FIBER REINFORCED POLYMER (FRP) PULTRUDED SHAPE STRUCTURAL CONNECTIONS

by

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Abstract

This report discusses the two main types of structural connections used for fiber reinforced polymer (FRP) pultruded shapes, which are mechanical and bonded connections. The most common types of mechanical and bonded connections for FRP pultruded shapes are bolted and adhesively bonded joints respectively, and the advantages and disadvantages of each are discussed. Bolted connections are the most common type of connection used for FRP pultruded shapes and are therefore the focus of this report. Limit states and critical stresses for FRP bolted connections are explained along with the appropriate material properties that are needed to determine them. A simplified mechanics approach to determining the stresses in the FRP material and connection is presented along with a design procedure for FRP connections. A design example is given for a simple beam-to-column shear connection using three materials: FRP pultruded shapes, W-flange steel shapes, and wood sawn lumber in which the beam-to-column shear connection is compared.

It is found that the FRP connection is comparable to the steel and wood connections, and all three are able to meet the requirements for the loading conditions given with reasonable results. Possible uses for FRP that would be more ideal than using steel or wood members are presented and areas that still need to be developed or require further research are discussed.

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A_b	Area of bolt (in^2).
A_{heel}	Area of the heel of the angle. (in^2).
A_{net}	Net tensile area in the base material (in^2).
A_{shear}	Area along the failure path that experiences shear (in^2).
A_{tens}	Area along the failure path that experiences tension (in^2).
b_{leg}	Width of the leg of the angle (*in.*).
d	moment arm of bolts due to eccentricity which cause prying action (*in.*).
d_b	Diameter of bolt (*in.*).
d_h	Diameter of bolt hole (*in.*).
d_w	Diameter of washer (*in.*).
e	End distance between the centerline of the bolts to the edge of the base material in the direction of the load (*in.*).
e_v	Eccentricity of fasteners in connection (*in.*).
g	Gage. Center to center spacing of bolts in the direction perpendicular to the load (*in.*).
$l_{\measuredangle}$,	Length of the angle (*in.*).
M	Maximun moment in a beam (*lbft*).
m_{leg}	Resulting moment applied to the leg of an angle caused by eccentricity and prying action (*lbin*).
M_v	Moment due to connection eccentricity (*lbin*).
n	Number of bolts in connection.
P	Axial factored load on a column (*lbs*).
p	Pitch. Center to center spacing of bolts in the direction of the load (*in.*).
P_b	Load transferred to an individual bolt (*lbs*).
P_n	Nominal tensile capacity of the base material (*lbs*).
P_t	Tensile load that is transferred by the entire joint (*lbs*).
R	Resultant load due to eccentricity and prying action (*lbs*).
R_b	Resultant load for bearing (*lbs*).

R_x	Horizontal resultant load due to eccentricity and prying action (*lbs*).
R_v	Vertical resultant load (*lbs*).
SF	Safety factor
s	Side distance between the centerline of the bolts to the edge of the base material in the direction perpendicular to the load (*in.*).
σ_{allow}^{*}	Allowable normal stress with safety factor applied (*psi*).
σ_{br}	Average bearing stress at the bolt hole of the base material (*psi*).
σ_{cr}^{brg}	Critical bearing stress (*psi*).
$\sigma_{L,br}$	Bearing strength of the base material in the longitudinal direction (*psi*).
$\sigma_{T,br}$	Bearing strength of the base material in the transverse direction (*psi*).
σ_{cr}^{b}	Critical tensile stress of bolt (*psi*).
σ_{trans}	Transverse tensile stress of a web flange junction (*psi*).
σ_{net}	net-tension design stress at the bolt hole in the base pultruded material (*psi*).
$\sigma_{cr}^{net-tens}$	Critical net-tensile stress (*psi*).
$\sigma_{L,t}$	Longitudinal tensile strength of the base material (*psi*).
$\sigma_{T,t}$	Transverse tensile strength of the base material (*psi*).
σ_{flex}	Flexural stress in the leg of the angle caused by eccentricity and prying action (*psi*).
σ_{cr}^{flex}	Critical flexural strength (*psi*).
$\sigma_{L,flex}$	Longitudinal flexural strength of the base material (*psi*).
$\sigma_{T,flex}$	Transverse flexural strength of the base material (*psi*).
τ_{allow}^{*}	Allowable shearing stress with safety factor applied (*psi*).
τ_b	Shear stress of a bolt (*psi*).
$\tau_{shear-out}$	Shear out stress in the base material (*psi*).
τ_{cr}	Critical shear or shear out stress (*psi*).
τ_{leg}	Shear stress in the leg of an angle due to eccentricity and prying action (*psi*).
τ_{LT}	In-plane shear strength of the base material (*psi*).
τ_{cr}^{TT}	Critical interlaminar shear stress (*psi*).
τ_{TT}	Interlaminar shear strength of the base material (*psi*).
τ_{pl}	Longitudinal shear stress along the heel of an angle (*psi*).
τ_{cr}^{b}	Critical shear stress of bolt (*psi*).

τ_{ult}^{b}	Ultimate shear strength of bolt (*psi*).
T_{max}	Maximum tensile load of bolt (*lbs*).
t_{col}	Thickness of the column (*in.*).
$t_{\measuredangle}$	Thickness of the angle (*in.*).
t_{bm}	Thickness of the beam (*in.*).
t_{pl}	Thickness of the base material (*in.*).
t_{web}	Thickness of the web of a W or I shape section (*in.*).
V	Total factored shear force (*lbs*).
V_b	Shear force acting on an individual bolt (*lbs*).
v_{leg}	Shear force in the leg of the angle caused by eccentricity and prying action (*lbs*).
W	Distributed factored load on a beam (*lbs/ft*).
w	Plate width of the base material in the direction perpendicular to the load (*in.*).

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Chapter 1 - Introduction

This report covers a study of different types of pinned connections for glass fiber reinforced polymer (FRP) pultruded structural shapes. A structural connection must be able to develop the required strength and stiffness necessary for the structure to perform over the service life of the building (Clarke, 1996). If a primary structural connection fails, the performance of the structure is affected as a whole. Therefore, it is important to understand the mechanics of a connection, and what will be required of that connection. A structural connection should be able to safely transfer forces from one member in the structural system to another. This can be accomplished by ensuring that the connection strength is, at a minimum, equal to the weakest member framing into it (ASCE, 1984).

Connections are the most demanding part of an FRP structure due to the physical properties of the material. Two major concerns when designing FRP connections are brittle failures, and the proper orientation of the material. The characteristics and performance of FRP pultruded structural shapes are not yet fully understood, which makes the design of their connections that much more critical.

This report discusses the two main connection types that are currently used in FRP structures today, and which type is the most advantageous to building structures and why. Some possible limit states and critical stresses that could develop in FRP connections are discussed and then a pinned connection example is presented and designed. This connection is designed using three different types of materials to meet specific loading requirements. It is designed using FRP structural shapes, steel members, and wood members. The results of each of these connections are compared and discussed.

Chapter 2 - Connection Types

The three main types of connections for FRP pultruded structural shapes are: mechanical, bonded, and combined connections. Each connection type has distinct characteristics which create different stress and strain concentrations in the pultruded members. All three are viable options for structural connections when they are developed properly. Some criteria to help determine which type of connection to use are (ASCE, 1984):

- Loads to be transferred
- Joint efficiency requirements
- Geometry of joining members
- Fabrication requirements (quantity, dimensions)
- Service environment and life of the structure
- Disassembly requirements
- Sealant/water-tightness restrictions
- Cost/weight considerations

Each type of connection has advantages and disadvantages for the different criteria listed that will aid in the decision making process.

Mechanical Connections

Mechanical connections are the most common type of connection for FRP shapes. Different mechanical connections include bolts, rivets, clamped joints, and threaded joints, among many others (Clarke, 1996). Bolted connections are the most common mechanical connection in building structures.

Bolted connections are advantageous to use for many reasons. They are very similar to steel connections, making them an easy transition for both engineers and fabricators alike. Field or shop assembly are both possible with unsophisticated tooling, and they can be easily inspected. They also have a relatively low cost and a quicker assembly as compared to bonded connections (Chen, 1997).

Some disadvantages associated with bolted connections are the stress concentrations that they create in the members at the bolt holes, and their relatively low structural efficiency [Chen, 1997]. Due to the unidirectional characteristic of FRP pultruded shapes and an incomplete understanding of the material behavior, these high concentrations at the bolt holes require large safety factors, which lead to the low structural efficiency of the connection. There is also a lack of high-strength FRP fasteners, so steel bolts are sometimes used instead of FRP fasteners (Chen, 1997).

The geometry of a bolted connection is important to maximize the structural efficiency and capacity of a connection. Lawrence C. Bank presents some recommended geometries for lap joints that were determined through an extensive research database and are shown in Table 2.1 and Figure 2.1 below (Bank, 2006). These values are based on normal room temperature and normal humidity level service conditions.

Table 2.1 Recommended Geometric Parameters for Lap Joint Connections

		Research Data		Manufacturer[a]	
		Recommended	Minimum	Recommended	Minimum
End[b] distance to bolt diameter	e/d_b	≥ 3	2	≥ 3	2
Plate width to bolt diameter	w/d_b	≥ 5	3	≥ 4	3
Side distance to bolt diameter	s/d_b	≥ 2	1.5	≥ 2	1.5
Longitudinal spacing (pitch) to bolt diameter	p/d_b	≥ 4	3	≥ 5	4
Transverse spacing (gage) to bolt diameter	g/d_b	≥ 4	3	≥ 5	4
Bolt diameter to plate thickness	d_b/t_{pl}	≥ 1	0.5	2	1
Washer diameter to bolt diameter	d_w/d_b	≥ 2	2	NR	NR
Hole size clearance	$d_h - d_b$	tight fit ($0.05d_b$)	$1/16$ in.[c]	$1/16$ in.	NA

[a] NR, no recommendation; NA, Not applicable.

[b] Also called edge distance.

[c] Maximum clearance.

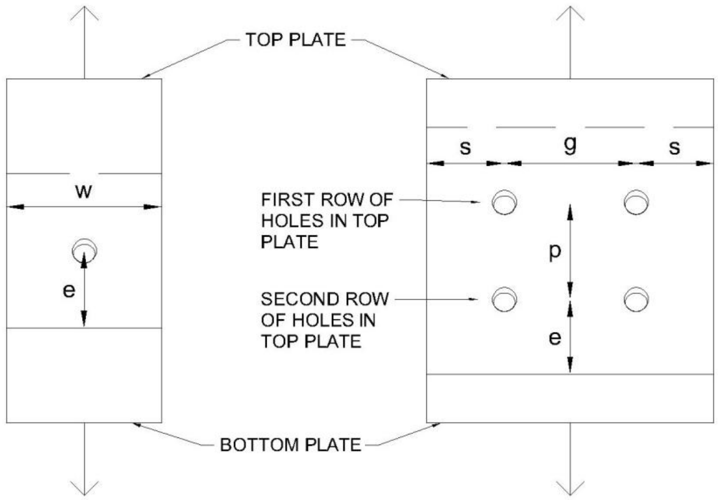


Figure 2.1 Geometric Parameters for Single- and Multi-bolt Lap Joints

A recent investigation by Coolinig Technology Institute (Troutman & Mostoller, 2010) has shown several characteristics pertaining to the geometries of bolts and bolt holes in a member. Troutman & Mostoller (2010) conducted a set of experiments on three different pultruded profiles with thicknesses of 0.25”, 0.30”, and 0.50” in standard operating and laboratory conditions. They performed a series of pin bearing strength tests on the three profiles using ¼”, ½”, and ⅝” diameter pins. The profiles using the ¼” diameter pins were tested using the ASTM D 953 compression test method and the profiles using the larger diameter pins were tested using the modified version of the ASTM D 953 compression protocol in order to accept the larger pin diameters. Through these tests they have shown that an increased bolt diameter will actually decrease the bearing strength, and as the clearance hole size increases the bearing strength will decrease (Troutman & Mostoller, 2010). This is true for both longitudinal and transverse orientations.

However, the solution to these findings is not to use the smallest bolt possible and as many as required to develop the appropriate strength. For one, it would be hard to make them all fit in the available space, and two, the load will not distribute evenly among all of the bolts. The outer bolts, or the bolts closest to, and farthest from the applied load will experience larger loads than the bolts in the middle of the row. Due to this distribution of load, it is recommended that FRP pultruded connections have at least two bolts in a row, but no more than four bolts in a row (Bank, 2006). Any more than that is an inefficient use of material because the load difference between the inner and outer bolts would be so great. For rows with more than four bolts, the outer bolts could see almost twice the amount of load as the inner bolts. Eventually, the inner bolts would have very little contribution to the strength of the connection. This load distribution between bolts is similar to what is seen in wood connections that is accounted for when the group action factor is applied (American Forest & Paper Association, 2005). This is unlike what is seen in steel connections though, where it is assumed that the load is distributed evenly among all of the bolts in all of the rows (Bank, 2006). Connections are also typically designed for the governing fastener, or the fastener that sees the largest load, and all of the bolts are sized according to this fastener. This would mean that the inner bolts would become greatly oversized for the load that they are actually going to see. Therefore, a bolted connection design is a balancing act between the quantity and size of bolts used.

Cooling Technology Institute also found that the bearing capacity is decreased when the threads are in the bearing zone (Troutman & Mostoller, 2010). Therefore, it is desirable to design connections without threads in the bearing zones or shearing planes whenever possible to get the most out of a connection.

Bolted connections are the most common type of connection used in FRP structures because of their ease and familiarity to both design engineers and fabricators (Bank, 2006). They can be assembled in the field and do not require a controlled environment like bonded connections.

Bonded Connections

Bonded connections consist of adhesively bonded joints, laminated joints, and cast-in joints, among others (Clarke, 1996). Adhesively bonded joints are the most common bonded connection in composite structures.

Adhesive connections have a higher joint efficiency compared to bolted connections due to the more uniform distribution of the load to the member. They have a more uniform stress distribution to the connecting members, so their stress concentrations are not as significant as they would be in bolted connections. Adhesive bonds are also generally stiffer than their counterparts in mechanical connections and they can provide a water-tight seal (Clarke, 1996; Keller & Vallée, 2005).

The design of an adhesive connection requires careful selection and consideration. When developing a connection, it is important to specify an appropriate adhesive that will properly bond to the FRP material and develop the required strength and ductility (ASCE, 1984). It should also be certain that the adhesive can withstand the service environment that it will be exposed to so that it will not degrade over time, and an appropriate thickness should be specified to ensure adequate strength. In order for the adhesive to bond properly to the FRP, the members should be roughened and cleaned. Any dirt or oil on the surface of the FRP shape can substantially reduce the quality of the connection. During the fabrication process, service conditions such as temperature, humidity, and pressure should be monitored to ensure proper curing and strength development (ASCE, 1984).

For these reasons, adhesive connections are best developed in a controlled environment and not fabricated in the field. This quality control helps ensure that the connection that was designed is the connection that is developed.

It is also advisable that adhesive bonds be limited to shear and compression connections and not subjected to tensile loads for FRP pultruded shapes. Tensile loading in the adhesive promotes peeling and cleavage action which could cause the joint to fail (ASCE, 1984).

Although bonded connections are commonly used with composite material structures of all kinds, their use in pultruded structures is very limited (Bank, 2006). Engineers rarely rely on bonded materials to transfer all of the loads between a connection because of a concern for the strength of the connection over the life of the structure. Engineers also like to avoid having all of their connections required to be prefabricated to make construction easier.

Combined Connections

Combined connections are a combination of both mechanical connections and bonded connections. For example, a connection that uses both bolts and adhesives is a combined connection.

The advantage to using this type of connection is to combine the advantages of the individual types of connections. In a combination bolt and adhesive connection, the stress concentrations at the bolt holes would not be as high as they would be without the adhesive bond, and it will still be a water-tight connection (Clarke, 1996). The sensitivity to peeling and cleavage from a tensile load is also reduced (Clarke, 1996). However, this type of connection is still difficult to inspect because of the adhesive bond, and a controlled environment and cure time are still required to develop full strength.

The bolts can also provide support and apply pressure during the assembly and cure time of the adhesive bond, and can therefore help prevent any defects that might occur between the contact surface of the adhesive bond and the FRP member. Such defects at the contact surface are otherwise known as bondline defects (Clarke, 1996). However, this connection type is a redundant design because both the bond and the fasteners are designed to carry the load as if they were supporting the load on their own. In an intact joint, the bolts are technically carrying zero load and are therefore unused during the life of the structure. They will only be utilized if the bonded joint becomes inadequate (Clarke, 1996).

Since combined connections initially rely on the adhesive bond for the strength of the connection and the bolts act as merely a backup system, this type of connection is not typically used in FRP pultruded structures.

Mechanical fasteners are the connection type of choice for FRP pultruded structural shapes and are the focus of this report. In order to design a mechanical connection for FRP shapes, an understanding of the properties and characteristics, specifically limit states and critical stresses, is required.

Chapter 3 - FRP Properties and Limit States

An understanding of limit states and critical stresses for FRP pultruded shape connections requires an understanding of the physical and characteristic properties of the material. FRP pultruded shapes have many characteristics that are advantageous for use in structural systems, but they also have characteristics that could be very detrimental to the structure.

Material Properties

FRP is made up of fibers and resin which are bonded together creating a composite material. The fibers provide the strength and stiffness for the material and the resin is environmentally resistant which protects the fibers and holds everything together (Davalos, 2006). Glass fibers are the most common type of fiber in pultruded structural shapes due to their high strength and relatively low cost. Other fibers that can be used are carbon or aramid fibers which have a higher strength and modulus of elasticity, but have a higher cost (Creative Pultrusions Inc., 2002). These fibers are typically only used in FRP pultruded shapes under special circumstances where the higher cost is outweighed by the added benefits. Typical resins used are polyester and vinyl ester, depending on the strength requirements (Creative Pultrusions Inc., 2007). These are also more common than other types of resins, such as epoxies, phenolics and urethanes, because of their relatively low cost (Creative Pultrusions Inc., 2002; Davalos, 2006).

The resin acts as a protectant for the fibers as well as a bonding agent. Resins are resistant to corrosion from such things as harsh chemicals, salts, moisture, and decay and therefore protect the fibers which are not as resistant (Davalos, 2006). Resins can be susceptible to UV damage, but there are additives and surfacing veils that can be added when exposure is expected (Creative Pultrusions Inc., 2002). The common resins are thermosetting resins that hold their shape permanently once they have set. Thermoplastics can also be used in FRP shapes, but this type of resin will liquefy when reheated which is not ideal in typical building applications (Daniel & Ishai, 2006).

Fiberglass reinforcement can come in the form of rovings, continuous strand mats, woven, or stitched fabrics, and all can be used in one structural shape (Creative Pultrusions Inc., 2002). Roving layers consist of continuous unidirectional fiber bundles, while continuous strand mats contain randomly oriented continuous fibers, and woven or stitched fabrics contain fibers that have been intertwined to form a fabric (Davalos, 2006). These rovings, mats, and fabrics are assembled in layers to create the desired thickness and strength requirements. A typical cross section of an FRP Pultruded shape is shown in Figure 3.1 below (Davalos, 2006). The orientation of the different fiber layers affects the material properties in the different directions of the material. When stitched fabrics are laid with their primary fibers perpendicular to the axis of the member, they can increase the transverse strength of the material (Davalos, 2006). They can also be laid at angles to the member, giving the section more strength in off-axis directions, or in shear. Due to the fabrication process of pultrusion, the roving layers can only be oriented along the axis of the member and therefore, will have the most effect along this axis, and very little effect in other directions (Davalos, 2006).

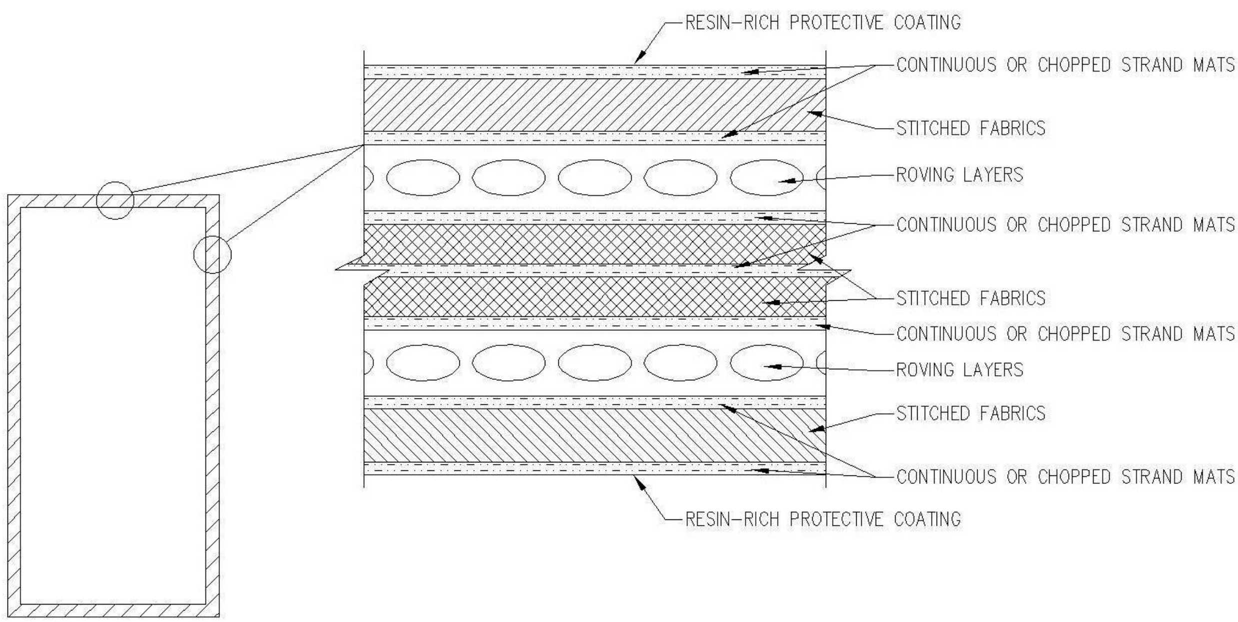


Figure 3.1 Typical Layer Arrangement of FRP Pultruded Structural Shapes.

The majority of strength for an FRP pultruded structural shape comes from the roving layers. They are the strongest layer in the longitudinal direction, but they do not have as much strength in the transverse direction, making the shape much weaker in the transverse direction (Davalos, 2006). The roving layers give FRP pultruded shapes unidirectional characteristics, causing different material properties and characteristics in different directions. This makes it very important to know the direction of loading and orientation of a member and connection prior to the design of the connection.

Many possibilities for the design and strength of a single FRP pultruded shape exist. Designing just one member could be a task in itself, because of the multitude of ways to arrange the different layers to develop required strengths and properties in every direction. However, manufacturers develop their own standards and shapes for use in structural applications. Rather than designing each individual layer for each member, it is more efficient to choose a manufacturer and use their standard member sizes and properties, much the same way that you would for steel or wood members. The difference from steel or wood members though, is that there is no set standard that all manufacturers use. They develop their sizes and standards on their own, and are therefore incompatible with each other. It is not possible to switch from one manufacture to another without redesigning all of the members. It is important that a manufacturer be chosen carefully and at the beginning stages of design. This report uses the members and properties given by Creative Pultrusions, Inc. and a list of their material properties for both fasteners and structural profiles are listed below (Creative Pultrusions Inc., 2007). This manufacturer was chosen because of their national and international presence and large production capabilities. They also publish and provide access to their standards and design manuals for design use.

Table 3.1 Material Properties for FRP Fasteners Using Flanged Hex Nuts

	Diameter /Threads per Inch					
Property	3/8" 16 UNC	1/2" 13 UNC	5/8" 11 UNC	3/4" 10 UNC	1" 8 UNC	Units
Ultimate Thread Shear Capacity[1,2,6]	1,250	2,500	3,900	5,650	7,400	lbs
Max. Ultimate Design Tensile Load[1,2,5]	1,000	2,000	3,120	4,520	6,200	lbs
Flexural Strength[2,3]	60,000	60,000	60,000	60,000	60,000	psi
Flexural Modulus[2,3]	2.0	2.0	2.0	2.5	2.75	10^6psi
Compressive Strength (L)[2,3]	55,000	55,000	55,000	55,000	60000	psi
Ult. Transverse Shear[2,3]	4,200	7,400	11,600	17,200	27400	load lb
Transverse Shear Yeild[2,3]	2,100	3,300	4,500	7,500	12500	load lb
Coefficient of Thermal Expansion (L)	3.0	3.0	3.0	3.0	3.0	10^{-6}in/in/°F
Torque Strength[2,4,5,6]						
Ultimate	8	15	33	50	115	ft-lb
Recommended	4	8	16	24	50	ft-lb
Stud Weight[3]	0.760	0.129	0.209	0.315	0.592	lb/ft
Thickness of Hex Nut	0.750	0.855	1.220	1.590	1.750	in
Diameter of Flange	0.745	1.000	1.250	1.950	2.000	in

(L): Longitudinal Direction

[1]Applies to single nut only; multiple nuts do not yield corresponding results.

[2]Ultimate strength values are average obtained in design testing by Creative Pultrusions, Inc.

[3]Values are based on unthreaded rod.

[4]Torque results are dependent on several variable factors including the lubricant used, the length of stud between nuts, alignment, washer surfaces, etc. Therefore, if such results of torque tightening are important, it is vital that torque limits be determined experimentally for the exact installation conditions. Values shown for CP Hex Nut Lubricated With SAE 10W30 Motor Oil.

[5]Appropriate safety factors must be applied.

[6]Properties apply to Superstud!™ used with CP Hex Nut.

Table 3.2 Material Properties for FRP Wide Flange Sections and I-Sections

Property	Material		Units
	Polyester	Vinyl Ester	
Physical			
Density	0.060-0.070	0.060-0.070	lb/in^3
Specific Gravity	1.66-1.93	1.66-1.93	
Coefficient of Thermal Expansion (L)	4.4	4.4	10^{-6}in/in/°F
Full Section			
Modulus of Elasticity (½" thick)	3.9	3.9	10^6psi
Modulus of Elasticity (¼" and ⅜" thick)	4.0	4.0	10^6psi
Shear Modulus	0.5	0.5	10^6psi
Flexural Strength	33,000	33,000	psi
Flange Section			
Tensile Strength (L)	40,000	46,000	psi
Tensile Modulus (L)	4.16	4.16	10^6psi
Compressive Strength (L)	45,770	52,500	psi
Compressive Strength (T)	17,800	20,400	psi
Compressive Modulus (L)	3.85	3.85	10^6psi
Compressive Modulus (T)	1.9	1.9	10^6psi
Flexural Strength (L)	42,800	49,200	psi
Flexural Modulus (L)	2.0	2.0	10^6psi
Interlaminar Shear (L)	4,000	4,500	psi
Shear Strength by Punch (PF)	5,500	6,000	psi
Maximum Bearing Strength (L)	33,000	38,000	psi
Maximum Bearing Strength (T)	23,000	26,500	psi
Poisson's Ratio (L)	0.35	0.35	in/in
Poisson's Ratio (T)	0.12	0.12	in/in
Web Section			
Tensile Strength (L)	30,300	35,000	psi
Tensile Strength (T)	10,500	12,000	psi
Tensile Modulus (L)	3.1	3.1	10^6psi
Tensile Modulus (T)	1.4	1.4	10^6psi
Compressive Strength (L)	37,500	43,125	psi
Compressive Strength (T)	14,200	16,330	psi
Compressive Modulus (L)	2.8	2.8	10^6psi
Compressive Modulus (T)	1.9	1.9	10^6psi
Flexural Strength (L)	43,320	49,800	psi
Flexural Strength (T)	17,360	19,900	psi
Flexural Modulus (L)	1.9	1.9	10^6psi
Flexural Modulus (T)	1.75	1.75	10^6psi
Interlaminar Shear (L)	3,400	3,900	psi
Shear Strength by Punch (PF)	5,500	6,000	psi
In-plane Shear (L)	7,000	7,000	psi
Maximum Bearing Strength (L)	33,980	39,000	psi
Maximum Bearing Strength (T)	30,000	34,500	psi
Poisson's Ratio (L)	0.35	0.35	in/in
Poisson's Ratio (T)	0.12	0.12	in/in

(L): Longitudinal Direction
(T): Transverse Direction

Table 3.3 Material Properties for FRP Structural Profiles: Rectangular Tubes, Channels, Angles, Square Tubes, Round Tubes

Property	Material		Units
	Polyester	Vinyl Ester	
Density	0.060-0.070	0.060-0.071	lb/in^3
Specific Gravity	1.66-1.93	1.66-1.93	
Coefficient of Thermal Expansion (L)	4.4	4.4	10^{-6}in/in/°F
Tensile Strength (L)	33,000	37,500	psi
Tensile Strength (T)	7,500	8,000	psi
Tensile Modulus (L)	2.5	3.0	10^6psi
Tensile Modulus (T)	0.8	1.0	10^6psi
Compressive Strength (L)	33,000	37,500	psi
Compressive Strength (T)	16,500	20,000	psi
Compressive Modulus (L)	3.0	3.0	10^6psi
Compressive Modulus (T)	1.0	1.2	10^6psi
Flexural Strength (L)	33,000	37,500	psi
Flexural Strength (T)	11,000	12,500	psi
Flexural Modulus (L)	1.6	2.0	10^6psi
Flexural Modulus (T)	0.8	1.0	10^6psi
Modulus of Elasticity	2.8-3.2	2.8-3.2	10^6psi
Shear Modulus	0.42	0.42	10^6psi
Interlaminar Shear (L)	4,500	4,500	psi
In-Plane Shear (L)	7,000	7,000	psi
Maximum Bearing Strength (L)	30,000	30,000	psi
Maximum Bearing Strength (T)	18,000	18,000	psi
Poisson's Ratio (L)	0.35	0.35	in/in
Poisson's Ratio (T)	0.15	0.15	in/in

(L): Longitudinal Direction
(T): Transverse Direction

The properties from Table 3.1-Table 3.3 are used in the design of the FRP connection presented in this report.

Limit States

Several limit states must be considered in connection design. An FRP connection can fail in many different ways, many of which, steel or wood connections do not fail in. In FRP bolted mechanical connections, bearing failure in the base material, shear out failure in the base material, net-tension or net-section failure in the base material, or shear failure in the connecting elements may occur. These types of failures are shown in Figure 3.2 (Clarke, 1996). You can also have splitting failure, which is also called cleavage failure, and cleavage failure is also known as block shear failure. These types of failures do not typically govern the design of FRP pultruded shape connections, and are therefore not covered extensively in this report.

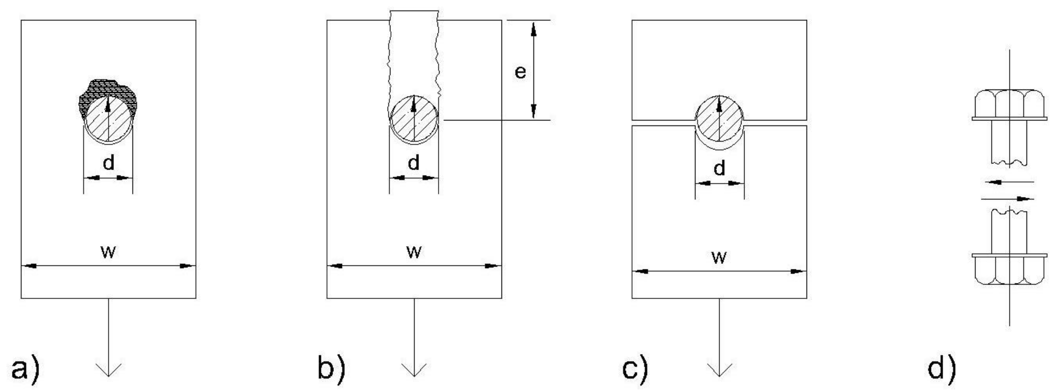


Figure 3.2 Failure Modes in Bolted Connections

a) Bearing failure; b) Shear-out failure; c) Net-tension failure; d) Fastener shear failure

The failures shown in Figure 3.2 are applicable to members loaded either perpendicular or parallel (transverse or longitudinal) to their major axis (pultrusion direction) and are not expected to occur in members loaded in an off-axis orientation (Bank, 2006). If a member is loaded in an off-axis direction, a combined net-tension and shear-out failure would most likely occur and tests should be conducted to determine the strength of this connection as opposed to calculations (Bank, 2006).

Bearing failure is caused by the bolt bearing on the base material and inducing a compressive stress (Clarke, 1996). This stress causes local crushing and delamination of the base pultruded material (Bank, 2006; Clarke, 1996). This type of failure is more likely to occur when the ratio of the width of the base material to the bolt diameter (w/d_b) is high (Clarke, 1996). Bearing failure is the preferred failure mode because it is a more ductile failure, unlike net-tension or shear-out failures which are brittle failures. A ductile bearing failure could lead to a progressive brittle failure however, as the connection is continually loaded. Once bearing failure has occurred, the connection could continue to slip and become a shear-out failure.

Shear-out failure occurs along shear-out planes that are tangential to the hole boundary and parallel with the direction of load. When the shear stress becomes too great, the material will shear along these planes and fail (Clarke, 1996). This failure is dependent on the end distance and orientation of the material (Clarke, 1996). Members loaded in the longitudinal direction are more susceptible to shear-out failures than those loaded in the transverse direction due to the unidirectional characteristics of the material. The shorter the end distance is, the more likely this failure is to occur.

Net tension failure forms along a plane that is perpendicular to the direction of loading and between bolt holes or the bolt hole and the edge of the material. This failure is caused by large compressive or tensile stresses at the edges of the hole and is more likely to occur when the ratio of the plate width to the hole diameter (w/d_b) is low (Clarke, 1996).

Bolt shear failure will occur when there are high shear stresses in the fastener (Clarke, 1996). The thicker the fastener is, the less likely this failure will occur.

When a connection is loaded so that it is subject to both shear and tensile loads, a block shear failure could occur. A good example of this is a beam that is coped at the location of the connection. This type of failure is a combination of a net-tension failure and a shear-out failure, and the two cannot be treated separately. An interaction equation between the two limit states is used in design.

The recommended geometric parameters for lap connections presented in Table 2.1 were developed with the intention of causing a bearing failure, which is a ductile failure as opposed to a brittle failure like the other types mentioned (Bank, 2006). Ductile failures are preferred in any structure because they are a progressive failure that happen over time and give warning, instead of brittle failures which happen suddenly and with very little warning. Even with these geometric recommendations though, if more than one bolt is in each row of fasteners, the connection could still have a net-tension failure instead of a bearing failure. Careful attention is required when designing a connection with multiple bolts per row.

Glass FRP pultruded structural shapes have low through-the-thickness strength and stiffness properties (Bank, 2006). If bolts are torqued, they could possibly crush or punch right through the FRP material. Also, due to the lack of stiffness, the FRP material could creep, and the tension from the torque could decrease over time from strain relaxation in both the base material and the FRP fasteners. For this reason, only bearing connections are used in FRP bolted connections and slip-critical or friction connections are not possible (Bank, 2006).

Connection Characteristics

In an FRP connection, the load is distributed differently between the different rows of fasteners. This is similar to wood connections but different from steel connections where it is assumed that the load is transferred evenly among all of the bolts. Research has shown that the rows closest to the edge of the material, or those closest to the applied load carry more load than rows further from the material edge or from the applied load (Bank, 2006). The table below shows the load distribution to bolts in each row (Clarke, 1996). The value shown should be multiplied by the average load that would be distributed to each fastener if they were to share the load equally. This will then give you the amount of load that is actually applied to each bolt. In connection design, it is recommended to have a minimum of 2 rows of fasteners but it is not recommended to have more than four rows (Clarke, 1996).

Table 3.4 Distribution of Load in Fasteners of Multi-row FRP Lap Joints

Number of Rows	Row 1	Row 2	Row 3	Row 4
1	1.0			
2	1.0	1.0		
3	1.1	0.8	1.1	
4	1.2	0.8	0.8	1.2

In an FRP connection, the bolt hole diameter is typically $^{1}/_{16}$” larger than the bolt diameter due to constructability reasons, similar to steel (Bank, 2006). This is not ideal for FRP shapes however, and should be avoided if possible. Since FRP connections are bearing connections, the gap created from the oversized hole allows the connection to slip before it can bear on the connected member. This slippage can cause added stress, rotation, and deflections to the FRP structure (Bank, 2006). It is possible to construct an FRP connection without oversized holes as long as the holes are drilled accurately. The fasteners would be relatively easy to insert with just a light tapping action (Bank, 2006).

Even though it is ideal not to oversize the bolt holes, in general construction it is still common to oversize them for ease of constructability. This allows for slight inaccuracies when members are fit together. For this reason, a $^{1}/_{16}$" oversized hole will be used in the following design examples.

When the load is transferred to the fasteners it is typically not concentric with the centroid of the connecting member. A typical beam to column connection using clip angles is a good example of this. The top and bottom fasteners in the clip angle to beam connection are transferring load above and below the centroid of the beam to column connection respectively, as shown in Figure 3.3. This causes an eccentricity in the plane of the web of the beam. This eccentricity will cause a localized bending moment in the plane of the member in addition to the other loads that it is experiencing. The clip angle to column connection also has added forces due to this eccentricity. The fasteners at the top of the clip angle experience tensile forces while the bottom fasteners experience compressive forces. The tensile forces could lead to failure due to prying of one material face away from the other. This could happen in one of two ways. The angle could pull away from the flange of the column, or the flange of the column could pry away from the web of the column. This separation of the web and flange is known as delamination of the material. These eccentric forces also occur in steel connections as well, but are not typically critical to the design of the connection. It is a critical limit state for FRP connections however, because of the materials unidirectional characteristics. These unidirectional characteristics make the material susceptible to delamination and prying forces.

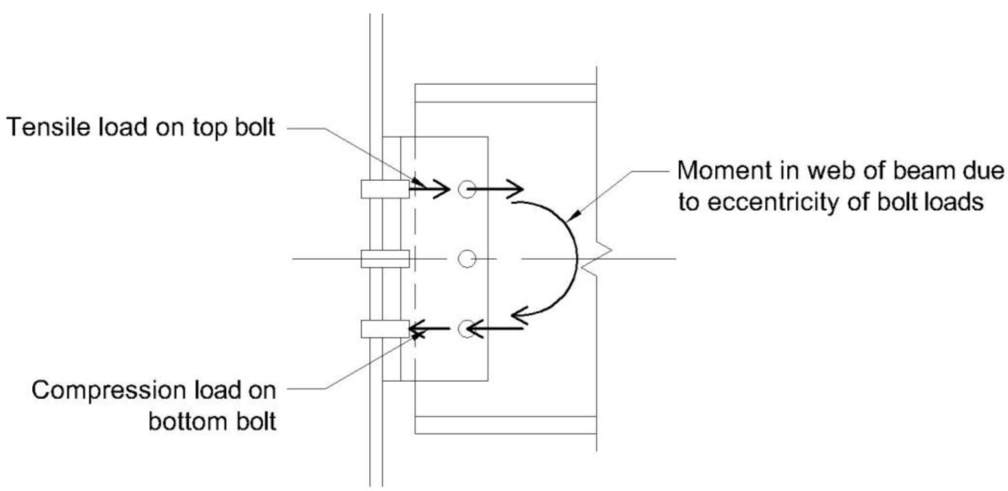


Figure 3.3 Forces Caused by Eccentricity of Beam-to-Column Connection

Critical Stresses

The critical stresses are the maximum stresses that the material can experience before failure. These stresses are found through testing of the material and are provided in manufacturer publications, or through testing arranged and analyzed by the engineer or other qualified professionals. These stresses are determined around the ultimate limit state of the material and not serviceability. Therefore, the critical stresses are determined by the ultimate strength for each limit state.

The critical stress depends on the orientation of material and the direction of loading. For the limit state of bearing stress, the critical stress will be equal to the bearing strength of the material in either the longitudinal or transverse direction (Bank, 2006).

$$\boldsymbol{\sigma_{cr}^{brg} = \sigma_{L,br}\ or\ \sigma_{T,br}}$$ **Equation 3.1**

where σ_{br} is the bearing strength of the pultruded material in either the longitudinal or transverse direction.

The critical strength for shear and shear-out failure is equal to the in-plane shear strength of the material (Bank, 2006):

$$\boldsymbol{\tau_{cr} = \tau_{LT}}$$ **Equation 3.2**

The critical strength for a net-tensile failure is equal to the tensile strength of the material in either the longitudinal or transverse direction depending on the orientation of the material multiplied by a strength reduction factor (Bank, 2006).

$$\boldsymbol{\sigma_{cr}^{net-tens} = .9\sigma_{L,t}\ or\ .9\sigma_{T,t}}$$ **Equation 3.3**

The reduction factor of .9 is suggested for conventional pultruded glass FRP materials and depends on the fiber layup, fiber and resin properties, and hole diameter to member width ratio, d_h/w. This suggested factor should only be used when d_h/w is below 0.2. If this requirement is not met, the strength reduction factor can be found experimentally using the ASTM D 5766 open-hole strength test (Bank, 2006).

The critical flexural stress, such as that that would occur in the angle of a beam to column connection from prying forces is equal to the longitudinal or transverse flexural strength of the material (Bank, 2006).

$$\boldsymbol{\sigma_{cr}^{flex} = \sigma_{L,flex}\ or\ \sigma_{T,flex}} \qquad \textbf{Equation 3.4}$$

The critical stress for interlaminar shear failure, which might also occur in the angle of a beam to column connection, is equal to the interlaminar shear strength of the FRP material (Bank, 2006).

$$\boldsymbol{\tau_{cr}^{TT} = \tau_{TT}} \qquad \textbf{Equation 3.5}$$

The critical shear for a bolt is the same as the ultimate shear strength of the bolt. This is true for both FRP and steel bolts used in a connection (Bank, 2006).

$$\boldsymbol{\tau_{cr}^{b} = \tau_{ult}^{b}} \qquad \textbf{Equation 3.6}$$

The critical tensile strength for a bolt is determined by the maximum tensile load, T_{max} (Bank, 2006). This value is determined by the maximum load that the bolt can take before the threads shear off.

$$\boldsymbol{\sigma_{cr}^{b} = \frac{T_{max}}{A_b}} \qquad \textbf{Equation 3.7}$$

The critical stress due to block shear failure requires an interaction equation between the tensile and shear strengths of the material. Since the limit states of net-tension and shear-out failures are both considered brittle failures, a linear interaction equation can be used. This relationship is given by Equation 3.8 (Bank, 2006).

$$\boldsymbol{P_n = \sigma_{cr}^{net-tens} A_{tens} + \tau_{LT} A_{shear}} \qquad \textbf{Equation 3.8}$$

where P_n is the nominal tensile capacity, A_{tens} is equal to the area along the failure path that undergoes tension, and A_{shear} is equal to the area along the failure path that experiences shear.

Chapter 4 - Connection Design

A simple beam to column connection is designed and analyzed using three different materials: FRP pultruded structural shapes, steel shapes, and wood members. This connection uses clip angles and bolts for the FRP and steel connections, and steel plates with bolts for the wood connection. A simple detail of the beam to column connection is illustrated in Figure 4.1; more detailed connections are shown with each material type.

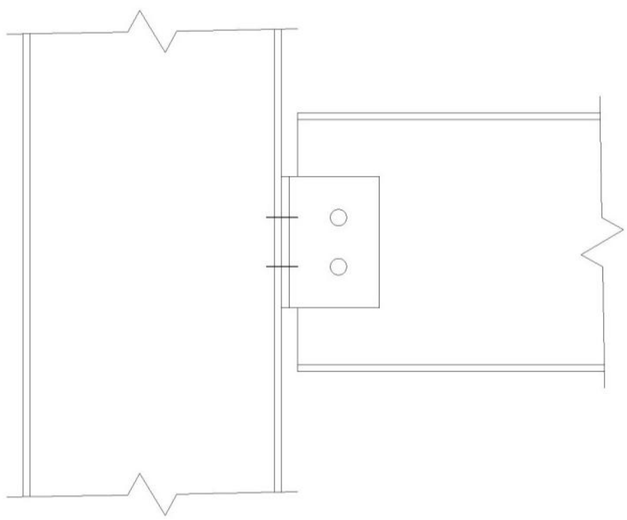


Figure 4.1 Beam to Column Connection Detail

The design method used was presented by Lawrence C. Bank in “Composites for Construction: Structural Design with FRP Materials”, and is based on simplified mechanics. In this simplified design, only uniaxial loads are applied to the connection and multi-axial loads are not considered. Therefore, combined in-plane tension and shear on bolts is not considered (Bank, 2006). Design methods for multi-axial loading exist, such as in the Eurocomp Design Code (1996), but the level of complexity required in the calculations are not usually considered appropriate for pultruded connections at this time, which is why a simplified mechanics approach is taken (Bank, 2006).

Bank’s design method is intended for specific types of connections. These connections are for in-plane lap-joint connections, such as overlapping joints or elements of beam to column connections, and out-of-plane beam to column connections for shear forces, such as web clip angle connections and flexible seated connections (Bank, 2006).

In the United States no national codes or standards for the design FRP pultruded connections currently exist. The American Society of Civil Engineers (ASCE) are currently developing a standard that will aid in the design of FRP pultruded connections, but has yet to be published. At the time of this report, the standard was being balloted and therefore was unavailable for review to aid in the research of this report. In late 2011, ASCE published a "Design Guide for FRP Composite Connections", but it is unknown how or if it will be referenced in the standard.

Until the ASCE standard is published, it is currently acceptable to use the basic principles used in steel-bolted simple shear connections while applying the FRP material properties and recommended geometries. However, since steel has isotropic material properties, it is important to also consider the orientation of the FRP member when using these basic principles in order to know which material properties to consider.

If it is determined that the FRP connection cannot reach the required capacity using the current member thicknesses, the connection can be reinforced by applying an adhesively bonded plate member to the inadequate members at the location of inadequate thickness. This will strengthen the member without requiring an increase in member size. The adhesive bond should be shop fabricated to ensure a proper bond and strength development. When plate stiffeners are added, it is recommended that the plates be oriented so that their longitudinal direction lies perpendicular to the members' longitudinal direction. This helps to provide a more isotropic section for the members with inadequate strength (Bank, 2006).

Determination of Stresses

The following equations for determining stresses in the FRP material are based on elementary one-dimensional mechanics and can be used for single- or multi-bolt lap joints. It should be noted as a reminder that whenever an equation uses a load for an individual bolt, the appropriate factor from Table 3.4 should be applied so that the actual load that the bolt will experience is accounted for. These equations are based on the assumption that the material is linear-elastic up to failure and experiences small deformations. The results of these equations can be compared to coupon-level material tests (Bank, 2006). FRP is designed for the ultimate limit state however, so the material does not behave linearly up until failure. As you approach the ultimate load, large deformations can change the stress distributions, and therefore the failure mode. As a result, these equations should be used as a guide to the strength of the material and not be relied upon to give exact results.

Bearing Stress in the Base Pultruded Material

Calculation of average bearing stress at the hole of the base pultruded material is given by:

$$\boldsymbol{\sigma_{br} = \frac{P_b}{d_b t_{pl}}} \qquad \textbf{Equation 4.1}$$

where P_b is the load that is transferred to an individual bolt, d_b is the bolt diameter, and t_{pl} is the thickness of the base material.

Net Tension Stress in the Base Pultruded Material

The net-tension design stress at the bolt hole in the base pultruded material is:

$$\boldsymbol{\sigma_{net} = \frac{P_t}{A_{net}}} \qquad \textbf{Equation 4.2}$$

where P_t is the tensile load that is transferred by the entire joint, and A_{net} is given as

$$\boldsymbol{A_{net} = t_{pl}(W - nd_h)} \qquad \textbf{Equation 4.3}$$

where n is equal to the number of bolts perpendicular to the load, W is the width of the plate, d_h is the hole diameter, and t_{pl} is the thickness of the base pultruded material. This equation assumes that all of the bolts in the effective area form a line that is perpendicular to the load. If the connection contains staggered rows whereas the line of failure could form at an angle that is not perpendicular to the load, then the following equation can be used to determine the effective net area,

$$A_{net} = t_{pl}\left(W - nd + \sum \frac{p^2}{4g}\right) \quad \textbf{Equation 4.4}$$

where p is the distance between bolts in the direction of the load, and g is the distance between bolts perpendicular to the load as shown in Figure 2.1. This equation sums the entire length of the possible failure path, including both the perpendicular and angled directions. Therefore, this equation assumes that the material has the same strength along all portions of the path. This is not true however, because the diagonal paths will have differing strengths from the perpendicular paths due to the unidirectional characteristics of the material. As a result, this equation provides a preliminary approximation of the tensile strength of the connection when using staggered bolts.

Shear-Out Stress in the Base Pultruded Material

The shear-out stress for the bolt located closest to the material edge that is in line with the direction of load is given as

$$\tau_{shear-out} = \frac{P_b}{2t_{pl}e} \quad \textbf{Equation 4.5}$$

where P_b is the load that is transferred to the single bolt, e is the end distance as shown in Figure 2.1, and t_{pl} is the thickness of the base pultruded material.

The shear-out stress between two bolts that lie in the direction of the load is given as

$$\tau_{shear-out} = \frac{P_b}{2t_{pl}p} \quad \textbf{Equation 4.6}$$

where P_b is the load that is transferred to the single bolt, p is the distance between two bolts that lie in the direction of the load as shown in Figure 2.1, and t_{pl} is the thickness of the base pultruded material. Equation 4.6 would only apply to locations where the edge shear-out stress is not a factor, such as in the flange of a column.

Shear Stress on a Bolt

The shear stress that acts on a single bolt is given by the equation 4.7.

$$\tau_b = \frac{V_b}{A_b} \qquad \textbf{Equation 4.7}$$

where V_b is the shear force acting on the individual bolt, and A_b is the cross-sectional area of the bolt shank.

The stresses shown above are for in-plane lap-joint connections. For out-of-plane shear connections, the following equations must be used in addition to those already discussed.

Longitudinal Shear Stress at the Heel of the Angle

The shear stress along the heel of an angle is given by

$$\tau_{pl} = \frac{V}{2A_{heel}} = \frac{V}{2t_{pl}l_{\measuredangle}} \qquad \textbf{Equation 4.8}$$

where V is the total shear force that the angle experiences, A_{heel} is the area of the heel, which is defined using t_{pl}, the thickness of the the angle and $l_{\measuredangle}$, the length of the angle. If this stress is greater than the ultimate shear strength of the material, then the connection will fail due to shear failure in the angle.

Flexural Stress in the Leg of an Angle Bolted to a Column

The flexural stress in the leg of the angle is the result of the tensile forces created due to the eccentricity of the connection which try to pry the angle away from the column. For the case where there are two angles on either side of a beam web, it can be assumed that the two angles act together as a fixed beam with a concentrated load at midspan. Refer to Figure 4.2. The resulting concentrated load is equal to the prying force and can be calculated as follows:

$$R = \frac{M_v d}{\sum d^2} \qquad \textbf{Equation 4.9}$$

where

$$M_v = Ve_v \qquad \textbf{Equation 4.10}$$

V is the total shear force, e_v is the eccentricity from the face of the column to the centroid of the bolts in the beam flange, and d is the distance of the bolts in the beam flange to the center of rotation, or the centerline of the beam. The resulting moment that is applied to the leg of each angle is

$$\boldsymbol{m_{leg} = \frac{R(b_{leg} - t_{pl} - s)}{8}} \qquad \textbf{Equation 4.11}$$

where b_{leg} is the width of the leg of the angle, and s is the distance from the center of the bolt hole to the edge of the leg of the angle. The shear force in the leg of the angle is given by

$$\boldsymbol{v_{leg} = \frac{R}{2}} \qquad \textbf{Equation 4.12}$$

which is used to determine the flexural stress in the leg of the angle as follows:

$$\boldsymbol{\sigma_{flex} = \frac{m_{leg}}{S} = \frac{m_{leg}}{2et_{pl}^2/6}} \qquad \textbf{Equation 4.13}$$

and the shear stress in the leg of the angle is

$$\boldsymbol{\tau_{leg} = \frac{V}{A} = \frac{v_{leg}}{t_{pl}(2e)}} \qquad \textbf{Equation 4.14}$$

This failure will occur when the shear stress reaches the interlaminar shear stress of the material and causes delamination in the fibers of the angle.

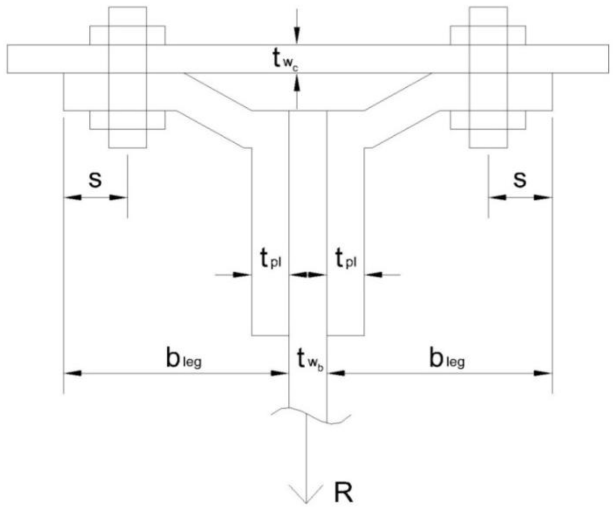


Figure 4.2 Prying Action of Angle to Column Flange Connection

Transverse Tensile Stress in the Web-Flange Junction of a Column

The junction between the web and flange of the column at the connection is also susceptible to the prying action caused by the connection eccentricity. The transverse tensile stress of the web-flange junction is given as

$$\boldsymbol{\sigma_{trans} = \frac{R}{t_{web}(l_{\measuredangle}/2)}}$$ **Equation 4.15**

Block Shear

Block shear is the result of a combined net-tension and shear-out failure and is given by the interaction equation below

$$\boldsymbol{P_t = \sigma_{net}A_{net} + \tau_{shear-out}A_{shear}}$$ **Equation 4.16**

where the values for net-tension and shear-out design can be found from Equations 4.2 – 4.6.

Design Procedure for FRP Pultruded Connections

An allowable stress design (ASD) will be used when designing the connection using FRP pultruded members. This is the most common design procedure used for FRP connections and is recommended by most manufacturers. A load and resistance factor design (LRFD) is not currently used because there has not been enough research into its development. A deeper understanding of the material and how it responds and reacts to loading is required before an LRFD design can be presented with confidence. The ASD design procedure used in this report follows the procedure presented in Lawrence C. Banks text (2006) and is outlined below.

- Step 1. Determine design loads and ASD factors.
  For FRP pultruded connections, a safety factor of 4 is used for all of the limit states in the connection. This is due to the assumptions made about the material and the limited knowledge of the material behavior. The loads applied to the connection are determined through statics, and the load to each member and fastener in the connection should be determined.
- Step 2. Select the connecting members and fasteners and dimension the connection.
  Determine the required member thicknesses of the connecting members based on the bearing strength of the pultruded material and the load that must be transferred. If the members have already been sized based on other loading criteria, such as bending for a beam or axial load for a column, start with these thicknesses. If they are not adequate, they can either be strengthened with an adhesively bonded plate or, if necessary, a thicker member may be chosen. The number of bolts required can be estimated based on the total shear or tensile force that must be transferred. A trial geometry must also be chosen at this stage and should follow the guidelines laid out in Table 2.1 and Figure 2.1
- Step 3. Determine the maximum design stresses.
  Use the equations presented earlier in this chapter and the geometry of the connection to determine the stresses in each part of the connection.
- Step 4. Determine the critical stresses and apply appropriate factors.
  The critical stresses can be found in manufacturers' publications or in Table 3.1 through Table 3.3 for the examples presented in this paper. Refer to Chapter 3 - Critical Stresses for information on which values are required for the different limit states and stresses. Note the orientation of the material to ensure that the appropriate strengths (longitudinal vs. transverse) are applied. Divide all of the critical stresses by the safety factor determined in Step 1.
- Step 5. Check the ultimate strength of the trial connection.

The design stresses found in Step 3 must be less than the factored critical stresses determined in Step 4. If this is not the case, return to Step 2 and adjust the trial connection design.

- Step 6. Dimension and detail all parts of the connection.
  Draw a detail of the design showing all dimensions, orientations, and member callouts for all parts of the connection.

Beam to Column Connection Example

In this section, the design of the beam-to-column connection as shown in Figure 4.1using FRP pultruded members, steel shapes, and wood members is presented. The structure is a one story building as shown in Figure 4.3. The section callout A-A is the detail shown in Figure 4.1 and is the connection that will be sized in this example. The top of the structure will be utilized as an elevated platform or walkway and supports floor grating and sub-purlins as well as a mechanical unit. The dead load for the structure accounts for the deck grating, sub-purlins, and the self-weight of the beam members. The sub-purlins are assumed to be 10 pounds per square foot (psf) and the self-weight of the beam members is also assumed to be 10psf. To determine the live load, the appropriate distributed live load from Table 4-1 of the ASCE 7-05 for elevated platforms and walkways is found. This value is 60psf and the mechanical unit is assumed to add an additional live load of 20psf. The mechanical unit is taken as a live load because it is assumed to vary with time as required by the use of the structure. In summary, there is a total dead load of 20psf and a total live load of 80psf, giving a total load of 100psf using the ASD method. All connections are assumed to be pinned connections and the beams are not laterally braced.

Using statics, the loads on each member have been determined as shown in Table 4.1.

Table 4.1 Factored Loads for Beam to Column Connection

Beam Distributed Load (15ft span):	$W = (100psf)x(10ft\ tributary\ width) = 1000\ {}^{lb}/_{ft}$
Column Axial Load:	$P = (100psf)x(20ft)x\left(\frac{15ft}{2}\right) = 15{,}000lb$
Maximum Beam Moment:	$M = \frac{1000^{lb}/_{ft}\ x\ (15ft)^2}{8} = 28.125kft$
End Shear Reaction in Beam:	$V = \frac{1000^{lb}/_{ft}\ x\ 15ft}{2} = 7500lb$

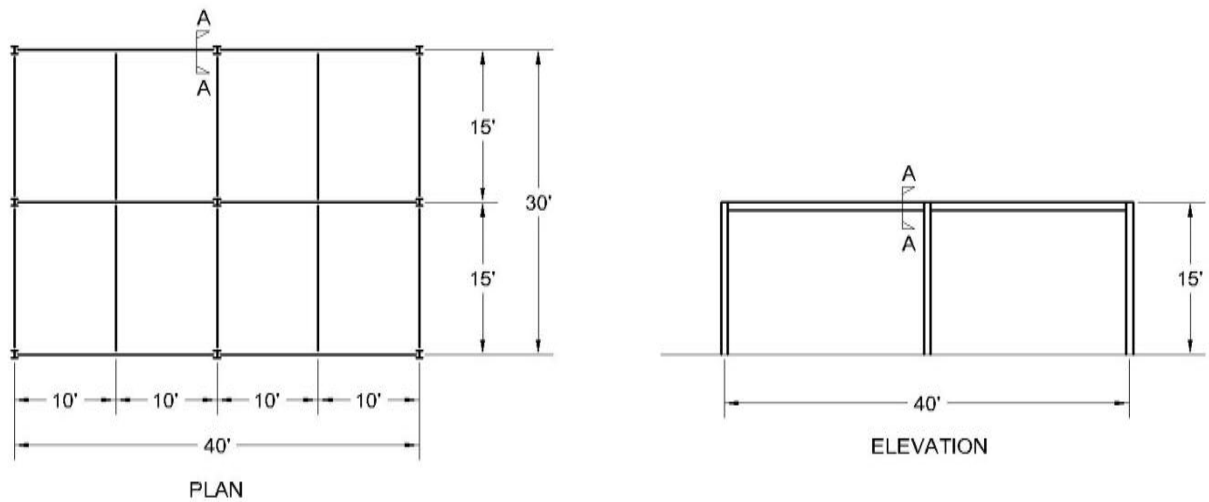


Figure 4.3 Framing Layout for Beam-to-Column Connection

FRP Connection Design

It is important to note the orientations of each member before any design calculations are performed. All of the longitudinal directions are located along the length of each member. Meaning, the beams longitudinal direction is oriented horizontally from left to right when looking at the detail in Figure 4.1, and the transverse direction for the web is up and down. The longitudinal direction for the column flange that the bolts will fasten into is also up and down, and the transverse direction will be in and out of the page. For the two angles connecting the beam and column together, the longitudinal direction is up and down, and the transverse direction is either into and out of the page, or horizontal from left to right depending on which leg of the angle is being considered.

All members will use the 1500/1525 series material properties unless these are found to be inadequate. If the 1500/1525 series are inadequate, the 1625 series may be analyzed. The beam and column for the FRP connection will be designed using the tables provided in Chapter 4 of The Pultex Pultrusion Design Manual published by Creative Pultrusions, Inc (Creative Pultrusions Inc., 2004). Using the distributed load of 1000 lb/ft and no lateral bracing, the required beam size due to bending was found to be a W12x12x½ FRP wide flange section. As found in the Pultex Pultrusion Design Manual, this beam is capable of supporting this distributed load with no lateral support. The column has a pin-pin connection, so the effective length is equal to the height of the column which is 15 ft. With an axial load of 15,000 lb, the required column size was found to be a W10x10x⅜ FRP wide flange section.

The design manual provided by Creative Pultrusions, Inc. also has a design section and tables devoted to simple beam to column connections. The connection will be sized using these tables as well as by hand calculations, as presented previously in this chapter. The two results will be compared by looking at the assumptions made by each method and by the overall outcome of the connection design.

Creative Pultrusion, Inc. Design Manual Method

The connection design tables are found on pages 4-91 – 4-95 of the Creative Pultrusion Design Manual (2004). A copy of these tables can be found in Appendix A of this report. With a max shear (V) of 7500 lbs as found previously in this chapter, a column flange size of 10 inches, and beam depth of 12 inches, a connection using a 3”x3”x½”x8¼” long angle with nine ½” diameter bolts is found to be sufficient.

Banks’ Method of Determining Stresses

The first step in the design method presented in Chapter 3 - is to determine the design loads and ASD safety factors for the connection. A safety factor of 4 will be applied for all parts of the connection as discussed previously. The design load affecting this connection is the end shear reaction in the beam which was found to be 7500lbs as shown in Table 4.1.

The next step is to dimension the connection and estimate the number of fasteners required. Since the column and angles are both loaded in the longitudinal direction and the beam is loaded in the transverse direction, the beams bearing capacity will most likely determine the number of fasteners required. The bearing stress will be used to estimate the number of fasteners since bearing provides a ductile failure which is what the connection is designed around. To estimate the number of fasteners required, set the critical bearing stress from Equation 3.1equal to the average bearing stress from Equation 4.1 and apply the safety factor.

$$\frac{\sigma_{cr}^{brg}}{SF} = \sigma_{br} \qquad \textbf{Equation 4.17}$$

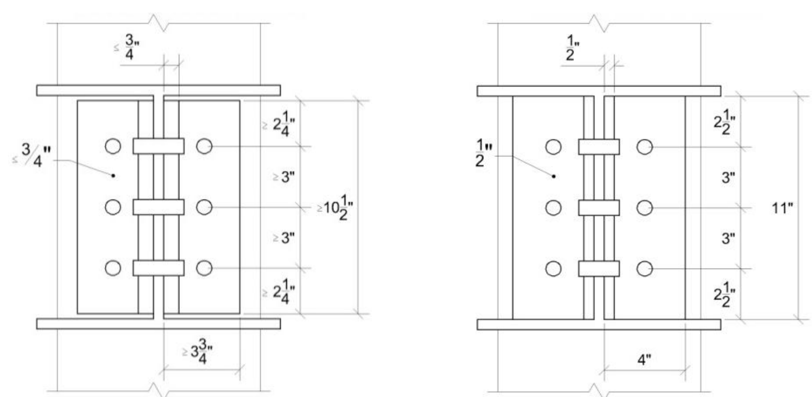
$$\frac{\sigma_{T,br}}{SF} = \frac{P_b}{d_b t_{pl}} = \frac{V}{n d_b t_{pl}} \qquad \textbf{Equation 4.18}$$

The transverse bearing strength of the material can be found in Table 3.2 under web section properties, and the load transferred to an individual bolt, P_b can be taken as the total shear load transferred to the connection divided by the number of fasteners required, n. The above equation can then be rearranged to determine the required number of fasteners. Assume an initial bolt diameter of ¾ inch and an angle thickness of ½ inch.

$$\boldsymbol{n = \frac{V\,SF}{d_b t_{pl} \sigma_{T,br}} = \frac{(7500lb)4}{\left(\frac{3}{4}in\right)\left(\frac{1}{2}in\right)(30{,}000psi)} = 2.67} \qquad \textbf{Equation 4.19}$$

Three bolts are required in the beam flange to prevent a bearing failure. Using the recommended geometries from Table 2.1 an initial layout of the connection is presented in Figure 4.4.

e	≥	3 d_b	=	2.25 in
w	≥	5 d_b	=	3.75 in
p	≥	4 d_b	=	3.00 in
t_{pl}	≤	1 d_b	=	0.75 in
d_w	≥	2 d_b	=	1.50 in



a) Recommended Geometries b) Recommended Detail c) Trial Connection

Figure 4.4 FRP Beam to Column Recommended Connection Geometries

To satisfy the recommended geometries for this example, a 4”x4”x½”x11 inch long double angle with nine bolts, (three through each angle leg) will be checked. Now that the connection is laid out, the stresses can be checked. The bearing stress must be checked again to determine if the connection is still adequate with the added load from the eccentricity of the fasteners. To do this, the resultant forces at the bolt holes need to be determined. The horizontal force, R_x, caused by the eccentricity can be determined from Equations 4.9 and 4.10, and the vertical force, R_v, is equal to the shear force on the bolt. This includes the load distribution factor found in Table 3.4. The moment caused by the eccentricity will be too large if the recommended end distance is used for the edge of the beam, so the minimum end distance of 1½” determined from Table 2.1 will be used. This distance is shown in Figure 4.5.

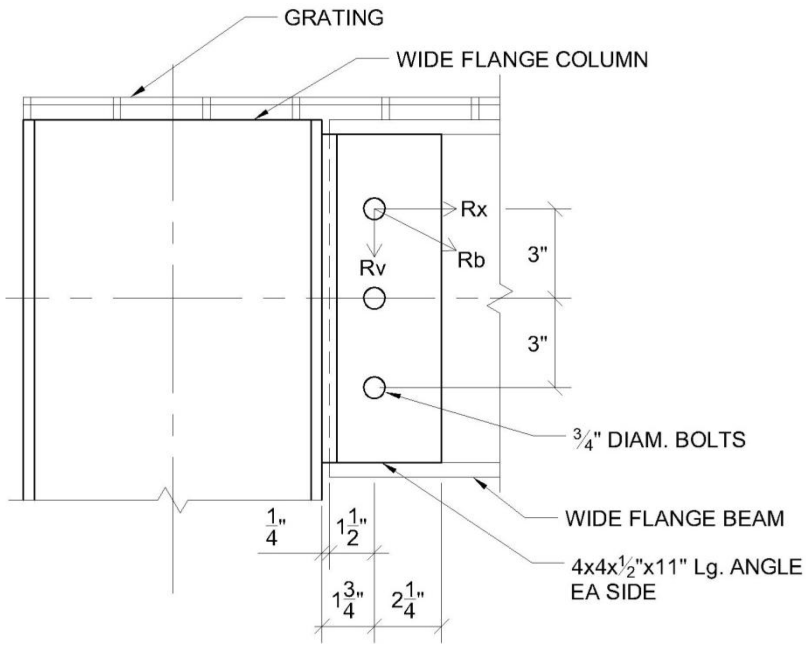


Figure 4.5 FRP Connection Eccentricity

$$\mathbf{M_{\mathit{v}} = Ve_v = (7500lb)\left(1\tfrac{3}{4}in\right) = 13,125lbin} \qquad \textbf{Equation 4.20}$$

$$\mathbf{R_x = \frac{M_{\mathit{v}}y}{\Sigma d^2} = \frac{(13{,}125lbin)(3in)}{2(3in)^2} = 2187.5lb} \qquad \textbf{Equation 4.21}$$

$$\mathbf{R_{\mathit{v}} = \frac{V}{n} = \frac{1.1(7500lb)}{3} = 2750lb} \qquad \textbf{Equation 4.22}$$

and the bearing force is equal to the resultant of these two loads

$$\boldsymbol{R_b = \sqrt{R_x^2 + R_v^2} = \sqrt{2187.5^2 + 2750^2} = 3514lb} \qquad \textbf{Equation 4.23}$$

acting at an angle of 51.5 degrees to the longitudinal direction of the beam. Using Equation 4.1, the bearing stress can now be checked for the beam web, column flange, and angle leg. Note that there are two angles so the total force is divided equally between them.

$$\sigma_{br_{beam}} = \frac{R_b}{d_b t_{bm}} = \frac{3514lb}{\left(\frac{3}{4}in\right)\left(\frac{1}{2}in\right)} = 9370psi$$ **Equation 4.24**

$$\sigma_{br_{\measuredangle}} = \frac{R_b/2}{d_b t_{\measuredangle}} = \frac{3514lb/2}{\left(\frac{3}{4}in\right)\left(\frac{1}{2}in\right)} = 4685psi$$ **Equation 4.25**

$$\sigma_{br_{col}} = \frac{V_b}{d_b t_{col}} = \frac{1.1(7500lb)/6bolts}{\left(\frac{3}{4}in\right)\left(\frac{3}{8}in\right)} = 4889psi$$ **Equation 4.26**

These design bearing stresses must be checked against the factored critical bearing stresses to determine if the connection in adequate. The critical stresses for the beam are taken from Table 3.2 and the safety factor is applied.

$$\sigma^{brg}_{allow_{L,beam}} = \frac{\sigma_{T,br}}{SF} = \frac{33{,}980psi}{4} = 8495psi$$ **Equation 4.27**

$$\sigma^{brg}_{allow_{T,beam}} = \frac{\sigma_{L,br}}{SF} = \frac{30{,}000psi}{4} = 7500psi$$ **Equation 4.28**

Since the bearing force in the beam web is not parallel or perpendicular to the transverse and longitudinal directions, the stress is divided into the resultant forces and compared. This is only an approximate check, but it will give an idea as to how close the material is to the critical bearing stress.

$$\sigma_{L,br_{beam}} = 9370\cos 51.5 = 5833psi$$ **Equation 4.29**

$$\sigma_{T,br_{beam}} = 9370\sin 51.5 = 7333psi$$ **Equation 4.30**

The components of the bearing stress are less than the allowable longitudinal and transverse bearing stresses, however the component for the transverse bearing stress is very close to the allowable bearing stress.

The material properties, and specifically, the orthotropic nature of FRP, slightly resemble that of wood. Because of this, Hankinson's formula, which is used in wood design to determine the stress when loaded at an angle to the grain, will also be used to check the bearing stress in the beam web. This correlation between wood and FRP was not found in the literature that was researched for FRP connection design, but has been included here to show that the actual bearing stress in this connection is most likely greater than the critical stress that the material can withstand.

$$\sigma_{allow_{beam}}^{brg} = \frac{\sigma_L \sigma_T}{\sigma_L sin\theta^2 + \sigma_T cos\theta^2} \qquad \textbf{Equation 4.31}$$

$$\sigma_{allow_{beam}}^{brg} = \frac{8495x7500}{8495\sin^2(51.5) + 7500\cos^2(51.5)} = 7857psi$$

The bearing stress in the beam is greater than the allowable bearing stress using Hankinson's formula. Due to these results, the beam web is not adequate for the applied load. Therefore, the beam web should be stiffened using adhesively bonded plates.

The angle must be treated the same way as the beam because the bearing stress is not in line with either the longitudinal or transverse direction. The critical stresses for the angle are taken from Table 3.3 and the safety factor is applied.

$$\sigma_{allow_{L,\measuredangle}}^{brg} = \frac{30{,}000psi}{4} = 7500psi \qquad \textbf{Equation 4.32}$$

$$\sigma_{allow_{T,\measuredangle}}^{brg} = \frac{18{,}000psi}{4} = 4500psi \qquad \textbf{Equation 4.33}$$

$$\sigma_{L,br_{\measuredangle}} = 4685\sin 51.5 = 3667psi \qquad \textbf{Equation 4.34}$$

$$\sigma_{T,br_{\measuredangle}} = 4685\cos 51.5 = 2917psi \qquad \textbf{Equation 4.35}$$

The design stresses are less than the critical stresses so the angle is adequate. The stress in the column flange is in the longitudinal direction of the column so the design stress is only compared to the critical longitudinal bearing stress. The critical stress for the column is taken from Table 3.2 and the safety factor is applied.

$$\sigma_{allow_{col}}^{brg} = \frac{33{,}000psi}{4} = 8250psi \geq 4889psi \qquad \textbf{Equation 4.36}$$

The column flange is adequate for bearing.

Next, the shear stress in the angle is checked. The angle could experience a shear failure either through shear-out at the bolt holes, or through longitudinal shear failure in the heel of the angle. Equations 4.5, 4.6, and 4.8 will be used to determine the design stresses and Table 3.3 is used to determine the allowable stress.

$$\tau_{shear-out_{edge}} = \frac{V_b}{2t_{\angle}e} = \frac{1.1(7500lb)/3}{2\left(\frac{1}{2}in\right)(2.5in)} = \mathbf{1100}psi \qquad \textbf{Equation 4.37}$$

$$\tau_{shear-out_{bolt}} = \frac{V_b}{2t_{\angle}p} = \frac{1.1(7500lb)/3}{2\left(\frac{1}{2}in\right)(3in)} = \mathbf{917}psi \qquad \textbf{Equation 4.38}$$

$$\tau_{pl} = \frac{V}{2A_{heel}} = \frac{7500lb}{2\left(\frac{1}{2}in\right)(11in)} = \mathbf{682}psi \qquad \textbf{Equation 4.39}$$

$$\tau_{allow} = \frac{\tau_{LT}}{SF} = \frac{4500psi}{4} = \mathbf{1125}psi \qquad \textbf{Equation 4.40}$$

The critical shear stress is greater than all of the design shear stresses so the angle is adequate for interlaminar shear.

Next the flexural and shear stresses in the angle caused by prying action are checked. Refer to Equations 4.11-4.14 and Table 3.3.

$$m_{leg} = \frac{R_x\left(b_{leg} - t_{\angle} - s\right)}{8} = \frac{(2187.5lb)\left(4in - \frac{1}{2}in - 2in\right)}{8} = \mathbf{410}lbin \qquad \textbf{Equation 4.41}$$

$$v_{leg} = \frac{R_x}{2} = \frac{2187.5lb}{2} = \mathbf{1094}lb \qquad \textbf{Equation 4.42}$$

$$\sigma_{flex} = \frac{m_{leg}}{\left(2et_{\angle}^2/6\right)} = \frac{410lbin}{\left(2(2.25in)\left(\frac{1}{2}in\right)^2/6\right)} = \mathbf{2188}psi \qquad \textbf{Equation 4.43}$$

$$\tau_{leg} = \frac{V_{leg}}{t_{\angle}2e} = \frac{1094lb}{\left(\frac{1}{2}in\right)2(2.25in)} = \mathbf{486}psi \qquad \textbf{Equation 4.44}$$

$$\sigma_{allow}^{flex} = \frac{\sigma_{T,flex}}{SF} = \frac{11{,}000psi}{4} = \mathbf{2750}psi \qquad \textbf{Equation 4.45}$$

$$\tau_{allow} = \frac{\tau_{LT}}{SF} = \frac{4500psi}{4} = \mathbf{1125}psi \qquad \textbf{Equation 4.46}$$

The angle is adequate for flexural and shear stresses due to prying forces.

The local stress at the web-flange junction in the column must also be checked. Equation 3.3 and 4.15 is used for this calculation.

$$\sigma_{trans} = \frac{R_x}{t_{web_{col}}\left(l_{\angle}/2\right)} = \frac{2187.5lb}{\left(\frac{3}{8}in\right)\left(11in/2\right)} = \mathbf{1061}psi \qquad \textbf{Equation 4.47}$$

$$\sigma_{allow}^{tens} = .9\sigma_{T,t} = \frac{.9(10{,}500psi)}{4} = 2363psi \geq 1061psi \qquad \textbf{Equation 4.48}$$

$\therefore$ local tensile strength is adequate

The shear stress on the bolts is checked using Equation 4.7 and Table 3.1.

Single shear in column flange: $$\tau_b = \frac{V}{A_b} = \frac{1.1(7500lb)/6bolts}{\left(\frac{\pi}{4}\right)\left(\frac{3}{4}in\right)^2} = 3112psi \qquad \textbf{Equation 4.49}$$

Double shear in beam web: $$\tau_b = \frac{R_b}{A_b} = \frac{3514lb/3}{\left(\frac{\pi}{4}\right)\left(\frac{3}{4}in\right)^2} = 2651psi \qquad \textbf{Equation 4.50}$$

$$\tau_{allow}^{b} = \frac{\tau_{ult}^{b}}{SF} = \frac{5650lb}{4\left[\left(\frac{\pi}{4}\right)\left(\frac{3}{4}in\right)^2\right]} = 3197psi \qquad \textbf{Equation 4.51}$$

$\therefore$ bolt shear strength is adequate

The final check is for the tensile stress on the bolts in the column flange which experience prying forces. It is assumed that the top bolt in each angle to column connection carries all of the tensile force between them, and no other bolts help in carrying the load. Equation 4.2 and Table 3.1 are used for this check.

$$\sigma_b = \frac{R_x}{2A_b} = \frac{2187.5lb}{2\left(\frac{\pi}{4}\right)\left(\frac{3}{4}in\right)^2} = 2476psi \qquad \textbf{Equation 4.52}$$

$$\sigma_{allow}^{b} = \frac{T_{max}}{SFA_b} = \frac{4520lb}{4\left(\frac{\pi}{4}\right)\left(\frac{3}{4}in\right)^2} = 2558psi \qquad \textbf{Equation 4.53}$$

$\therefore$ bolt tensile strength is adequate

This connection is adequate for the required loading. A W12x12x½ FRP beam with an adhesively bonded plate stiffener for the web is framing into a W10x10x⅜ FRP column with a 4”x4”x½”x11” long double angle and (9) ¾” diameter bolts.

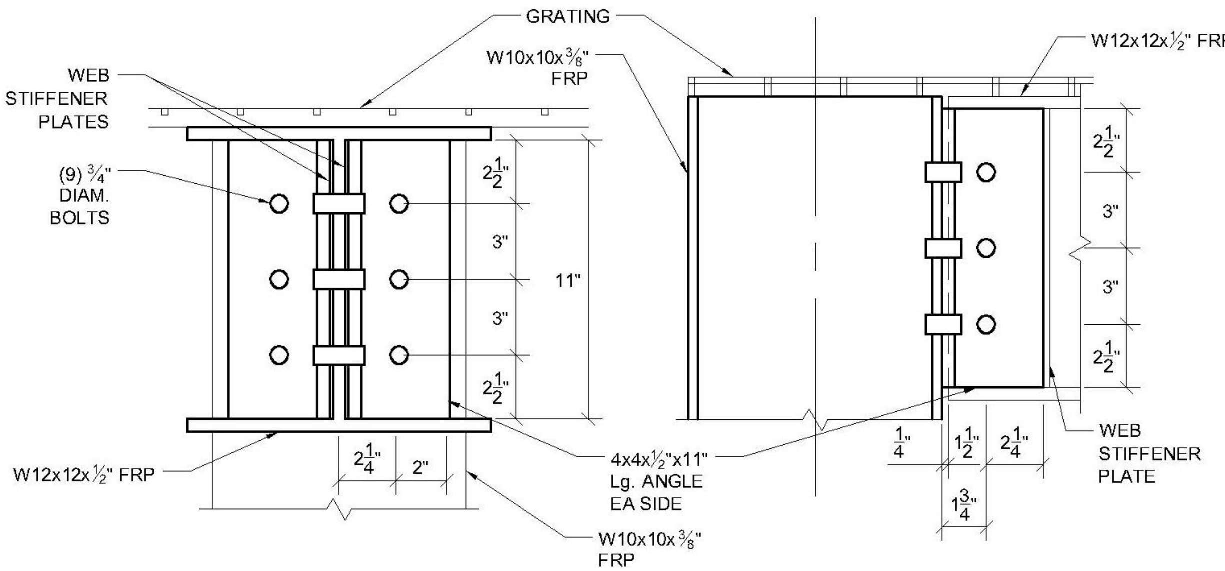


Figure 4.6 FRP Beam to Column Connection

Comparison of Creative Pultrusion Design Manual to Bank's Method

There is a difference in the design that used the Creative Pultrusion, Inc. Design Manual method and the method of determining stresses. This is the result of using different safety factors. The Pultex Pultrusion Design Manual uses a safety factor of 2.5 instead of 4 because Creative Pultrusion, Inc. has developed their design tables and safety factors based on their own testing and analysis. This difference in safety factors is the cause for the difference in the two design methods.

Steel Connection Design

The steel connection is designed using the ASD method presented in the 14th edition of the AISC Steel Construction Design Manual (American Institute of Steel Construction, 2011). The beam and column are to be A992 structural steel, the angle is A36 structural steel, and the bolts are to be grade A325-N. The beam and column are first sized using tables 3-10 and 4-1 of the Steel Construction Design Manual respectively. For a moment of 28.125 kft and an unbraced length of 15 ft, the required beam size was found to be a W8x21. With an axial load of 15,000 lbs and column length of 15 ft with a pinned-pinned connection, the required column size is a W8x31.

For this connection, only the shear strength of the bolts and the bearing strength need to be checked. The steel connection has the same eccentricity as the FRP connection, but for the steel connection, the added stresses that this will cause are negligible and are therefore not checked in this example. A trial connection using ½" diameter bolts and ¼" thick plate is checked for adequacy. The number of bolts required is determined from the shear strength which is found using Equation J3-1 of the AISC Construction Design Manual (American Institute of Steel Construction, 2011). The nominal stress from this equation is found in Table J3.2.

$$\boldsymbol{R_n = F_n A_b n \quad \Omega = 2} \qquad \textbf{Equation 4.54}$$

$$\boldsymbol{F_n = F_{nv} = 54ksi} \qquad \textbf{Equation 4.55}$$

$$\boldsymbol{n = \frac{R_n}{F_n A_b} = \frac{R_a \Omega}{F_n A_b} = \frac{(7.5k)2}{(54ksi)\left(\frac{\pi}{4}\right)\left(\frac{1}{2}in\right)^2} = 1.41} \qquad \textbf{Equation 4.56}$$

Two bolts are required for this connection, and they are spaced at typical dimensions for steel connections as shown in Figure 4.7. The bearing strength is checked next using Equation J3.6a of the AISC Steel Construction Design Manual. There are three possible areas where a bearing failure could occur: the beam web, column flange, or the leg of the angle. By inspection it can be determined that bearing in the angle leg will be the governing case. The beam web and the angle leg have the same thickness of ¼", but the beam web is made of A992 grade steel whereas the angle is made of A36 grade steel. The angle is made up of a weaker material, so even though the angle and beam web have the same thickness, the angle will have a lower capacity than the beam. The column flange is also made of A992 grade steel and is thicker than ¼" so it will not govern the design. Therefore, the angle will be checked for bearing failure, because if it is adequate, the column and beam will also be adequate.

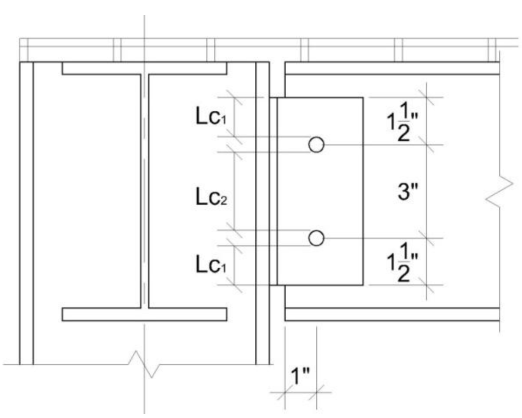


Figure 4.7 Steel Connection Layout

$$R_n = 1.2L_c t F_u \leq 2.4 d t F_u \quad \Omega = 2$$ **Equation 4.57**

$$L_{c_1} = 1.5in - \left(\frac{\frac{1}{2}in + \frac{1}{16}in}{2}\right) = 1.21875in$$ **Equation 4.58**

$$L_{c_2} = 3in - \left(\tfrac{1}{2}in + \tfrac{1}{16}in\right) = 2.4375in$$ **Equation 4.59**

$$L_c = min\left(L_{c_1}, L_{c_2}\right) = 1.21875in$$ **Equation 4.60**

$$R_n = 1.2(1.21875in)\left(\tfrac{1}{4}in\right)(58ksi) \leq 2.4\left(\tfrac{1}{2}in\right)\left(\tfrac{1}{4}in\right)(58ksi)$$ **Equation 4.61**

$$R_n = 21.2k \leq 17.4k$$ **Equation 4.62**

$$R_n = 17.4k$$ **Equation 4.63**

$$R_a \leq {R_n}/_{\Omega}$$ **Equation 4.64**

$$7.5k \leq \frac{17.4k}{2} = 8.7k$$ Equation **4.65**

∴ bearing strength is adequate

The steel connection will use a W8x21 beam that frames into a W8x31 column with a 3”x3”x¼”x6” long angle and six ½” diameter bolts.

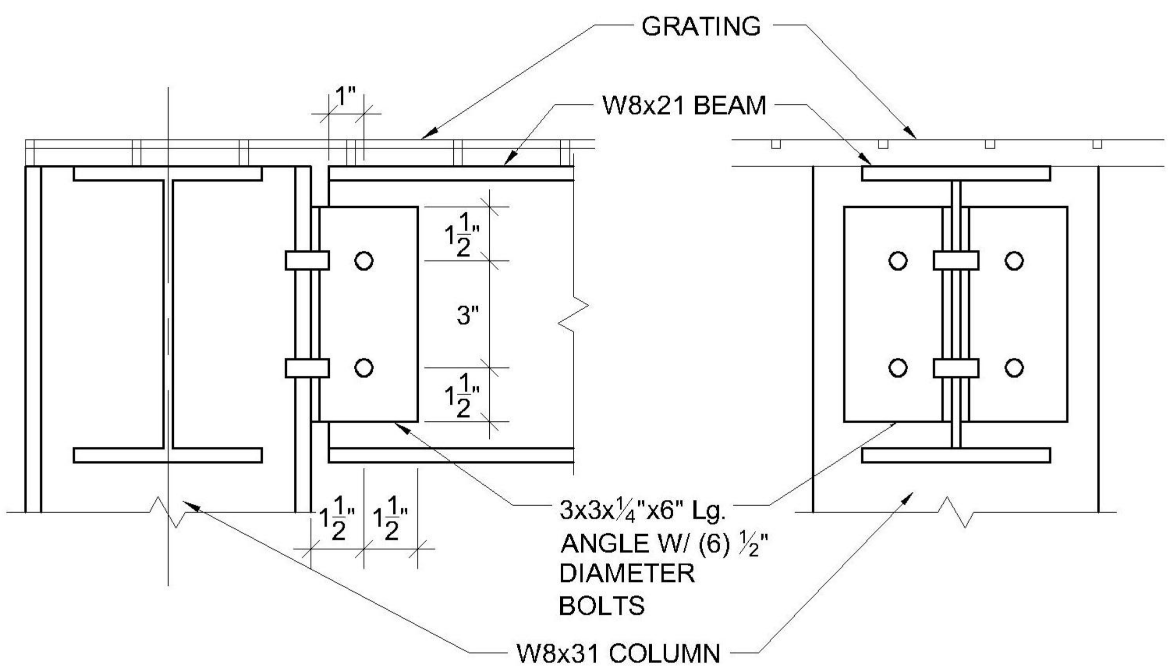


Figure 4.8 Steel Beam to Column Connection

Wood Connection Design

The wood connection is designed using the ASD method presented in the 2005 edition of the National Design Specification for Wood Construction, or NDS (American Forest & Paper Association, 2005). The wood is assumed to be Douglas Fir-Larch sawn lumber, Grade No. 1, and the beam and column are connected using a saddle connection. The beam will bear on the saddle and the saddle will be bolted to the column. The beam size is determined by the flexural stress as follows. Equation 3.3-2 and Table 4.3.1 of the NDS are used to ensure that the actual bending stress is less than the allowable.

$$f_b \leq F'_b \qquad \textbf{Equation 4.66}$$

$$f_b = \frac{M}{S} \qquad \textbf{Equation 4.67}$$

$$F'_b = F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r \qquad \textbf{Equation 4.68}$$

The load duration factor, C_D, is found in Table 2.3.2. of the NDS, and the wet service factor, C_M, is equal to one when the moisture content of the wood is less than 19 percent. The structure will operate under normal temperatures, and the beam is oriented so that the depth is greater than the width. The beam does not qualify for repetitive member use, and contains no incisions.

The bending stress of the timber, F_b, and the size factor, C_F, are found in Table 4D of the NDS, and a value for the beam stability factor, C_L, will be assumed. Since the beam will not be laterally supported, the beam stability factor will be less than one. A value of .95 will be assumed for this beam and then checked after a beam is selected.

$$C_D = C_M = C_t = C_{fu} = C_i = C_r = 1.0 \qquad \textbf{Equation 4.69}$$

$$C_F = 0.74 \qquad \textbf{Equation 4.70}$$

$$C_L = 0.95 \qquad \textbf{Equation 4.71}$$

$$F_b = 1350psi \qquad \textbf{Equation 4.72}$$

$$F'_b = (1350psi)x0.74x0.95 = 949psi \qquad \textbf{Equation 4.73}$$

The minimum section modulus, *S*, is determined and a beam with a greater section modulus is selected.

$$S \geq \frac{M}{F'_b} = \frac{(28125lbft)\left(12^{in}/_{ft}\right)}{949psi} = 356in^3 \qquad \text{Equation 4.74}$$

A 6x22 beam with a section modulus of 423.7in^3 is selected. Large timber, such as a 6x22, may be difficult to find, and a designer may choose to select a glulam rather than timber, but for the purposes of comparison in this report, and so that all of the wood members in this connection are of the same wood type, the 6x22 timber will be chosen. If a glulam member were to be selected, a 15x5½" glulam would be adequate.

The beam stability factor, C_L, is checked using Equation 3.3-6 of the NDS as follows. The minimum modulus of elasticity is found in Table 4D and Table 4.3.1 of the NDS and the effective length for the beam is found in Table 3.3.3 of the NDS.

$$C_L = \frac{1+(F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1+(F_{bE}/F_b^*)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}} \qquad \text{Equation 4.75}$$

where

$$F_b^* = F_b C_D C_M C_t C_F C_i C_r = (1350psi)x0.74 = 999psi \qquad \text{Equation 4.76}$$

$$F_{bE} = \frac{1.20E'_{min}}{R_B^2} \qquad \text{Equation 4.77}$$

$$E'_{min} = E_{min} C_M C_t C_i C_T \qquad \text{Equation 4.78}$$

$$E_{min} = 580,000psi \qquad \text{Equation 4.79}$$

$$C_T = 1.0 \qquad \text{Equation 4.80}$$

$$E'_{min} = (580,000psi)x1.0 = 580,000psi \qquad \text{Equation 4.81}$$

$$R_B = \sqrt{\frac{l_e d}{b^2}} \qquad \text{Equation 4.82}$$

$$l_e = 1.63l_u + 3d = 1.63\left(15ftx12\frac{in}{ft}\right) + 3(5.5in) = 352.5in \qquad \text{Equation 4.83}$$

$$R_B = \sqrt{\frac{(352.5in)(21.5in)}{(5.5in)^2}} = 15.828 \qquad \text{Equation 4.84}$$

$$F_{bE} = \frac{1.20(580,000psi)}{(15.828)^2} = 2778psi \qquad \text{Equation 4.85}$$

$$F_{bE}/F_b^* = \frac{2778psi}{999psi} = 2.78 \quad \textbf{Equation 4.86}$$

$$C_L = \frac{1+2.78}{1.9} - \sqrt{\left[\frac{1+2.78}{1.9}\right]^2 - \frac{2.78}{0.95}} = 0.97 \quad \textbf{Equation 4.87}$$

The assumed value of 0.95 for the beam stability factor is less than the actual value of 0.97, so the beam is adequate.

The column is determined from the compression stress as follows. Section 3.6.3 and Table 4.3.1 of the NDS are used to ensure that the actual compression stress is less than the allowable.

$$f_c \leq F'_c \quad \textbf{Equation 4.88}$$

$$f_c = \frac{P_n}{A_n} \quad \textbf{Equation 4.89}$$

$$F'_c = F_c C_D C_M C_t C_F C_i C_p \quad \textbf{Equation 4.90}$$

where,

$$C_D = C_M = C_t = C_F = C_i = 1.0 \quad \textbf{Equation 4.91}$$

The compression stress parallel to the grain, F_c, is found in Table 4D of the NDS, and a value for the column stability factor, C_p, will be assumed to be equal to one. This assumption will be checked once a column size is selected.

$$C_p = 1.0 \quad \textbf{Equation 4.92}$$

$$F_c = 1000psi \quad \textbf{Equation 4.93}$$

$$F'_c = 1000psi \quad \textbf{Equation 4.94}$$

The minimum area is determined and a column selected, and then the assumption that the column stability factor, C_p, equals one is checked.

$$A_n \geq \frac{P_n}{F'_c} = \frac{15{,}000lb}{1000psi} = 15in^2 \quad \textbf{Equation 4.95}$$

In order for the column width to be either equal to or greater than the beam width, a 6x6 column is selected with an area of 30.25in^2. The column stability factor is checked next using Equation 3.7-1 of the NDS.

$$C_p = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_C^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \qquad \textbf{Equation 4.96}$$

where,

$$F_{cE} = \frac{0.822E'_{min}}{(l_e/d)^2} \qquad \textbf{Equation 4.97}$$

The factored minimum modulus of elasticity, E'_{min}, is found using Tables 4.3.1 and 4D of the NDS. All of the factors can be taken as 1.0.

$$E'_{min} = E_{min}C_MC_tC_iC_T = (580,000psi)1.0 = 580,000psi \qquad \textbf{Equation 4.98}$$

The effective length factor for a pin-pin connection is equal to 1.0, and the effective length to depth ratio is determined. This value cannot exceed 50 as stated in section 3.7.1.4 of the NDS. Once this ratio is determined, all of the terms required to find the column stability factor can be found and the factor is calculated.

$$\frac{l_e}{d} = \frac{1.0(15ft)\left(12^{in}/_{ft}\right)}{5.5in} = 32.7 \qquad \textbf{Equation 4.99}$$

$$F_{cE} = \frac{0.822(580{,}000psi)}{(32.7)^2} = 445psi \qquad \textbf{Equation 4.100}$$

$$F_c^* = 1000psi \qquad \textbf{Equation 4.101}$$

$$F_{cE}/F_c^* = \frac{445psi}{1000psi} = .445 \qquad \textbf{Equation 4.102}$$

For sawn lumber:

$$c = 0.8 \qquad \textbf{Equation 4.103}$$

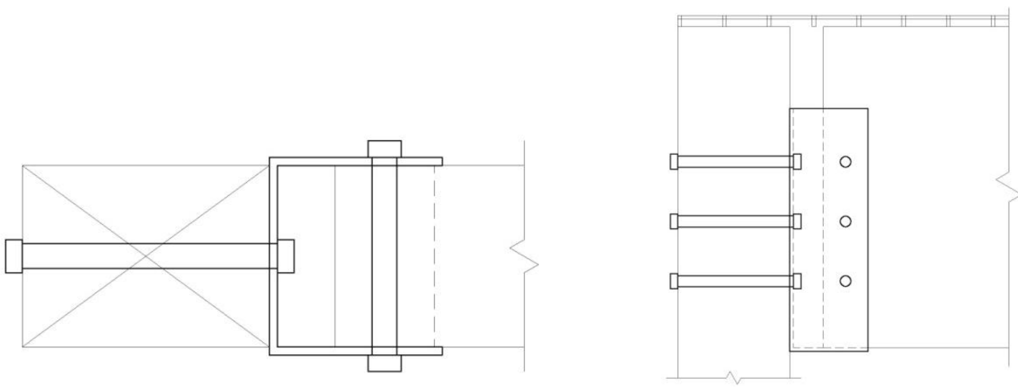
$$C_p = \frac{1+.445}{2x0.8} - \sqrt{\left[\frac{1+.445}{2x0.8}\right]^2 - \frac{0.445}{0.8}} = .39 \qquad \textbf{Equation 4.104}$$

$$F'_c = (1000psi)0.39 = 394psi \qquad \textbf{Equation 4.105}$$

$$A_n \geq \frac{15{,}000lb}{394psi} = 38.1in^2 \qquad \textbf{Equation 4.106}$$

A 6x6 column does not have the required area. Increasing the column size to a 6x8 provides an adequate design. A 6x8 column has an area equal to 41.24in^2 and the calculations for the stability factor remain the same since the 6in dimension does not change. Therefore this column is adequate for the required load.

The beam to column connection will use a saddle as shown in Figure 4.9. The beam will bear on the saddle with thru-bolts for stability and constructability reasons and the column will attach to the saddle with thru-bolts as shown. Note that the exterior beams framing into the sides of the column are not shown for clarity.



a) Plan Detail b) Section Detail

Figure 4.9 Wood Connection Details

The bearing connection for the beam is determined from the compression stress that is parallel to grain as follows. Section 3.10 and Table 4.3.1 of the NDS are used to ensure that the actual compression stress perpendicular to the grain is less than the allowable.

$$f_{c\perp} \leq F'_{c\perp} \quad \textbf{Equation 4.107}$$

$$f_{c\perp} = \frac{P_n}{A_b} \quad \textbf{Equation 4.108}$$

$$F'_{c\perp} = F_{c\perp} C_M C_t C_i C_b \quad \textbf{Equation 4.109}$$

where

$$F_{c\perp} = 625psi \quad \textbf{Equation 4.110}$$

$$C_M = C_t = C_i = 1.0 \quad \textbf{Equation 4.111}$$

The bearing area factor, C_b, can be conservatively assumed to be equal to one since it will only increase the allowable bearing stress. This can be checked after a bearing size is determined to decrease the bearing area if needed.

$$\boldsymbol{C_b = 1.0} \qquad \textbf{Equation 4.112}$$

$$\boldsymbol{F'_{c\perp} = (625psi)1.0 = 625psi} \qquad \textbf{Equation 4.113}$$

The bearing area, A_b, is equal to the width of the beam, b, multiplied by the bearing length, l_b. Using this information and Equations 4.107 and 4.108, the bearing length can be determined.

$$\boldsymbol{l_b \geq \frac{P_n}{F'_{c\perp}b} = \frac{(7500lbs)}{(625psi)(5.5in)} = 2.18in} \qquad \textbf{Equation 4.114}$$

A bearing length of 3in will be used. The connection of the column to the saddle is designed next. The side of the saddle connecting to the column will be designed as a ¼” A36 grade steel plate. This is a single shear connection in which there are six yield mechanisms that could occur. These mechanisms are: crushing in the wood member, crushing in the steel saddle, rotation of the fastener, a plastic hinge and crushing in the wood member, plastic hinge and crushing in the steel saddle, or two plastic hinges, one in the saddle and one in the wood, at the location of the shear plane. Each of these limits could be calculated using Table 11.3.1A of the NDS, or the governing limit case can be found in Table 11B of the NDS. Table 11B lists the limits for several different types of wood and bolt diameters for single shear connections using a ¼ inch, A36 steel plate. For the beam, the load is applied parallel to the grain, and the appropriate fastener strength is found. A ¾ inch diameter bolt is assumed.

$$\boldsymbol{Z_{\parallel} = 1670lbs} \qquad \textbf{Equation 4.115}$$

The factored strength for dowel type fasteners is determined using Table 10.3.1 as shown below.

$$Z' = ZC_DC_MC_tC_gC_\Delta C_{eg}C_{di}C_{tn} \qquad \textbf{Equation 4.116}$$

The load duration factor, C_D, and temperature factor, C_t, are the same as for the beam and column and the bolts are not bolted into the end grain of any member. The geometry factor, C_Δ, is taken as one and the final geometry of the connection will be laid out so that it meets this requirement. The fasteners are not used in diaphragm construction or in toe nail connections.

The group action factor, C_g, is required for connections that have at least one row of fasteners and a fastener diameter that is less than 1 inch. For the preliminary calculation this value will be assumed, and then checked once the connection is determined.

$$C_D = C_t = C_\Delta = C_{eg} = C_{di} = C_{tn} = 1.0 \qquad \textbf{Equation 4.117}$$

$$C_g = 0.96 \qquad \textbf{Equation 4.118}$$

$$Z' = (1670lbs)0.96 = 1603.2lbs \qquad \textbf{Equation 4.119}$$

The required number of fasteners can now be determined. The connection must be able to support the total shear load through the connection, so the required number of fasteners is determined as follows:

$$V \leq nZ' \qquad \textbf{Equation 4.120}$$

$$n \geq \frac{V}{Z'} = \frac{7500lbs}{1603.2.2lbs} = 4.66 \qquad \textbf{Equation 4.121}$$

Five bolts are required for the column to saddle connection. The group action factor is now checked using Equation 10.3-1 in the NDS

$$C_g = \left[\frac{m(1-m^{2n})}{n[(1+R_{EA}m^n)(1+m)-1+m^{2n}]}\right]\left[\frac{1+R_{EA}}{1-m}\right] \qquad \textbf{Equation 4.122}$$

$$R_{EA} = min\left(\frac{E_sA_s}{E_mA_m}, \frac{E_mA_m}{E_sA_s}\right) \qquad \textbf{Equation 4.123}$$

$$m = u - \sqrt{u^2 - 1} \qquad \textbf{Equation 4.124}$$

$$u = 1 + \gamma\frac{s}{2}\left[\frac{1}{E_mA_m} + \frac{1}{E_sA_s}\right] \qquad \textbf{Equation 4.125}$$

$$\boldsymbol{\gamma = 270,000(D^{1.5}) = 270,000\left(\frac{3}{4}in\right)^{1.5} = 175,370lbs/in}$$ **Equation 4.126**

The center to center spacing between the fasteners in the row is 3 inches and the modulus of elasticity and areas of the wood member and steel plate are as follows. The modulus of elasticity of the wood member is found in Table 4D of the NDS. Once these values are determined, the group action factor is found.

E_m = moldulus of elasticity of wood member = *1,600,000psi*

E_s = moldulus of elasticity of steel plate = *29,000,000psi*

A_m = gross cross-sectional area of wood member = *5.5in x 21.5in = 118.25in*^{2}

A_s = gross cross-sectional area of steel plate = *0.25in x 16in = 4.in*^{2}

Therefore,

$$\boldsymbol{C_g = 0.956}$$ **Equation 4.127**

The assumed group action factor of 0.96 is very close to the actual factor so the connection is adequate.

The wood connection consists of a 6x22 beam framing into a 6x6 column with a 16" long, A36 steel saddle, and (5) ¾" diameter thru-bolts in the column. (2) ¾" diameter thru-bolts are also included in the beam for stability and constructability reasons.

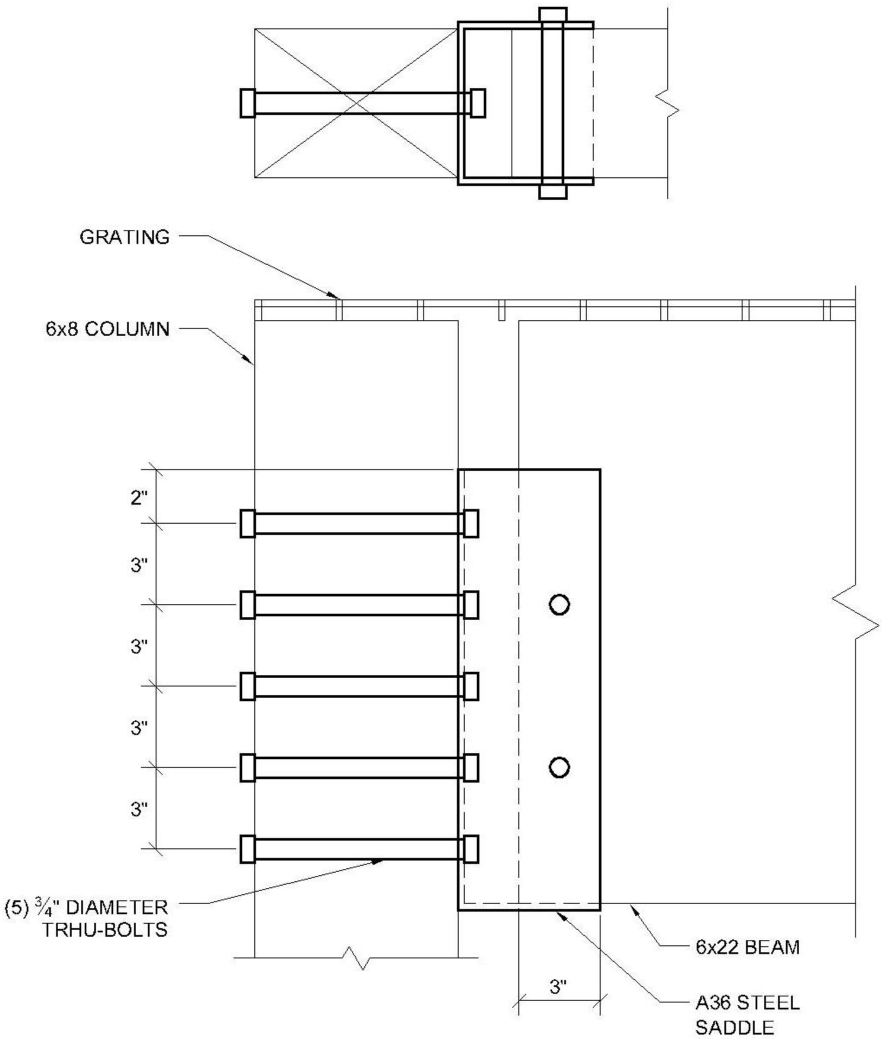


Figure 4.10 Wood Beam to Column Connection

Exterior beams framing into sides of column not shown for clarity.

Chapter 5 - Results and Conclusion

A simple beam to column connection has been designed using three different material types: FRP pultruded structural shapes, steel shapes, and wood. The results for each connection type are summarized in the tables below. Due to the deflection in the beam, there would be some rotation at the connection location. This is not considered in the comparison because it is assumed to be relatively small compared to the actual deflection and that it would be about the same for each beam type.

Table 5.1 Beam Comparison

	Beam				
Material	Size	Weight (lb/ft)	Moment Capacity (lbft)	Shear Capacity (lbs)	Deflection (in)
FRP	12x12x½	13.25	40,556	28,005	0.75
Steel	W8x21	21.00	31,000	41,400	0.52
Wood	6x22	24.64	33,509	13,402	0.18
					L/240 = 0.75

Table 5.2 Column Comparison

	Column			
Material	Size	Weight (lb/ft)	Axial Load Capacity (lbs)	Slenderness Ratio (KL/r)
FRP	10x10x⅜	8.30	18,941	75.9
Steel	W8x31	31	153,000	89.1
Wood	6x8	8.59	16,088	83.1

Table 5.3 Connection Comparison

	Connection						
Material	Connector	Bolts (Quantity & Diameter)		Beam to Connector		Column to Connector	
		Beam	Column	Capacity	Failure Mode	Capacity	Failure Mode
FRP	4x4x½	(3) ¾"	(6) ¾"	7670 lbs	Shear out in angle	7704 lbs	Bolt Shear
Steel	3x3x¼	(2) ½"	(4) ½"	8700 lbs	Bearing	8700 lbs	Bearing
Wood	A36 Grade Saddle	(2) ¾"	(5) ¾"	9844 lbs	Bearing	8114 lbs	Plastic Hinge and Crushing in Saddle

In comparing the beams used for each connection, the FRP beam has the lightest cross section and the largest allowable moment as shown in Table 5.1. The moment capacity of the FRP beam is 31% greater than that of the steel beam and 21% greater than the wood beam. This moment is well over the required moment capacity of 28,125 pound-feet; however, the FRP beam also has the largest deflection of the three which just meets the L/240 requirements. The wood beam has the heaviest cross section, but the deflection is well below the other two beam types. The steel beam has the smallest moment capacity of the three used, and weighs less than the wood beam but more than the FRP beam. All three beams meet the design requirements, but vary greatly in their capacities compared to their actual loading conditions.

As can be seen in Table 5.2, the wood and FRP columns have very similar weights, and the FRP beam has a much smaller cross sectional area. However, the wood section is more compact and has smaller outer dimensions, so it will not take up as much space as the FRP column will. The steel column is the largest and heaviest of the three and the capacity is much greater than required. The required load of 15,000 pounds is a rather small load for steel construction, so it would be expected that the column has a much larger capacity than is required.

Table 5.3 summarizes the connections and the connection capacities. Of the three connections, the FRP connection has a capacity that is the closest to the required capacity, and the wood connections capacity is farthest from the required capacity.

The FRP and the wood connections have two different governing failure modes for the beam to angle connection and the column to angle connection, unlike the steel connection which has the same governing failure mode for both. The beam to angle connection capacity for the FRP connection is limited by a shear-out failure in the angle leg. The capacity would have been limited by the bearing capacity of the beam web, but once the web was strengthened using an adhesively bonded plate, it was no longer the governing failure mode. The column to angle connection capacity is determined by the shear capacity of the bolts. The bolts in the column to angle connection only had a single shear plane to transfer the load into the column. This limits the capacity of bolts for transferring load. Had the beam to angle connection only used a single shear plane, the bolt shear would also have been the limiting failure mode. Since the beam to angle connection used a double shear plane, the bolts could transfer more shear, and therefore a different failure mode is the limiting case. The shear out failure is the governing failure mode in the beam to angle connection and not the column to angle connection because there is more area to transfer the shear in the column connection. The column has twice as much area at the edge of the angle to distribute the shear because there are two bolts carrying the load instead of just one.

The steel connection capacity is limited by bearing failure in the angle. This is true for both the beam to angle and column to angle connection. The angles are made of A36 steel as opposed to A992, which means that the angle is made of a weaker material than the beam and column. Therefore, even though the angle and the beam web are of the same thickness, the angle still governed the capacity of the connection.

The governing failure mode in the wood beam to saddle connection is crushing of the wood in the beam member due to bearing. This is different from the other connections because this connection was designed as a bearing connection rather than a bolted connection. Due to framing restraints, this type of connection was more feasible and constructible than a bolted connection. The failure mode for the column to saddle connection is the formation of a plastic hinge in the bolt and crushing due to bearing in the steel saddle. This type of failure occurs when the dowel bends and a plastic hinge forms near the shear plane and the steel plate crushes due to bearing at the bolt locations (Breyer, Fridley, Cobeen, & Pollock, 2007).

All three connections have different governing failure modes, due to the difference in their material properties. However, they are all adequate connections that will hold the load that they were designed for.

Three connections have been presented and proven to be adequate for the required loading conditions. This report has shown that FRP is a viable option for building structures, since it can be designed to meet the same criteria as the more common materials of steel and wood. An instance where FRP would be a good choice over steel or wood would be in hazardous environments, because of its resistance to corrosion. Steel and wood can break down or corrode in harsh environments, such as when they are exposed to chemicals, salts, and moisture, but FRP will not (Davalos, 2006).

Further development must be made into the design of FRP pultruded structural shape connections however; such as the publication of a design standard which ASCE is currently developing. Once this standard is published, many engineers may feel more comfortable in designing with this material. Standard shapes and material properties should also be developed so that engineers can become more familiar with the design and that their design will not depend on which manufacturer is chosen. Further research should also be conducted to better determine how FRP pultruded shapes act under loading so that an LRFD design procedure can be developed with confidence. FRP has many areas for improvement, but it seems to be a very promising material that must be further developed, and engineers must increase their understanding in, in order to reach its full potential.

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Appendix A – Clip Connection Load Tables From The Pultex Pultrusion Design Manual

These are provided as a comparison to Bank's method for determining stresses and designing FRP pultruded shape connections as presented in this report.

Clip Connection Load Tables with Pultex® SuperStructural Angles

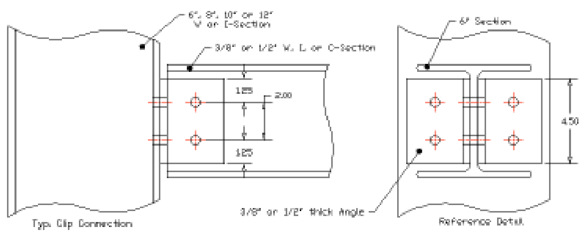
The following Clip Connection Load Tables were developed based on full section testing of the Pultex® **SuperStructural** angles. The experimental procedure can be located in Appendix A. The load tables were developed based on the following:

1. Experimental test results
2. Room temperature (73°)
3. A safety factor has **not** been applied. All loads are ultimate unless noted.
4. An ultimate bearing strength of 33,000 psi
5. A 4% hole deformation bearing stress of 12,000 psi
6. No damage to the composite materials
7. No chemical exposure
8. Bolted connection only; no adhesive.
9. Full section shear strength through angle heel = 3,400 psi (1500/1525 Series); 3,900 psi (1625 Series)

Note: Clip angle connection dimensions are governed by internal dimensions of beams and external dimensions of the flanges on the columns.

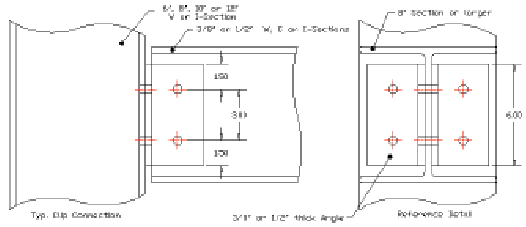
1500/1525 Series Pultex® SuperStructural Angles

Clip Angle Thickness (in)	Angle Connection Length (in)	Bolt Hole Dia. (in)	Bolt Dia. (in)	# of Angles in Connection	# of Bolts in Shear	# of Bolts through Beam Web	# of Bolts Connected to Column	Ultimate Shear Load through Angle Heels (lbs.)	Ultimate Bearing Load of Connection (lbs.)	Allowable Bearing Load for 4% Hole Deformation (lbs.)	Clip Angle Ultimate Load before Failure (lbs.)
3/8	4 1/2	9/16	1/2	2	4	2	4	11,475	24,750	9,000	11,475
3/8	4 1/2	11/16	5/8	2	4	2	4	11,475	30,938	11,250	11,475
1/2	4 1/2	9/16	1/2	2	4	2	4	15,300	33,000	12,000	15,300
1/2	4 1/2	11/16	5/8	2	4	2	4	15,300	41,250	15,000	15,300

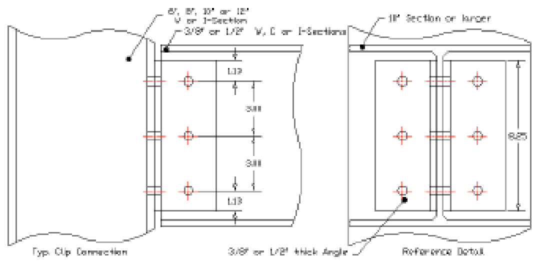


1500/1525 Series Pultex® SuperStructural Angles

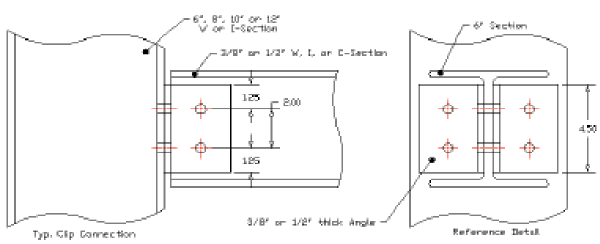
Clip Angle Thickness (in)	Angle Connection Length (in)	Bolt Hole Dia. (in)	Bolt Dia. (in)	# of Angles in Connection	# of Bolts in Shear	# of Bolts through Beam Web	# of Bolts Connected to Column	Ultimate Shear Load through Angle Heels (lbs.)	Ultimate Bearing Load of Connection (lbs.)	Allowable Bearing Load for 4% Hole Deformation (lbs.)	Clip Angle Ultimate Load before Failure (lbs.)
3/8	6	9/16	1/2	2	4	2	4	15,300	24,750	9,000	15,300
3/8	6	11/16	5/8	2	4	2	4	15,300	30,938	11,250	15,300
1/2	6	9/16	1/2	2	4	2	4	20,400	33,000	12,000	20,400
1/2	6	11/16	5/8	2	4	2	4	20,400	41,250	15,000	20,400



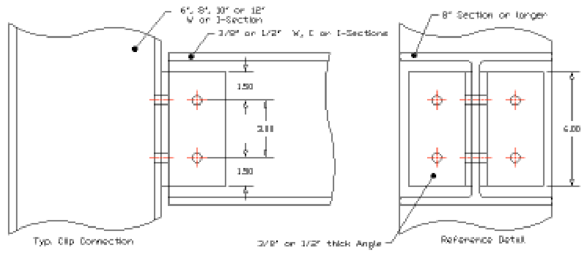
1500/1525 Series Pultex® SuperStructural Angles											
Clip Angle Thickness (in)	Angle Connection Length (in)	Bolt Hole Dia. (in)	Bolt Dia. (in)	# of Angles in Connection	# of Bolts in Shear	# of Bolts through Beam Web	# of Bolts Connected to Column	Ultimate Shear Load through Angle Heels (lbs.)	Ultimate Bearing Load of Connection (lbs.)	Allowable Bearing Load for 4% Hole Deformation (lbs.)	Clip Angle Ultimate Load before Failure (lbs.)
3/8	8 1/4	9/16	1/2	2	6	3	6	21,038	37,125	13,500	21,038
3/8	8 1/4	11/16	5/8	2	6	3	6	21,038	46,406	16,875	21,038
1/2	8 1/4	9/16	1/2	2	6	3	6	28,050	49,500	18,000	28,050
1/2	8 1/4	11/16	5/8	2	6	3	6	28,050	61,875	22,500	28,050



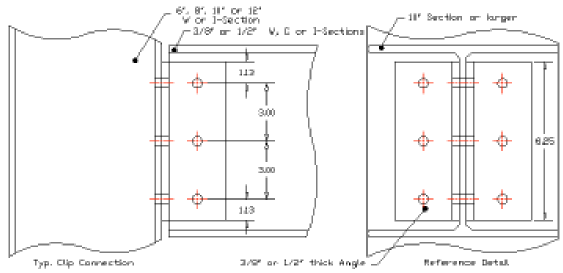
1625 Series Pultex® SuperStructural Angles											
Clip Angle Thickness (in)	Angle Connection Length (in)	Bolt Hole Dia. (in)	Bolt Dia. (in)	# of Angles in Connection	# of Bolts in Shear	# of Bolts through Beam Web	# of Bolts Connected to Column	Ultimate Shear Load through Angle Heels (lbs.)	Ultimate Bearing Load of Connection (lbs.)	Allowable Bearing Load for 4%Hole Deformation (lbs.)	Clip Angle Ultimate Load before Failure (lbs.)
3/8	4 1/2	9/16	1/2	2	4	2	4	13,163	24,750	9,000	13,163
3/8	4 1/2	11/16	5/8	2	4	2	4	13,163	30,938	11,250	13,163
1/2	4 1/2	9/16	1/2	2	4	2	4	17,550	33,000	12,000	17,550
1/2	4 1/2	11/16	5/8	2	4	2	4	17,550	41,250	15,000	17,550



1625 Series Pultex® **SuperStructural** Angles											
Clip Angle Thickness (in)	Angle Connection Length (in)	Bolt Hole Dia. (in)	Bolt Dia. (in)	# of Angles in Connection	# of Bolts in Shear	# of Bolts through Beam Web	# of Bolts Connected to Column	Ultimate Shear Load through Angle Heels (lbs.)	Ultimate Bearing Load of Connection (lbs.)	Allowable Bearing Load for 4% Hole Deformation (lbs.)	Clip Angle Ultimate Load before Failure (lbs.)
3/8	6	9/16	1/2	2	4	2	4	17,550	24,750	9,000	17,550
3/8	6	11/16	5/8	2	4	2	4	17,550	30,938	11,250	17,550
1/2	6	9/16	1/2	2	4	2	4	23,400	33,000	12,000	23,400
1/2	6	11/16	5/8	2	4	2	4	23,400	41,250	15,000	23,400



1625 Series Pultex® **SuperStructural** Angles											
Clip Angle Thickness (in)	Angle Connection Length (in)	Bolt Hole Dia. (in)	Bolt Dia. (in)	# of Angles in Connection	# of Bolts in Shear	# of Bolts through Beam Web	# of Bolts Connected to Column	Ultimate Shear Load through Angle Heels (lbs.)	Ultimate Bearing Load of Connection (lbs.)	Allowable Bearing Load for 4% Hole Deformation (lbs.)	Clip Angle Ultimate Load before Failure (lbs.)
3/8	8 1/4	9/16	1/2	2	6	3	6	24,131	37,125	13,500	24,131
3/8	8 1/4	11/16	5/8	2	6	3	6	24,131	46,406	16,875	24,131
1/2	8 1/4	9/16	1/2	2	6	3	6	32,175	49,500	18,000	32,175
1/2	8 1/4	11/16	5/8	2	6	3	6	32,175	61,875	22,500	32,175



Note: The maximum slip of the connection at failure is approximately 0.35"without adhesive. The maximum slip of the connection at failure is approximately 0.101" with adhesive. The adhesive does not affect the ultimate load capacity of the connection, but does affect the amount of slip of the connection.

Bolt Hole Bearing Capacity of Pultex® SuperStructural Beams and Columns

Tables 4-3 and 4-4 represent the bearing capacity of the flange and web sections of various beams and columns. The load table is based on the following:

1. The ultimate lengthwise bearing strength of the flange sections is 33,000 psi.
2. The ultimate crosswise bearing strength of the web section is 30,000 psi.
3. The 4% hole deformation longitudinal bearing strength of the flange is 12,000 psi.
4. The 4% hole deformation transverse bearing strength of the web is 11,000 psi.
5. No safety factors are applied.
6. Testing performed at room temperature (73°F)
7. No adhesive is used.
8. Bolt torque is not considered.

Table 4-3
Bolt Hole Bearing Capacity of Pultex® SuperStructural Columns and Beams for 1500/1525 Series
Design Chart

Member Thickness (inches)	Bolt Diameter (inches)	# of Bolts in Column	# of Bolts in Beam	4% Hole Deformation Beam Web Bearing Load in Transverse Direction (lbs.)	Ultimate Beam Web Bearing Load in Transverse Direction (lbs.)	4% Hole Deformation Column Flange Bearing Load in Longitudinal Direction (lbs.)	Ultimate Column Flange Bearing Load in Longitudinal Direction (lbs.)
1/4	1/2	2	1	938	2,563	3,000	8,250
1/4	1/2	4	2	1,875	5,125	6,000	16,500
1/4	1/2	6	3	2,813	7,688	9,000	24,750
3/8	1/2	2	1	2,063	5,625	4,500	12,375
3/8	1/2	4	2	4,125	11,250	9,000	24,750
3/8	1/2	6	3	6,188	16,875	13,500	37,125
1/2	1/2	2	1	2,750	7,500	6,000	16,500
1/2	1/2	4	2	5,500	15,000	12,000	33,000
1/2	1/2	6	3	8,250	22,500	18,000	49,500
1/4	5/8	2	1	1,172	3,203	3,750	10,313
1/4	5/8	4	2	2,344	6,406	7,500	20,625
1/4	5/8	6	3	3,516	9,609	11,250	30,938
3/8	5/8	2	1	2,578	7,031	5,625	15,469
3/8	5/8	4	2	5,156	14,063	11,250	30,938
3/8	5/8	6	3	7,734	21,094	16,875	46,406
1/2	5/8	2	1	3,438	9,375	7,500	20,625
1/2	5/8	4	2	6,875	18,750	15,000	41,250
1/2	5/8	6	3	10,313	28,125	22,500	61,875

Note: Values are representative for bolted connections without adhesives.
Transverse bearing stress of 1/4" Pultex® SuperStructural profiles, Web sections = 20,500 psi., 4% Elongation = 7,500 psi

Table 4-4
Hole Bearing Capacity of Pultex® SuperStructural Columns and Beams for 1625 Series
Design Chart

Member Thickness (inches)	Bolt Diameter (inches)	# of Bolts in Column	# of Bolts in Beam	4% Hole Deformation Beam Web Bearing Load in Transverse Direction (lbs.)	Ultimate Beam Web Bearing Load in Transverse Direction (lbs.)	4% Hole Deformation Column Flange Bearing Load in Longitudinal Direction (lbs.)	Ultimate Column Flange Bearing Load in Longitudinal Direction (lbs.)
1/4	1/2	2	1	1,175	2,938	3,500	9,500
1/4	1/2	4	2	2,350	5,875	7,000	19,000
1/4	1/2	6	3	3,525	8,813	10,500	28,500
3/8	1/2	2	1	2,372	6,469	5,250	14,250
3/8	1/2	4	2	4,744	12,938	10,500	28,500
3/8	1/2	6	3	7,116	19,406	15,750	42,750
1/2	1/2	2	1	3,163	8,625	7,000	19,000
1/2	1/2	4	2	6,325	17,250	14,000	38,000
1/2	1/2	6	3	9,488	25,875	21,000	57,000
1/4	5/8	2	1	1,469	3,672	4,375	11,875
1/4	5/8	4	2	2,938	7,344	8,750	23,750
1/4	5/8	6	3	4,406	11,016	13,125	35,625
3/8	5/8	2	1	2,965	8,086	6,563	17,813
3/8	5/8	4	2	5,930	16,172	13,125	35,625
3/8	5/8	6	3	8,895	24,258	19,688	53,438
1/2	5/8	2	1	3,953	10,781	8,750	23,750
1/2	5/8	4	2	7,906	21,563	17,500	47,500
1/2	5/8	6	3	11,859	32,344	26,250	71,250

Note: Values are representative for bolted connections without adhesives.

Transverse bearing stress of 1/4" Pultex® **SuperStructural** profiles 1625 Series, Web sections = 23,500 psi., 4% elongation = 9,400 psi

Connection Fastener Edge Distance and Torque Level Recommendations[2]

Ratio of distance to fastener diameter		
	Range	Recommended
Edge Distance- End	2.0-4.5	3.0
Edge Distance- Side	1.5-3.5	2.0
Pitch (Distance between holes)	4.0-5.0	5.0

Reference: Structural Plastics Design Manual

ASTM A325	Low Torque 37.5% of Bolt Proof Load	High Torque 75% of Bolt Proof Load
Bolt Size	Torque ft-lbs. (N m)	Torque ft-lbs. (N m)
1/2" -13	29 (39)	57 (77)
5/8" - 11	57 (77)	113 (153)

Bolt recommendations based on ASTM A325 grade five coarse threaded bolts
Connection testing performed with grade 8 oversized washers (2.5 times hole dia.)
Proof strength for ASTM A325 bolt equals 75,000 psi (0.5 GPa)

Appendix B – Permission for Reuse

From : Shawna Holler <sholler@pultrude.com> Mon, Oct 03, 2011 11:59 AM
Subject : RE: Permission for reuse
To : Renee Sommer <rsommer@k-state.edu>

Renee,

I spoke with the Director of Marketing and Product Development regarding your request. You have permission to use the pages in your report.

Regards,

Shawna L. Holler
Marketing Coordinator

Creative Pultrusions, Inc.
214 Industrial Lane, Alum Bank, PA 15521
Phone: 814-839-4186 Ext. 221 Fax: 814-839-4276
www.creativepultrusions.com

From: Renee Sommer [mailto:rsommer@k-state.edu]
Sent: Monday, October 03, 2011 12:17 PM
To: sholler@pultrude.com
Subject: Permission for reuse

Shawna Holler,

Hello, my name is Renee Sommer and I am a graduate student at Kansas State University in the Department of Architectural Engineering. I am researching FRP pultruded shape structural connections and I have been using Creative Pultrusions, Inc. for shapes and material properties. In my report I have referenced a few pages from 'The Pultex Pultrusion Design Manual' to size a beam to column clip angle connection. With your permission, I would like to provide a copy of these pages (4-91 - 4-95) in the appendix of my report.

Renee Sommer
Department of Architectural Engineering
Kansas State University
rsommer@ksu.edu
