

# The Quick-Connect moment connection for portal frame buildings – An introduction and case study

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**ABSTRACT:** This paper introduces the Quick-Connect portal frame connection for timber portal frame buildings which has recently been developed at the University of Auckland. The rod based connection which utilizes fully threaded timber screws has been developed to allow timber construction to move from the traditional model of onsite connection assembly to a construction methodology whereby much of the connection is manufactured offsite. Further, this paper provides a case study of the connection in use in a large industrial and commercial building in New Zealand. The authors highlight the challenges as well as advantages encountered when commercializing such a connection.

**KEYWORDS:** Moment connection, portal frames, fully threaded screws, versatile, expedient

## 1 INTRODUCTION

Buildings which require maximum use of interior space are often designed with portal frames as the structural system [1]. The structural system for timber portal frames varies depending on span, with short-span structures often using solid sawn or Glulam members, whilst long-span structures of over 50m clear span utilize plywood or Laminated veneer lumber box beams [2].

Design effort in timber structures is heavily focused on the connections. The nailed plywood gusset connection as introduced by Batchelar [3], is one of the most commonly utilized connections for timber portal frames in Australasia. Whilst the nailed gusset connection has been used over many years, changes in construction methods and ongoing competition between the steel and timber industries in this space has driven the development of new connection types. These connections take into account the critical factors which drive modern construction.

## 2 FACTORS WHICH AFFECTED CONNECTION DEVELOPMENT

The timber industry is lacking moment connections which are easily specified by the designer. To become more competitive with the concrete and steel industries, the timber industry must develop and adopt connections which are easy to specify and provide guidance through all stages of the build process. Connections which lend themselves to design via the use of electronic aids such as spread sheets are required if the timber industry wishes to remain competitive in this space.

Onsite construction path length is a critical factor. Currently most large portal frames are erected using the grid lift procedure. This allows the columns to be erected and much of the roof assembly to be completed on the

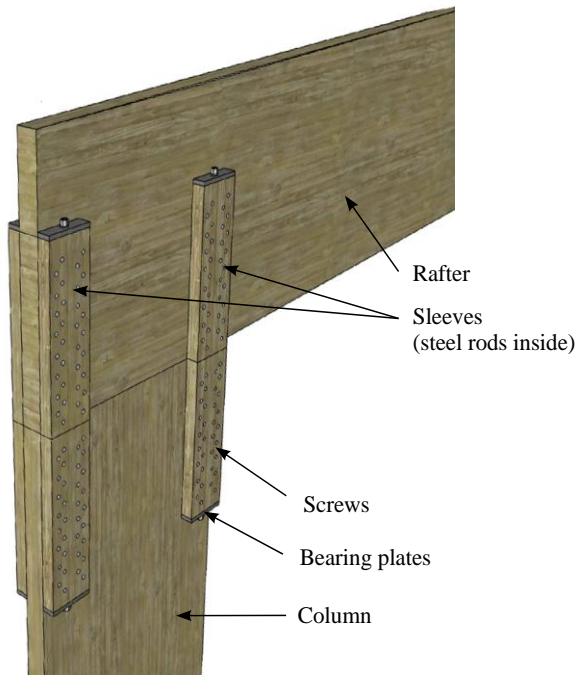
ground before a whole roof bay or a number of roof bays (a roof section) are lifted into place. Considering one of the most common current portal frame connections, the nailed gusset as an example, half of the connection nailing can be completed on the ground before the connection is lifted. The other half must be completed at height resulting in high crane and labour times. Adopting a design approach for portal connections which is similar to bolted connections commonly used in the steel industry will result in a reduction of crane and labour times, an overall reduction in the critical construction path times, and therefore an overall reduction in onsite costs.

Many of the current connections for portal frame timber buildings utilize large steel plates or many fasteners to ensure adequate strength and stiffness. As a result, connections are often not aesthetically pleasing and costly. They draw the eye of the observer from the main timber structure to the steel components. By striving to design connections which have less fasteners and more timber componentry the designer reinforces the positive effects which timber has on the overall appearance of a building. Although portal frame structures of the type which are being considered in this paper do not generally require a fire rating, a reduction in steel components in connections would make a fire rating easier to achieve if needed.

To date the vast majority of timber structures are fabricated onsite. With numerous manufacturers and fabricators now investing in modern computer numerical control machinery (CNC), the prospect of pre-fabricating in the factory and providing kitset solutions has become a real possibility. Pre-fabrication has the advantage that quality control issues onsite are greatly reduced.

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**Figure 1:** The Quick Connect Portal Frame Connection

Manufacturing offsite allows timber buildings to follow the same construction model which is used for steel buildings resulting in a reduction of construction cost.

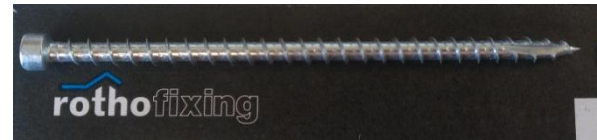
### 3 THE QUICK-CONNECT

The Quick Connect joint is a semi-rigid moment connection which has been developed as an alternative to current nailed solutions. The connection bears some conceptual similarity to the partially restrained bolted connections often used in steel construction. The joint consists of a rod based system as shown in Figure 1.

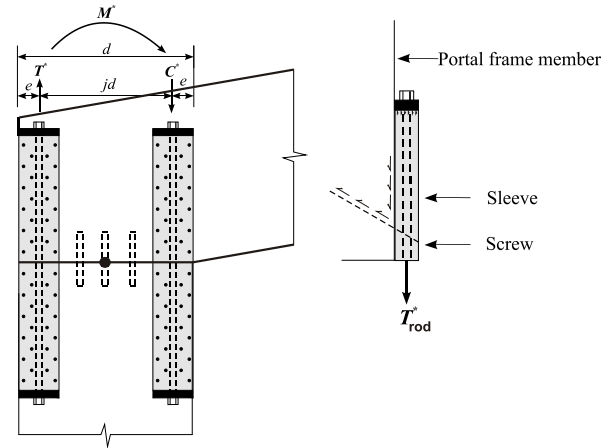
When the structure is loaded, a tensile force is applied to one set of rods whilst the other set remains idle. The compressive force in the connection is transferred in elastic bearing at the interface between the main members. This allows a moment couple to be developed which facilitates the transfer of load across the joint. The rods are housed in U-shaped timber members, hereafter referred to as timber sleeves. Placing the rods on the exterior of the portal members allows for the full bending moment capacity of the members to be developed at the joint.

The timber sleeves are fixed to the main portal members by way of continuously threaded timber screws. The availability of these long, high strength, fully threaded screws which have been designed specifically for high load applications in timber allows for the creation of efficient connections between the timber sleeves and main members. The Rothoblaas VGZ screw is an example of such a screw and is shown in Figure 2.

Unlike traditional timber screws, these screws are hardened after the thread has been rolled. The hardening process increases the screws bending and torsional



**Figure 2:** Rothoblaas VGZ 7180 fully threaded screw



**Figure 3:** Force transfer in the Quick Connect

capacities. Additionally, a self-tapping tip is cut or moulded into each screw which allows for application of the screws without pre-drilling. These attributes make self-tapping timber screws ideal fasteners for high load timber to timber connections [4]. The use of traditional fasteners such as nails and bolts would result in reduced strength and increased slip at the timber sleeve to main member interface, reducing the efficiency of the connection.

Blass and Bejtka [4] performed testing in glued laminated timber whereby four self-tapping timber screws were used to connect a central member to two side members. The angle of the screws to the applied force was adjusted from 45 to 90 degrees in order to ascertain the effect of withdrawal strength of the screws on the overall strength and stiffness of the connection. The maximum load carrying capacity of the inclined versus perpendicular to load screws increased by approximately 53% at an angle of 60 degrees. An increase in connection stiffness was observed, with a 12 fold increase in stiffness observed when screws are oriented at 45 degrees to load versus perpendicular [4]. Subsequent to performing tests on New Zealand timbers, an angle of 60 degrees to the applied force was initially adopted for the Quick-Connect joint. After consultation with Blass and further testing this angle has now been amended to 45 degrees. In placing the screws at this angle connection stiffness and strength characteristics are increased for the screw connection.

The force transfer system for a timber beam to timber column knee joint is shown in Figure 3. The rafter is placed atop the column as is common in portal frame structures.

The rods then extend vertically on either side of the joint and are contained within the timber sleeves. The sleeves

are fixed to the portal beam and column with self-tapping timber screws before the connection is taken to site, leaving only the rod to insert and tighten insitu.

Due to the limited components which are required to form the connection, it is easily adapted for use in all parts of the structure. The basic connection configuration always remains the same. A shear transfer system which consists of a number of short rods which are set into the main portal members is added if required. Furthermore, the connection can be adapted for use with box beams. The design of the box beam connection is very similar to that applied to solid members. However, the rods are placed in the gap between the members of the box beam, allowing for use in architectural applications in which it is not desirable for the connection components (sleeves, plates, rods and nuts) to be visible.

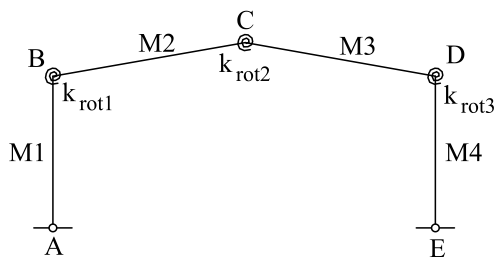
In practical terms, the connection can be designed and manufactured without special training. Pre-manufacturing of the connection offsite allows for reduced crane and labour requirements during erection. The slightly higher materials costs when compared to the nailed gusset connection are negligible when compared to the savings in plant and labour onsite.

## 4 QUICK CONNECT JOINT DESIGN PROCEDURE

The Quick Connect design procedure [5] consists of an iterative process which is to be used to determine the required characteristics for the sleeves, main tension rods, screws and bearing plates. The connection is designed to resist a given moment, once the components have been specified, joint rotation can be determined.

As moment is transferred, the members and connection components show limited rotation. This rotation must be quantified in the portal frame analysis [6] as it directly affects the bending moments in the structural members and thus the member sizes. Furthermore, joint rotation must be strictly controlled as small deflections at the joint lead to much larger mid span deflections.

The procedure assumes that a portal frame analysis has been performed in which the stiffness of the connections  $k_{rot}$ , as shown in Figure 4, was assumed to be infinity. The designer will have calculated the moment, shear and axial forces acting on the connection and will have used these to perform a preliminary member design.



**Figure 4:** Simple structural model showing  $k_{rot}$  for a two pin portal frame

In order to determine the appropriate tension and compression forces present in the joint, the connections centre of rotation must be determined. However, this requires the connection stiffness and deflection at the tension edge to be known. The determination of the deflection at the tension edge is only possible once the attributes of the sleeve, rod, screws and bearing plate have been specified. Therefore the procedure initially uses a simplified assumption that the centre of rotation lies at the midpoint of the beam as shown in Figure 3. This assumption is used to carry out the initial sizing of the connection. Depending on the accuracy of this assumption the connection must be resized once a more accurate centre of rotation location has been determined later in the procedure.

### 4.1 DESIGN OF CONNECTION DETAIL ASSUMING CENTRE OF ROTATION LIES AT CENTRE OF MEMBER

The sleeves are sized to allow for correct screw geometry in the connection (i.e. screw spacing and end distances). Initially the designer should assume a conservative sleeve size. The sleeve size can then be reduced in a later iteration of the design.

Sleeves must be designed to resist the compression force applied to the end grain. To determine the applied compression force, the moment arm  $jd$  of the rods to be inserted is calculated using equation (1).

$$jd = d - 2e \quad (1)$$

where

- $jd$  = moment arm between the steel rods, as shown in Figure 3
- $d$  = width of the main member
- $e$  = distance from the extreme fibre to the centre of the rod, as shown in Figure 3

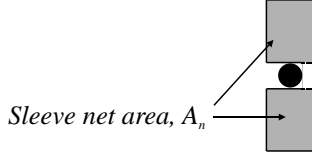
Equation (2) allows preliminary tension and compression forces to be calculated for each set of rods. One set of the rods will act in tension, whilst the other set will not have any load applied. This loading is reversed when the applied force direction is reversed. The sleeves are sized for axial compression loads resulting from the applied moment as well as the tension force in the member.

$$C_{sleeve}^* = T_{rod}^* = \frac{M^*}{2jd} + \frac{N^*}{4} \quad (2)$$

where

- $M^*$  = design moment
- $N^*$  = design tensile force at the members interface
- $C_{sleeve}^*$  = design compression force in one sleeve
- $T_{rod}^*$  = design tensile force in one rod

The timber sleeve must be capable of resisting the maximum applied compression force (equal to the tension force in the rod). The compression capacity must



**Figure 5:** Net cross sectional area of sleeve acting in compression

be designed in accordance with the applicable design standard, taking into account all applicable modification factors such as load duration, service conditions and so forth. The sleeve is considered to be fully laterally restrained therefore buckling may be ignored.

$$C_{sleeve}^* \leq N_{nc,sleeve} \quad (3)$$

$$N_{nc,sleeve} = f_c A_n \quad (4)$$

where

- $f_c$  = compression strength of timber
- $A_n$  = net cross sectional area of sleeve acting in compression. Refer to Figure 5

Various screw capacities  $P_{screw}$  are available for Glulam and LVL. These screw capacities are only valid for those screw types which have been tested such as the Rothoblaas VGZ screw. Alternatively screw capacities can be calculated using the screw manufacturers design information. To calculate the number of screws required to resist the tension at each timber sleeve the following equation is used:

$$n_s = \frac{T_{rod}^*}{P_{screw}} \quad (5)$$

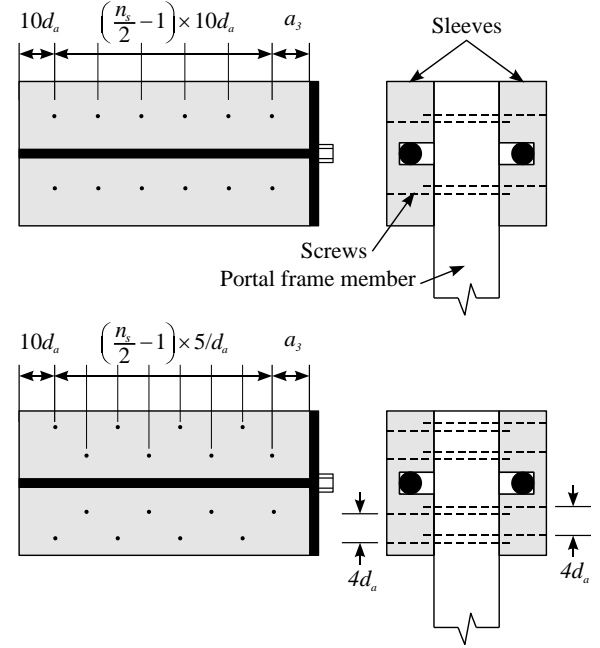
where

- $n_s$  = number of screws per sleeve required to resist tension forces in the connection
- $T_{rod}^*$  = tension force carried by each steel rod
- $P_{screw}$  = characteristic strength of a single screw

Screws are arranged in rows along the length of the sleeve. Screw spacing requirements are shown in Figure 6. The authors recommend a minimum spacing of  $10d_a$  between screws in a row. Rows of screws should be staggered with a minimum spacing of  $4d_a$  between rows.

To ensure screws do not interfere with those on the opposite side of the connection, spacing between rows should be offset on alternate sides of the joint. The characteristic load capacity of the screws varies depending on whether they are installed parallel or perpendicular to the grain. This is particularly important in knee joints as different numbers of screws may be required in the portal timber rafter and timber column members.

Block tear-out resistance in main portal member must be checked. The sleeves must be large enough to prevent block tear-out type failure of the main member. It is



**Figure 6:** Recommended screw spacing

assumed for simplicity that the resistance in shear and tension are additive. This design check is required only for the case where the timber sleeves are fixed to the portal member loaded parallel to grain.

The shear resistance is first calculated as:

$$V = f_s A_s \quad (6)$$

where

- $f_s$  = shear resistance of portal frame members
- $A_s$  = shear area

The shear area  $A_s$  is:

$$A_s = t(l_s - a_3) \quad (7)$$

where

- $t$  = thickness of the main member being considered
- $l_s$  = length of the timber sleeve
- $a_3$  = end distance, as shown in Figure 6

The dimension  $l_s$  is taken as the length of the sleeve associated with the parallel-to-grain member. The sleeve must be long enough to accommodate the required number of screws for the connection, adopting the recommended spacing shown in Figure 6.

The tearing resistance of the main member is given by:

$$T = f_t A_t \quad (8)$$

where

- $f_t$  = tensile strength of main portal frame

$A_t$  = member  
= tensile area

And

$$A_t = t(w - \text{edge distance}) \quad (9)$$

where

w = depth of timber sleeve  
t = thickness of main member

The resistances obtained from equations (6) and (8) are then added to give the final tearing and shear resistance of the main member where the timber sleeves are attached.

$$2T_{rod}^* \leq T + V \quad (10)$$

The bearing plates are designed as if each timber sleeve consists of two individual rectangular pieces. This approach is conservative. The bearing plate length is specified by first calculating the required bearing area using:

$$A_p = \frac{T_{rod}^*}{f_c} \quad (11)$$

where

$T_{rod}^*$  = tension force carried by each steel rod  
 $f_c$  = characteristic compression stress timber of sleeves

The dimensions of the plate are limited by the thickness and depth of the timber sleeve. To ease calculations, the bearing plate is specified with the same width as the timber sleeve. The total bearing area required can then be divided by this width to obtain the depth of the bearing plate.

The required thickness of the bearing plates must be calculated considering the serviceability and ultimate limit state load cases. It is likely that the serviceability limit state will govern the design of the plate. It is also desirable to set a self-imposed bearing plate deflection of 0.1 mm as deformations at the knee connections lead to large displacement in the structure as a whole. This limiting value is at the discretion of the designer and experience should be used to determine an appropriate value for the design.

To calculate the deflection of the plate, it must be considered as a beam element with two cantilevers mirrored at the rod axis, the deflection equation is therefore:

$$\Delta = \frac{wl^4}{8EI} \quad (12)$$

where

w = uniformly distributed load acting on bearing plate =  $T_{rod}^*/2l$   
l = outstand length of bearing plate either side of the rod  
E = modulus of elasticity of steel bearing plate

and the moment of inertia I is:

$$I = \frac{bt^3}{12} \quad (13)$$

where

b = height of the bearing plate (mm)  
t = thickness of bearing plate (mm)

A final check is conducted to ensure the plate satisfies strength requirements using:

$$t_u \geq \sqrt{\frac{3IT_{rod}^*}{2f_y b}} \quad (14)$$

The rods in the connection must be checked primarily for tension strength (elongation of the rod is included in later checks). To evaluate the tension strength of each rod it is assumed that the rods do not carry any shear force. This can be ensured by utilizing a shear transfer system in the connection such as embedded steel dowels or a corbel.

The tension resistance of the rod must be larger than or equal to the applied tension force:

$$T_{rod}^* \leq N_{tf} \quad (15)$$

where  $N_{tf}$  is the tension resistance of the rod given by:

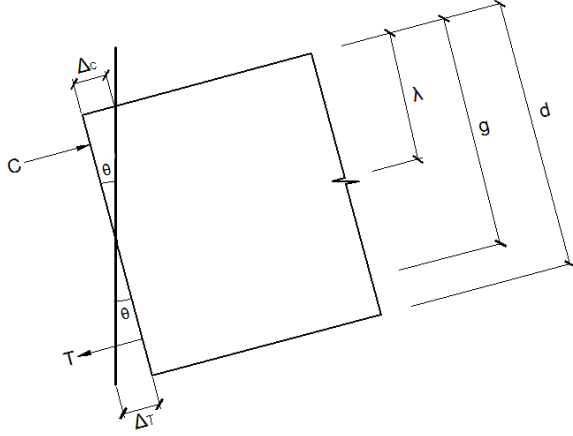
$$N_{tf} = A_s f_{uf} \quad (16)$$

$A_s$  is the required tensile area of the rod. This can be used to obtain the diameter of rod required.

The initial sizing of the connection is now complete and the second stage of the procedure can be used to calculate a better approximation of the centre of rotation which can be used to determine the joint rotation.

#### 4.2 EVALUATION OF APPROXIMATE POSITION OF CENTRE OF ROTATION AND ROTATIONAL STIFFNESS $k_{rot}$

The following provides a brief overview only. Full details are published in a separate paper [5]. For an explanation of the variables used please consult Figure 7.



**Figure 7: Rotation at joint**

For this procedure to be valid, the connection is designed to ensure that all component deformations remain in the elastic range. The moment is calculated as part of the preliminary design for the portal frame structure. The rotational stiffness of the joint is calculated using equation (17):

$$k_{rot} = \frac{M^*}{\theta} \quad (17)$$

The rotation  $\theta$  is given by the following equation:

$$\theta = (\Delta_C + \Delta_T) / d \quad (18)$$

where

$$\Delta_C = \lambda \left( \frac{\Delta_T}{d - \lambda} \right) \quad (19)$$

and

$$\Delta_T = \Delta_{@T_{rod}} \left( \frac{d - \lambda}{g - \lambda} \right) \quad (20)$$

$\Delta_C$  is the elastic compression of the main member on the compression side of the connection and  $\Delta_T$  is the sum of the elastic deflections at the extreme fibre on the tension side of the connection. The value of  $\Delta_T$  is a function of  $\Delta_{@T_{rod}}$  which is calculated using equation (21).

$$\Delta_{@T_{rod}} = \Delta_{rod} + \Delta_{sleeve} + \Delta_{screws} + \Delta_{plate} \quad (21)$$

The revised position of the centre of rotation  $\lambda$ , required in equations (19) and (20) is calculated using equation (22).

$$\lambda = \left( \frac{2k_{system}g}{(E_{c,wood}b) + (2k_{system})} \right) \quad (22)$$

where

$$k_{system} = \frac{n_{tension\ rods}}{\left( \frac{1}{k_{rod}} + \frac{1}{k_{screws}} + \frac{1}{k_{sleeve}} + \frac{1}{k_{plate}} \right)} \quad (23)$$

The value of the  $k_{system}$  parameter is governed by the individual stiffness characteristics of the individual connection components, namely the stiffness of the rod, screws, timber sleeve and plate.

The deflection calculations for the individual components in equation (21) are dependent on the revised  $T_{rod}^*$  which is calculated using equation (24).

$$T_{rod}^* = \frac{M^*}{2 \left( g - \frac{1}{3} \lambda \right)} \quad (24)$$

The revised  $T_{rod}^*$  determined using the revised centre of rotation of the connection should be compared to the  $T_{rod}^*$  which was initially calculated by assuming that the centre of rotation was located at the centre of the main timber element. If the initial assumption was conservative then the designer may make the choice to leave the connection design as is. If however the initial estimate of  $T_{rod}^*$  is non-conservative, then an iterative process must be adapted whereby the designer inputs the more accurate revised  $T_{rod}^*$  value found by using the closer approximation of the centre of rotation and repeats the design procedure.

Assuming that the designer has either followed an iterative process or accepted the conservatism of the connection which has resulted from using the initial assumption it is possible to use equation (19) to calculate the elastic deformation,  $\Delta_C$ , of the main member. By adding the value for  $\Delta_C$  and the value for  $\Delta_T$  found by using equation (20) and substituting these values into equation (18), it is possible to determine the connection rotation. This rotation value is then input into equation (17) to give the  $k_{rot}$  value for the connection. The  $k_{rot}$  value found must be input into the initial structural model to determine the revised moment loading. If the moment calculated is different to that calculated when assuming that  $k_{rot} = \infty$ , then the designer should use an iterative approach to determine a connection size which gives comparative moment values.

## 5 TUMU ITM NAPIER– A QUICK CONNECT FIRST

### 5.1 INTRODUCTION

This hybrid material structure is an excellent example of how different building materials can be effectively and harmoniously used in a single project. The building structure consists of a solid LVL rafter roof structure supported by steel columns, some of which are placed atop concrete perimeter walls. Solid timber members were chosen for the rafters and purlin members as these require a minimum of bird proofing. The designers Stratagroup, and the building owners Tumu ITM, made





**Figure 8:** A section of roof being prepared on the ground prior to being lifted

an early decision to avoid timber columns for security and commercial reasons. The building is situated on an exposed industrial site and tilt up panels form a sturdy building envelope which will provide protection to the stores contents once complete. Steel columns offer a reduced cross section when compared to that required when using timber columns. Reduced column cross section was an important consideration in this case as the owner had particular requirements for sales racking within the store. The building is encased in a standard iron sheet envelope.

For the remainder of this paper only the timber roof structure will be considered with the main emphasis on the knee, splice and apex portal connections.

## 5.2 TIMBER ROOF STRUCTURE

At its widest point, the roof structure spans a total of approximately 60 metres with a central support. Portals are spaced at a maximum of 7.88 metres. The roof

structure consists of 12 bays with an overall length of 90.3 metres. The rafters used consist of 1220mm deep by 90mm wide LVL supplied by Nelson Pine. Lateral restraint to the top of the rafter is ensured by 300mm deep by 45mm wide purlins spaced at 1.5 metre centres.

The Quick Connect joint was used for the steel column to timber rafter knee connection, the rafter splice connection and the timber rafter to steel column or timber rafter to timber rafter apex connections. Connection sizing was standardized when possible.

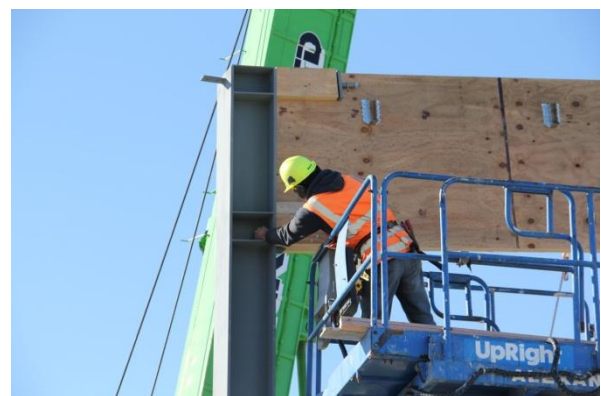
## 5.3 GENERAL CONSTRUCTION SEQUENCE FOR THE QUICK CONNECT JOINT

The construction sequence for the Quick Connect joint allows much of the work to be completed on the ground. The construction sequence for all connection areas, i.e. the knee, splice and apex joint is similar. To illustrate the general sequence a steel column to timber rafter knee joint is considered.

The sleeves are cut to length and the channels for the main tension rods are either cut or drilled. Screw holes can then be predrilled to ease screw installation. If an appropriate screw jig is available then this step can be



**Figure 9:** Roof section being lowered onto steel corbels ready for Quick Connect tension rod installation



**Figure 10:** Installer completing the Quick Connect joint by placing bearing plate and tightening nut on main tension rods

omitted. The completed sleeves are installed on the rafter or column members using fully threaded screws. Once all sleeves have been installed, the roof section is lifted into place and supported by crane until the main tension rods have been inserted, bearing plates have been put in place and nuts tightened.

A roof section being prepared for the lifting process is shown in Figure 8. The majority of the purlins have been fitted on the ground leaving only the inter-section purlins to be installed once the roof section has been secured. Figure 9 shows the roof section being lowered onto the steel corbels of the steel columns ready for the tension rods to be installed.

For larger structures, such as the Tumu ITM building, the remainder of the Quick Connect is most often assembled using a scissor hoist. The installer simply threads the bearing plates onto the main tension rods and tightens the nuts as shown in Figure 10 to complete the connection. This procedure is much more expedient than that required for the nailed gusset and results in saving in both labour and crane time.

## 5.4 KEY POINTS FOR FUTURE PROJECTS

Upon the completion of the major works for the Tumu ITM project a debrief was held onsite to discuss any difficulties encountered during the construction process

and detail any changes which could be applied to the Quick Connect joint. Members present at the meeting included representatives from Stratagroup, the engineering firm responsible for the overall project, Alexanders Construction and the client Tumu ITM. The debrief therefore resulted in feedback on the connection from all facets of the process including the views of the client.

#### 5.4.1 Screwed connections

The screwed connections between the sleeves and the main rafter members utilized 10mm fully threaded self-tapping screws with a length of 200mm. Screws were set in four rows as shown in Figure 11. Sleeves had a depth of 200mm resulting in a vertical centre to centre interline spacing of 40mm or  $4d$ , where  $d$  is the diameter of the screw. The close proximity of the screw lines, along with the length of the screws, caused some screws from opposing screw lines to interact. This interaction is undesirable and should be avoided in future projects. Solutions to this problem include increasing the depth of the sleeve or decreasing the diameter of the fasteners to be used.

The 10mm diameter of the screws resulted in difficulties during insertion and a decision was made soon after the start of construction to predrill the sleeves and rafter before inserting the screws. The predrilling process is time consuming and requires double handling of the members. A recommendation has therefore been made that screw diameters are limited to 8mm to ensure ease of installation.

The length of the screws resulted in the screw exiting the main rafter member and penetrating the opposing sleeve. Whilst this provides some benefit in terms of screw connection strength and stiffness it causes the opposing sleeve to be pushed away from the main member as the screw passes through the member interface. This

tendency was counteracted in this project by clamping the sleeves. However, for ease of construction, it is recommended that the screw connection for all future projects be designed so that the screw tip terminates in the main rafter.

This can be achieved by employing a shorter screw, which may not be possible for some screw diameters. Alternatively the angle between the screw axis and the direction of applied force can be modified. After obtaining advice from the Karlsruhe Institute of Technology (KIT) further testing has been performed on selected screws with differing angles of insertion. For practical reasons, an angle of 45 degrees has been adopted. Using this angle, rather than the 60 degree angle which was specified in the original connection design guidance, results in improved strength and stiffness characteristics of the screw connection.

The screws used in the projects were the flared head type. These required countersinking to avoid the screw heads snapping off on contact with the timber sleeve. Using screws with cylindrical heads would eliminate this requirement and result in a cleaner look. Examples of both screw types are shown in Figure 12.

#### 5.4.2 Tolerances

To avoid unnecessary deflections and rotation at the joints tight tolerances were specified for the project. These tolerances affected the sizing of the embedment holes for the shear transfer system (in the apex and splice joints), the diameter of the rod channels in the sleeves and the diameter of the receiving holes in the steel column members. The tolerances applied, usually a maximum of  $\pm 2\text{mm}$ , resulted in some difficulties during erection. The majority of the difficulties faced during erection may be attributed to a small amount of cupping in the main members which occurred as a result of weather exposure and slight differences in the billet



**Figure 11:** A splice joint with four screw rows, per side, per sleeve



**Figure 12:** At top, flared head screw (SPAX) At bottom, cylindrical head screw (ROTHOBLAAS)



dimensions of the main LVL members. These issues did not significantly impact the shear transfer system or the rod channels in the sleeves. There was however a significant impact at the timber rafter to steel column interfaces as the cupping and dimensional differences of the timber rafter members resulted in difficulties in positioning the main tension rods in relation to the receiving holes in the steel column flanges. After some discussion it was noted that oversized receiving holes or slots in the steel member flanges would not affect the overall rigidity or integrity of the connection as the shear forces in this connection are transferred through a steel corbel.

#### **5.4.3 Design and installation of the shear transfer connection**

Two different shear transfer systems were used in the Quick Connect joints in this building. For the steel column to timber connections a simple steel corbel was used. For the timber to timber splice and apex connections embedded shear dowels were added to the central region of the member to avoid any significant loss in moment capacity in the main member.

For this building the construction company decided to glue the shear rods at the splice joint. The glue used required heat curing which proved difficult onsite. In future, a clear directive stating that no gluing is required will be given as part of the preliminary design notes. If a client wishes to use glue then a cold curing glue should be used.

The specification for the shear transfer system utilizing rods did not include direct guidance on the use of threaded or smooth rods. Alexander Construction, after discussion with the project engineers Stratagroup, decided to use smooth bars. It was deemed likely that the use of threaded bars would increase the likelihood of the bars jamming during erection.

#### **5.4.4 Standardization of the Quick Connect joint**

The joints used in this building were standardized for the different joint regions but not for the overall project. Providing a standardized connection detail for a given main member size (based on the main member ultimate capacity) would reduce the amount of quality control required onsite and reduce the risk of inadvertent use of wrong sized components.

For this particular project, the main point of concern was the bearing plates. Due to the small differences in thickness these needed constant checking to ensure that they were mounted on the appropriate connection. Standardized connection sizing would allow for more accurate costing of projects.

#### **5.4.5 Prefabrication of connection components**

Whilst the prefabrication of connection components for the Quick Connect system is recommended it is not a requirement. For the Tumu ITM project, the construction company, Alexanders Construction, opted to manufacture onsite. During the debrief it was noted that

whilst this approach was a success in this project, substantial reductions in onsite quality control measures would result from pre manufacturing the sleeves offsite. Ideally, sleeves would arrive onsite cut to length, with the channels for the main tension rods and screw holes predrilled. The predrilling in the sleeves in this case is not a requirement for splitting but serves to ease construction by ensuring accurate placement of the screws.

#### **5.5 General discussion and feedback on Tumu ITM Quick Connect joints**

The client and industry members present at the debrief session were asked to comment on all aspects of the project. The overall opinion was that the Quick Connect joint resulted in a much cleaner look than a steel plate gusset. Rather than the eye being drawn to the steel in the connections the Quick Connect promotes the aesthetic appeal of the timber roof structure. An overall reduction in erection time was observed when comparing the Quick Connect building to a building with nailed gussets.

### **6 CONCLUSIONS**

To be competitive in today's industrial and commercial buildings sectors designers and construction firms of timber buildings must maximize the advantages which timber offers over concrete and steel.

This paper has introduced a connection designed in response to a need for timber portal frame buildings to become more expedient and economically competitive. The Quick Connect joint allows much of the fabrication and assembly work to be completed offsite. The offsite fabrication approach results in a reduction in plant and labour time and a reduction in critical construction path lengths.

The first use of the Quick Connect in the Tumu ITM facility in Napier is described. Advantages and disadvantages are discussed and solutions presented which will eliminate the disadvantages encountered in this project for future projects.

The main issues identified are the tolerances and screw dimensions, screw placement and angle of insertion. The recommendations included in this paper give details of modified screw connection geometries and a revision to the initial insertion angle of 60 degrees between the normal angle of the screw to the applied load to an angle of 45 degrees.

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