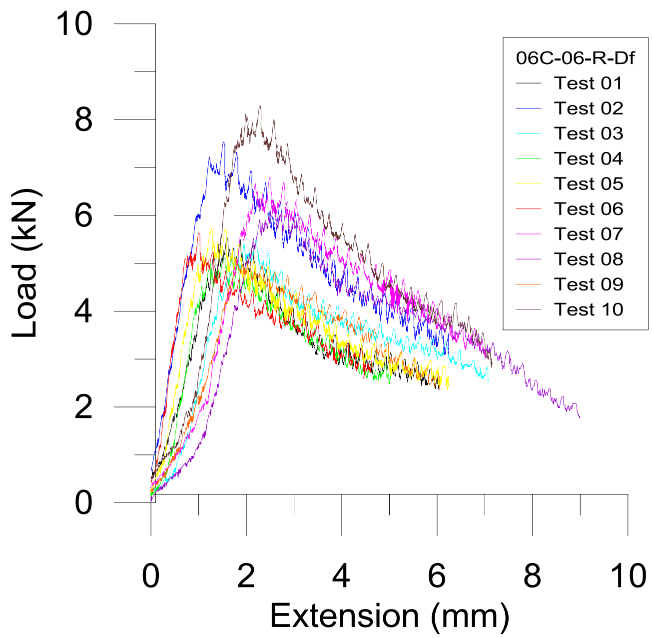


Figure B4 06C-06d-Df load vs. extension for side screws

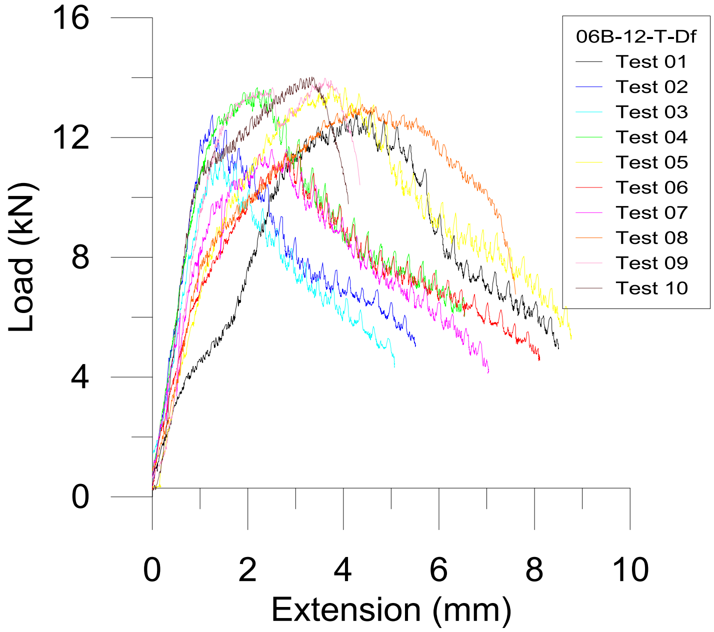


Figure B5 06B-12d-Df load vs. extension for top screws

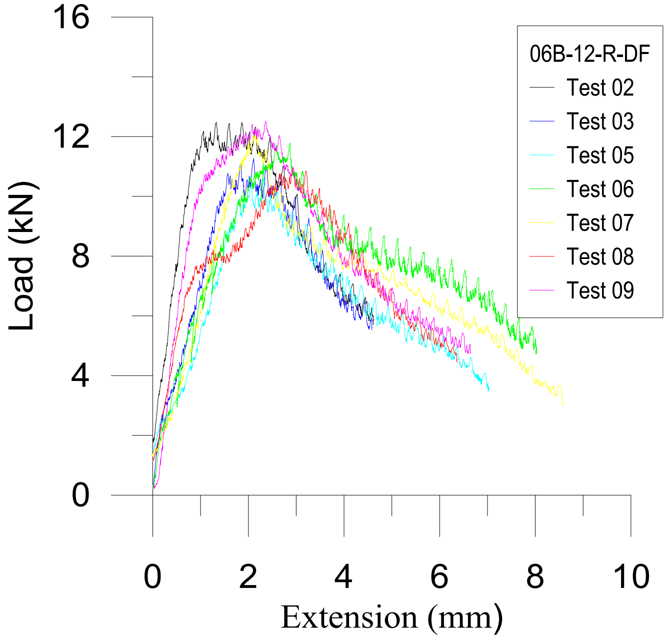


Figure B6 06B-12d-Df load vs. extension for side screws

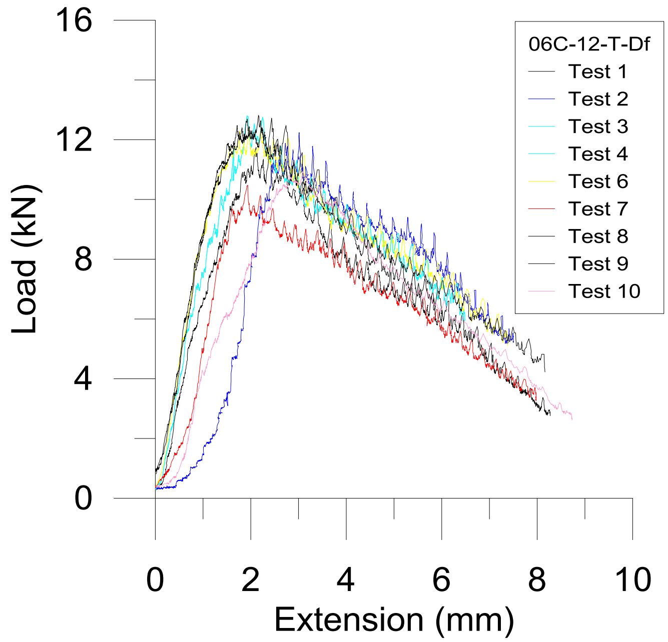


Figure B7 06C-12d-Df load vs. extension for top screws

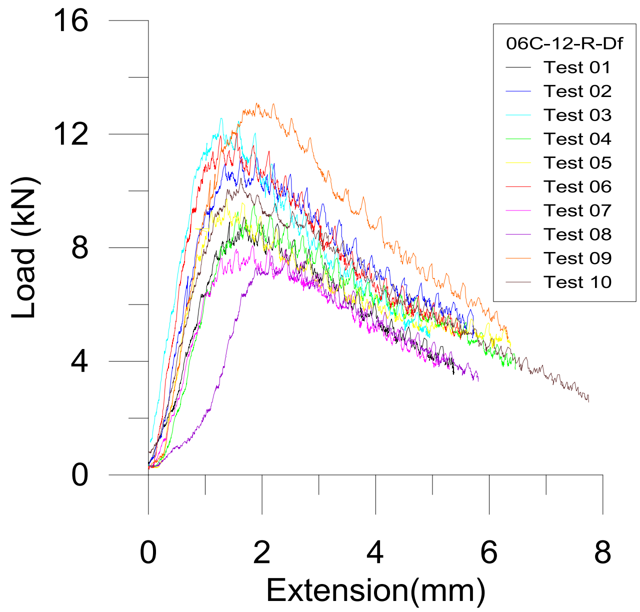


Figure B8 06C-12d-Df load vs. extension for side screws

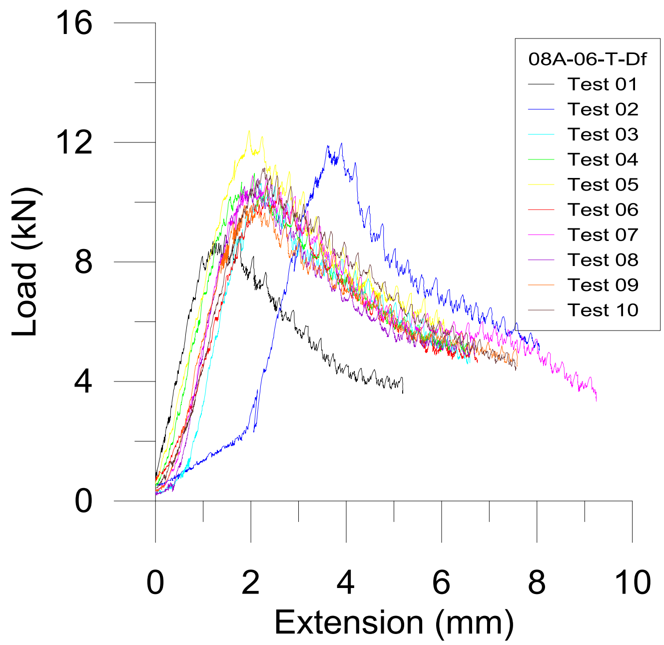


Figure B9 08A-06d-Df load vs. extension for top screws

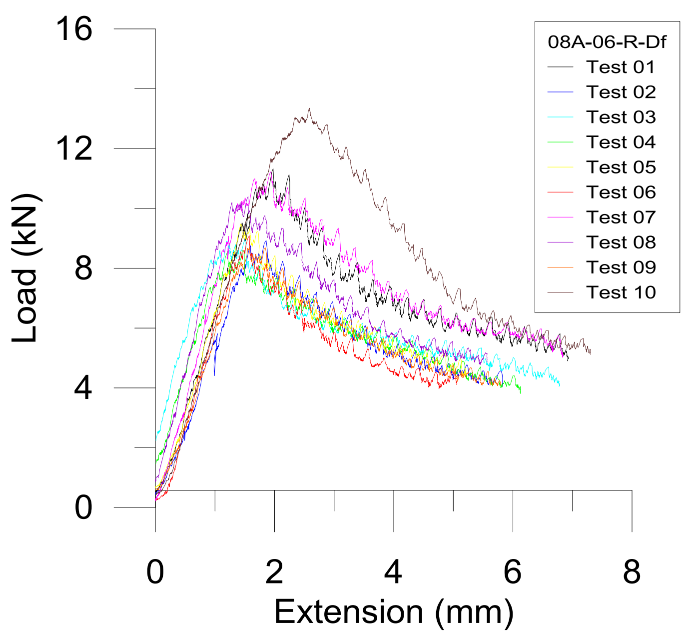


Figure B10 08A-06d-Df load vs. extension for side screws

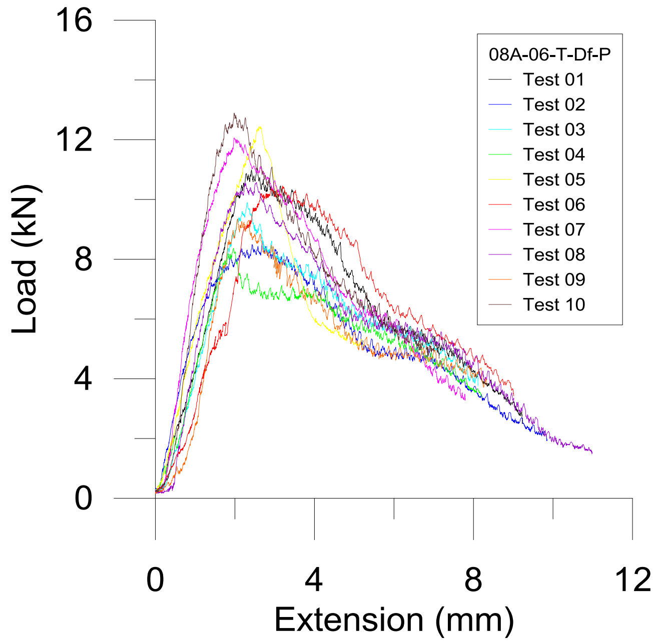


Figure B11 08A-06d-Df-P load vs. extension for top screws

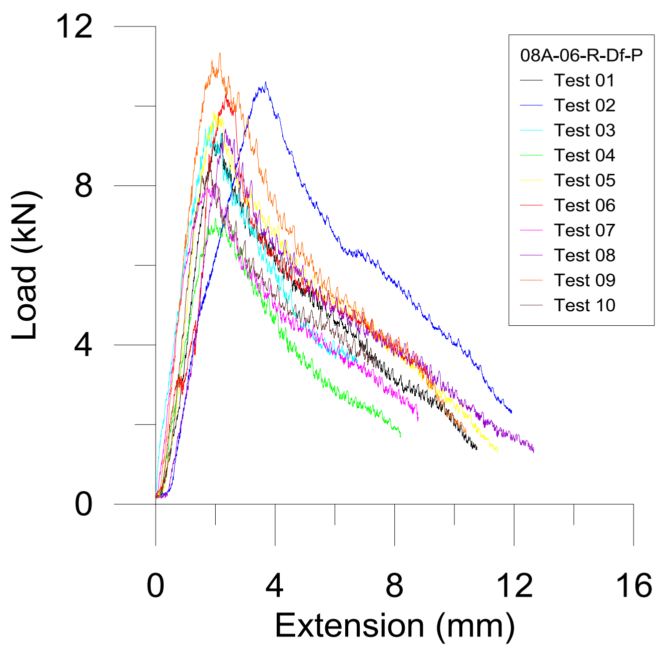


Figure B12 08A-06d-Df-P load vs. extension for side screws

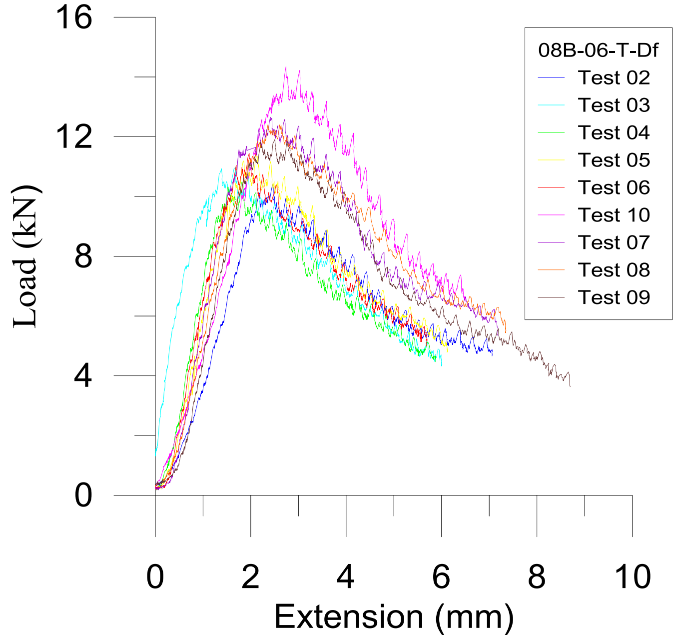


Figure B13 08B-06d-Df load vs. extension for top screws

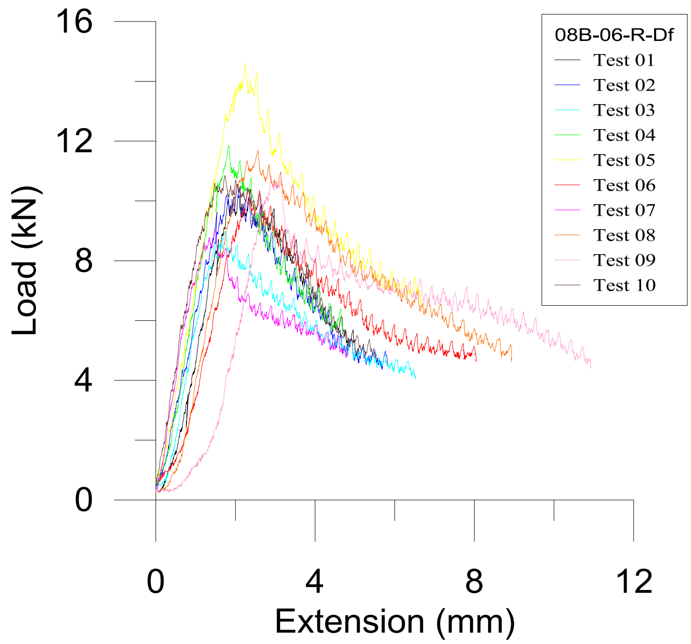


Figure B14 08B-06d-Df load vs. extension for side screws

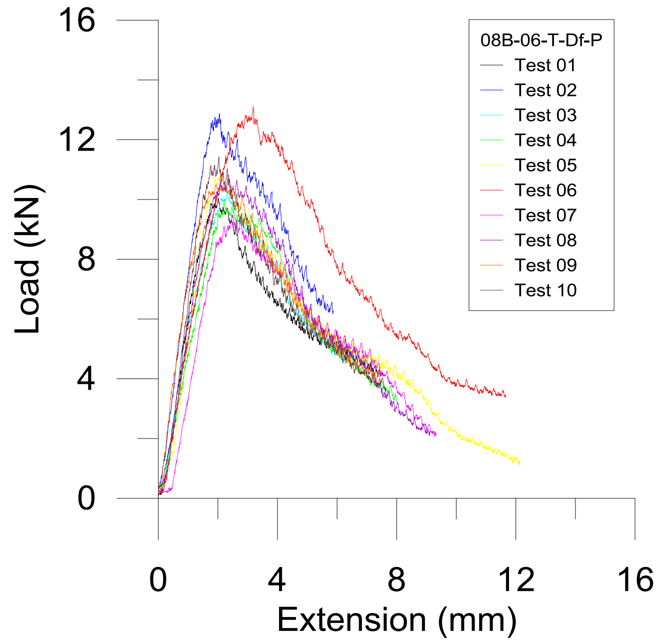


Figure B15 08B-06d-Df-P load vs. extension for top screws

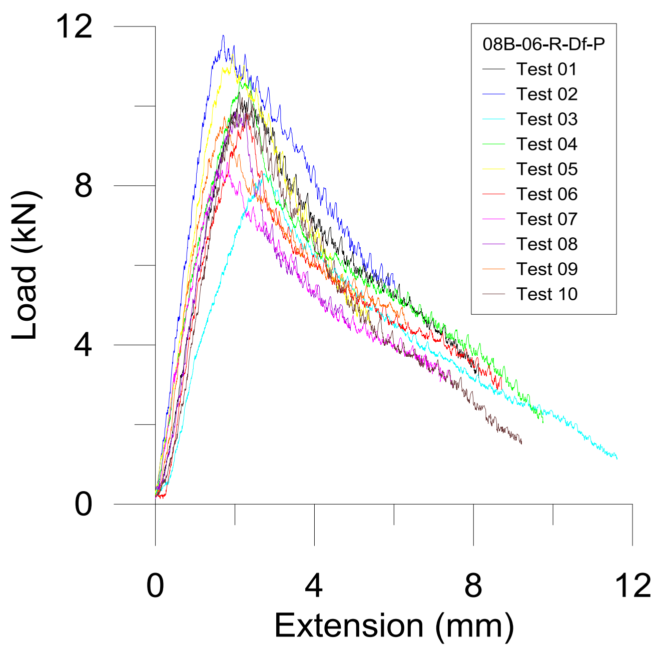


Figure B16 08B-06d-Df-P load vs. extension for side screws

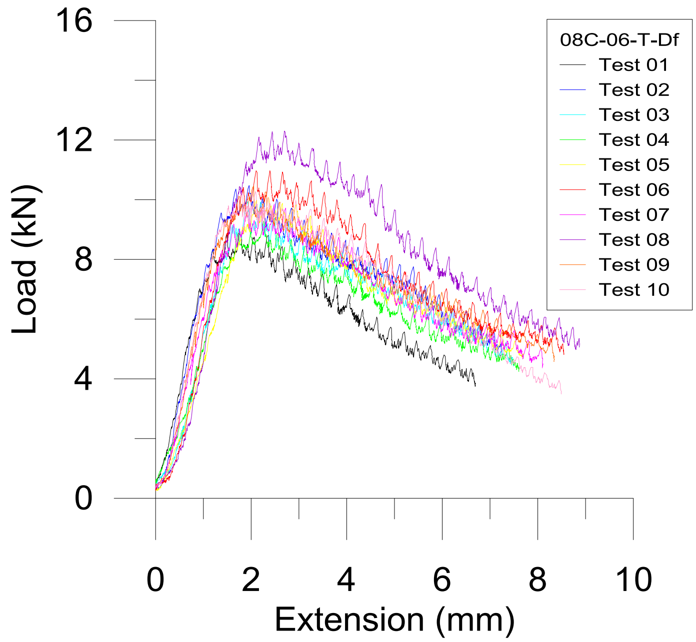


Figure B17 08C-06d load vs. extension for top screws

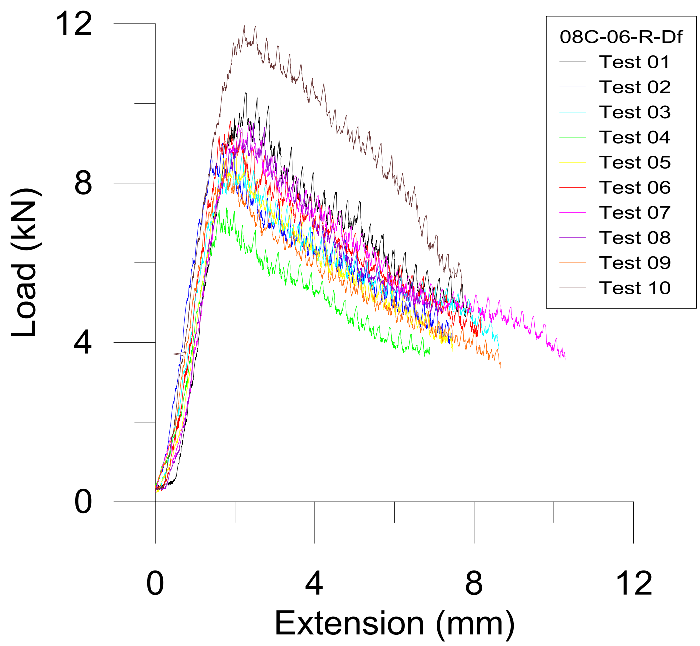


Figure B18 08C-06d load vs. extension for side screws

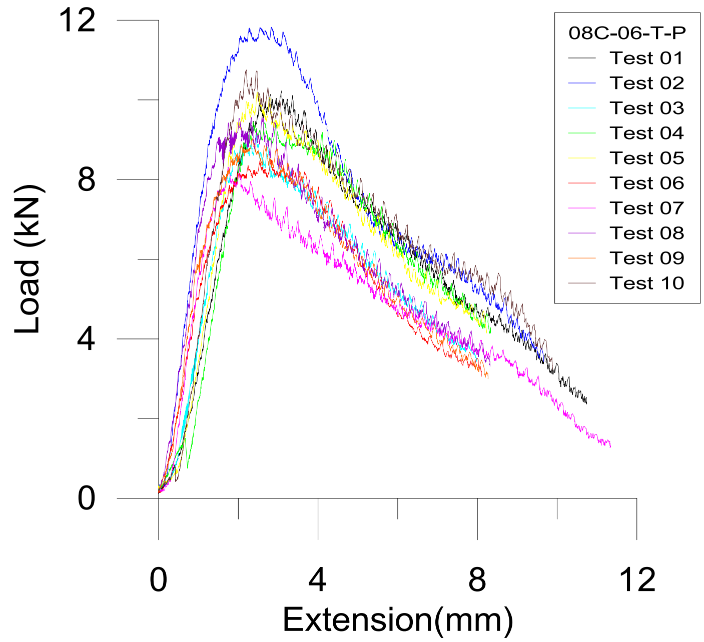


Figure B19 08C-06d-P load vs. extension for top screws

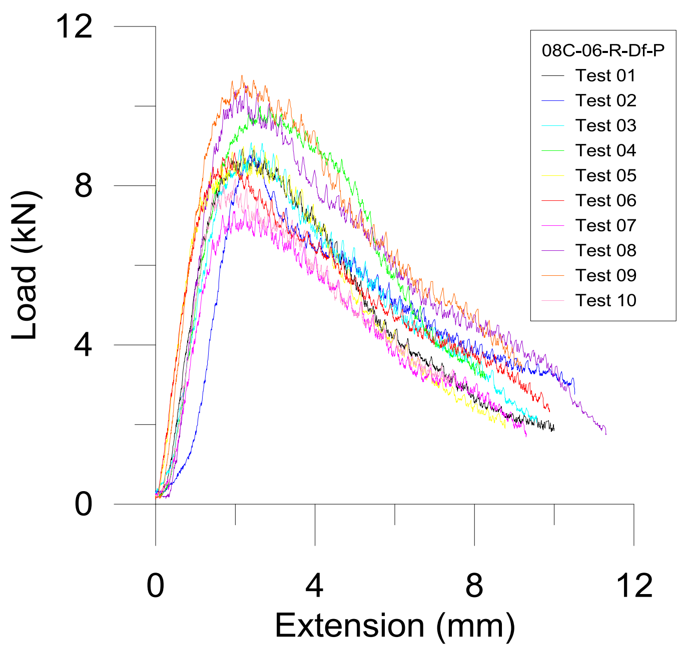


Figure B20 08C-06d-P load vs. extension for side screws

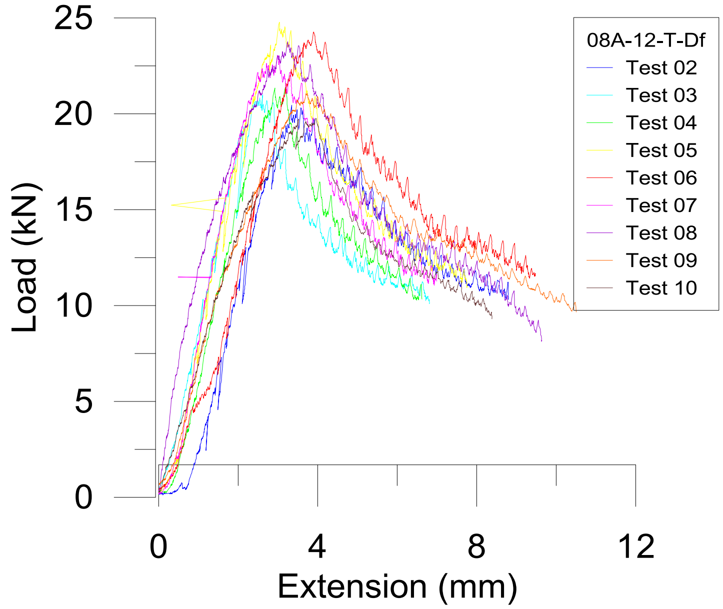


Figure B21 08A-12d-Df load vs. extension for top screws

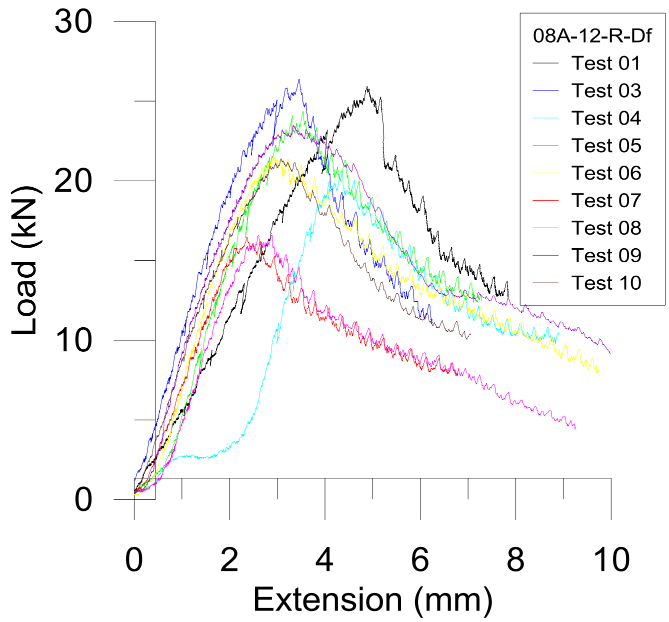


Figure B22 08A-12d-Df load vs. extension for side screws

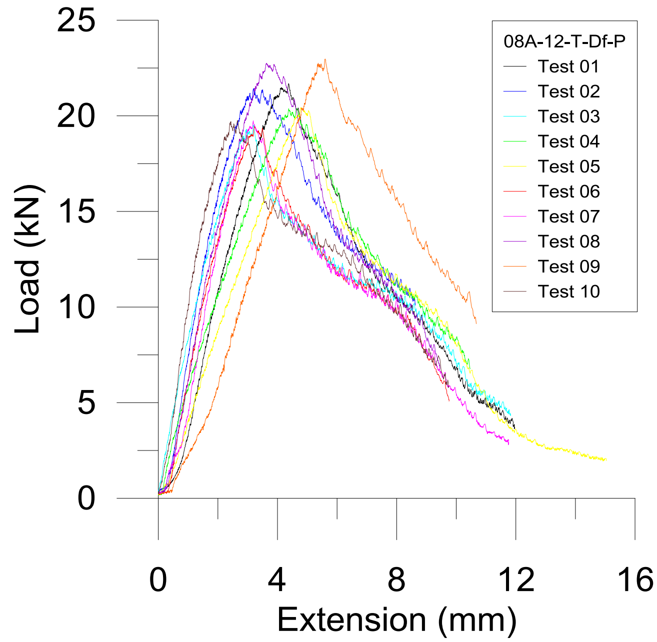


Figure B23 08A-12d-Df- P load vs. extension for top screws

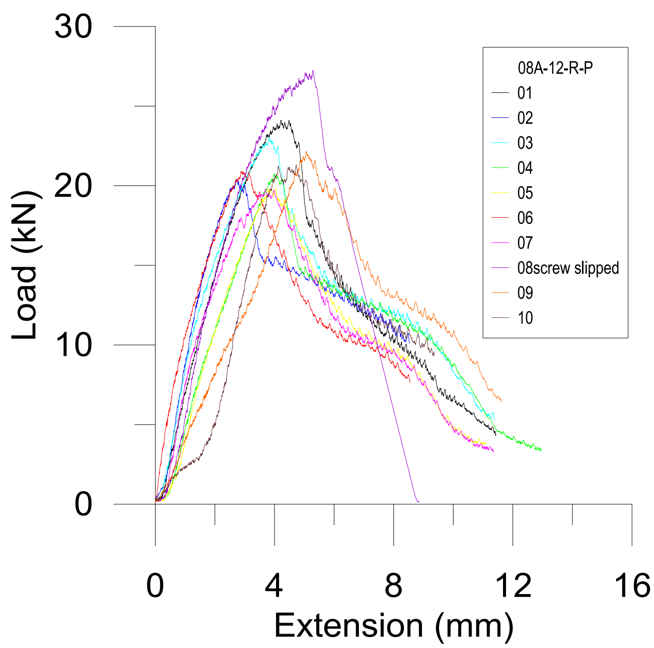


Figure B24 08A-12d-Df- P load vs. extension for side screws

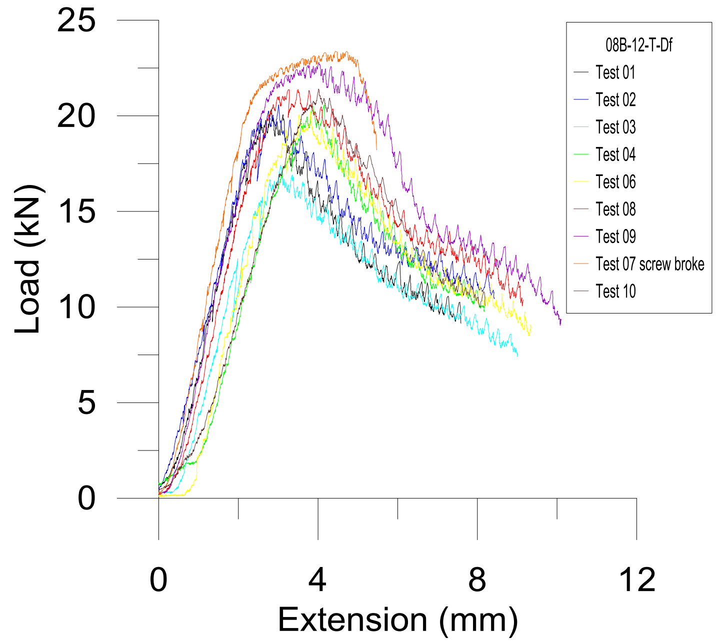


Figure B25 08B-12d-Df load vs. extension for top screws

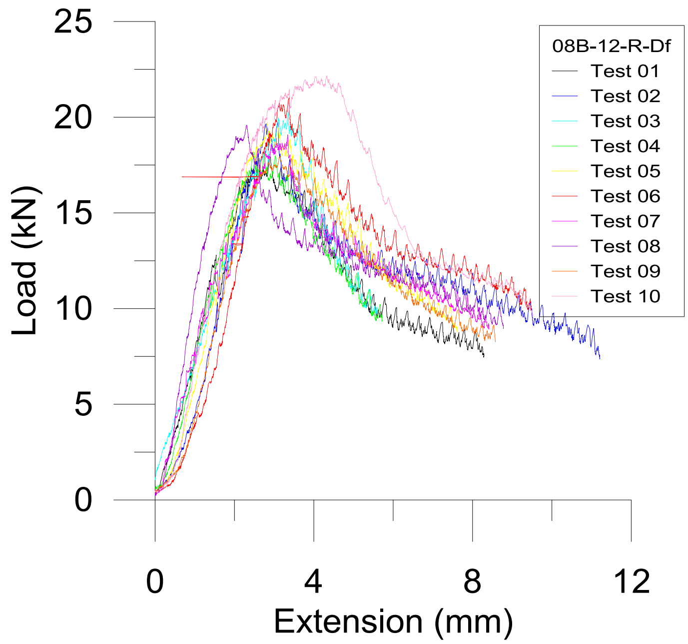


Figure B26 08B-12d-Df load vs. extension for side screws

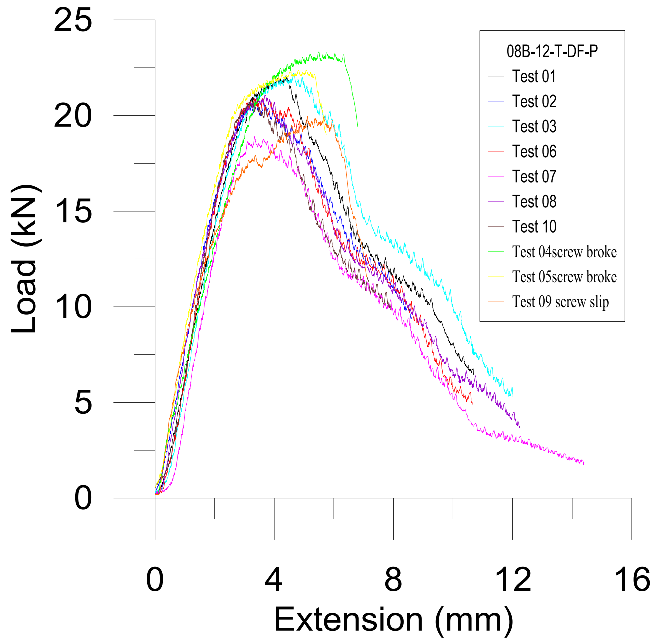


Figure B27 08B-12-DF-P load vs. extension for top screws

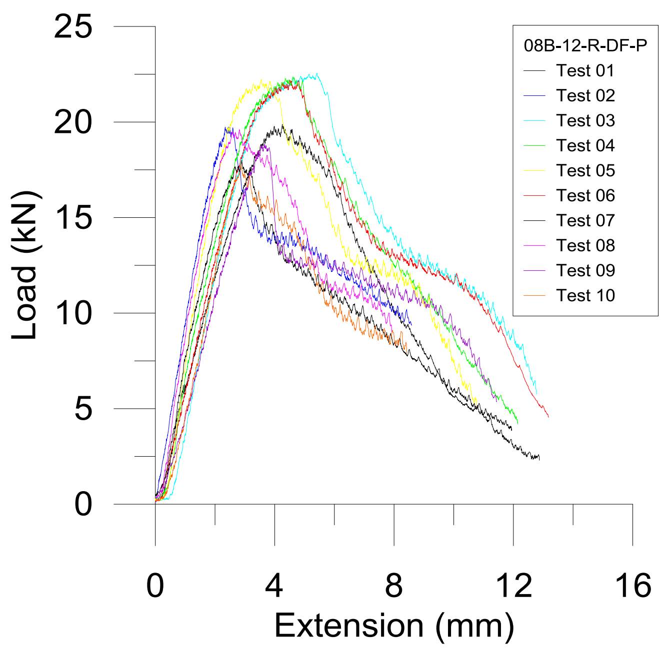


Figure B28 08B-12-DF-P load vs. extension for side screws

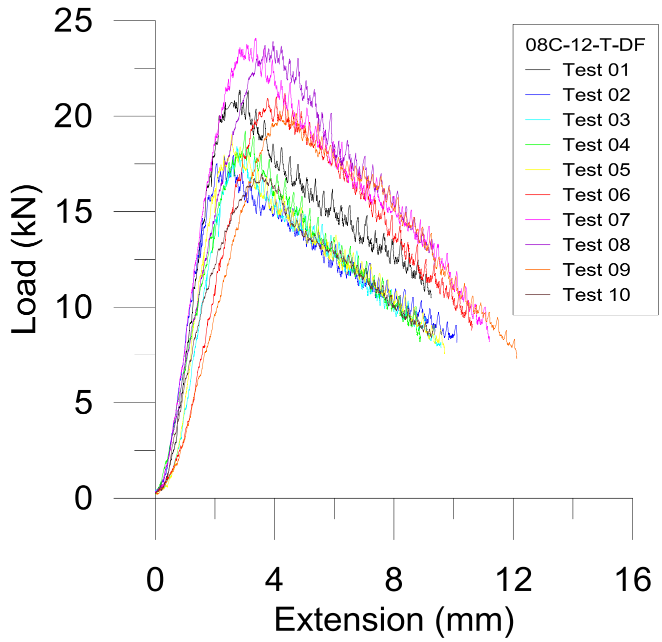


Figure B29 08C-12d-DF load vs. extension for top screws

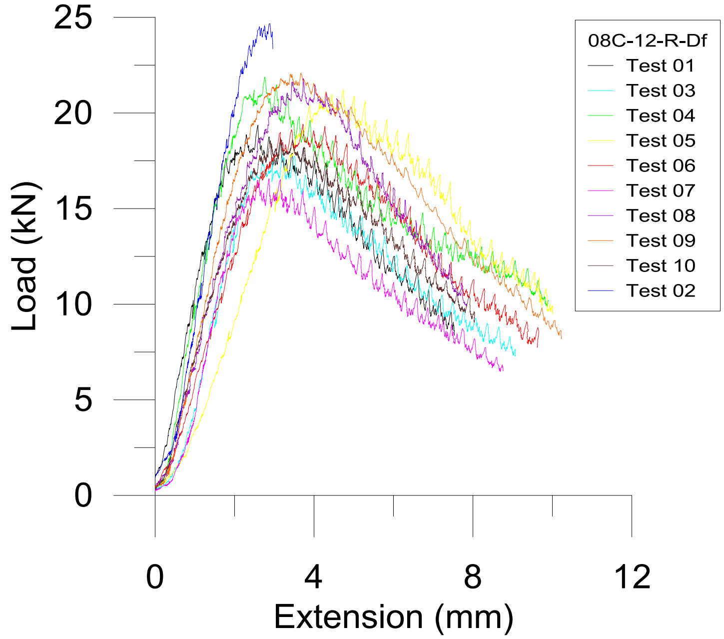


Figure B30 08C-12d-DF load vs. extension for side screws

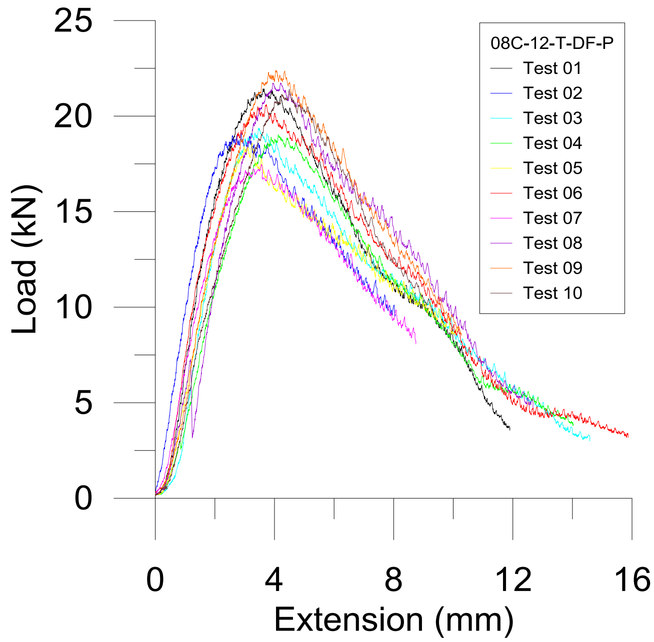


Figure B31 08C-12d-Df-P load vs. extension for top screws

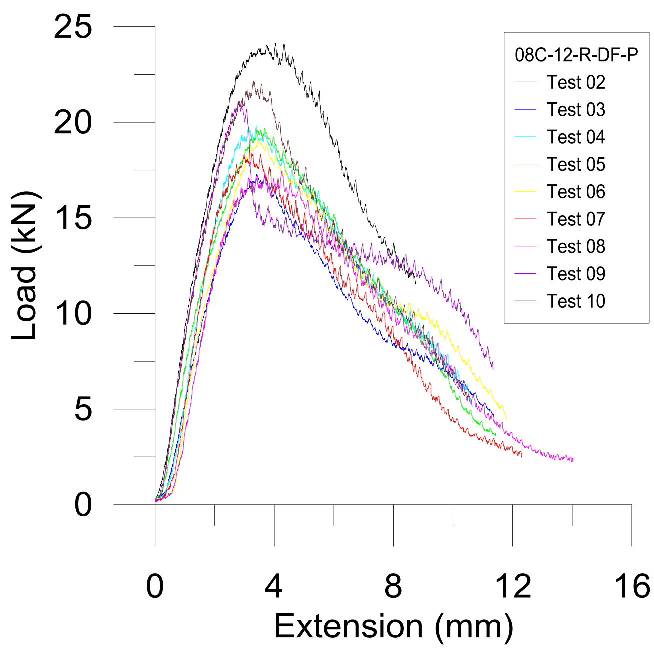


Figure B32 08C-12d-Df-P load vs. extension for side screws

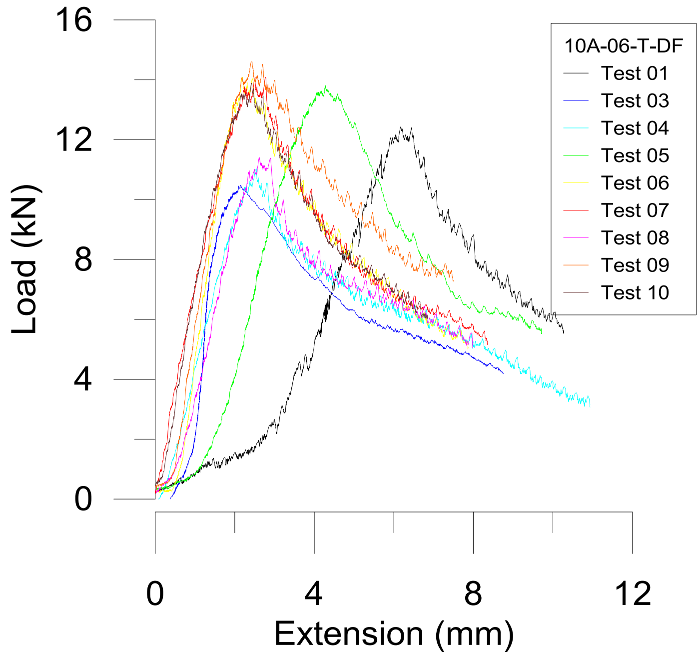


Figure B33 10A-06-R-DF load vs. extension for top screws

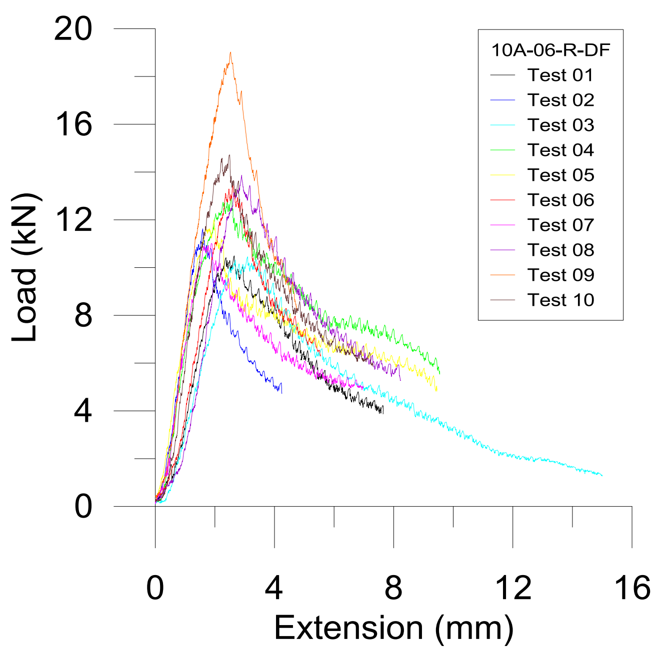


Figure B34 10A-06-R-DF load vs. extension for side screws

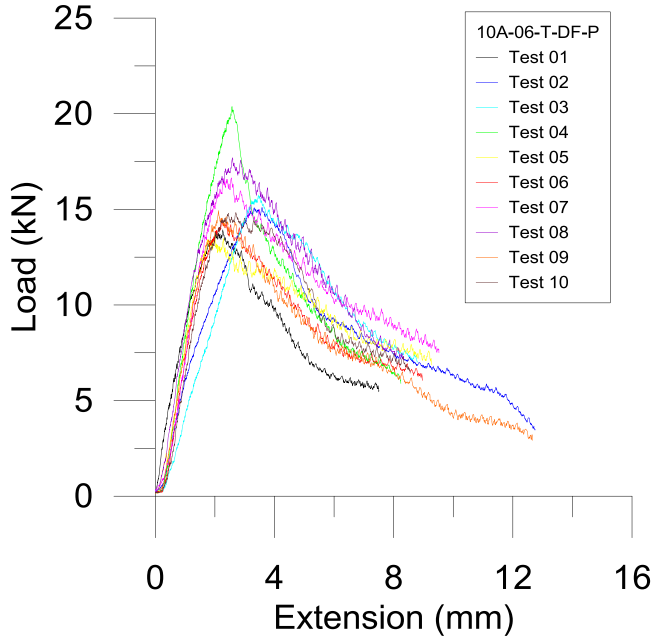


Figure B35 10A-06-R-DF-P load vs. extension for top screws

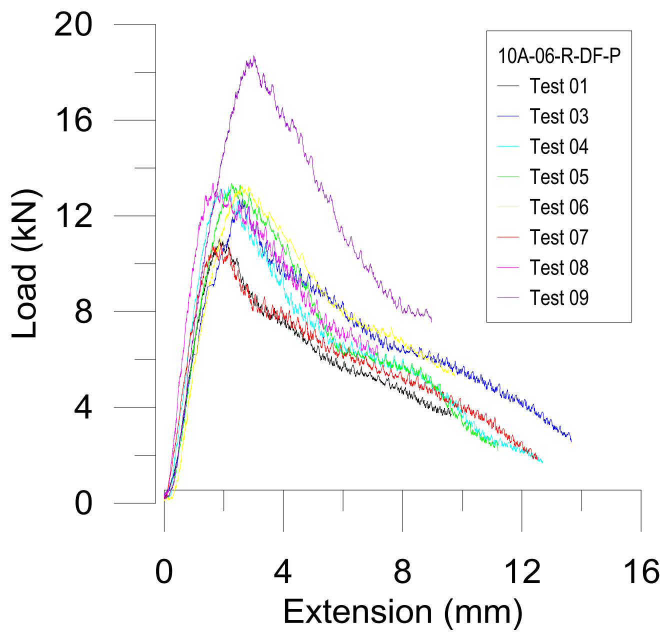


Figure B36 10A-06-R-DF-P load vs. extension for side screws

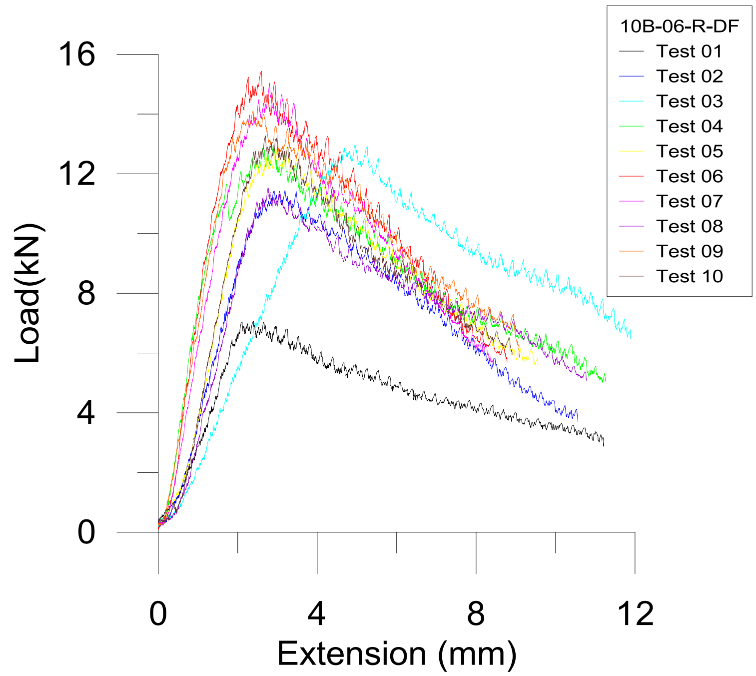


Figure B37 10B-06-T-Df load vs. extension for top screws

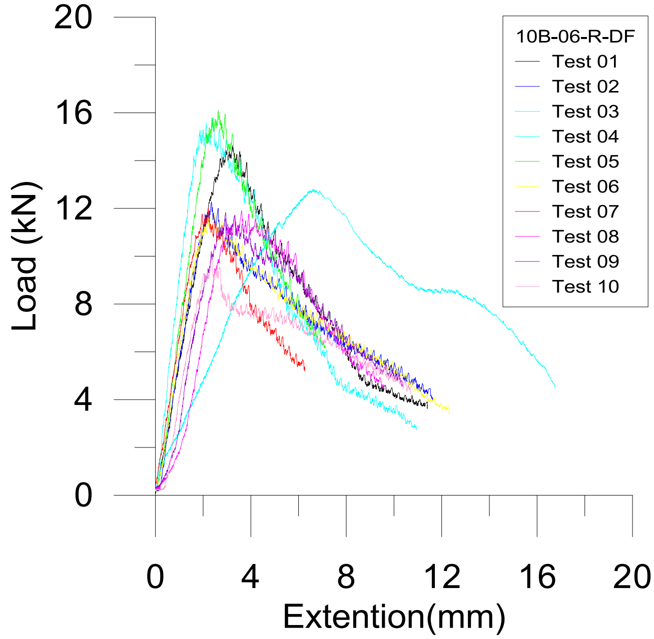


Figure B38 10B-06-T-Df load vs. extension for side screws

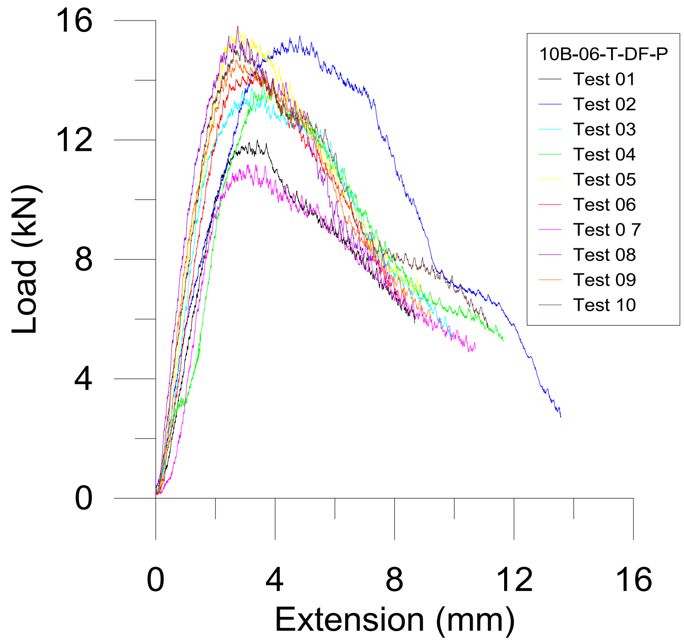


Figure B39 10B-06-R-Df-P load vs. extension for top screws

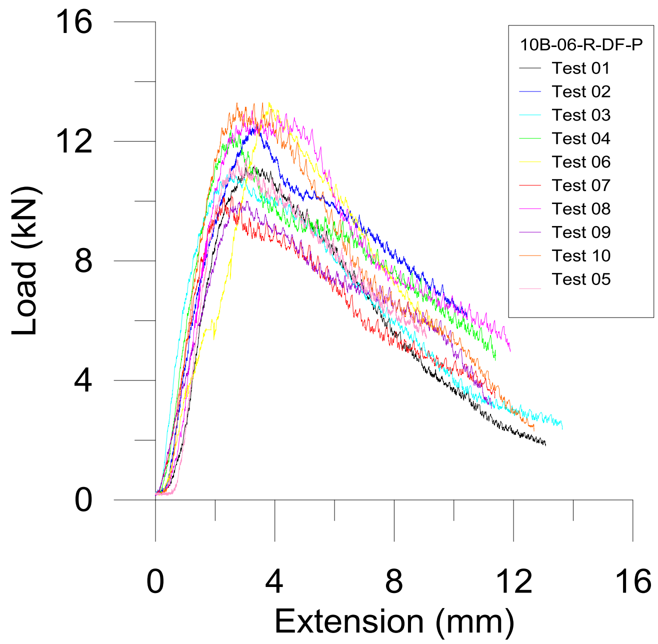


Figure B40 10B-06-R-Df-P load vs. extension for side screws

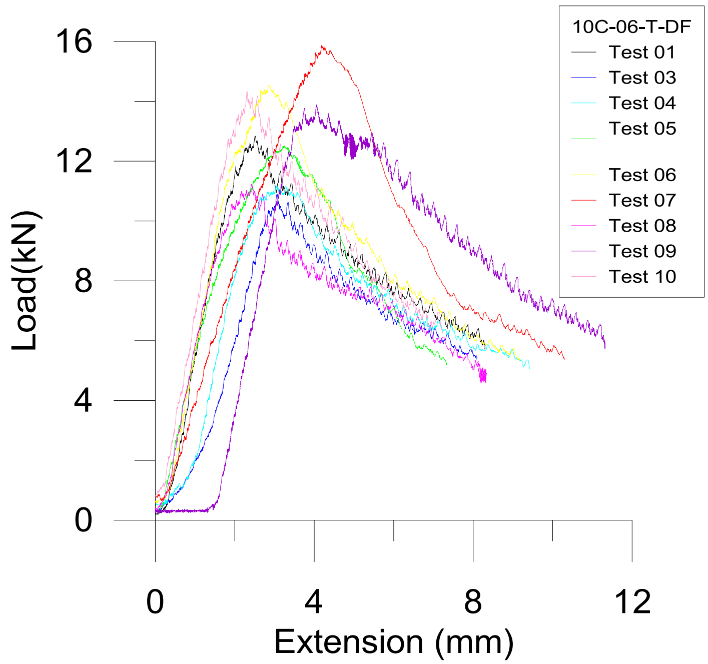


Figure B41 10C-06-R-Df load vs. extension for top screws

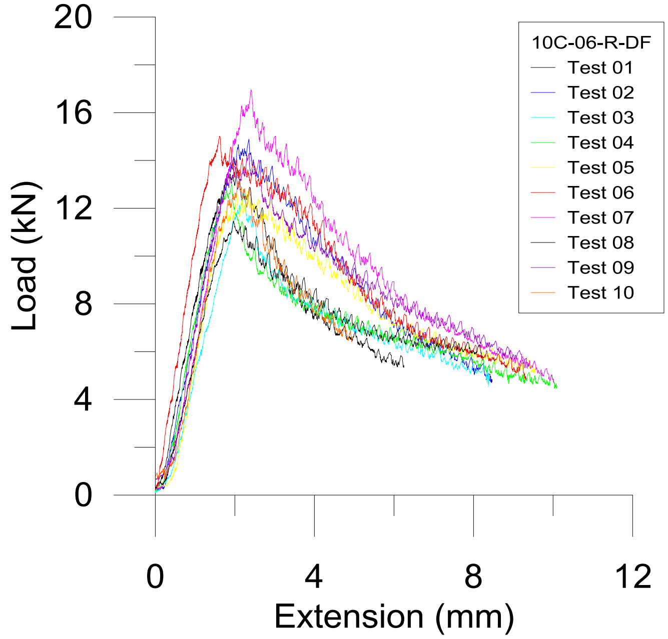


Figure B42 10C-06-R-Df load vs. extension for side screws

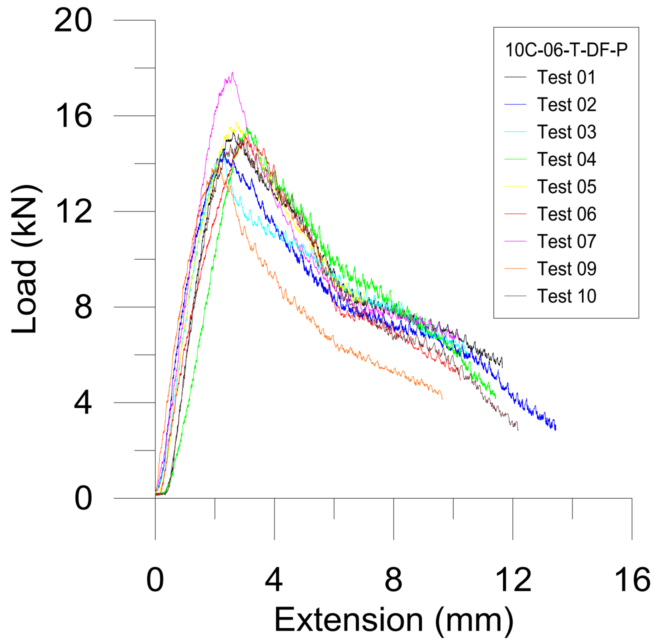


Figure B43 10C-06-R-Df-P load vs. extension for top screws

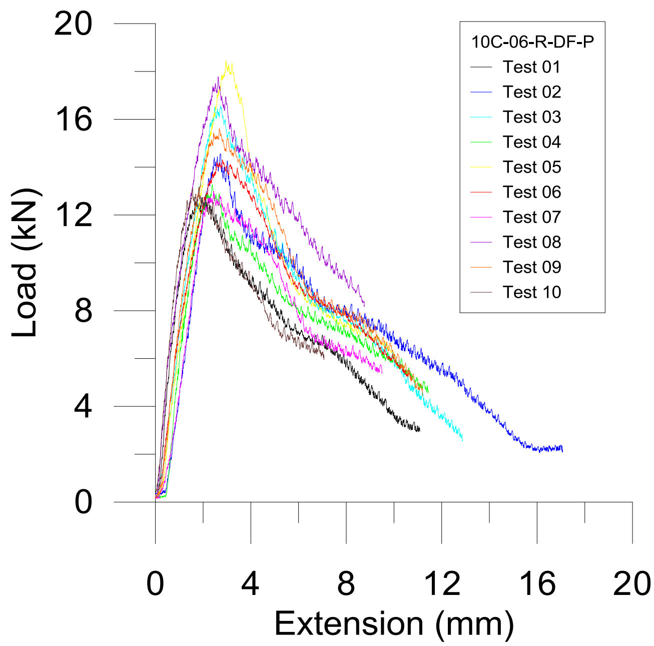


Figure B44 10C-06-R-Df-P load vs. extension for side screws

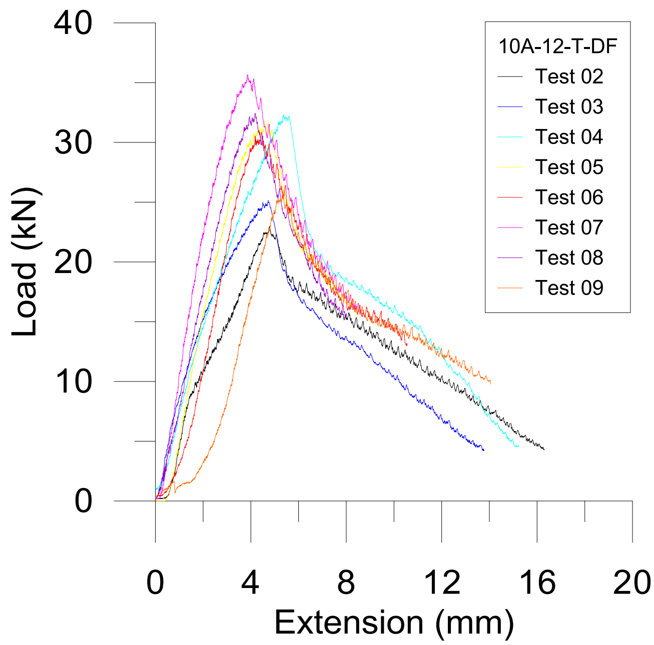


Figure B45 10A-12-DF load vs. extension for top screws

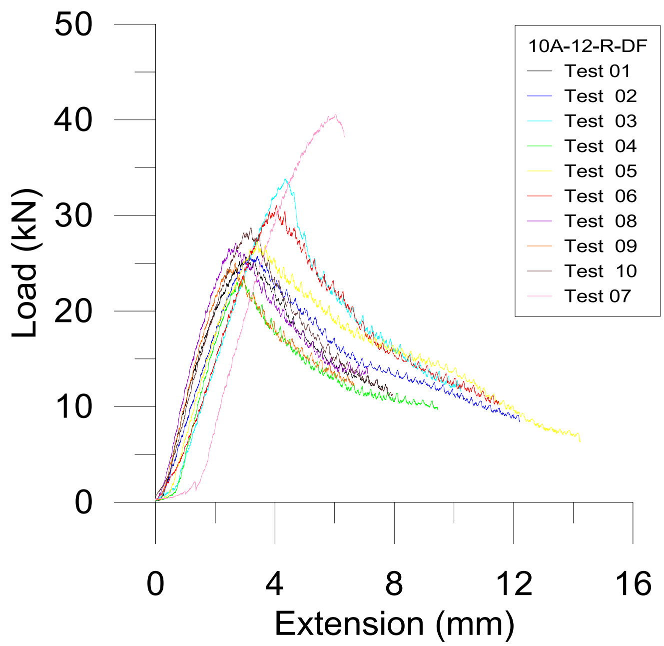


Figure B46 10A-12-DF load vs. extension for side screws

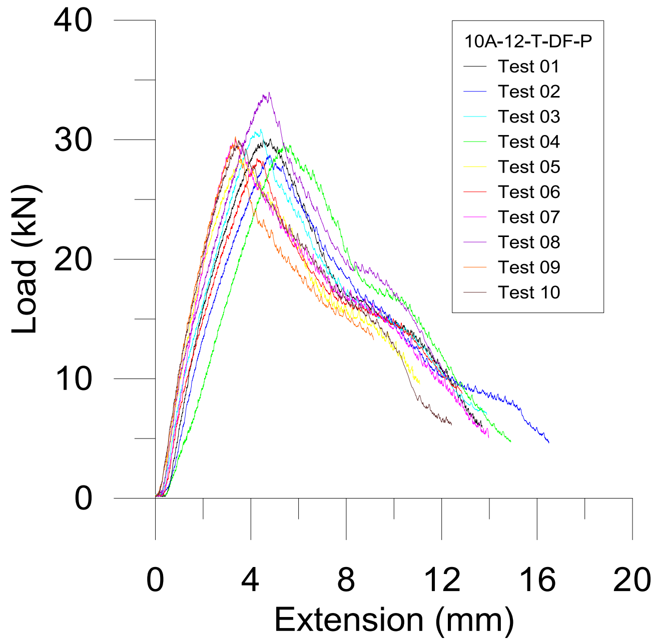


Figure B47 10A-12-DF-P load vs. extension for top screws

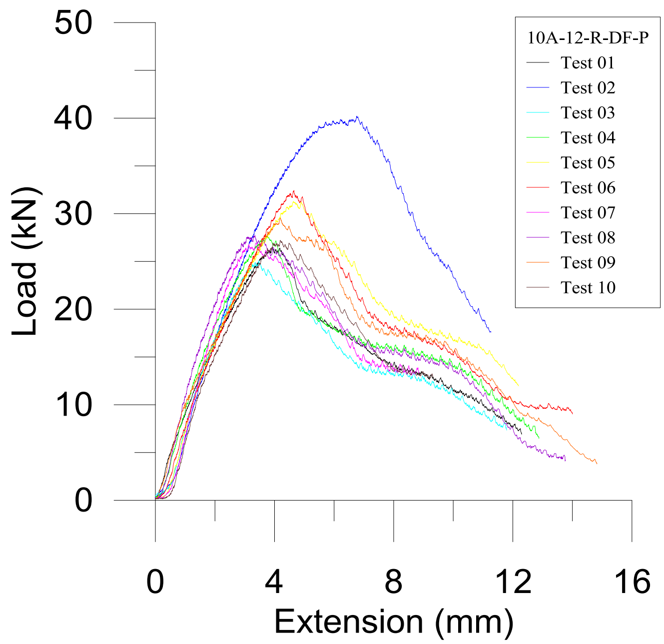


Figure B48 10A-12-DF-P load vs. extension for side screws

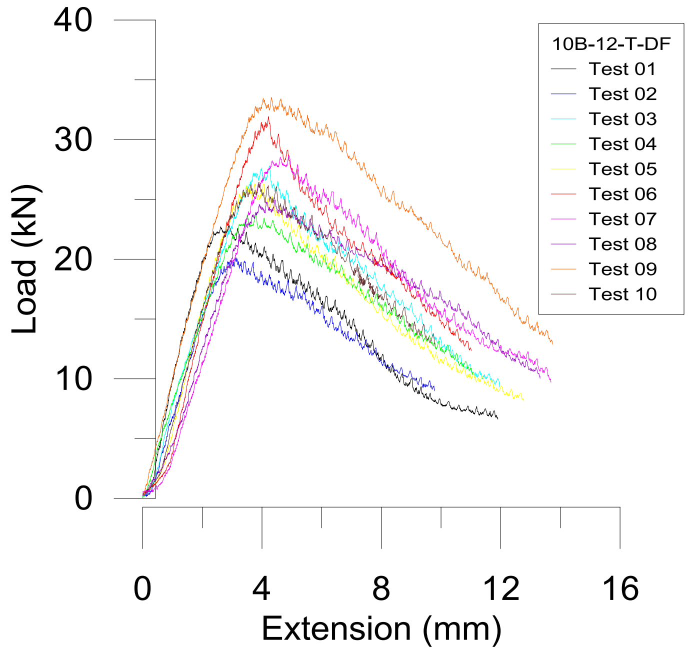


Figure B49 10B-12-Df load vs. extension for top screws

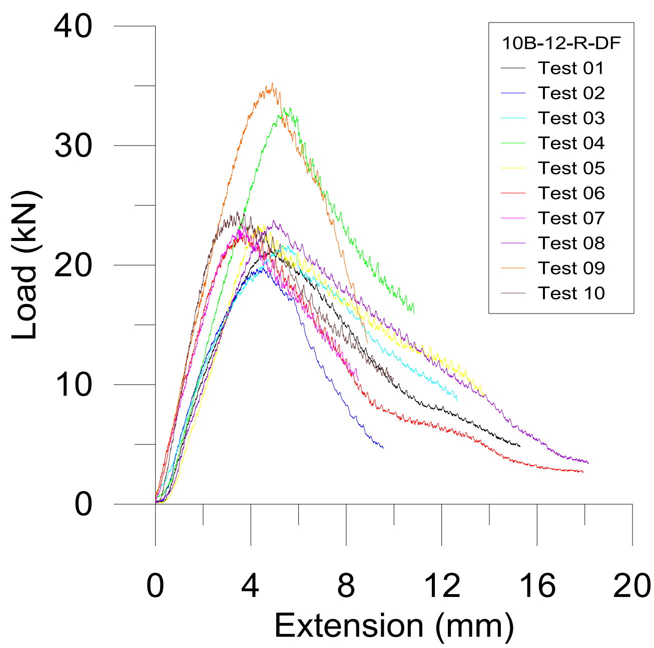


Figure B50 10B-12-Df load vs. extension for side screws

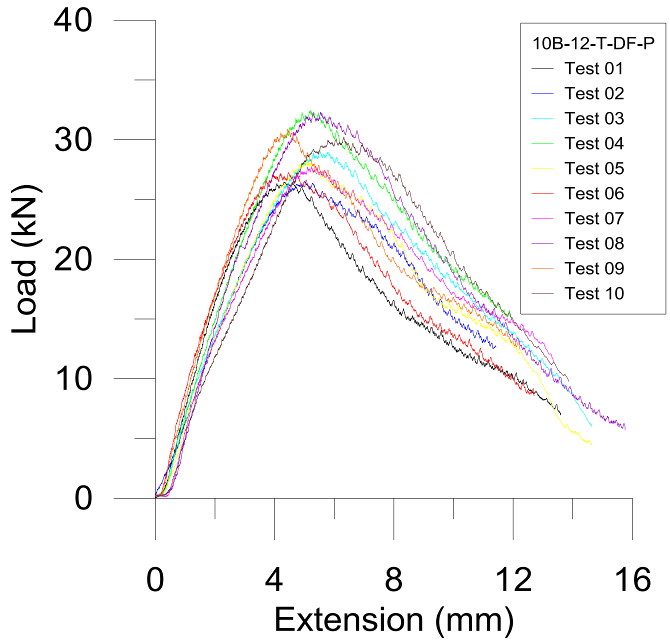


Figure B51 10B-12-Df-P load vs. extension for top screws

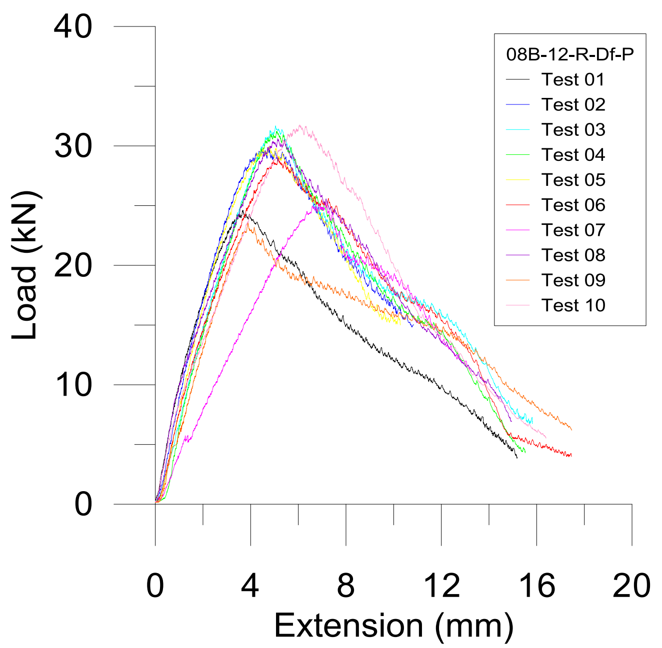


Figure B52 10B-12-Df-P load vs. extension for side screws

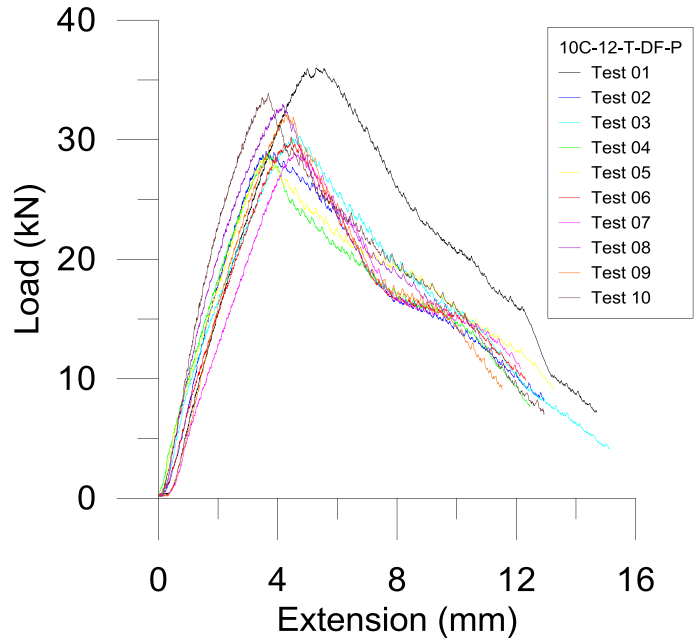


Figure B53 10C-12-Df-P load vs. extension for top screws

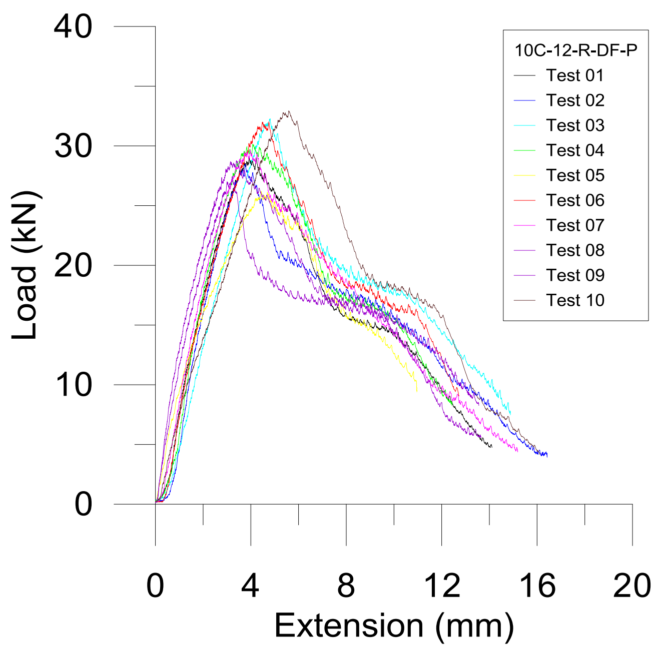


Figure B54 10C-12-Df-p load vs. extension for side screws

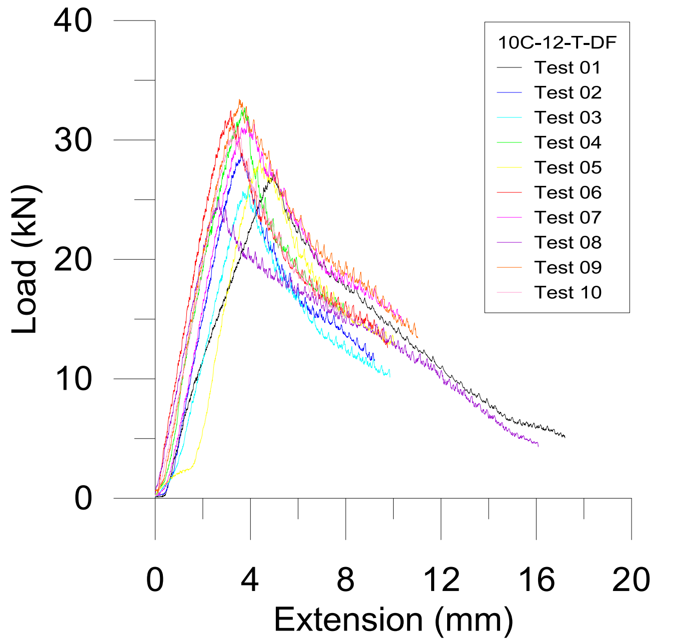


Figure B55 10C-12-Df load vs. extension for top screws

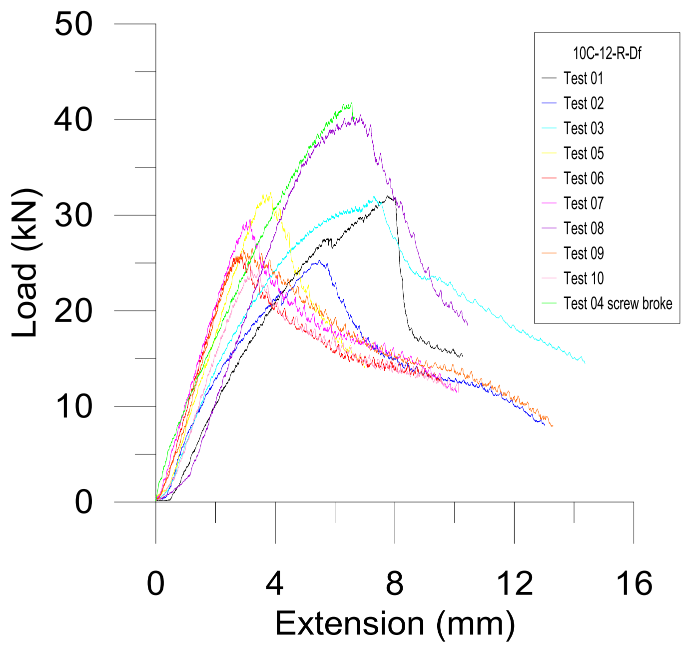


Figure B56 10C-12-Df load vs. extension for side screws

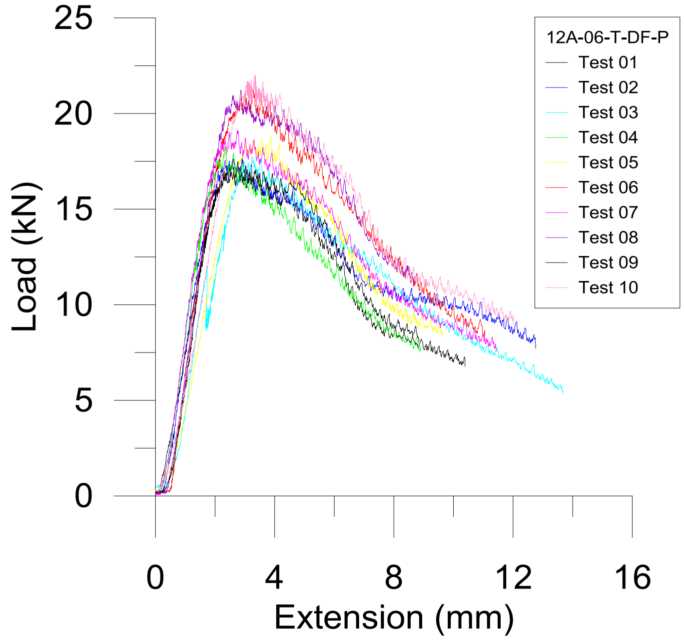


Figure B57 12A-06-Df-P load vs. extension for top screws

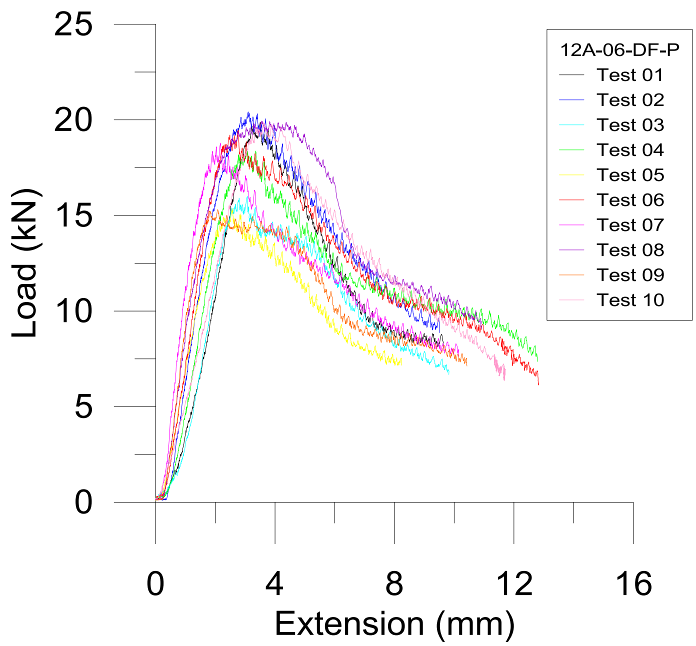


Figure B58 12A-06-Df-P load vs. extension for side screws

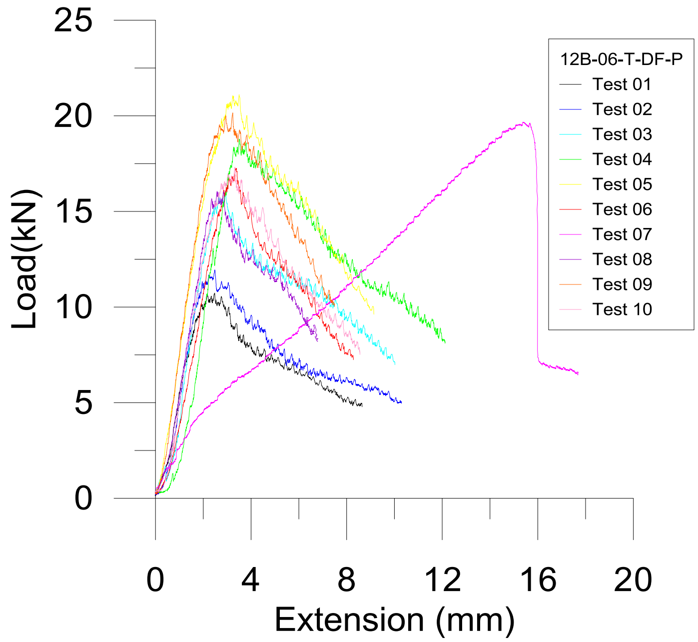


Figure B59 12B-06-Df-P load vs. extension for top screws

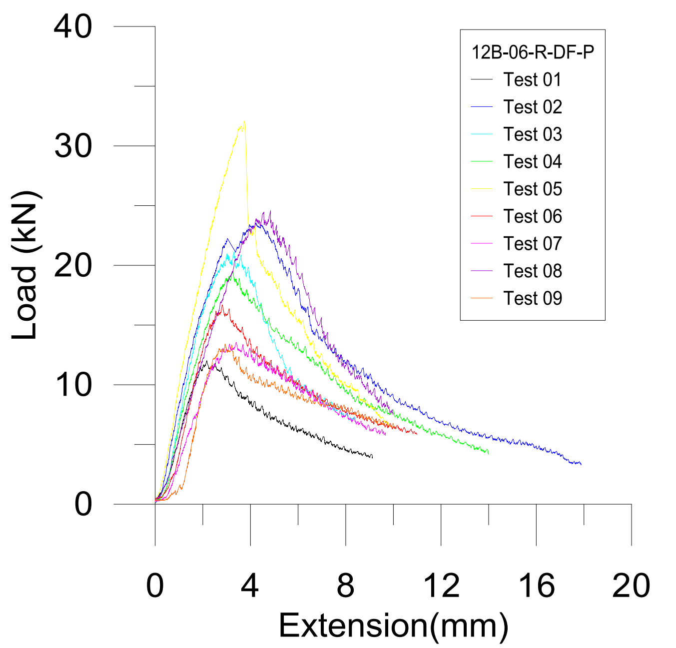


Figure B60 12B-06-Df-P load vs. extension for side screws

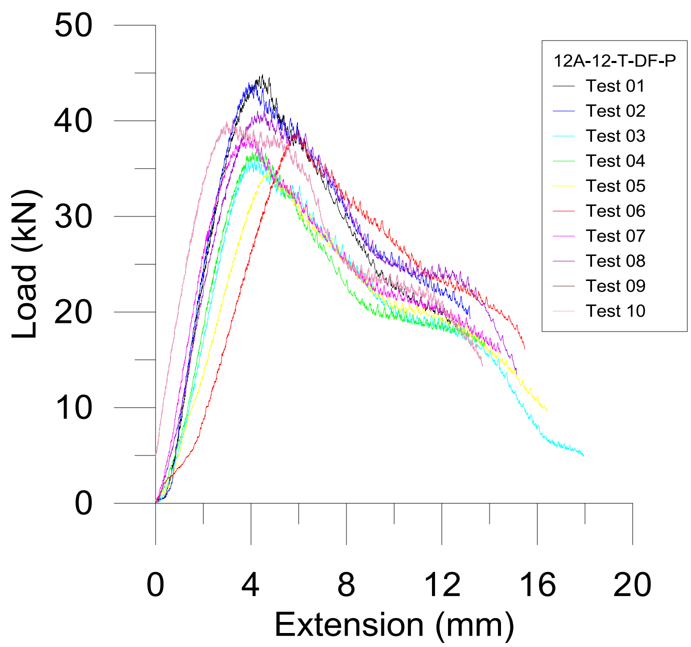


Figure B61 12A-12-DF-P load vs. extension for top screws

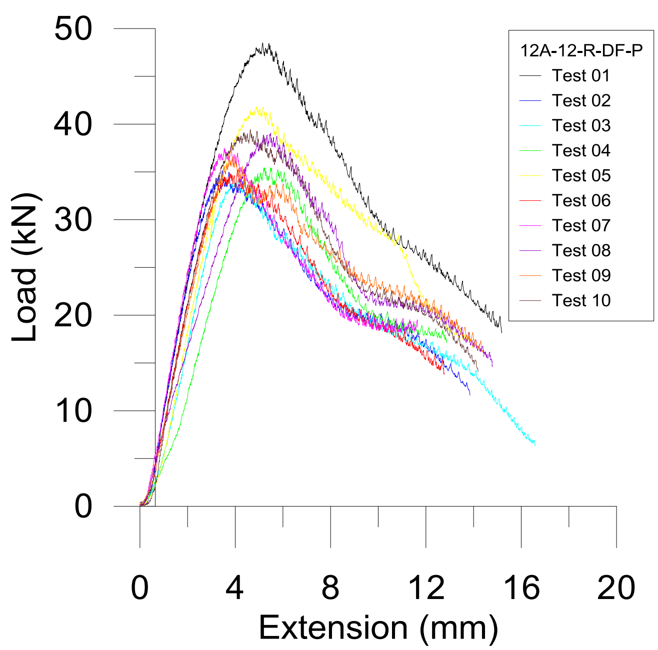


Figure B62 12A-12-DF-P load vs. extension for side screws

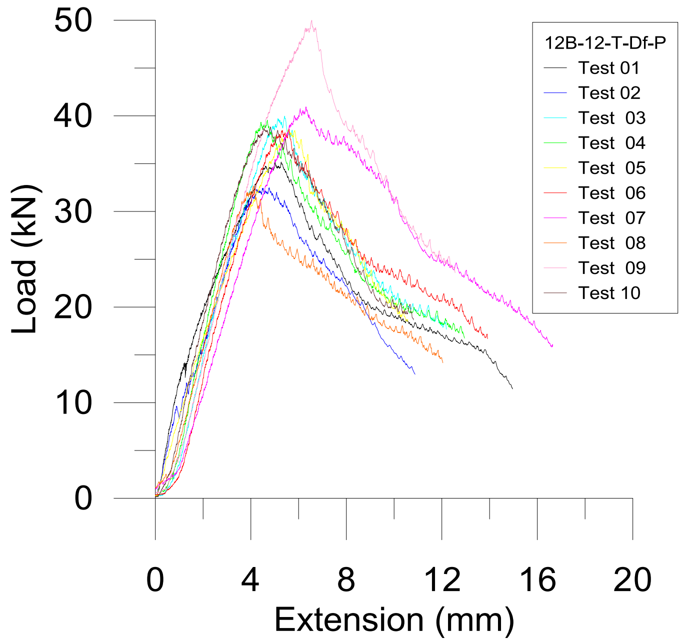


Figure B63 12B-12-DF-P load vs. extension for top screws

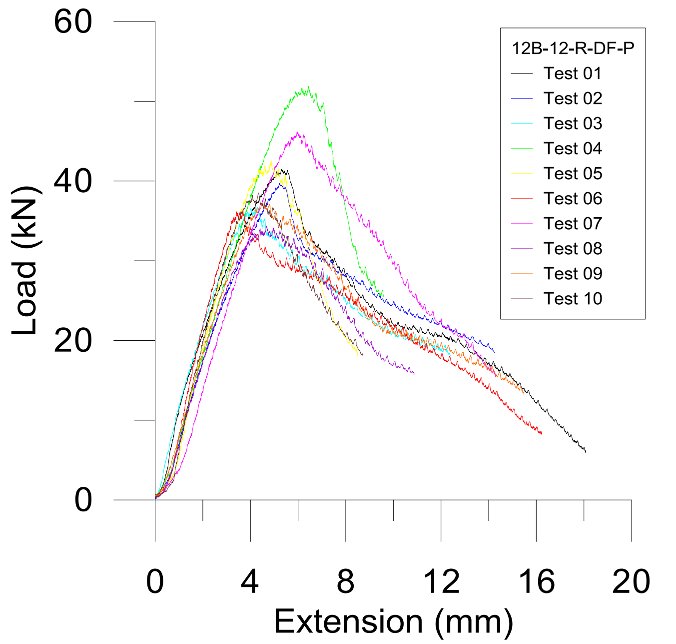


Figure B64 12B-12-DF-P load vs. extension for side screws

## Spruce Pine Glulam

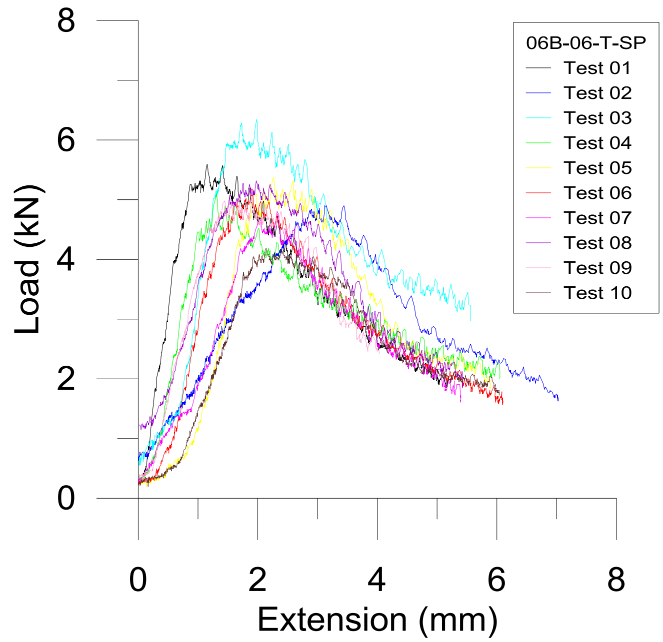


Figure B65 06B-06-SP load vs. extension for top screws

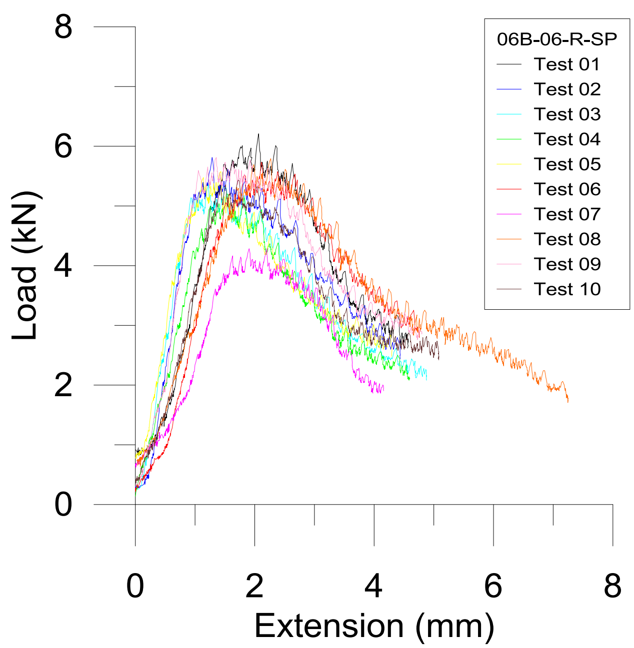


Figure B66 06B-06-SP load vs. extension for side screws

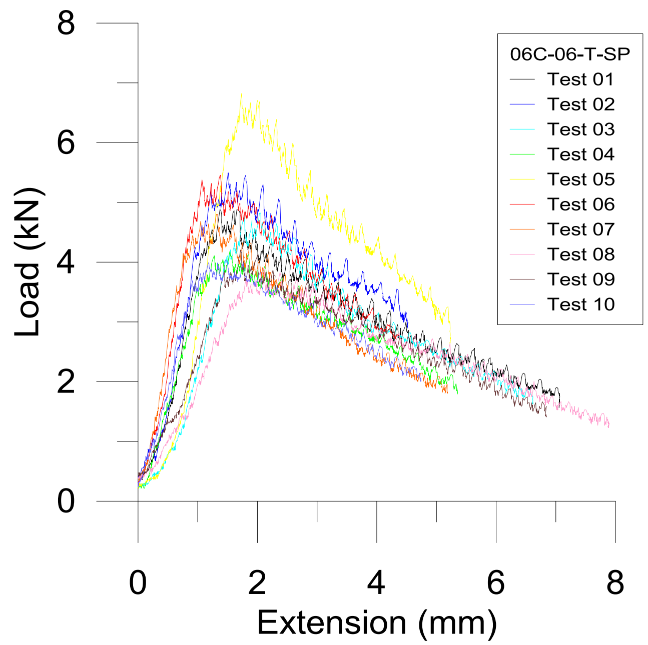


Figure B67 06C-06-SP load vs. extension for top screws

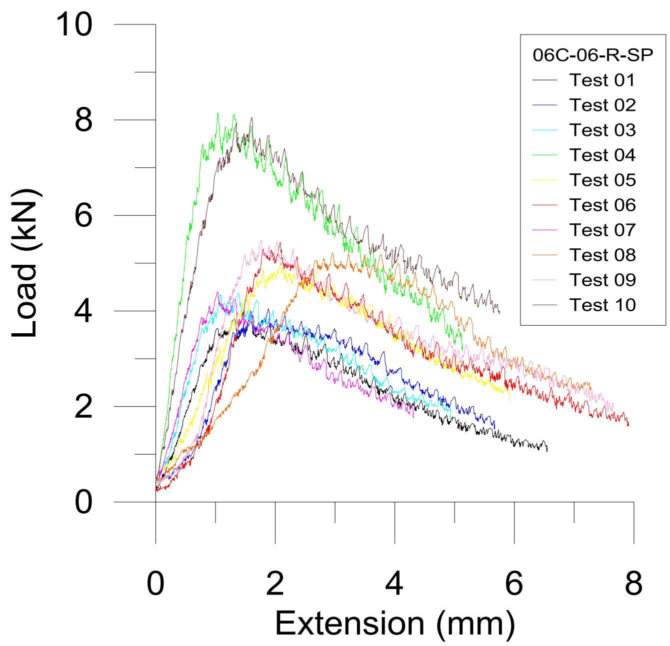


Figure B68 06C-06-SP load vs. extension for side screws

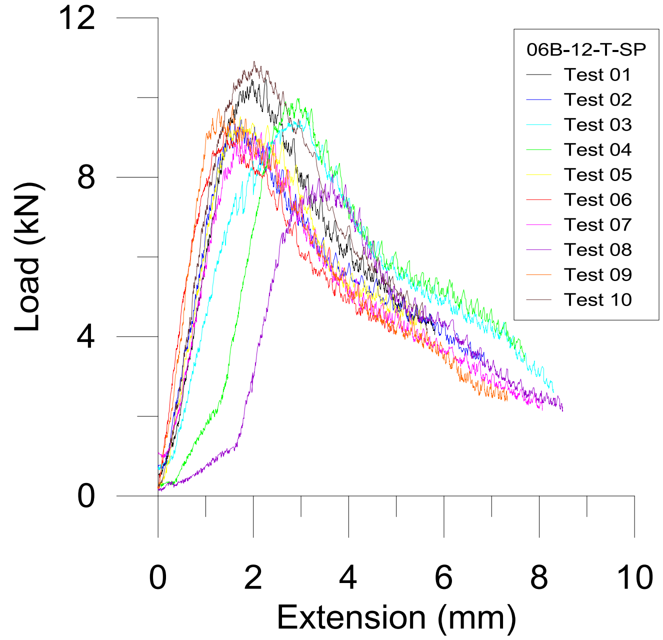


Figure B69 06-12-SP load vs. extension for top screws

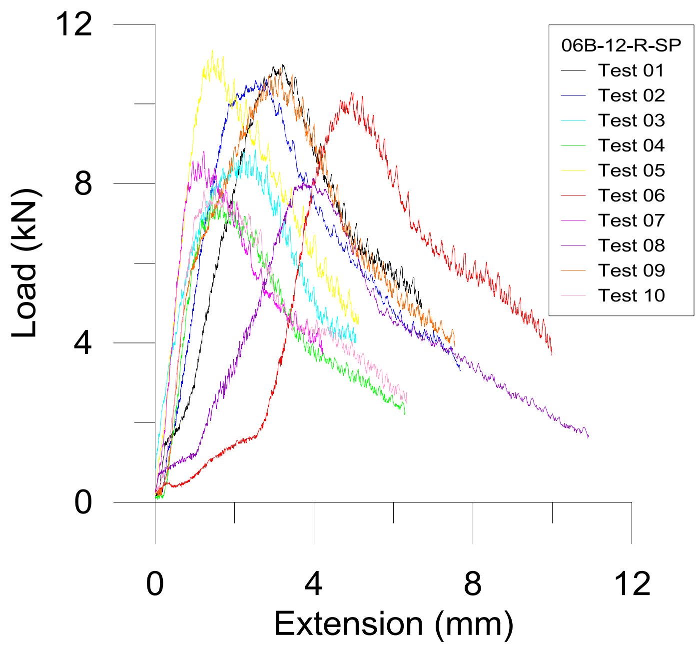


Figure B70 06-12-SP load vs. extension for side screws

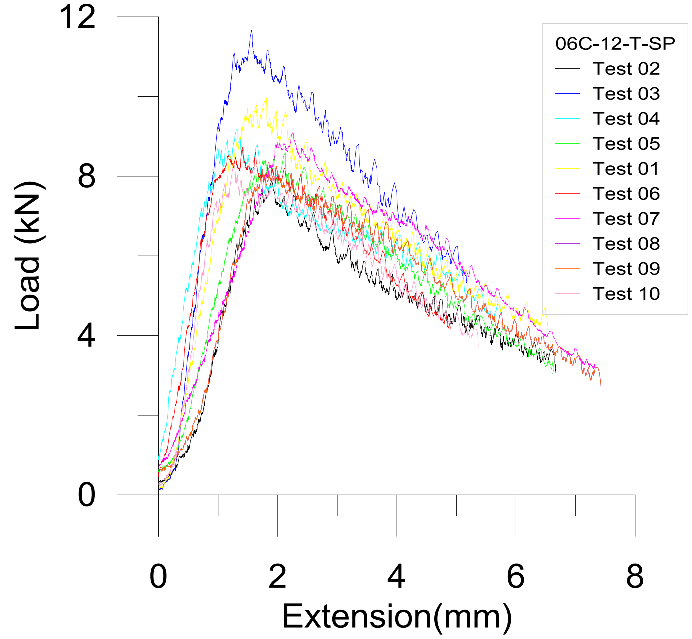


Figure B71 06C-12-SP load vs. extension for top screws

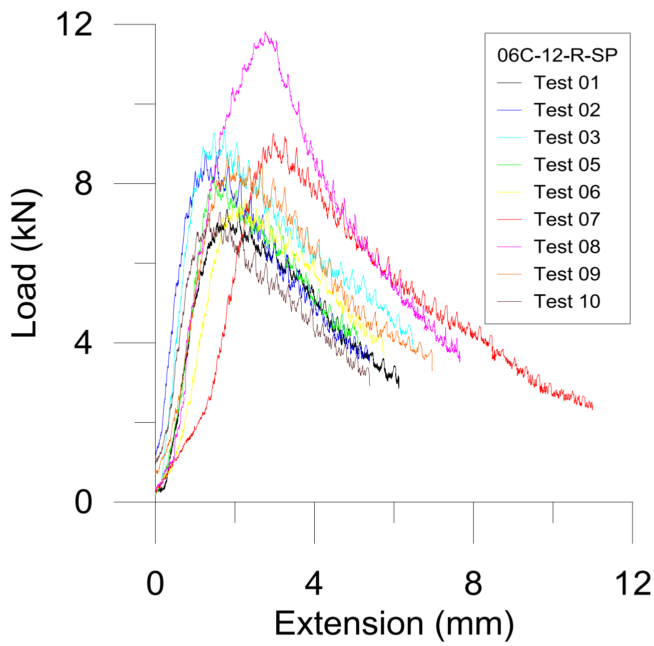


Figure B72 06C-12-SP load vs. extension for side screws

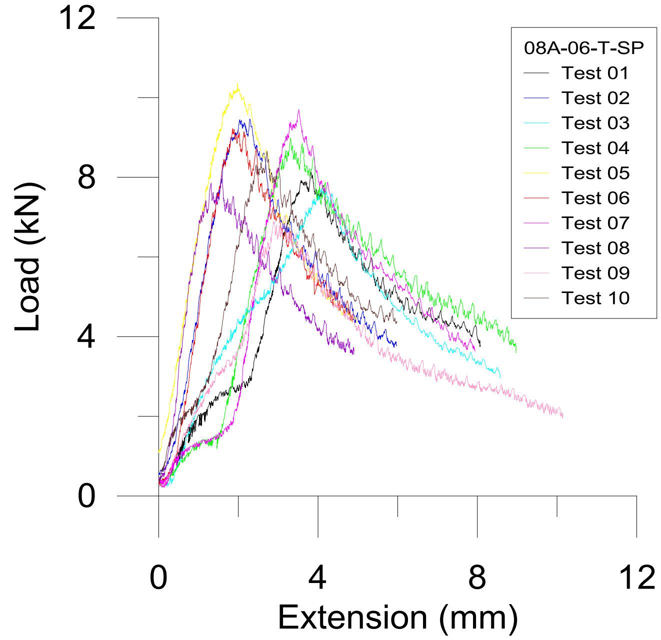


Figure B73 08A-06-SP load vs. extension for top screws

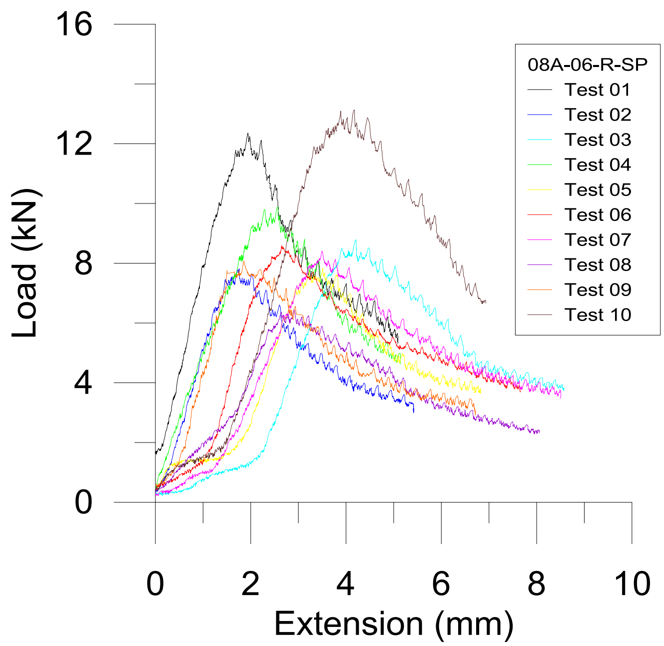


Figure B74 08A-06-SP load vs. extension for side screws

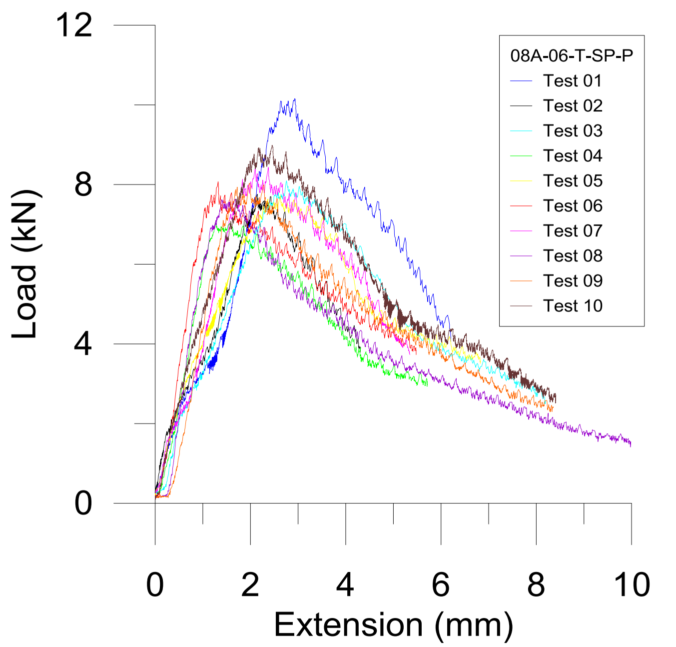


Figure B75 08A-06-SP-P load vs. extension for top screws

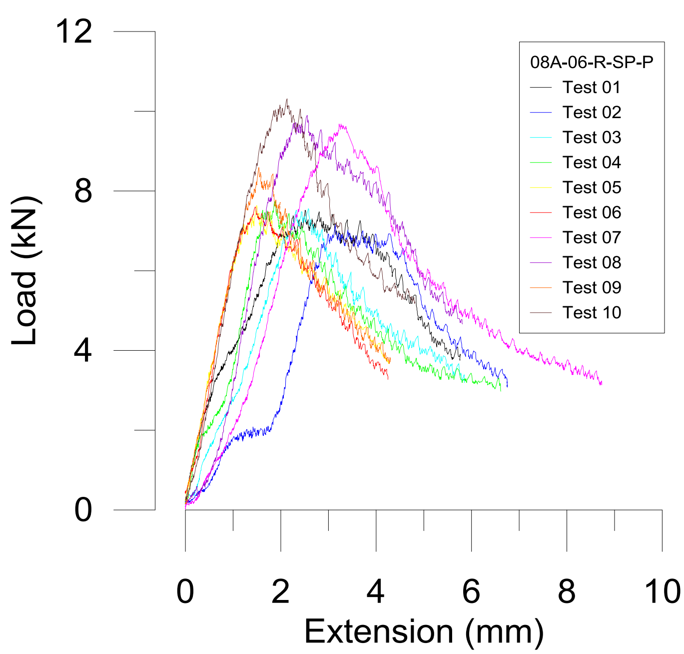


Figure B76 08A-06-SP-P load vs. extension for side screws

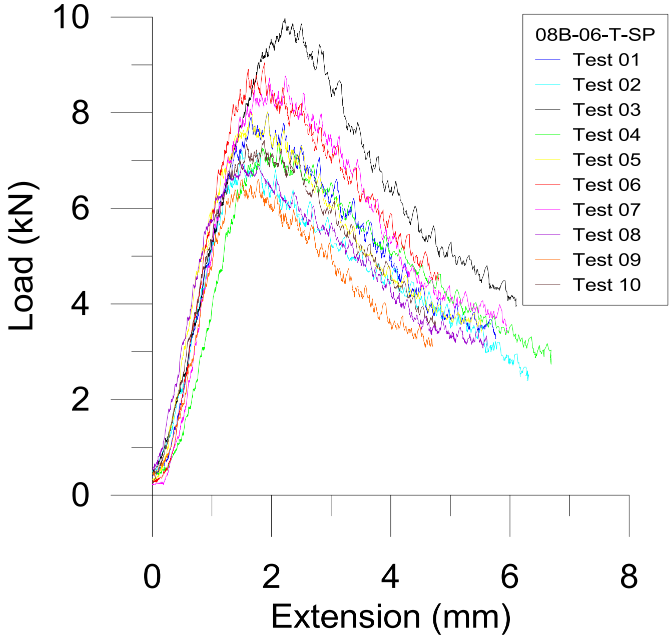


Figure B77 08B-06-SP load vs. extension for top screws

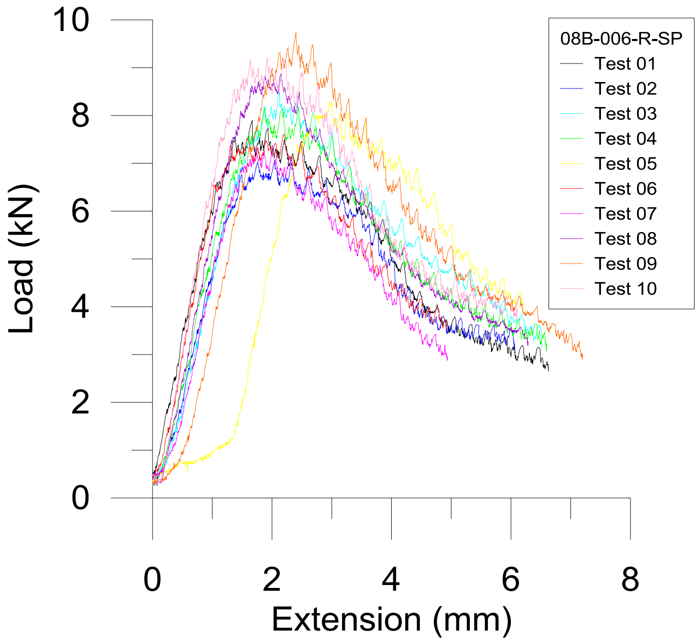


Figure B78 08B-06-SP load vs. extension for side screws

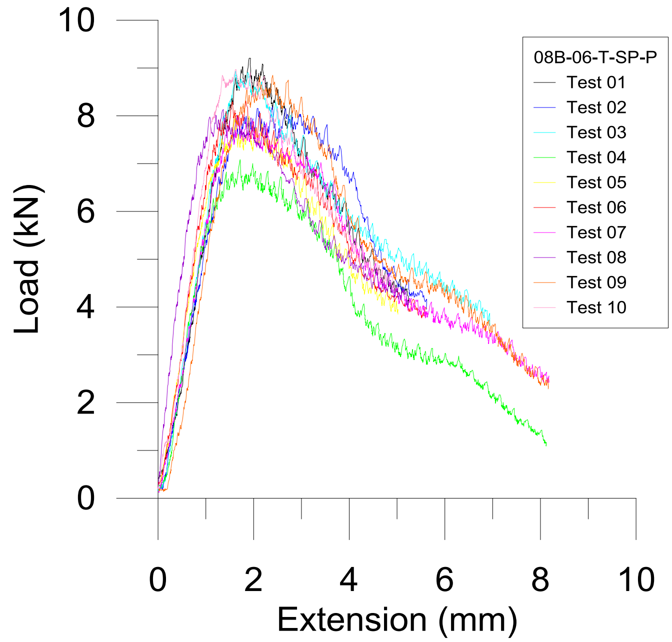


Figure B79 08B-06-SP-P load vs. extension for top screws

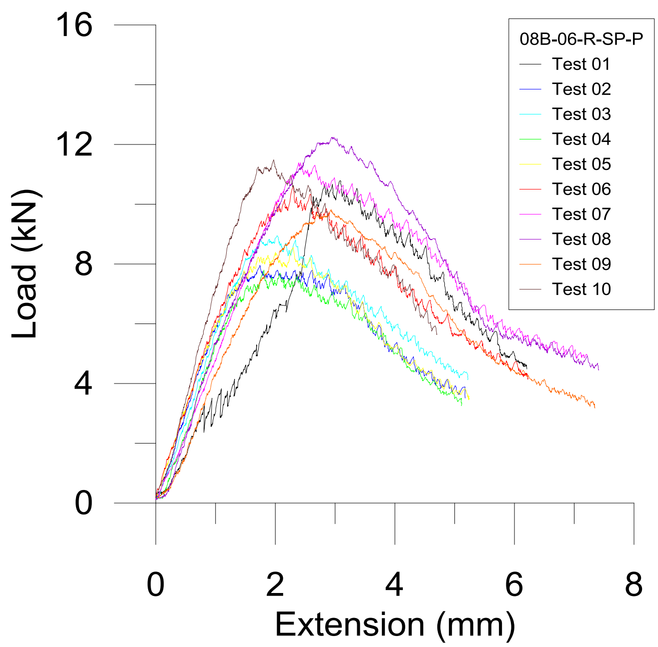


Figure B80 08B-06-SP-P load vs. extension for side screws

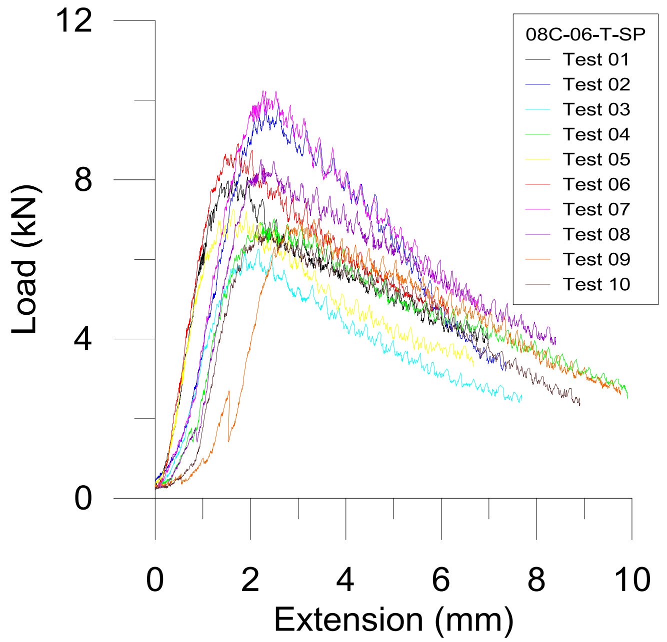


Figure B81 08C-06-SP load vs. extension for top screws

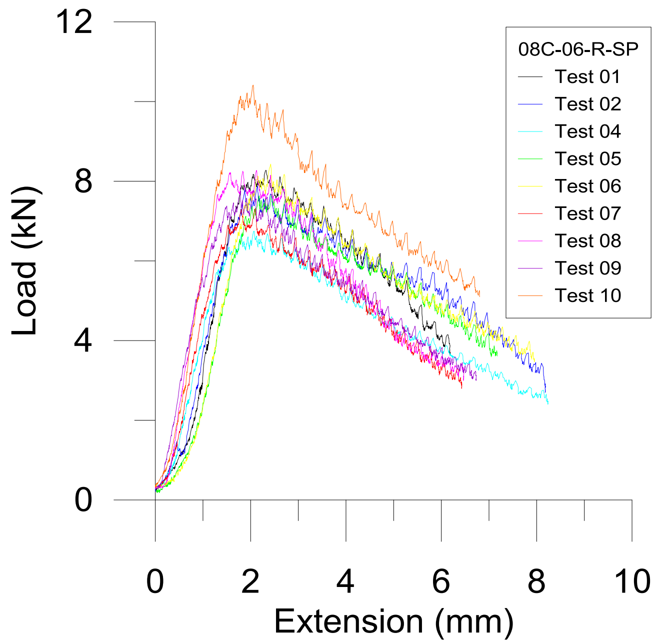


Figure B82 08C-06-SP load vs. extension for side screws

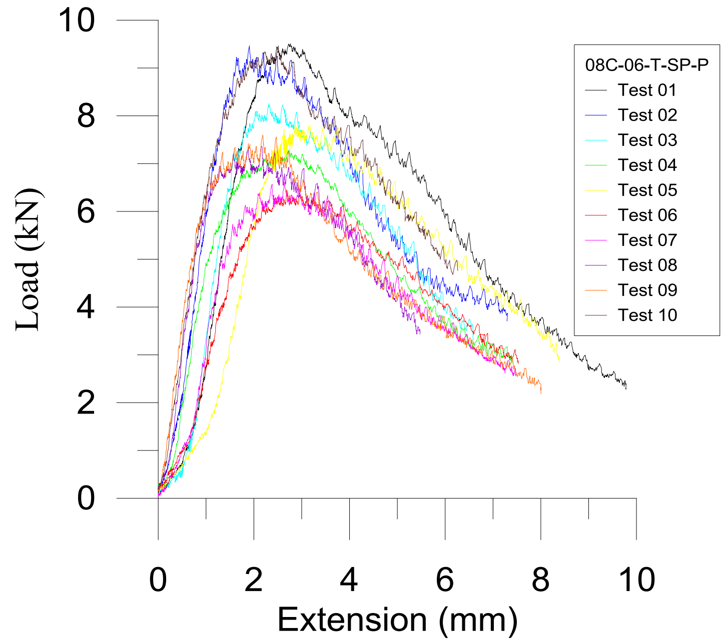


Figure B83 08C-06-SP-P load vs. extension for top screws

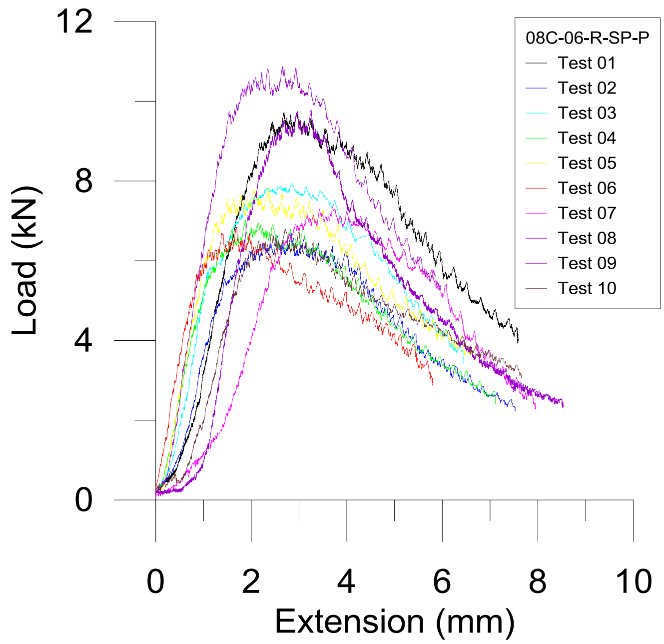


Figure B84 08C-06-SP-P load vs. extension for side screws

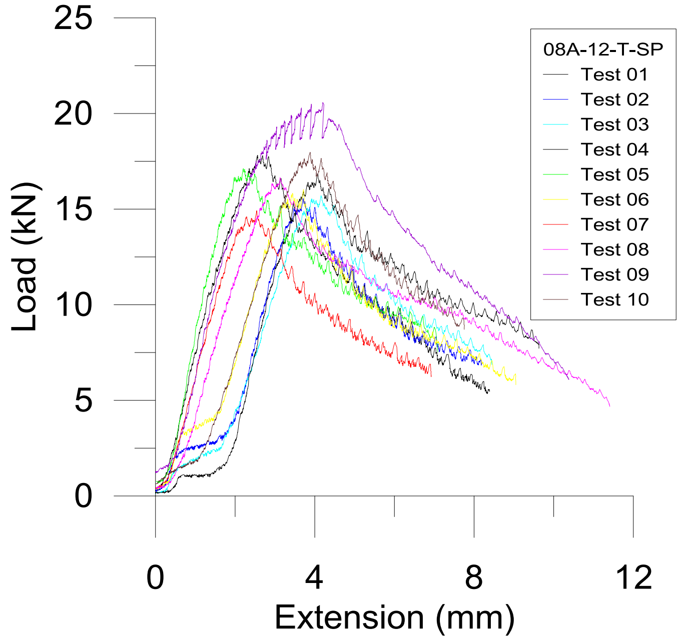


Figure B85 08A-12-SP load vs. extension for top screws

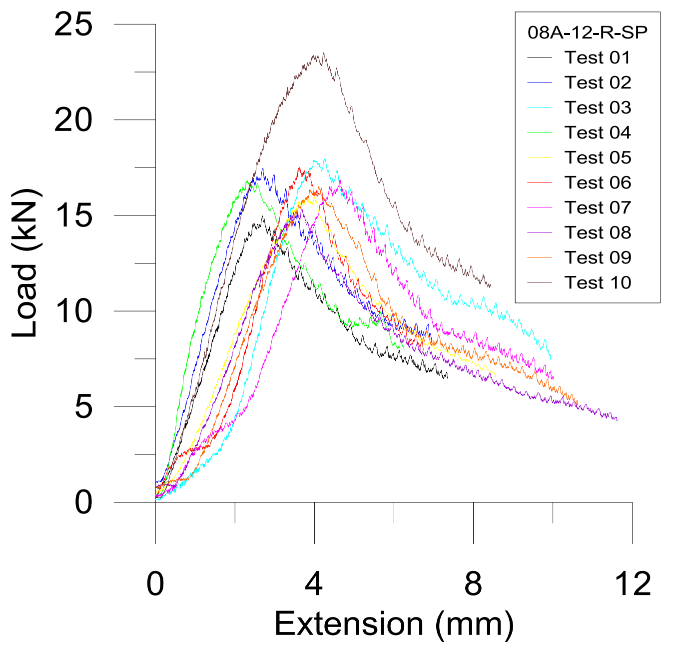


Figure B86 08A-12-SP load vs. extension for side screws

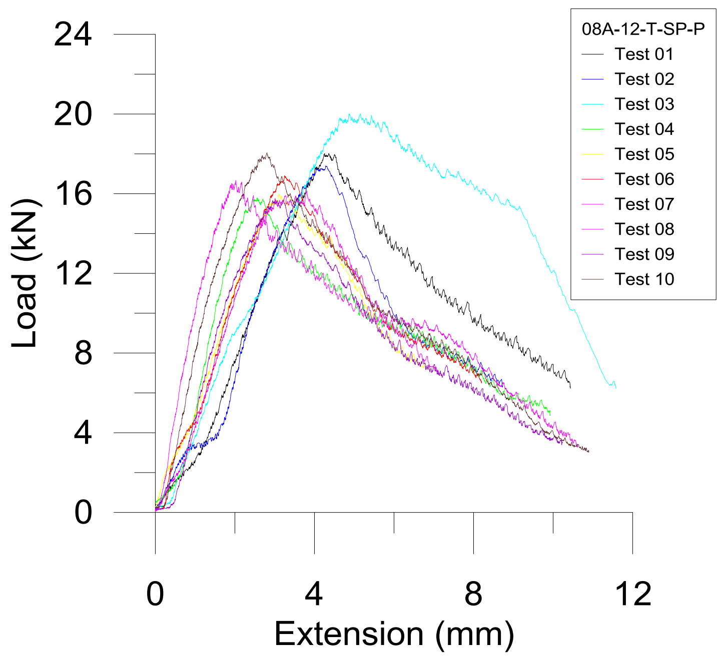


Figure B87 08A-12-SP-P load vs. extension for top screws

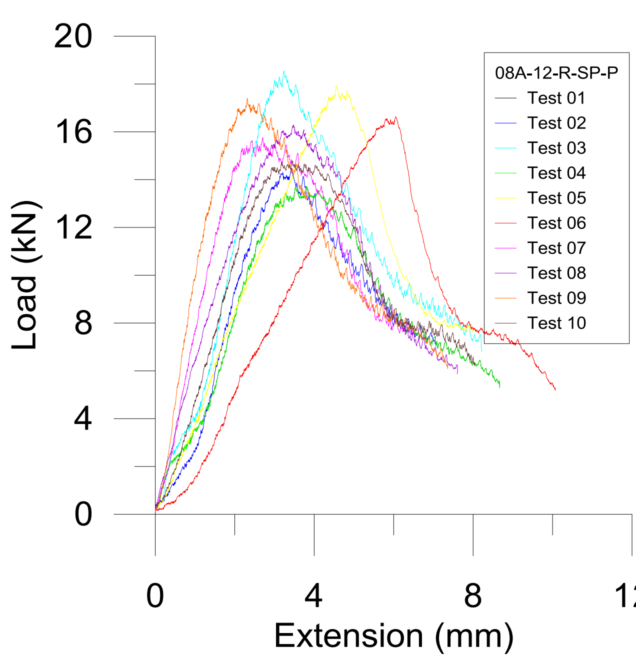


Figure B88 08A-12-SP-P load vs. extension for side screws

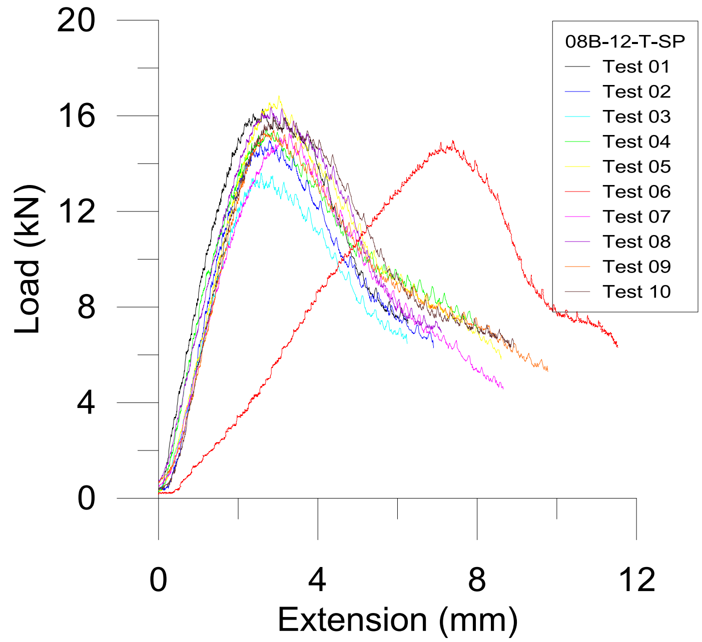


Figure B89 08B-12-SP load vs. extension for top screws

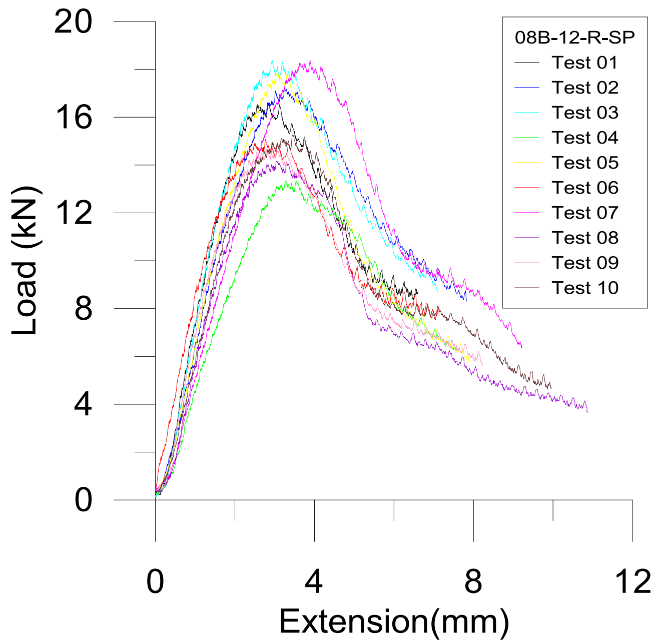


Figure B90 08B-12-SP load vs. extension for side screws

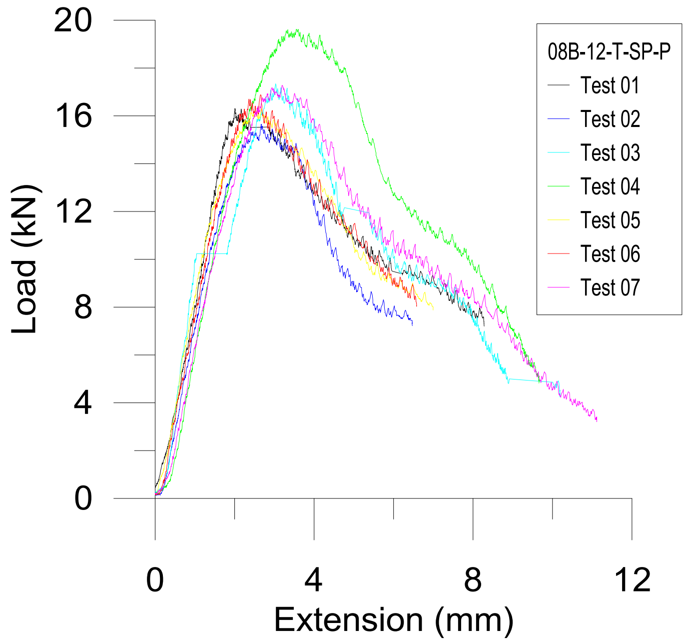


Figure B91 08B-12-SP-P load vs. extension for top screws

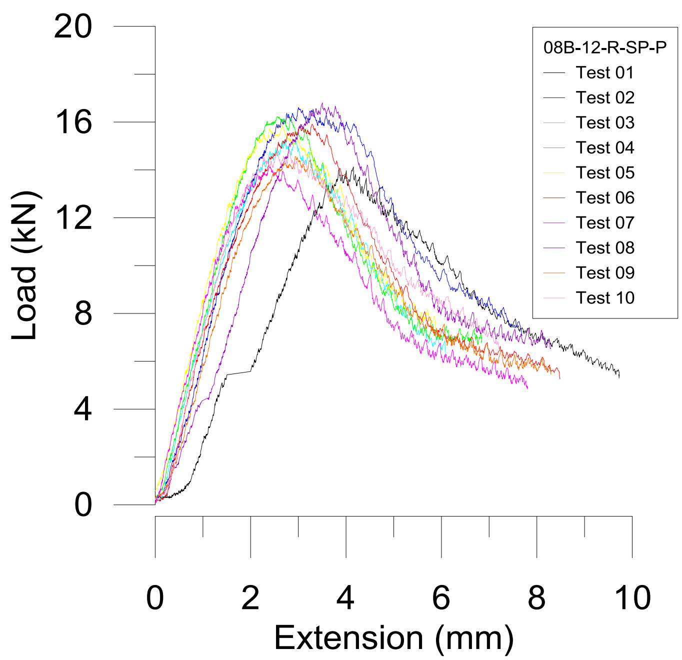


Figure B92 08B-12-SP-P load vs. extension for side screws

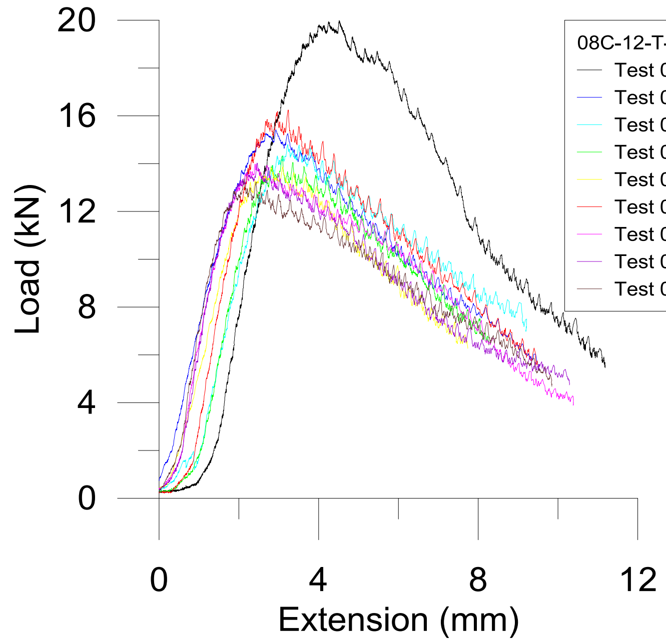


Figure B93 08C-12-R-SP load vs. extension for top screws

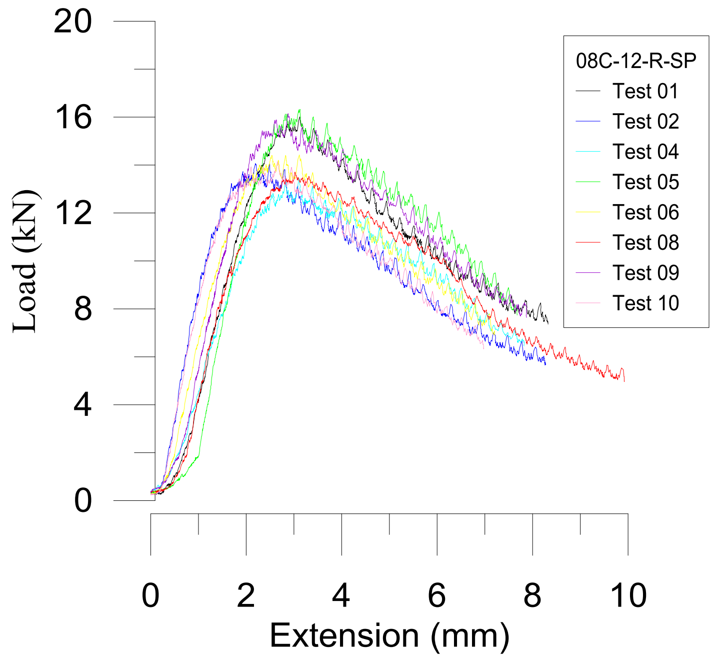


Figure B94 08C-12-R-SP load vs. extension for side screws

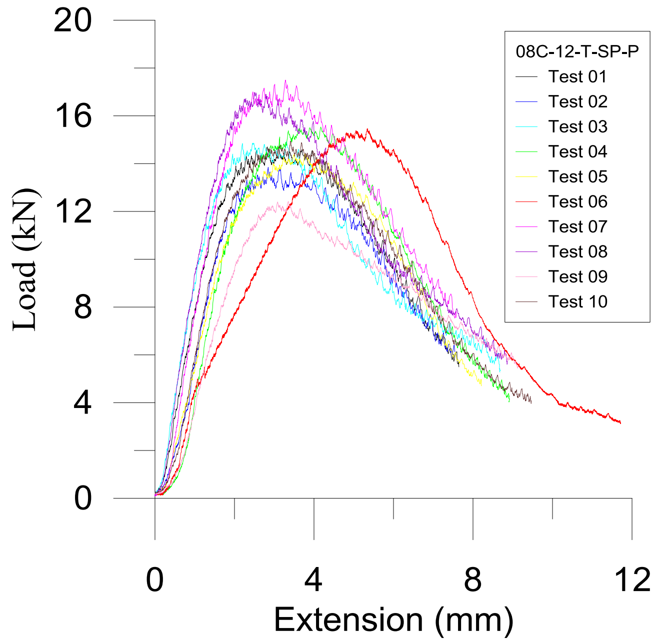


Figure B95 08C-12-SP-P load vs. extension for top screws

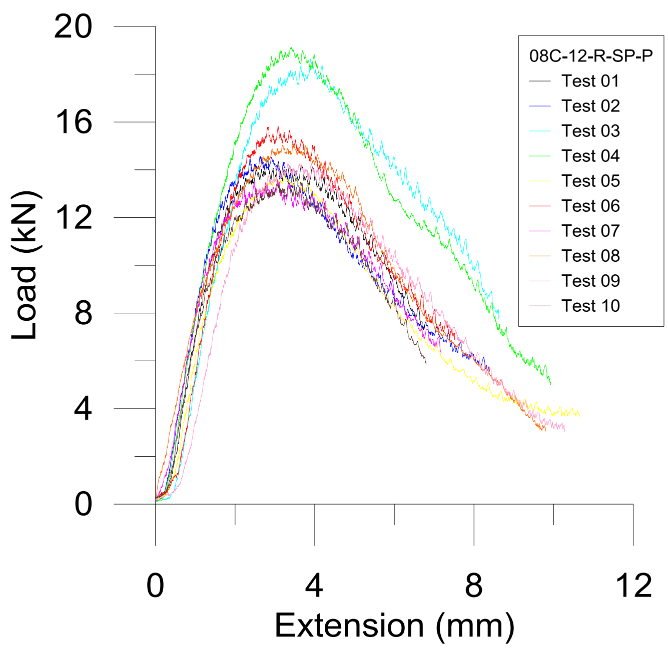


Figure B96 08C-12-SP-P load vs. extension for side screws

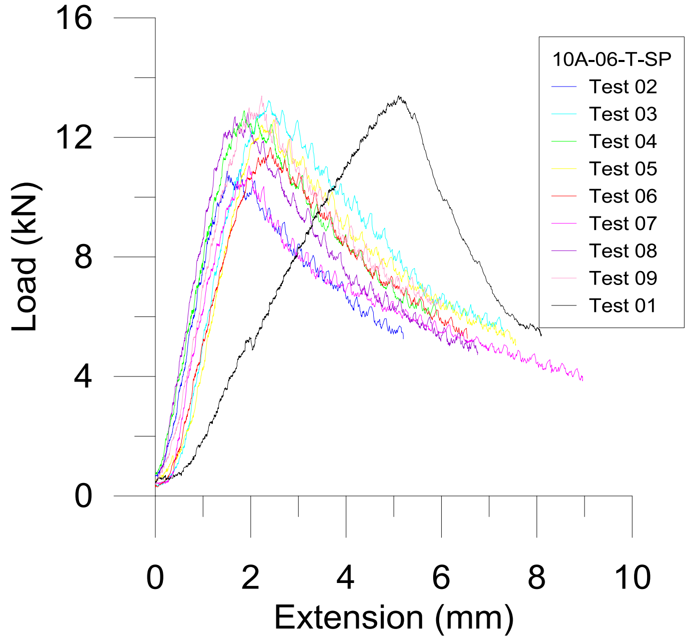


Figure B97 10A-06-SP load vs. extension for top screws

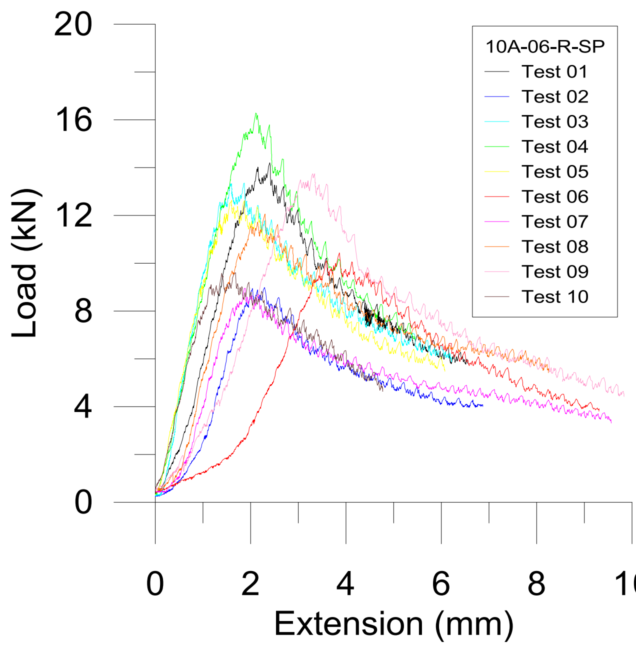


Figure B98 10A-06-SP load vs. extension for side screws

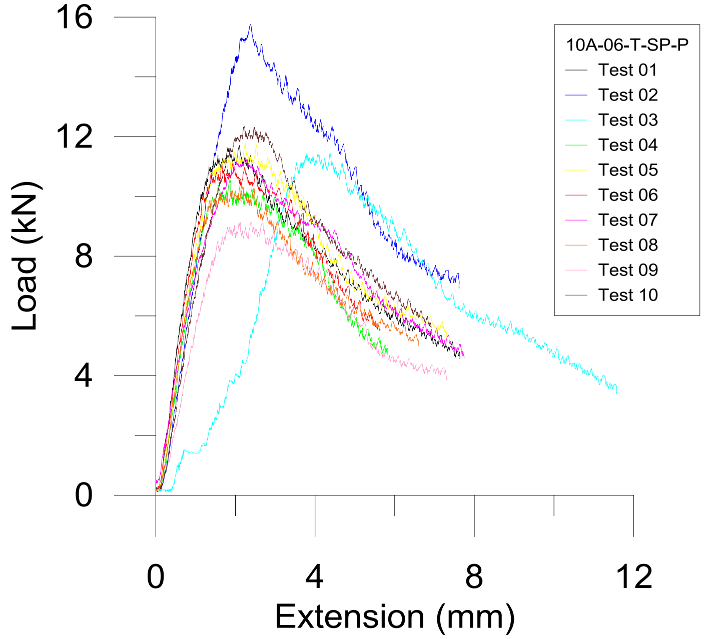


Figure B99 10A-06-SP-P load vs. extension for top screws

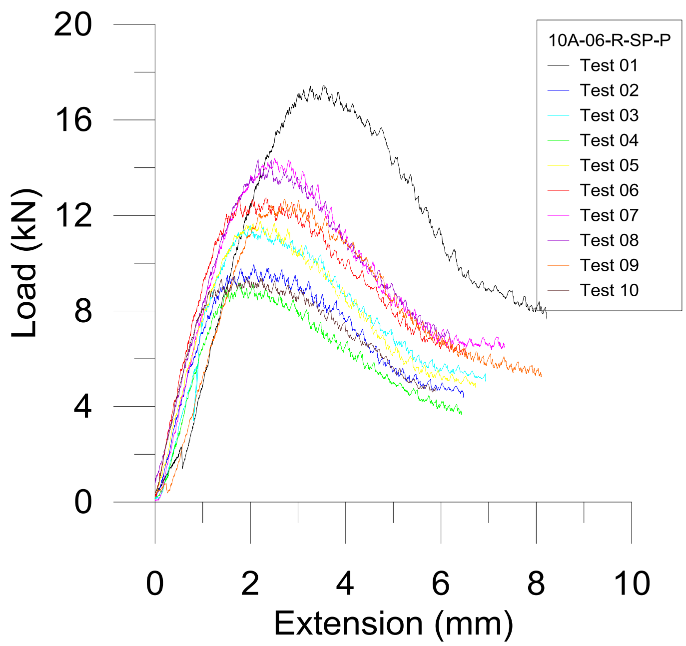


Figure B100 10A-06-SP-P load vs. extension for side screws

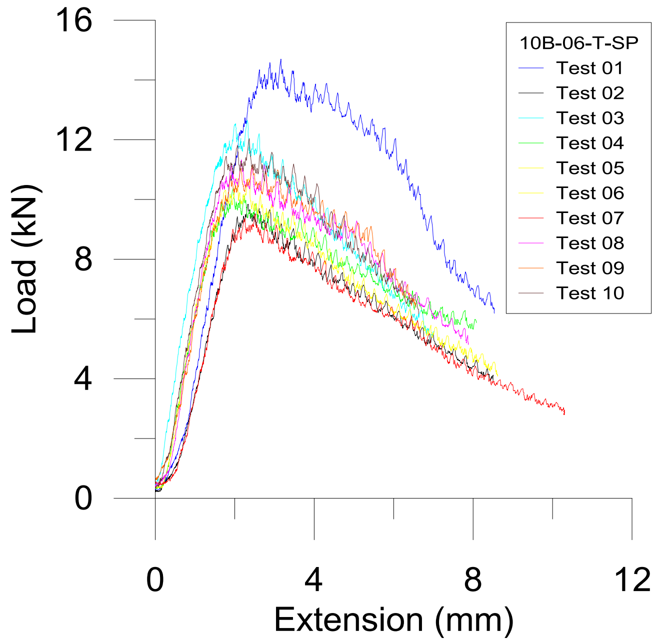


Figure B101 10B-06-SP load vs. extension for top screws

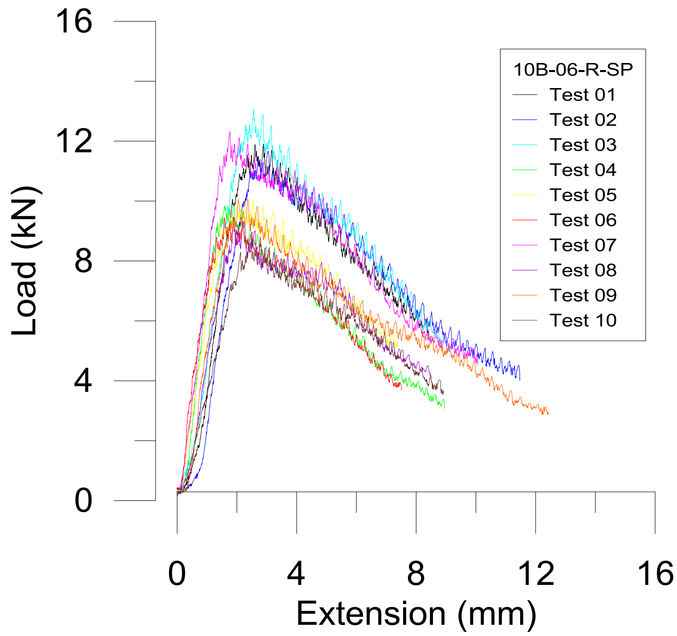


Figure B102 10B-06-SP load vs. extension for side screws

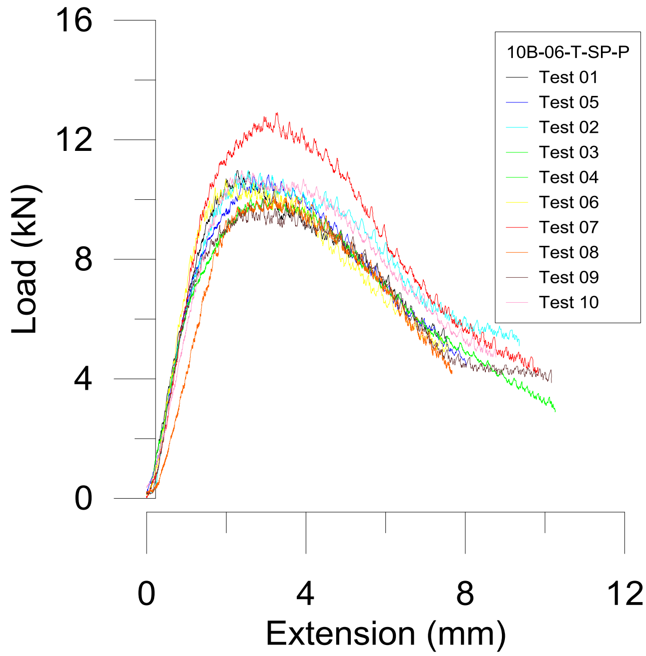


Figure B103 10B-06-SP-P load vs. extension for top screws

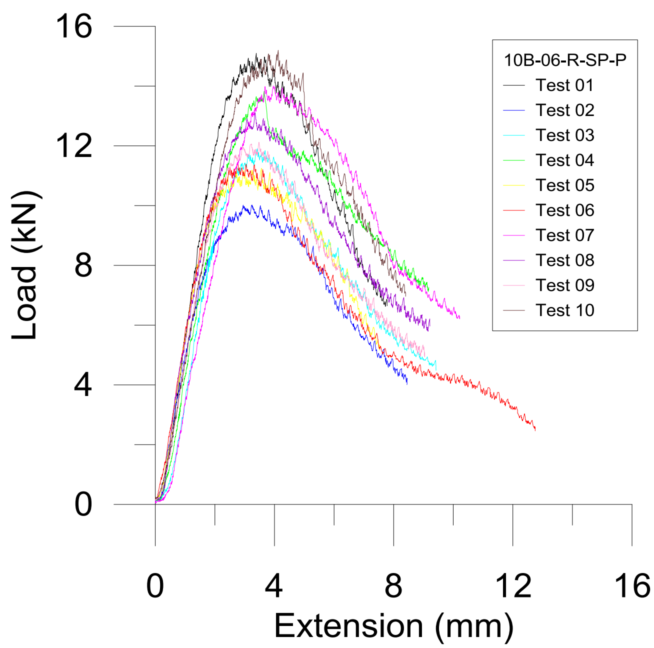


Figure B104 10B-06-SP-P load vs. extension for side screws

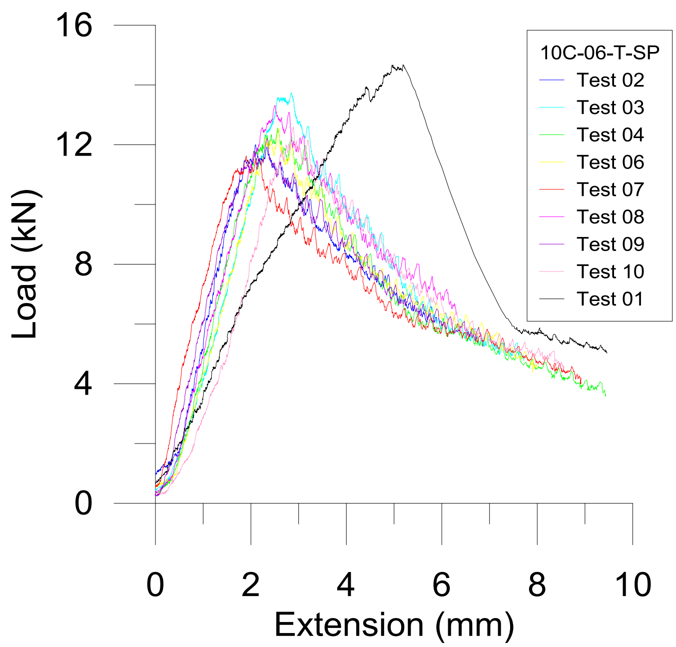


Figure B105 10C-06-SP load vs. extension for top screws

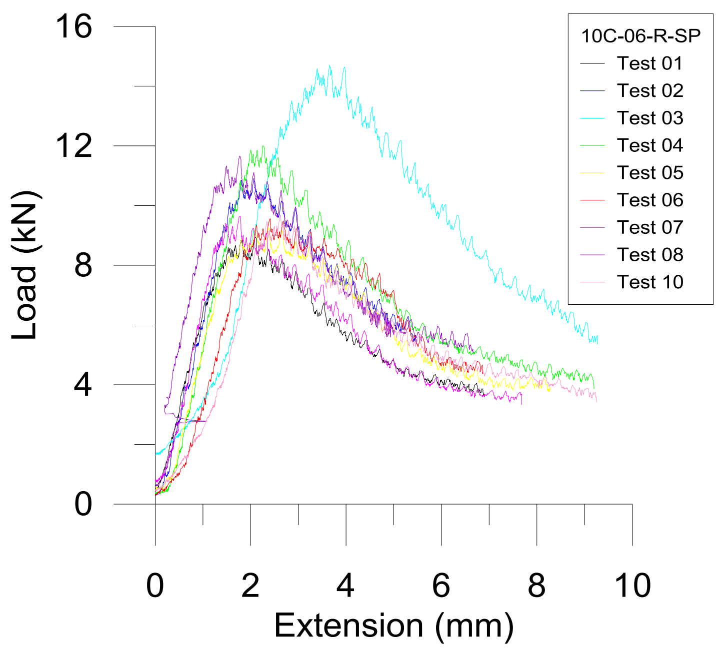


Figure B106 10C-06-SP load vs. extension for side screws

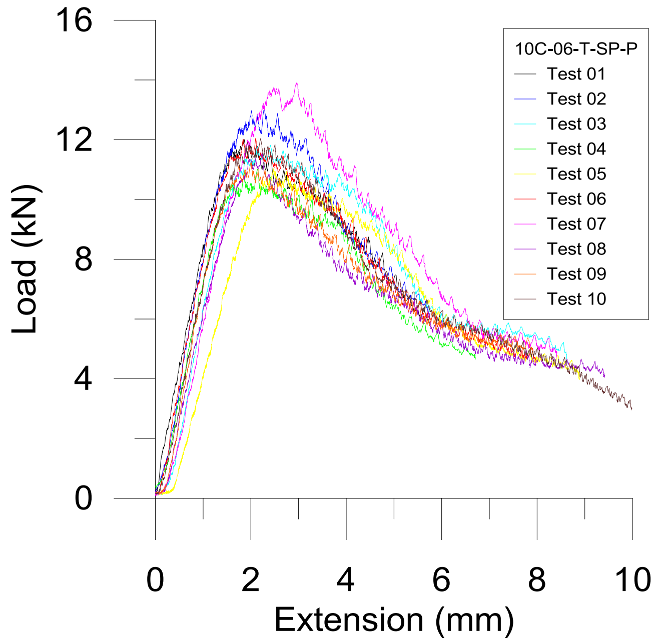


Figure B107 10C-06-SP-P load vs. extension for top screws

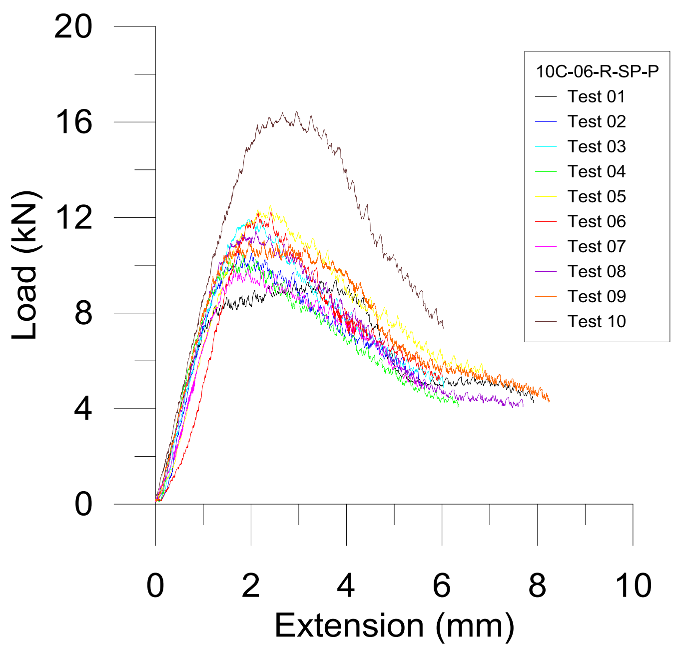


Figure B108 10C-06-SP-P load vs. extension for side screws

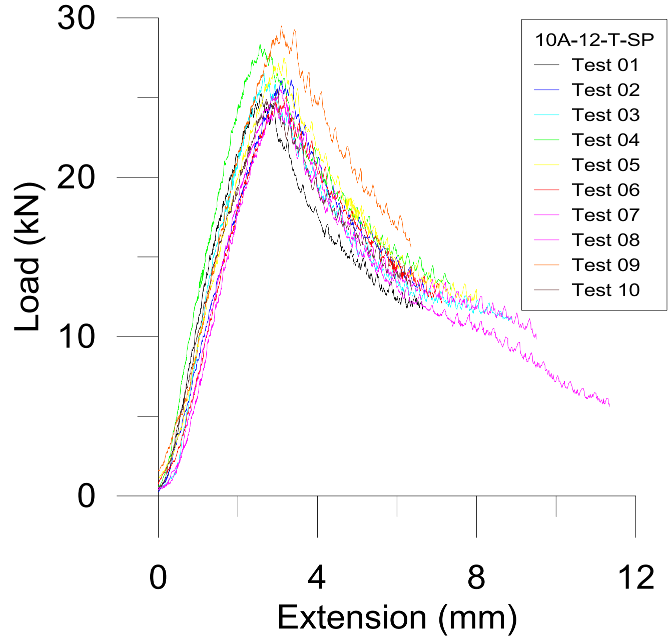


Figure B109 10A-12-SP load vs. extension for top screws

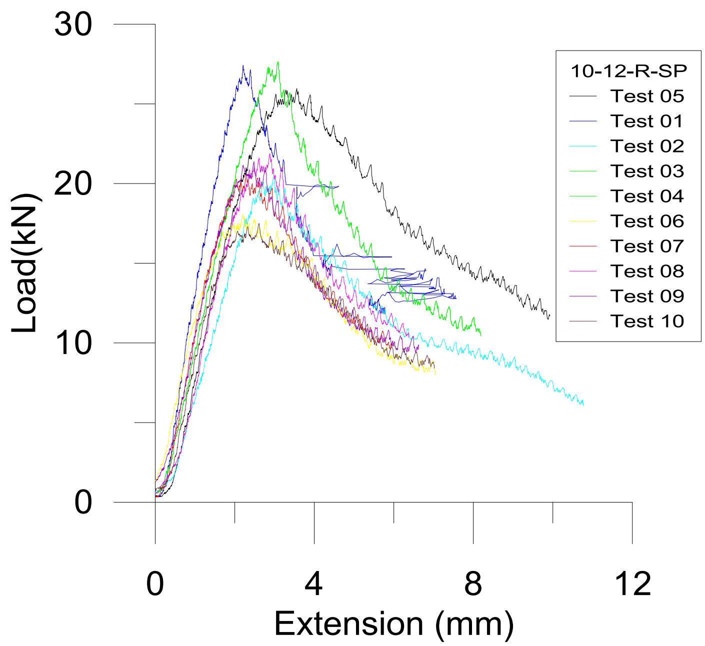


Figure B110 10A-12-SP load vs. extension for side screws

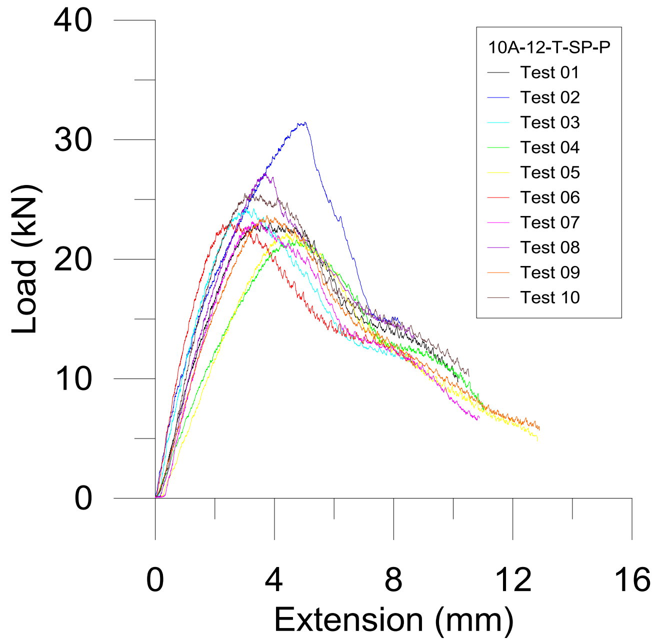


Figure B111 10A-12-SP-P load vs. extension for top screws

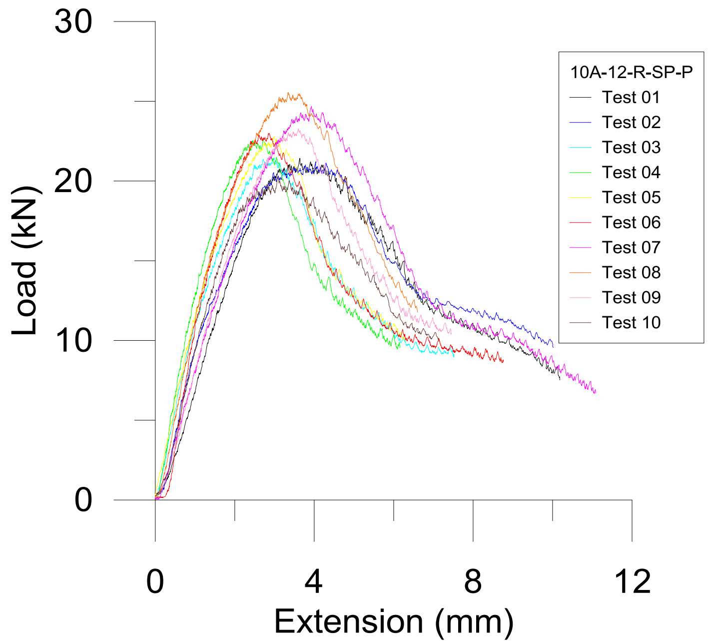


Figure B112 10A-12-SP-P load vs. extension for side screws

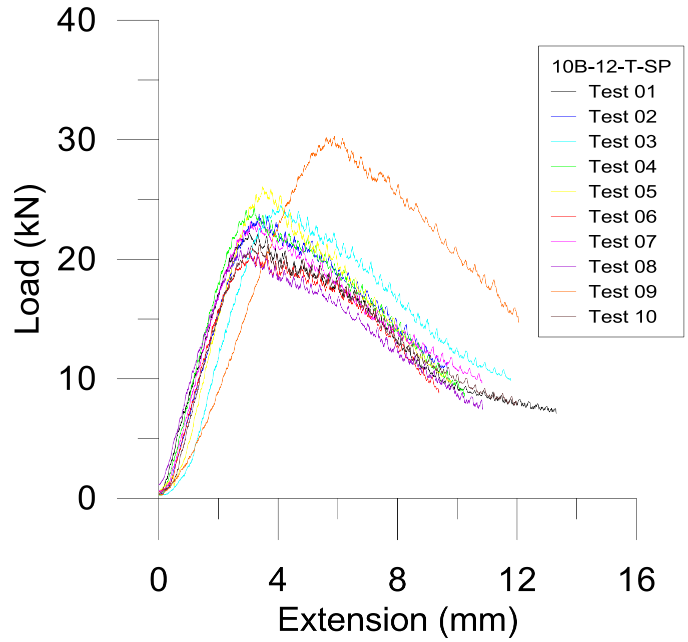


Figure B113 10B-12-SP load vs. extension for top screws

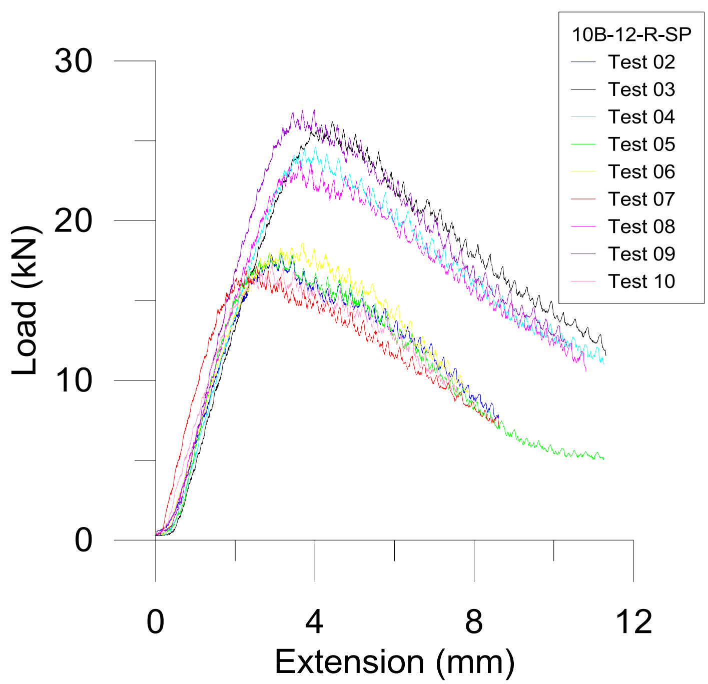


Figure B114 10B-12-SP load vs. extension for side screws

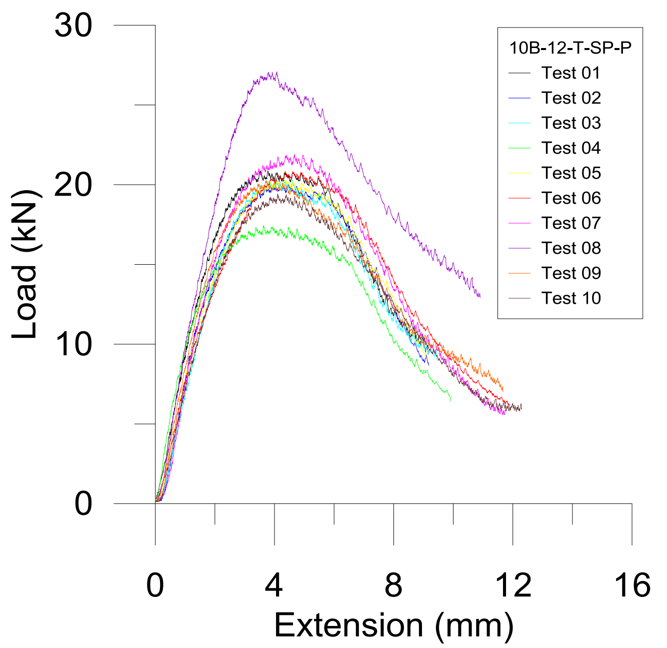


Figure B115 10B-12-SP-P load vs. extension for top screws

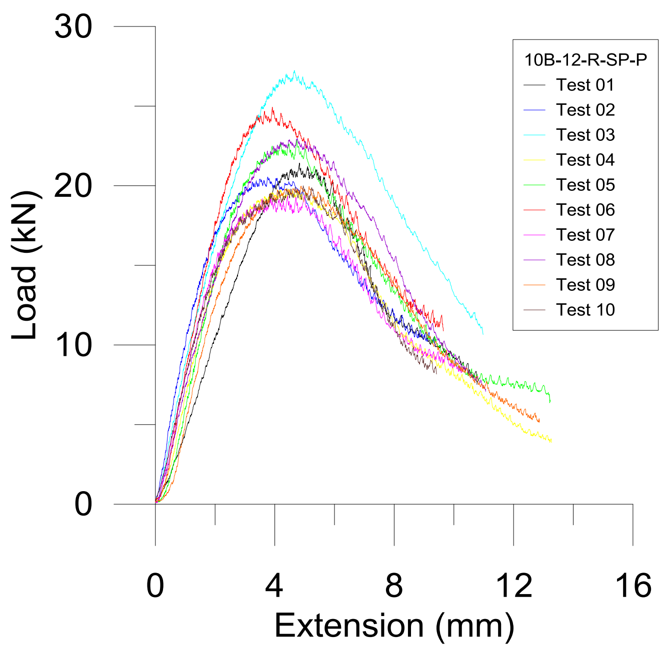


Figure B116 10B-12-SP-P load vs. extension for side screws

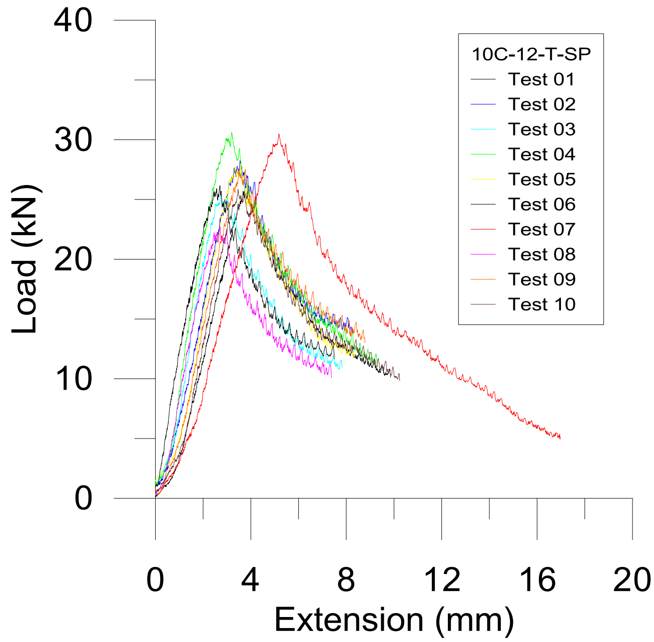


Figure B117 10C-12-SP load vs. extension for top screws

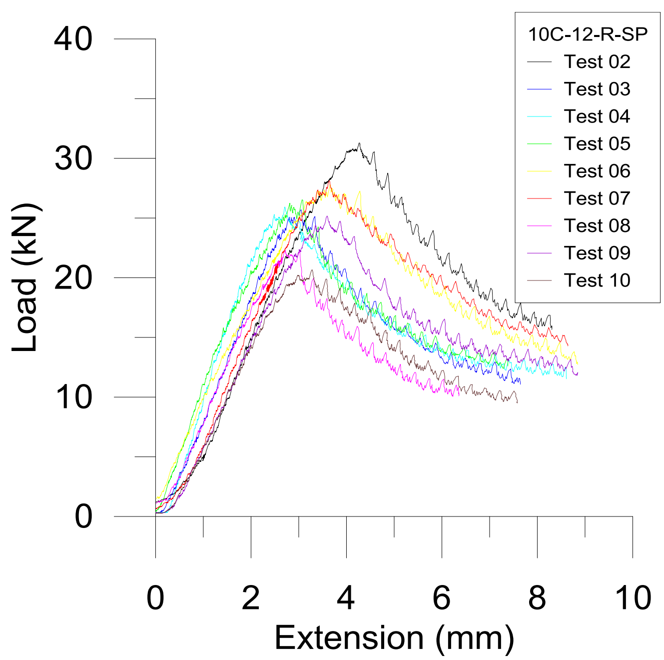


Figure B118 10C-12-SP load vs. extension for side screws

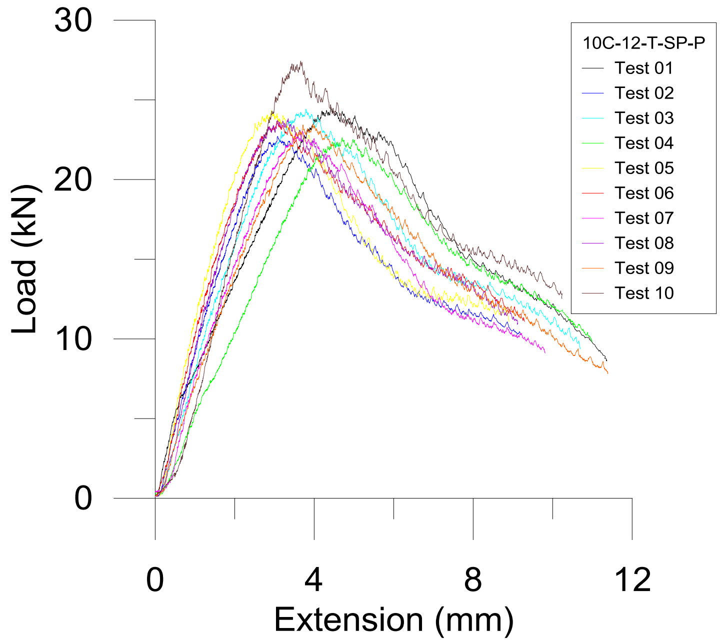


Figure B119 10C-12-SP-P load vs. extension for top screws

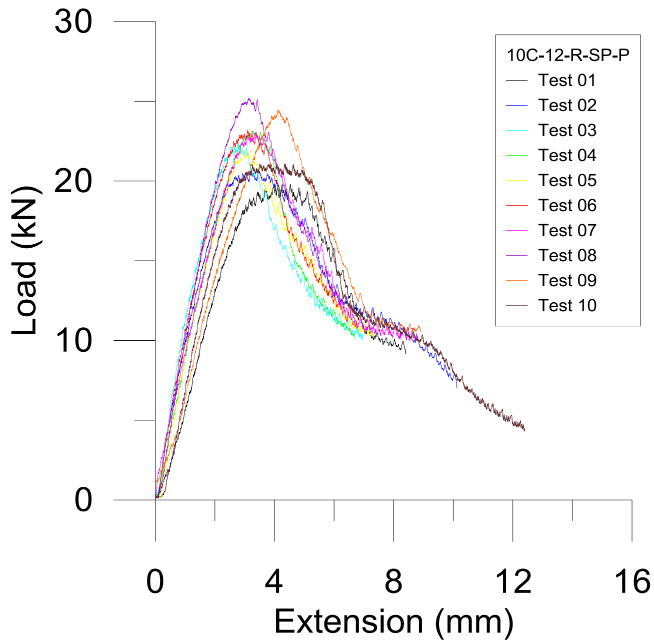


Figure B120 10C-12-SP-P load vs. extension for side screws

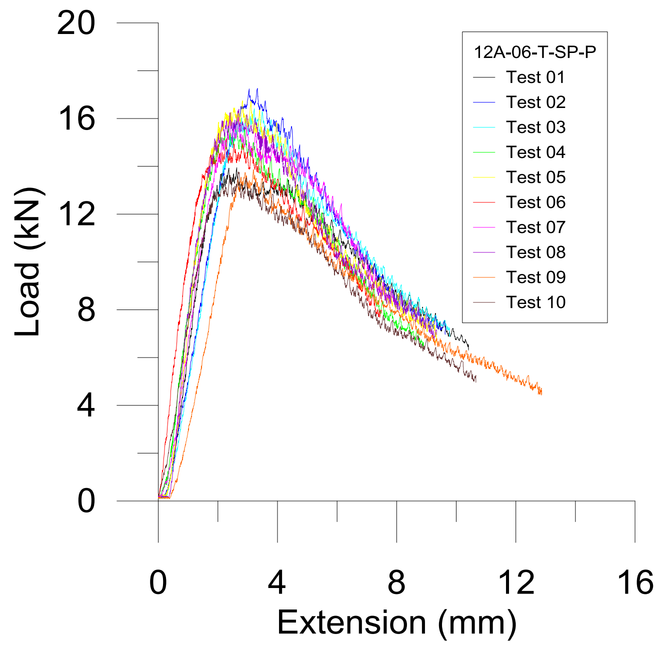


Figure B121 12A-06-SP-P load vs. extension for top screws

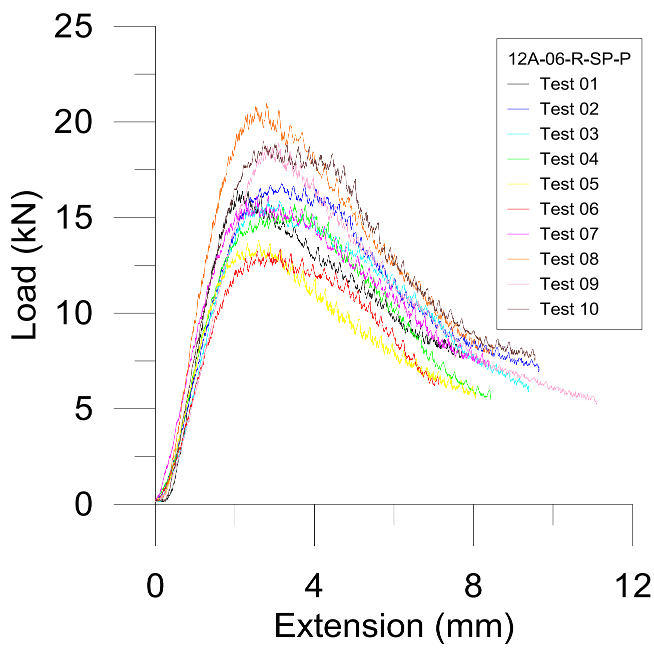


Figure B122 12A-06-SP-P load vs. extension for side screws

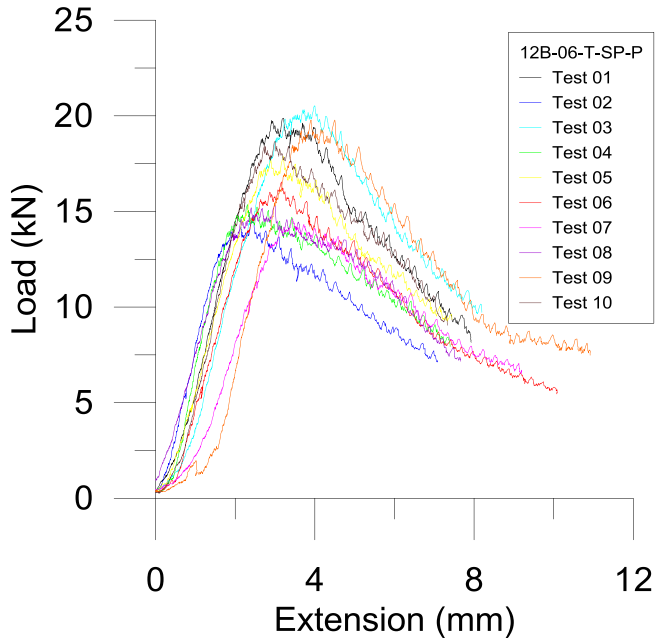


Figure B123 12B-06-SP-P load vs. extension for top screws

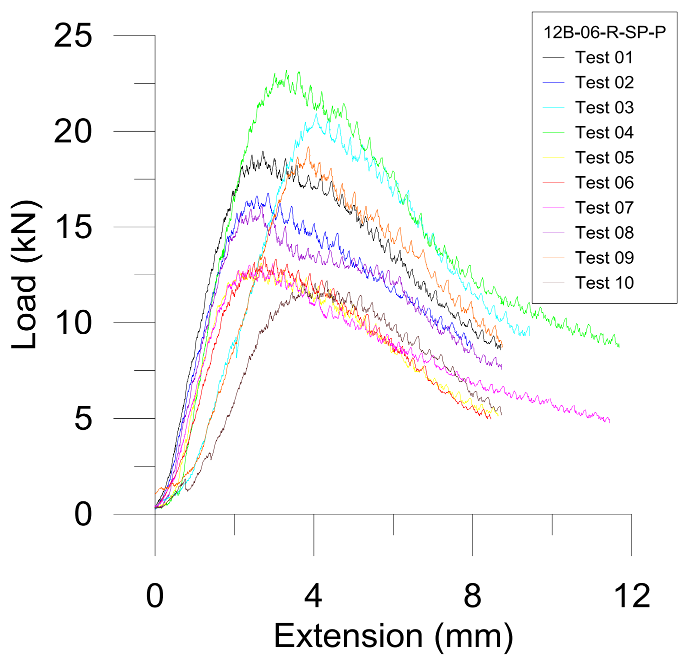


Figure B124 12B-06-SP-P load vs. extension for side screws

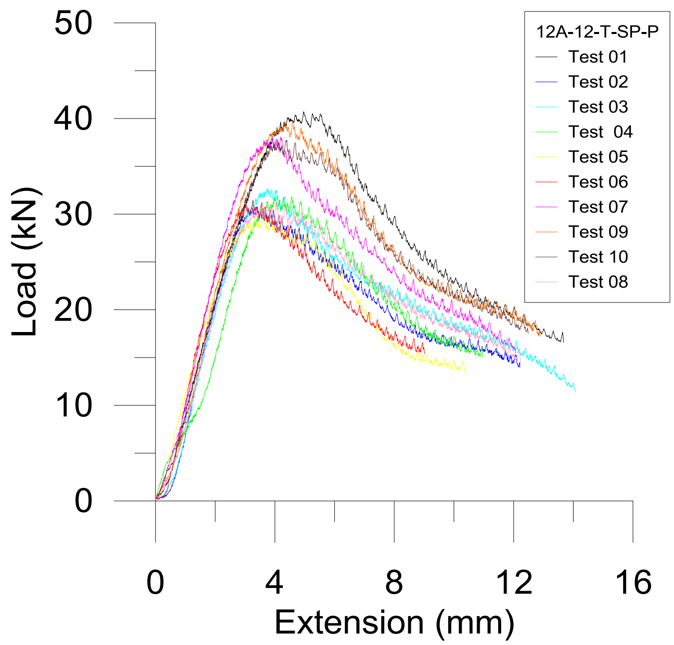


Figure B125 12A-12-SP-P load vs. extension for top screws

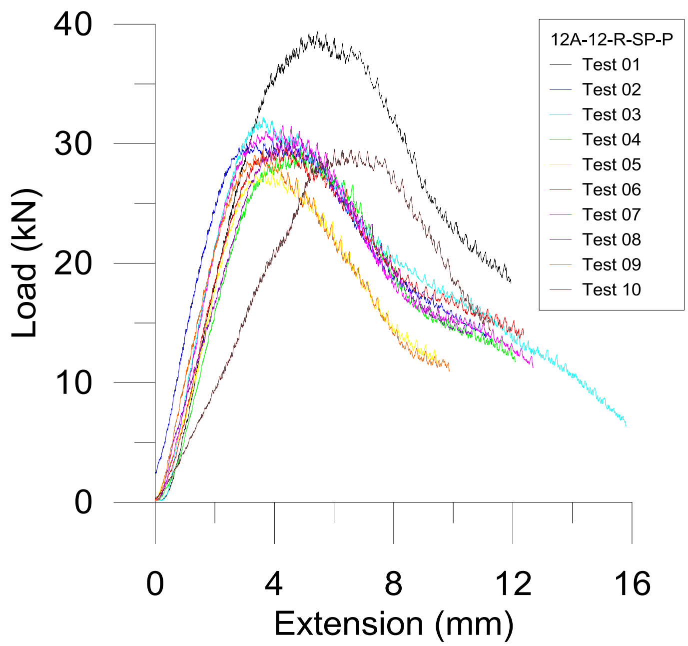


Figure B126 12A-12-SP-P load vs. extension for side screws

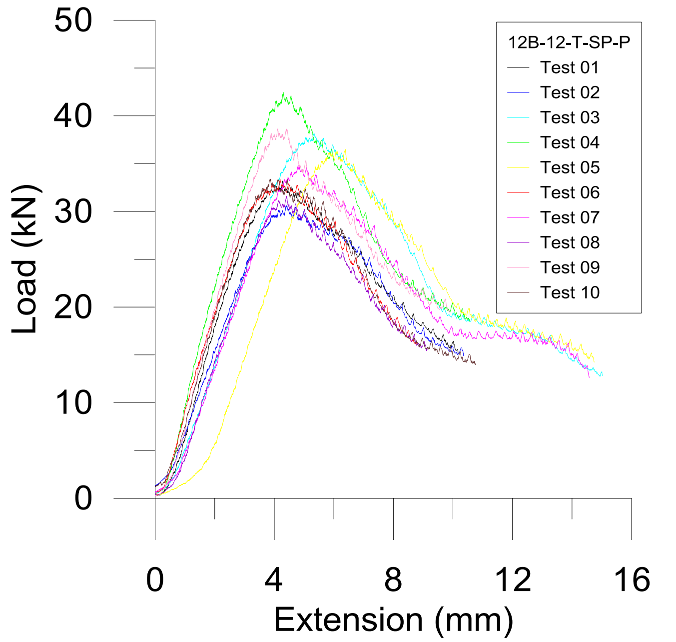


Figure B127 12B-12-SP-P load vs. extension for top screws

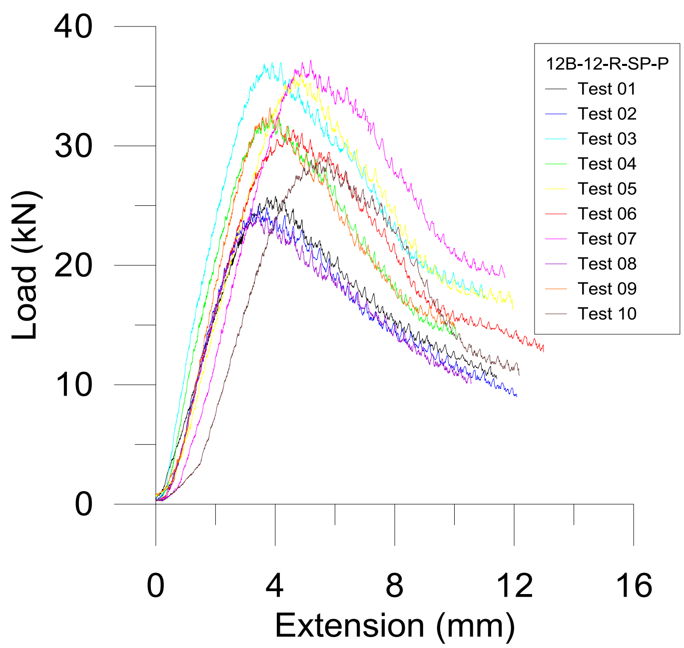


Figure B128 12B-12-SP-P load vs. extension for side screws

## Nordic Lam Glulam

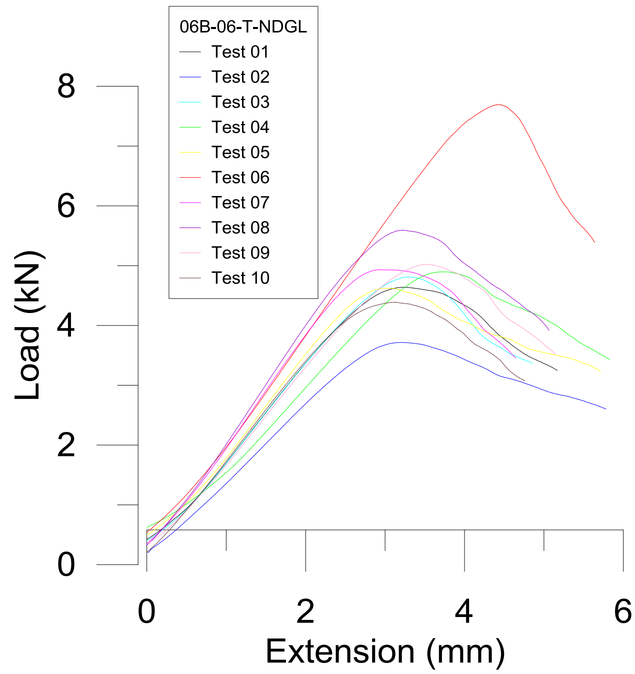


Figure B129 06B-06-NDGL load vs. extension for top screws

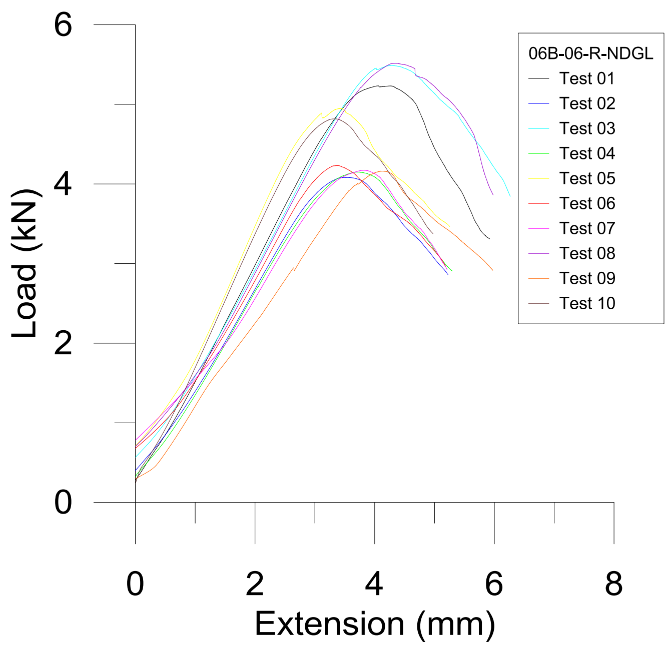


Figure B130 06B-06-NDGL load vs. extension for side screws

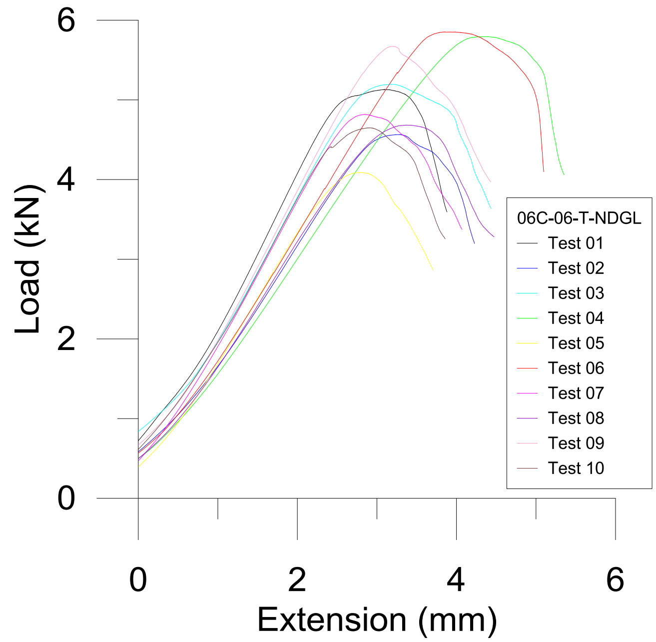


Figure B131 06C-06-NDGL load vs. extension for top screws

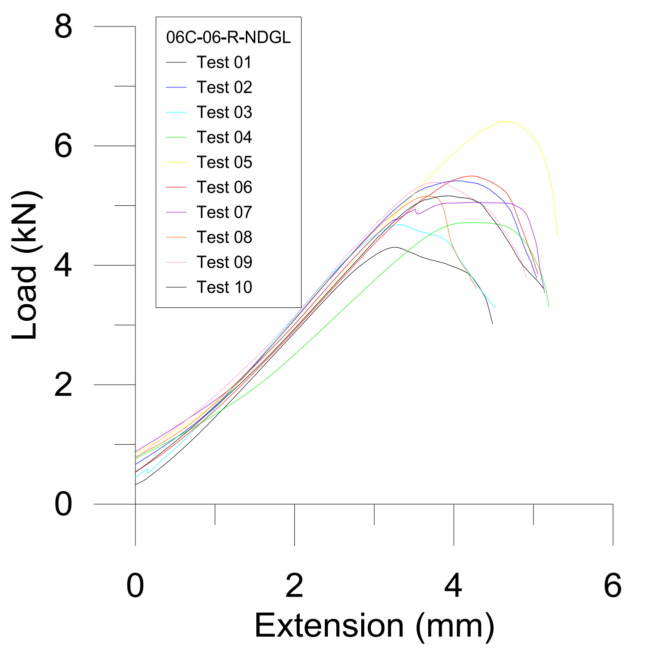


Figure B132 06C-06-NDGL load vs. extension for side screws

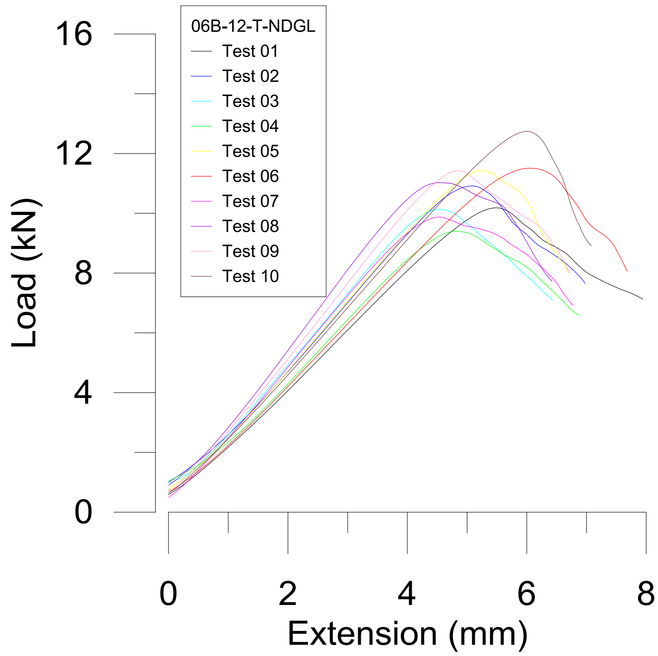


Figure B133 06B-12-NDGL load vs. extension for top screws

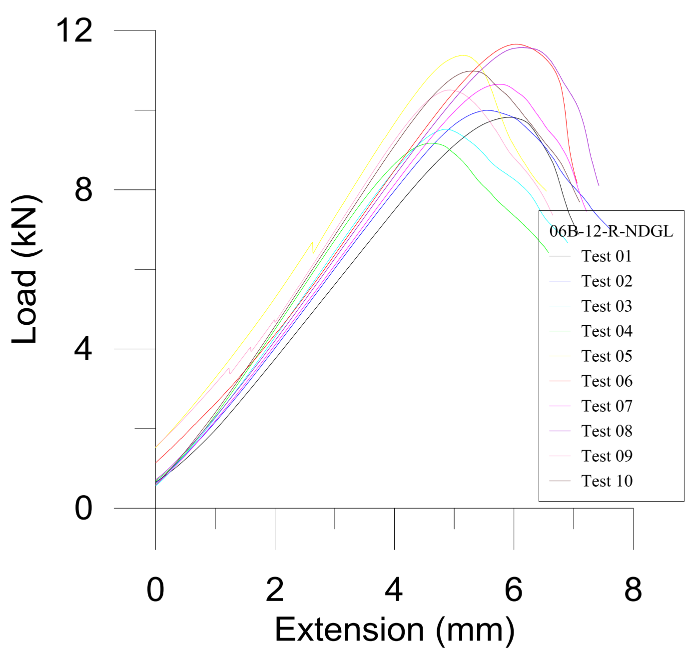


Figure B134 06B-12-NDGL load vs. extension for side screws

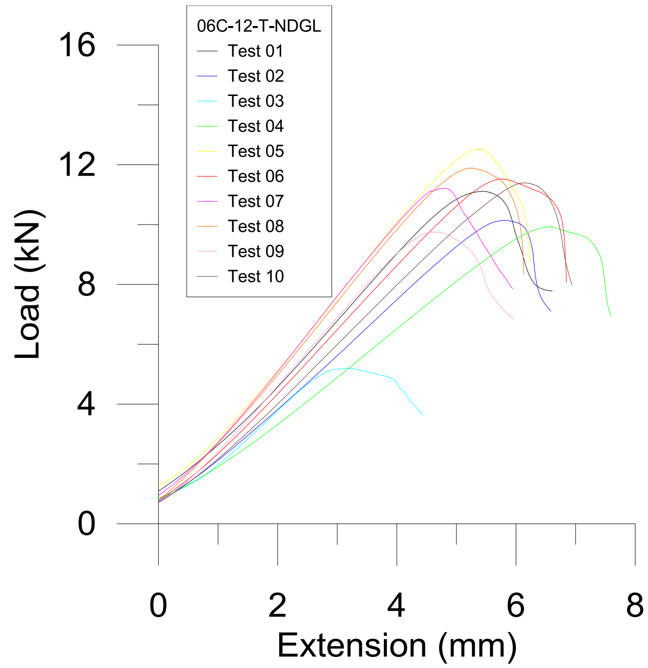


Figure B135 06C-12-NDGL load vs. extension for top screws

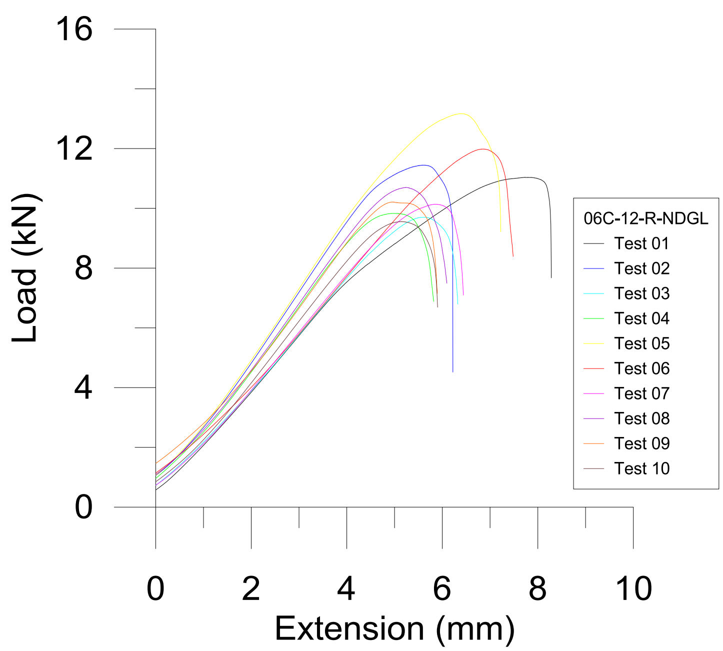


Figure B136 06C-12-NDGL load vs. extension for side screws

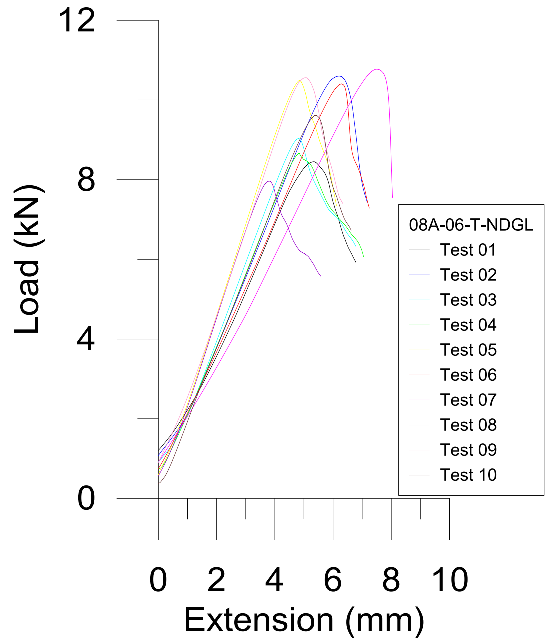


Figure B137 08A-06-NDGL load vs. extension for top screws

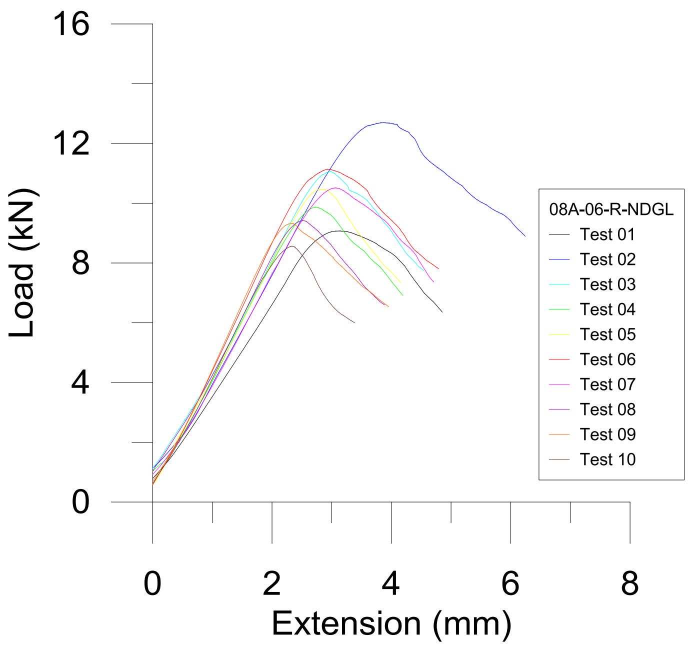


Figure B138 08A-06-NDGL load vs. extension for side screws

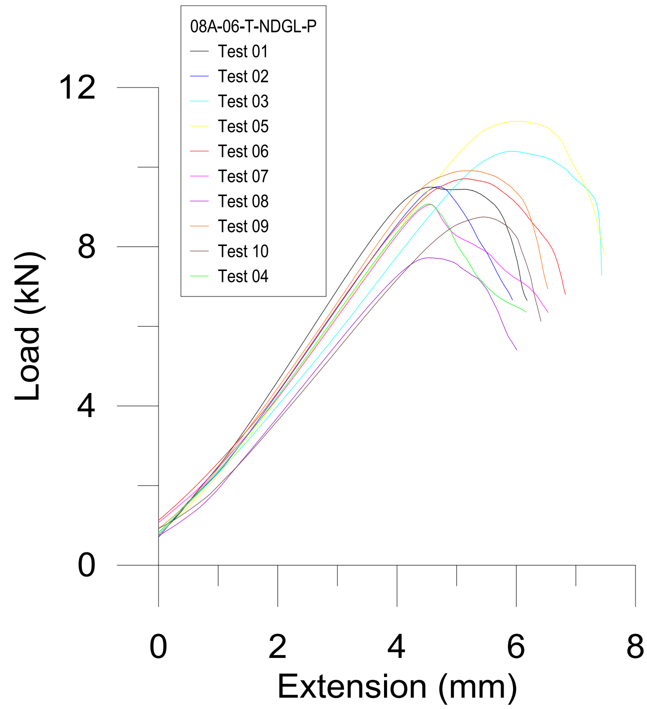


Figure B139 08A-06-NDGL-P load vs. extension for top screws

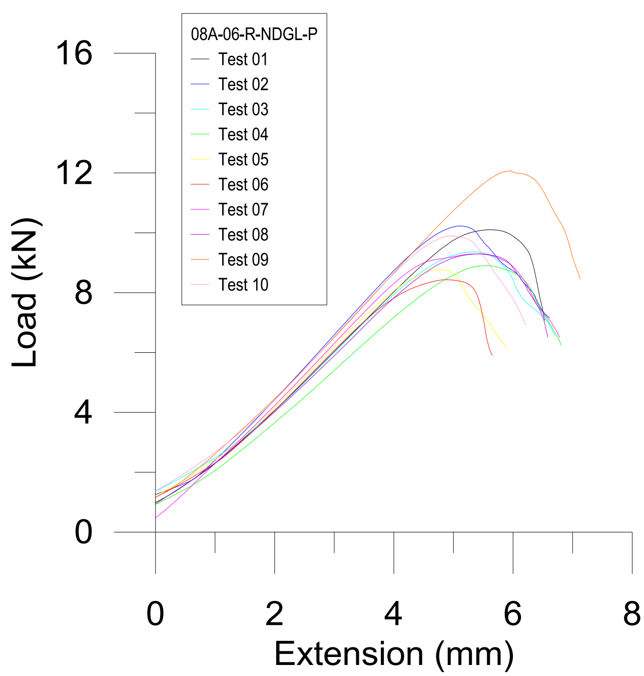


Figure B140 08A-06-NDGL-P load vs. extension for side screws

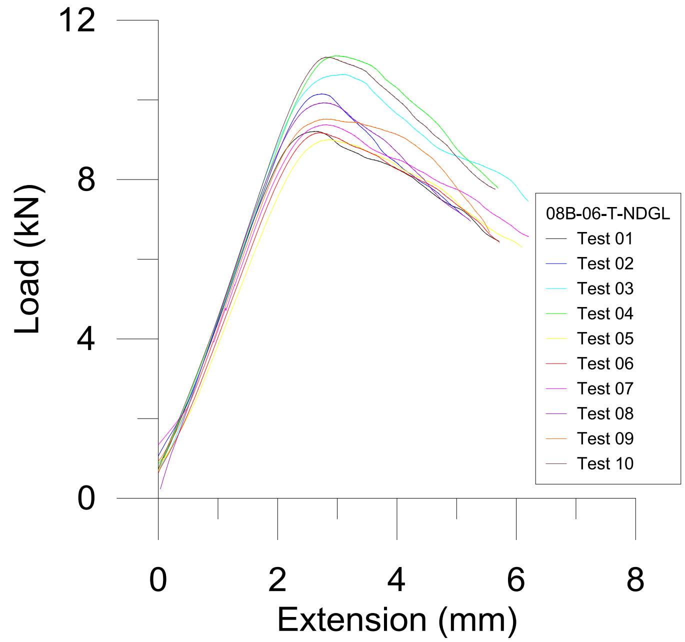


Figure B141 08B-06-NDGL-P load vs. extension for a) top screws and b) side screws

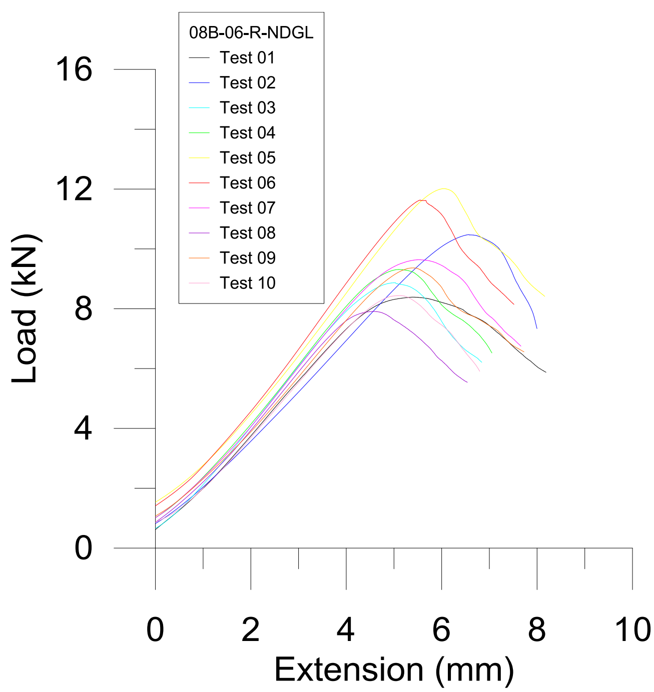


Figure B142 08B-06-NDGL-P load vs. extension for a) top screws and b) side screws

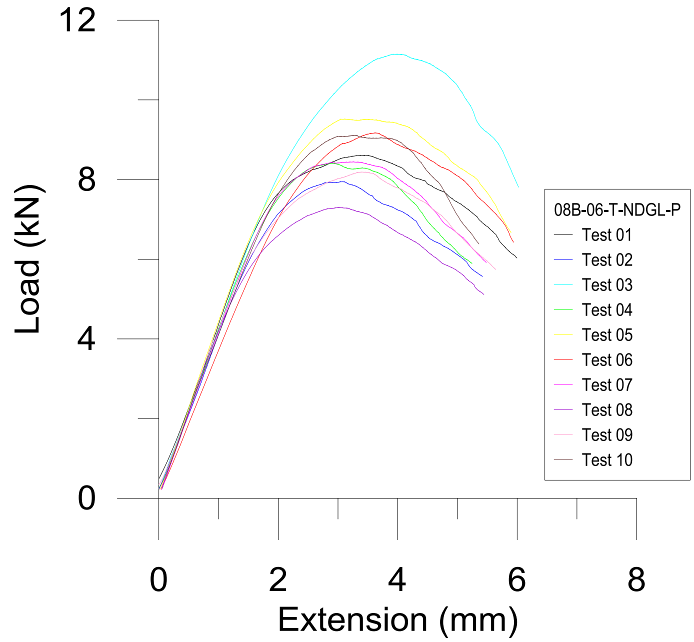


Figure B143 08B-06-NDGL-P load vs. extension for top screws

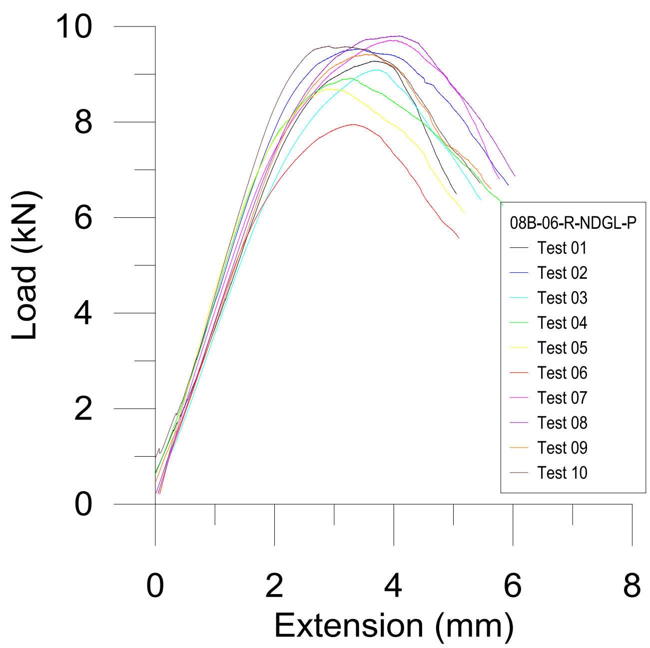


Figure B144 08B-06-NDGL-P load vs. extension for side screws

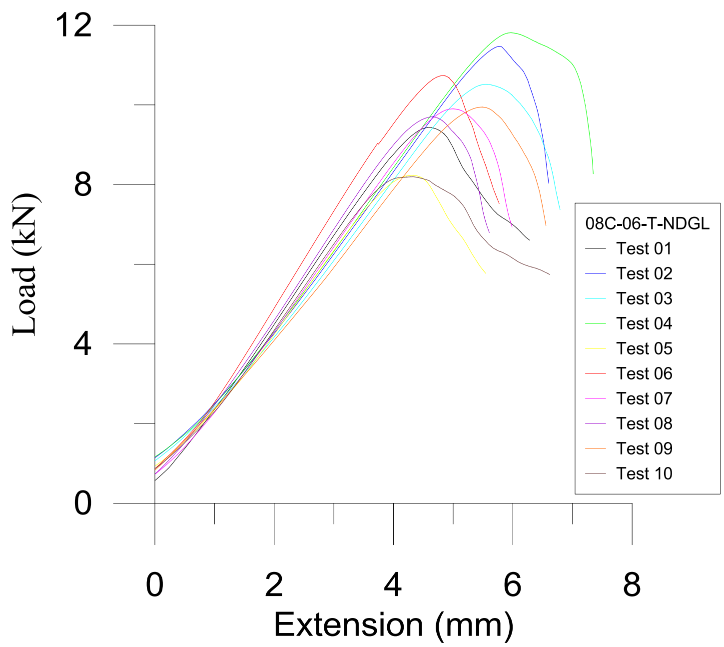


Figure B145 08C-06-NDGL load vs. extension for top screws

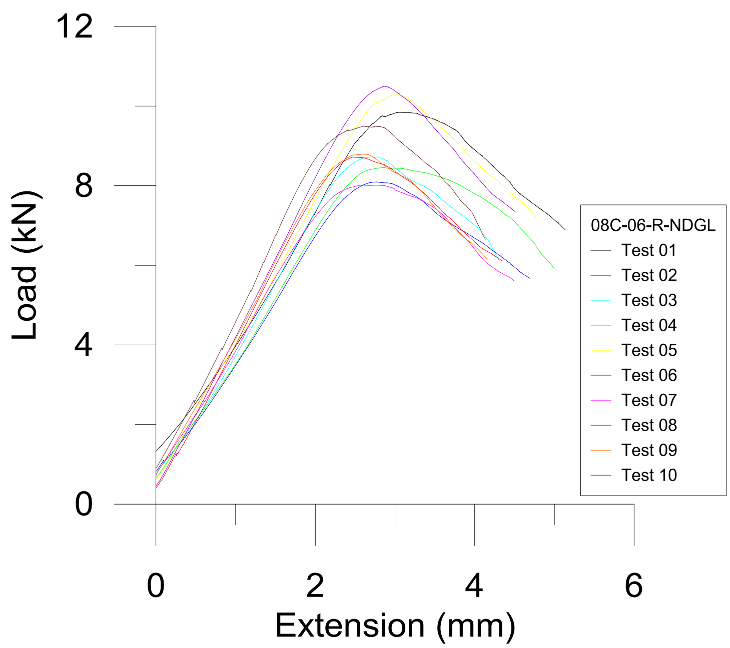


Figure B146 08C-06-NDGL load vs. extension for side screws

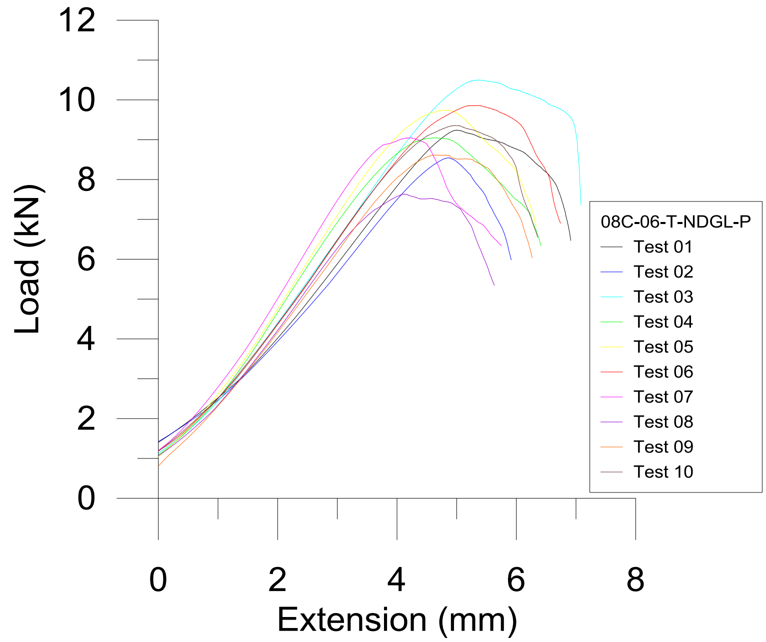


Figure B147 08C-06-NDGL-P load vs. extension for top screws

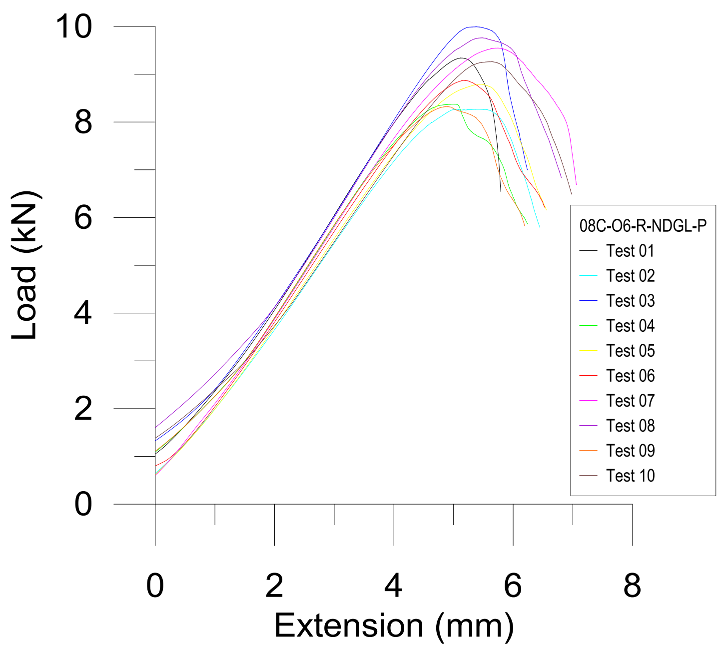


Figure B148 08C-06-NDGL-P load vs. extension for side screws

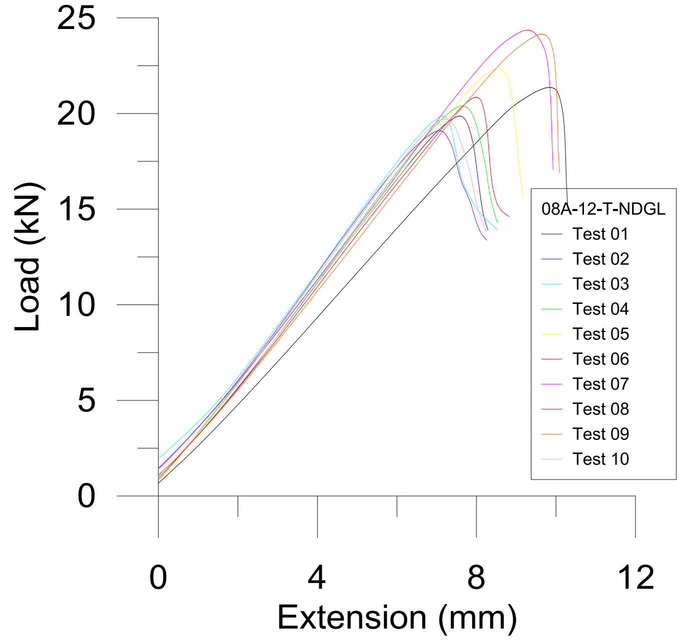


Figure B149 08A-12-NDGL load vs. extension for top screws

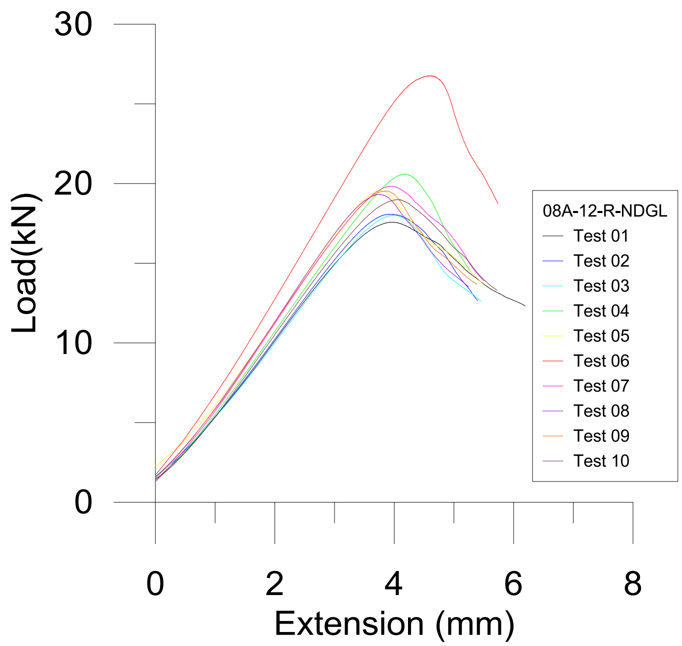


Figure B150 08A-12-NDGL load vs. extension for side screws

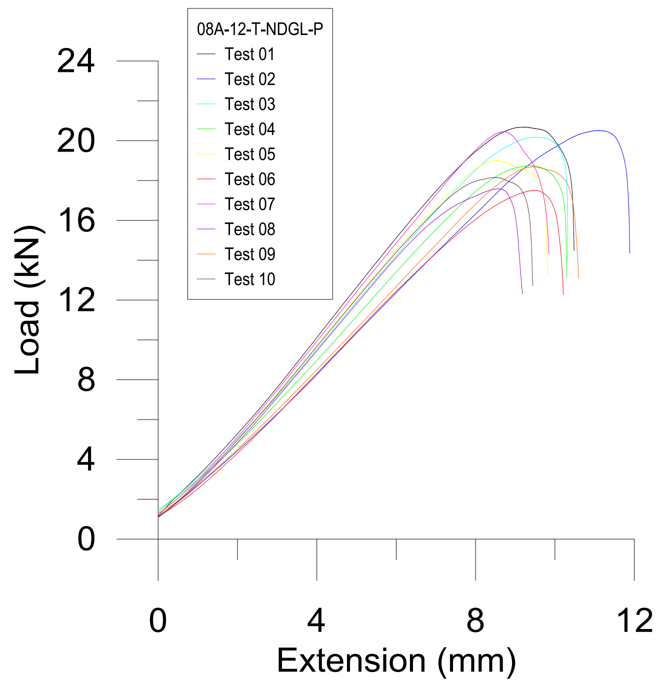


Figure B151 08A-12-NDGL-P load vs. extension for top screws

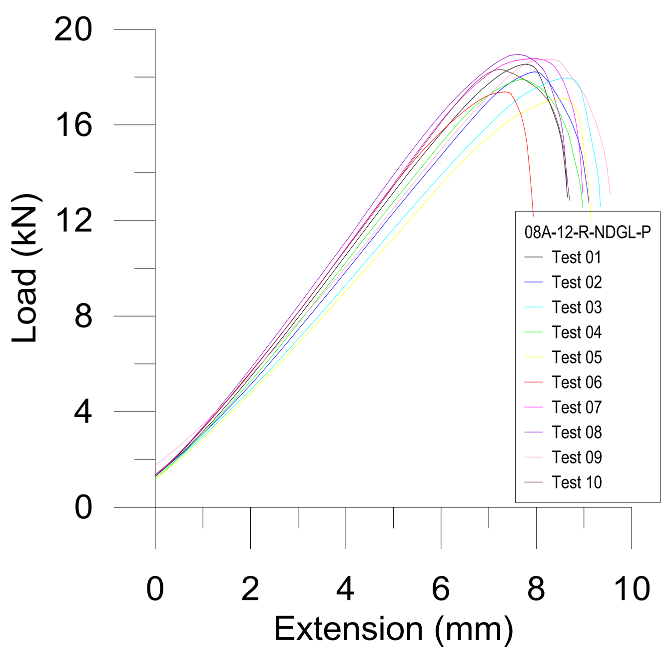


Figure B152 08A-12-NDGL-P load vs. extension for side screws

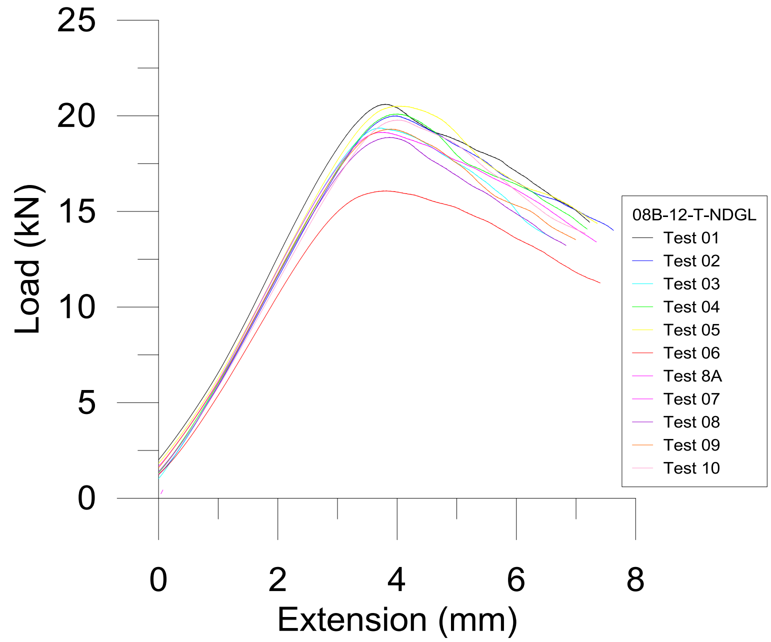


Figure B153 08B-12-NDGL load vs. extension for top screws

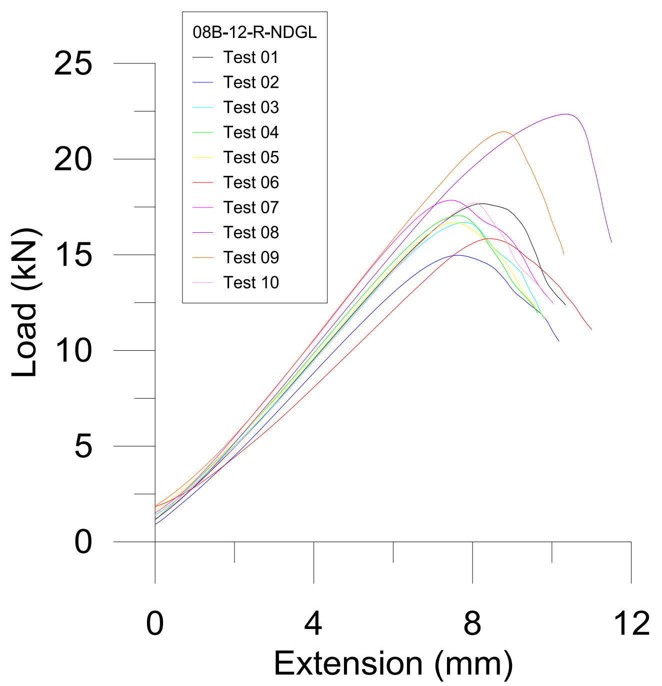


Figure B154 08B-12-NDGL load vs. extension for side screws

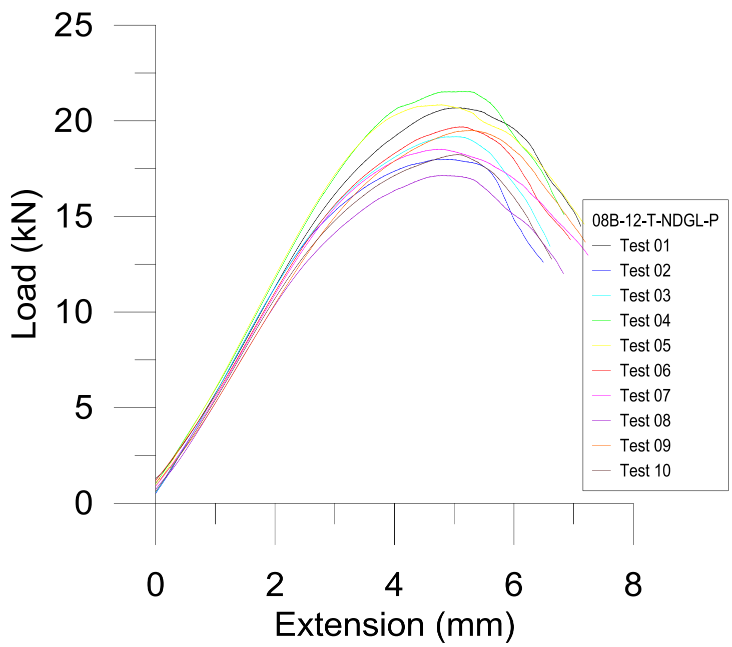


Figure B155 08B-12-NDGL-P load vs. extension for top screws

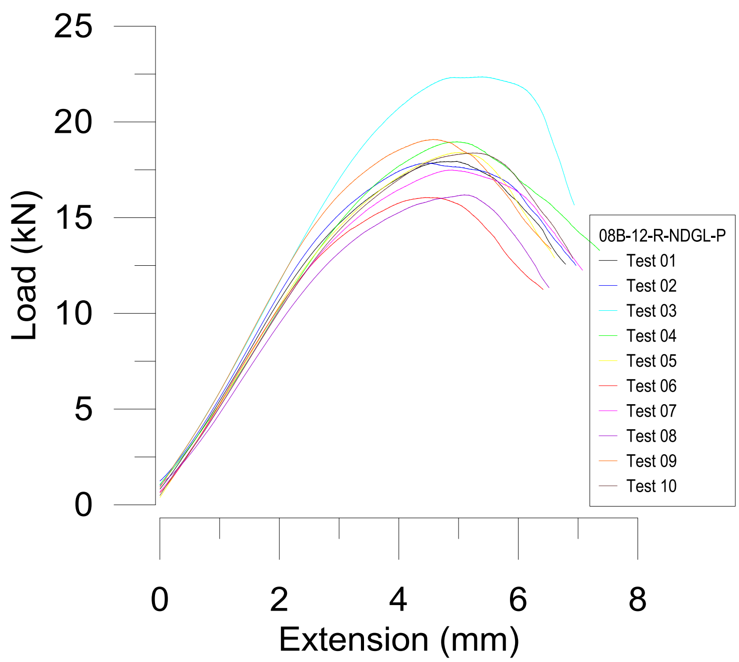


Figure B156 08B-12-NDGL-P load vs. extension for side screws

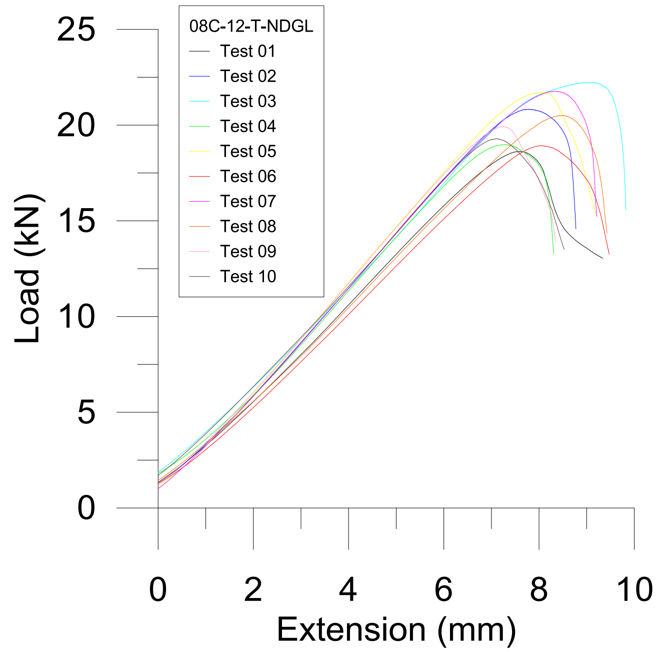


Figure B157 08C-12-NDGL load vs. extension for top screws

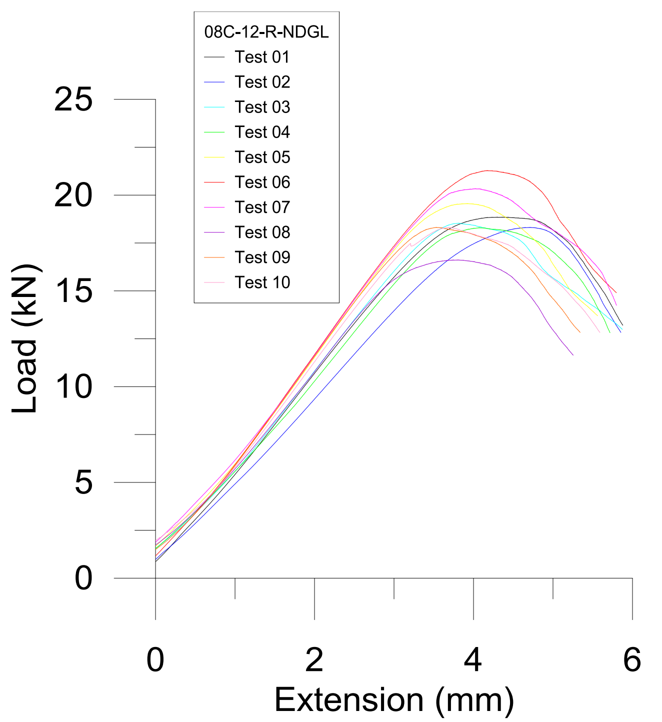


Figure B158 08C-12-NDGL load vs. extension for side screws

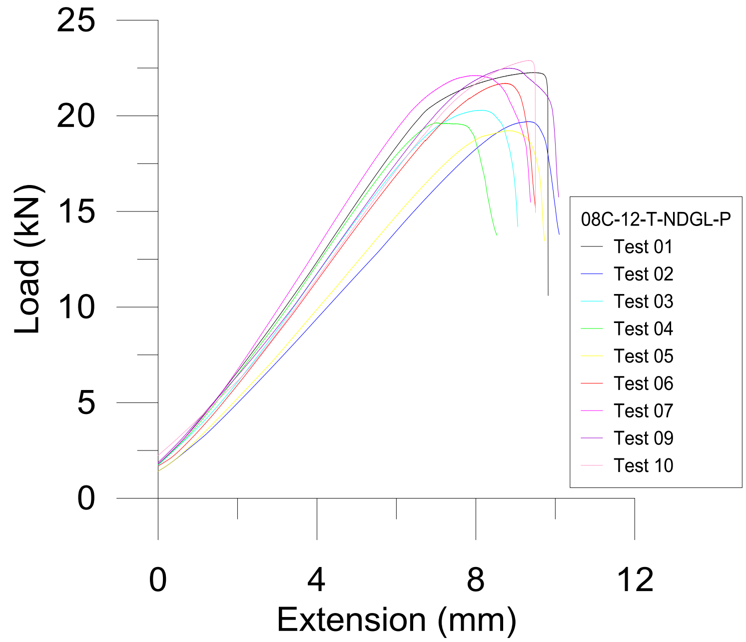


Figure B159 08C-12-NDGL-P load vs. extension for top screws

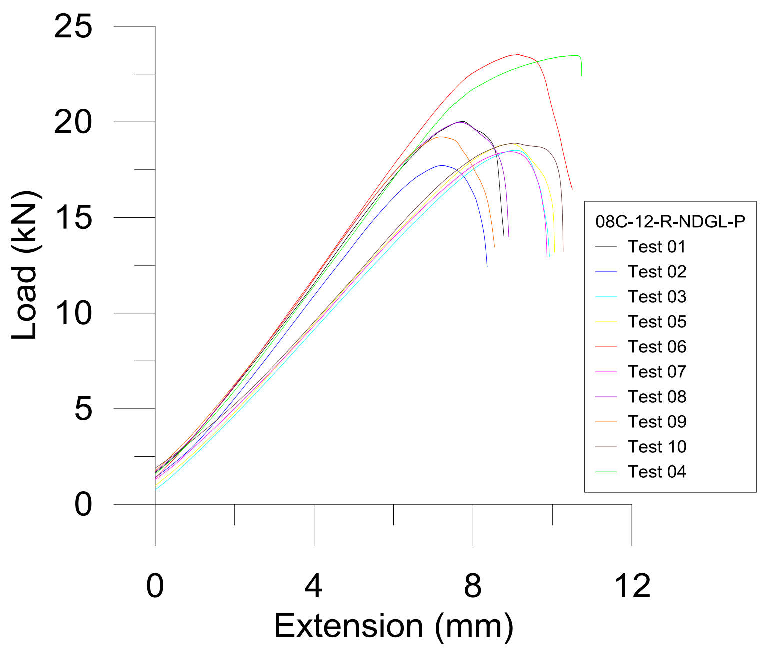


Figure B160 08C-12-NDGL-P load vs. extension for side screws

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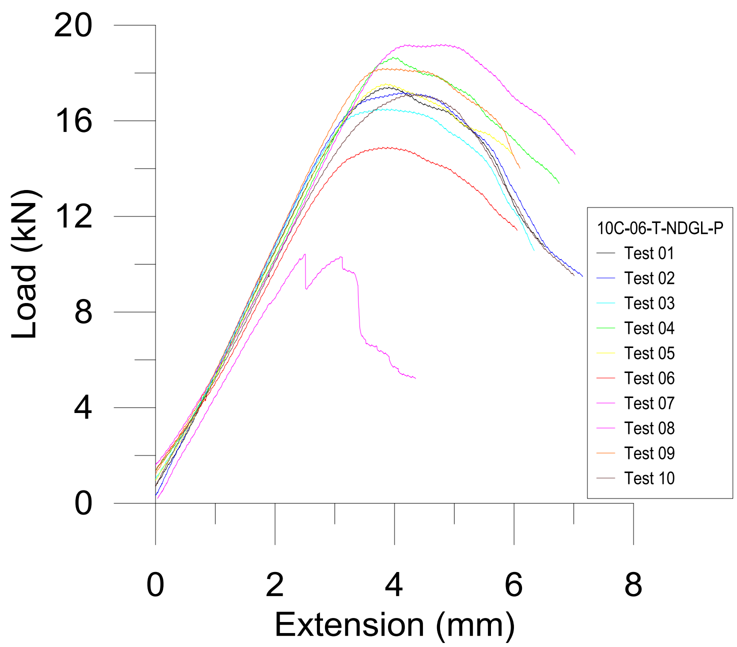


Figure B161 10C-06-NDGL-P load vs. extension for top screws

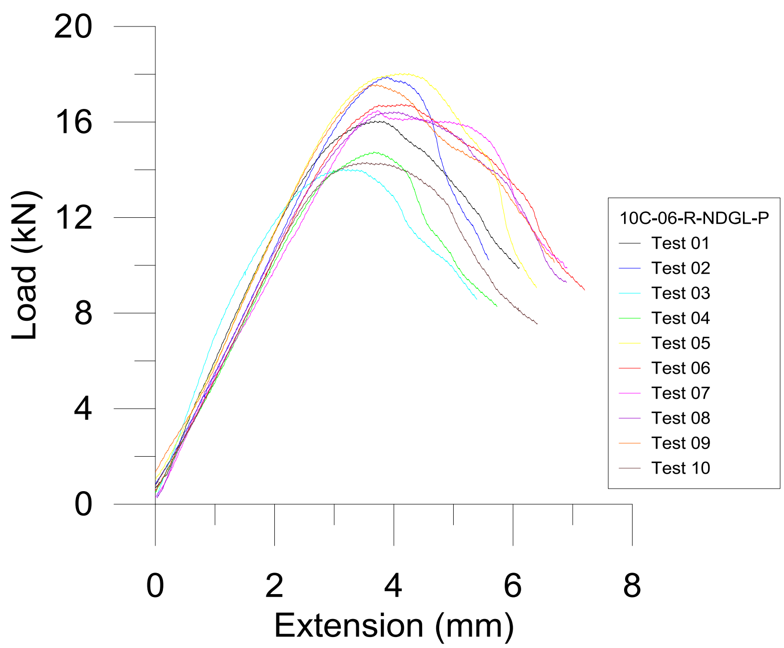


Figure B162 10C-06-NDGL-P load vs. extension for side screws

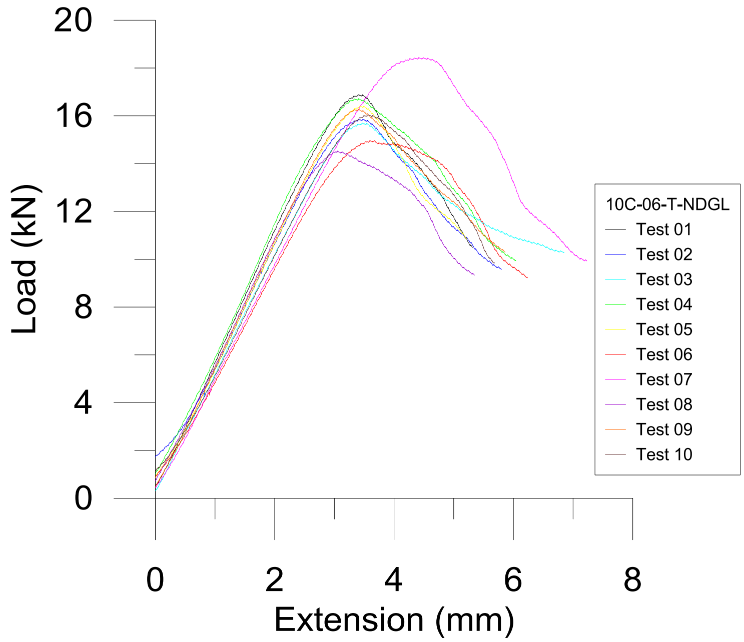


Figure B163 10C-06-NDGL load vs. extension for top screws

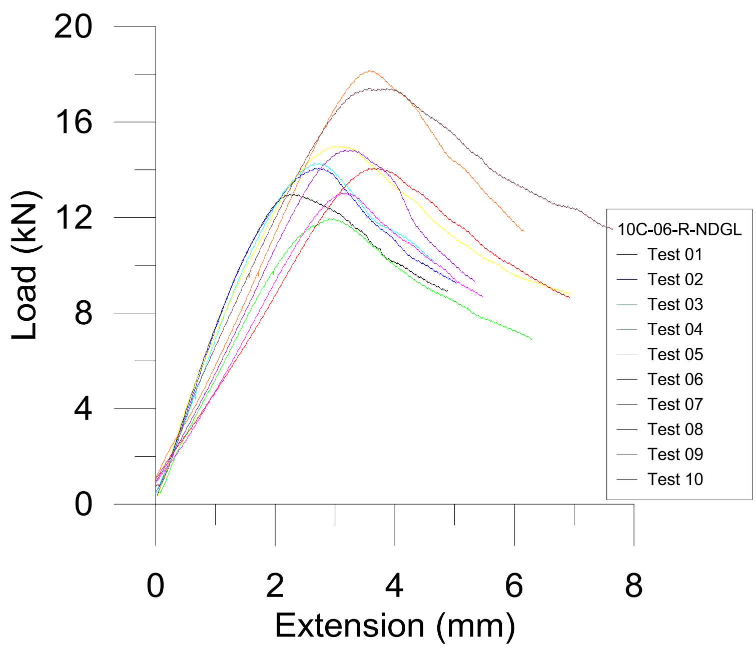


Figure B164 10C-06-NDGL load vs. extension for side screws

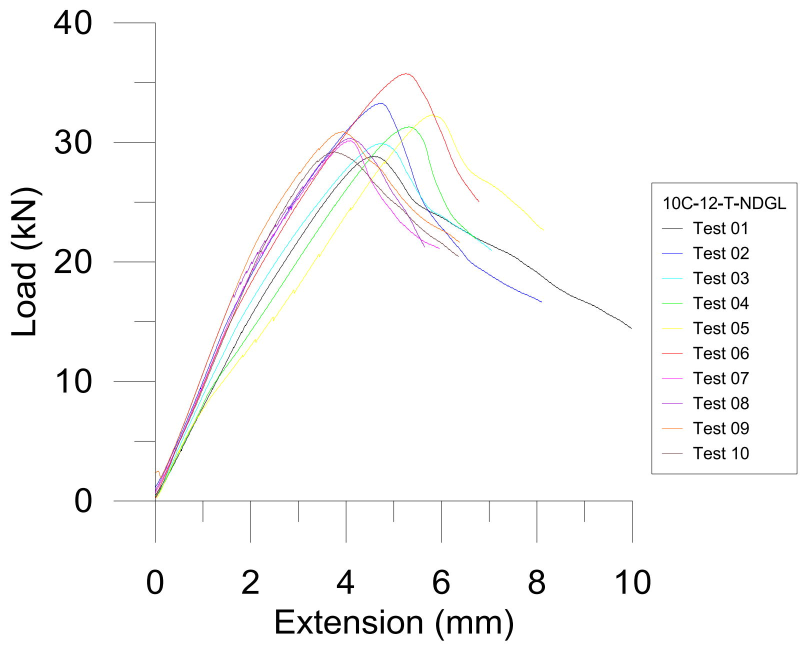


Figure B165 10C-12-NDGL load vs. extension for top screws

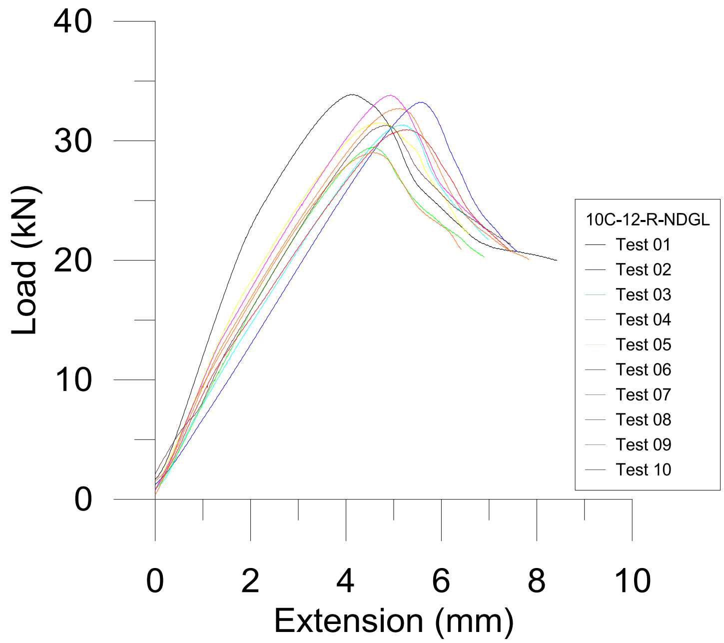


Figure B166 10C-12-NDGL load vs. extension for side screws

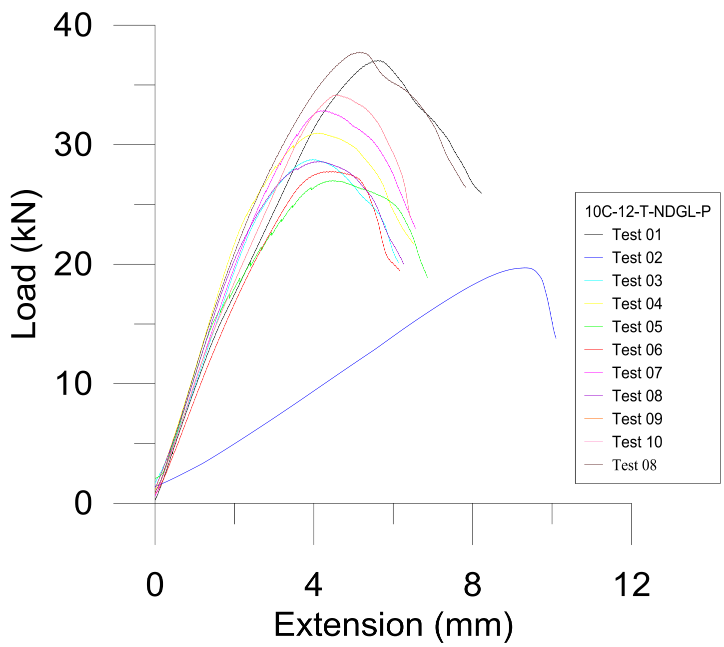


Figure B167 10C-12-NDGL-P load vs. extension for top screws

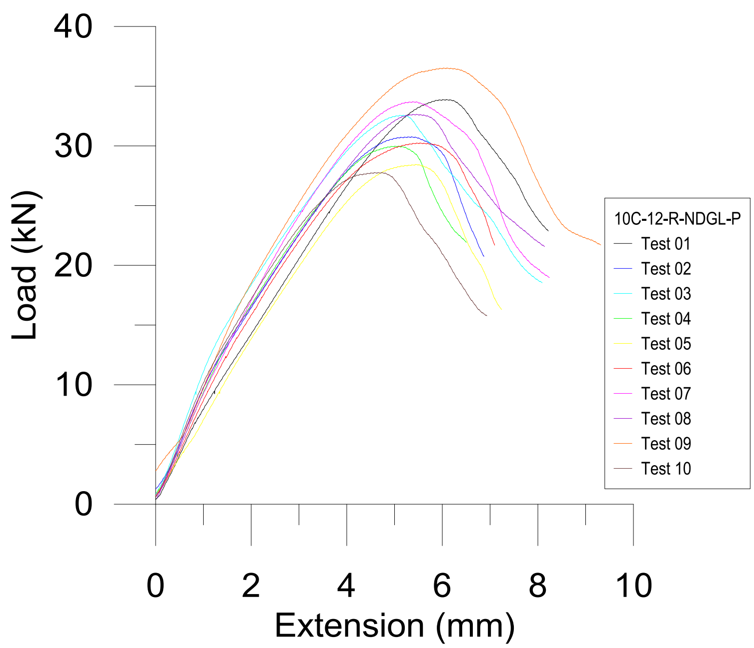


Figure B168 10C-12-NDGL-P load vs. extension for side screws

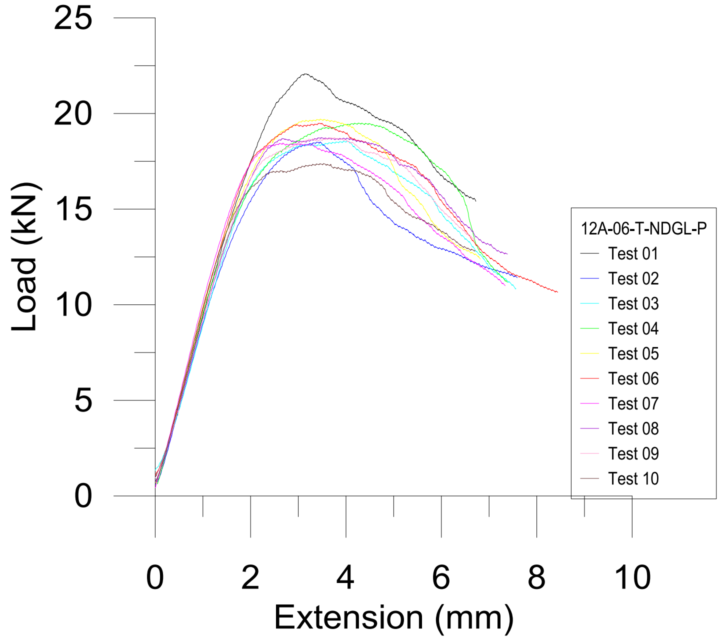


Figure B169 12A-06-NDGL load vs. extension for top screws

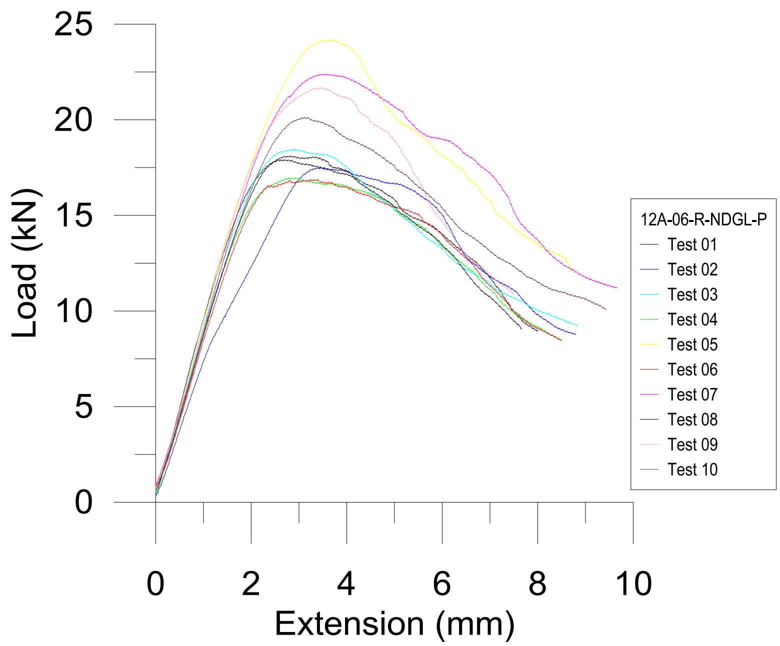


Figure B170 12A-06-NDGL load vs. extension for side screws

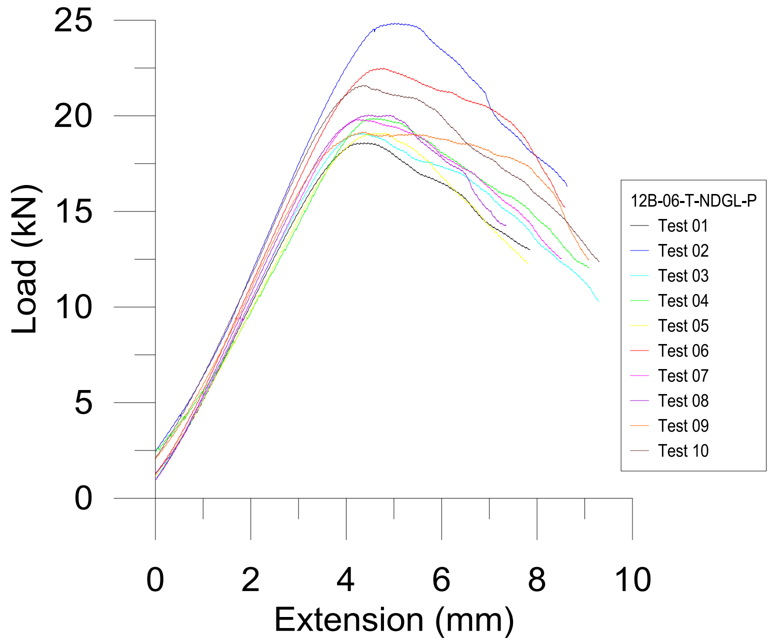


Figure B171 12B-06-NDGL-P load vs. extension for a) top screws and b) side screws

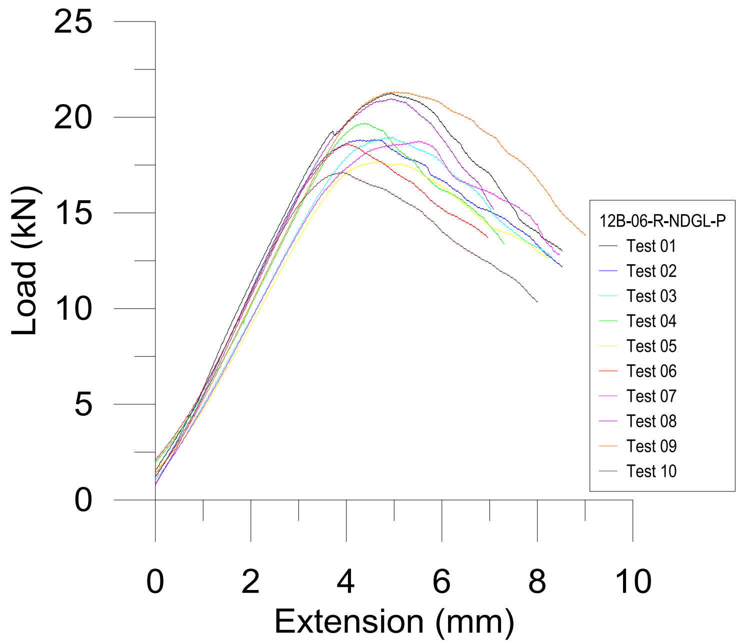


Figure B172 12B-06-NDGL-P load vs. extension for a) top screws and b) side screws

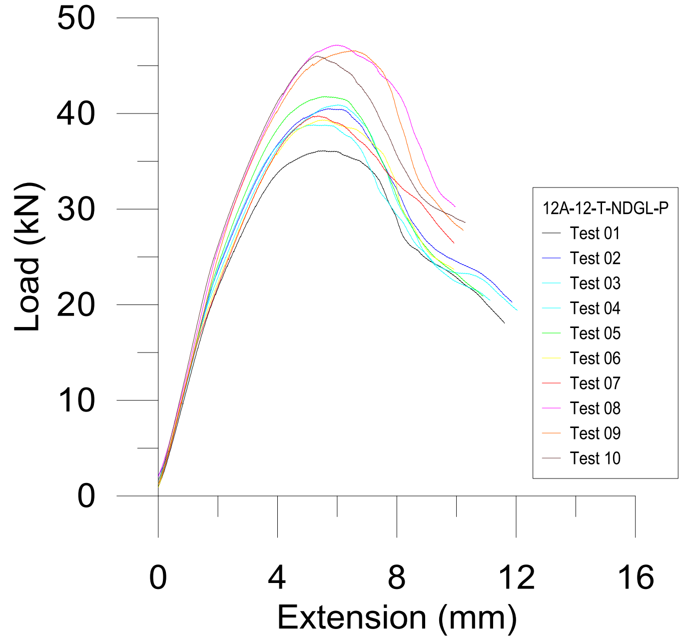


Figure B173 12A-12-NDGL-P load vs. extension for top screws

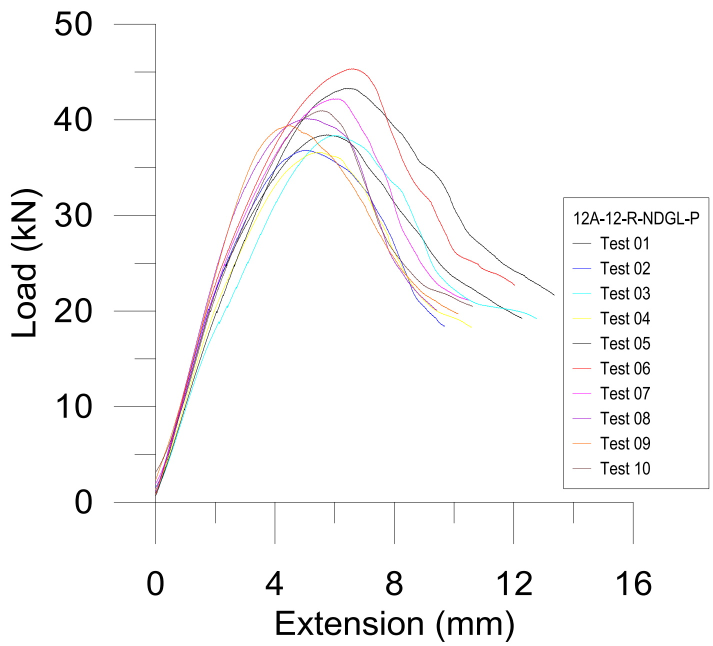


Figure B174 12A-12-NDGL-P load vs. extension for side screws

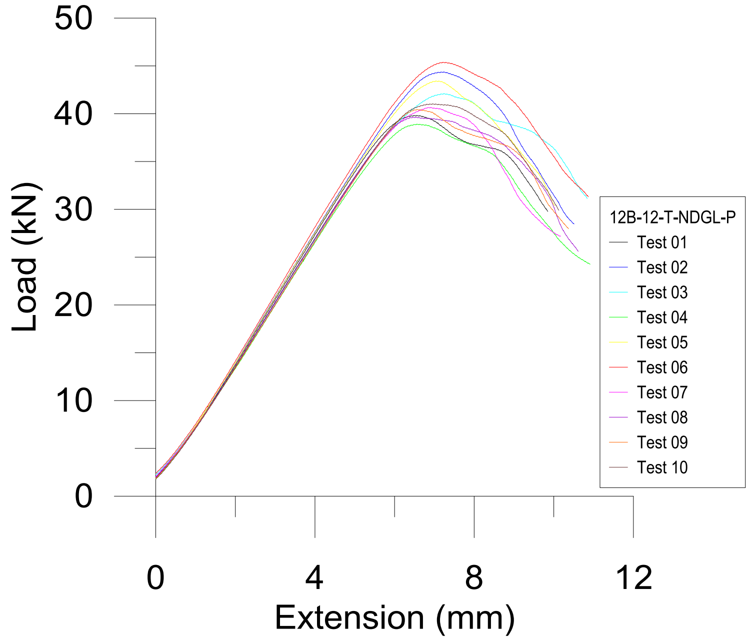


Figure B175 12B-12-NDGL-P load vs. extension for top screws

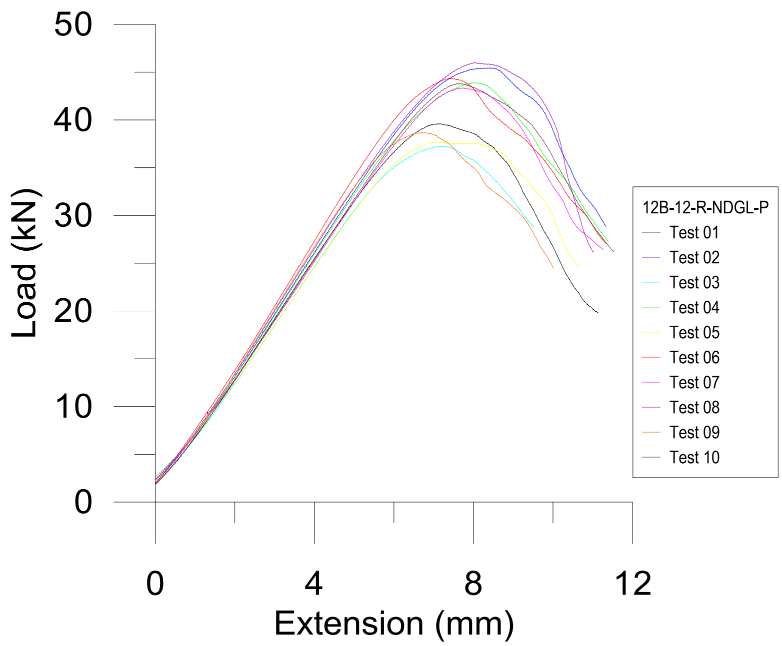


Figure B176 12B-12-NDGL-P load vs. extension for side screws

# Appendix C Details of Screws Used for Test

Table C1 Test screws details

| Type | Total length, (mm) | Length of threaded part,(mm) | Diameter of screw, (mm) | Details |
|---|---|---|---|---|
| Wurth Assy VG, **6B** zylinderkopf,AW30 | 100 | 100 | 6 | Fully threaded, small cylindrical head, |
| Wurth Assy SK,**6C** Schneibenkopf-AW30 | 120 | 70 | 6 | Partially threaded, pointed tip, secondary rough thread above threaded part |
| SFS WFD-T-H12, **8A** | 220 | 100 | 8 | Partially threaded, pointed tip, hexagonal countersunk head, secondary rough thread above threaded part, sharp threads |
| Wurth Assy VG,**8B** Schrauben Senkfraskopf AW40,$40^{O}$ | 200 | 200 | 8 | Fully threaded, blunt end, countersunk round head. |
| Wurth Assy Ecofast, **8C** | 220 | 100 | 8 | Partially threaded, large thread spacing as compared to screws A&B, pointed tip, large countersunk head, secondary rough thread above threaded part |

| **Type** | **Total length, (mm)** | **Length of threaded part,(mm)** | **Diameter of screw, (mm)** | **Details** |
|---|---|---|---|---|
| Wurth Assy Ecofast 3.0 Schrauben, **10B** | 320 | 125 | 10 | Partially threaded, pointed tip, round countersunk head, secondary rough thread above threaded part, long screw |
| Schmid scharauben heinfeld,**10C** | 200 | 125 | 10 | Partially threaded, pointed tip, round head countersunk head, NO secondary rough thread above threaded part, difficult to install |
| SFS WFD,**12A** | | | 12 | Partially threaded, pointed tip, hexagonal countunk head, secondary rough thread above threaded part, sharp threaded part |
| Wurth Assy 3.0 SK Schrauben,AW50 **12B** | 380 | 145 | 12 | Partially threaded, large thread spacing as compared to screws 12A pointed tip, large, long screw countersunk head, secondary rough thread above threaded part |

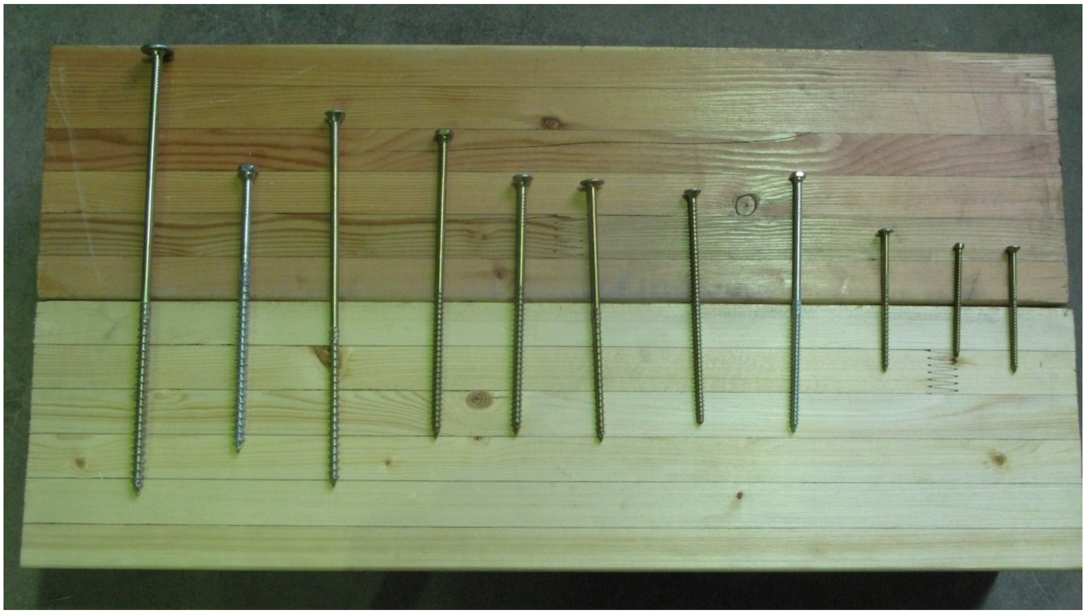

Figure C1 All screws used in testing

As shown in Fig  C1,the first screw from right was not tested because the threaded part was too short hence starting from second screw from the right of picture **6B,6C,8A,8B,8C,(10C- from Schmid was the one only tested at Laval, we changed label to C),10A,10B,12A,12B**

| **Diameter/Type** | **A** | **B** | **C** |
|---|---|---|---|
| 6mm | | Würth Assy plus VG | Würth Assy 3.0Schr SK |
| 8mm | SFS WFD | Würth Assy VG | Würth Assy SK |
| 10mm | Sonderfertigung | Würth Assy | Schmid Schrauben |
| 12mm | SFS WFD | Würth Assy | |
| WT-T-8.2mm,SFS Intec screws used for the cross screw joints tests | | | |

a)  b) 

Figure C2 Screws a) 6B b) 6C

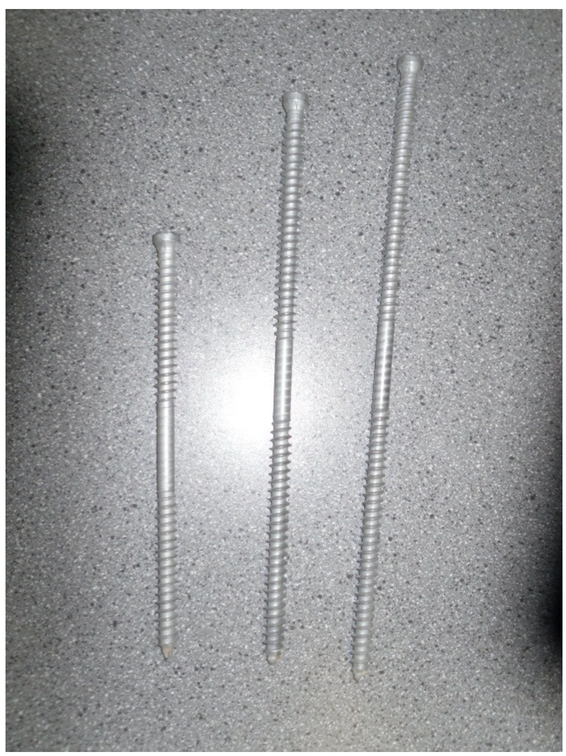

Figure C3 WT-T-8.2 screws (160, 220 and 245 respectively)

a)

b)


Figure C4 Screws a) 8A b) 8B

a)

b)


Figure C5 Screws a) 8C b) 10A

a)  b) 

Figure C6 Screws a) 10B b) 10C

a)  b) 

Figure C7 Screws a) 12A b) 12B