PERMA-COLUMN®

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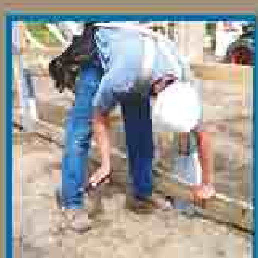
Deliver

Place

Level



Attach Skirt

"THE PERMANENT SOLUTION"

Why should you have to choose between a foundation that is economical and one that is permanent? PERMA-COLUMNS® are the first product to combine the economy and speed of post-frame construction with the durability of a concrete foundation.

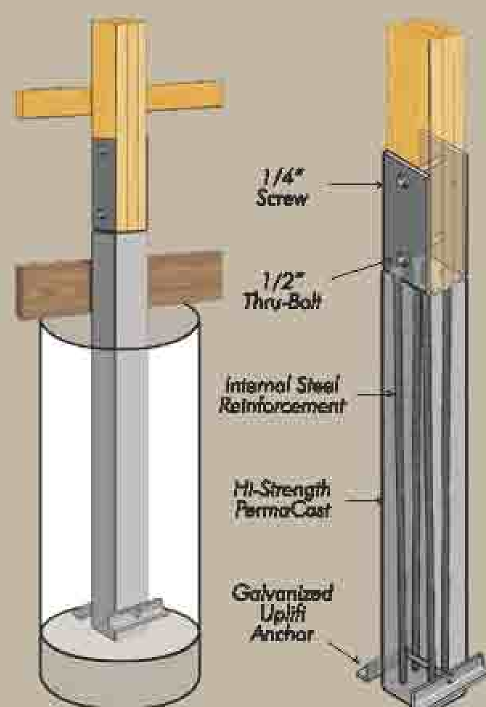
Proven - Concrete is the most proven and durable foundation material known to man!

Economical - PERMA-COLUMNS are the most economical method of constructing a post-frame building on concrete!

Safe - PERMA-COLUMNS are safe and have no harsh chemicals or preservatives like treated wood!

A building is only as permanent as the foundation on which it is built.

What are YOU building on?



PERMA COLUMN

phone: 1-800-622-7190
sales@permacolumn.com

Out with the old.

In with the new!

The Permanent Solution For Rotten Posts

Introducing the new PERMA-COLUMN® PC6600. The same pre-cast technology available in new construction can now be used to repair rotten posts. Don't let deteriorated posts compromise a building with many useful years left in it! The PC6600 is a long awaited answer to an old and ongoing problem.

A building is only as permanent as the foundation on which it is built. Concrete... durable, economical, proven; it only makes sense to choose *"The Permanent Solution."*

PERMA COLUMN

phone: 1-800-622-7190
sales@permacolumn.com

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POST REPLACEMENT
step-by step guide

1. Locate rotten post to be replaced. Detach siding and remove existing skirt board.

2. Dig down next to face and sides of rotten post with auger.

3. Support truss with brace and hydraulic jack.

4. Cut rotten post off at ground level. No measuring necessary at this point.

5. Remove rotten portion of post.

6. Clean out post hole to provide a level, compacted base for the pre-cast concrete pad. Sakrete or ready mix concrete can also be used for post base.

7. Install pad and measure the Perma-Column for exact clearance needed. *Note: Perma-Column length may vary.

8. Mark existing post and cut to desired clearance for Perma-Column installation.

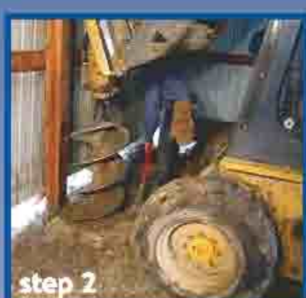
9. Install and position Perma-Column.

10. Install 1/4" x 3" wood lags. Drill and install (2) 1/2" bolts.

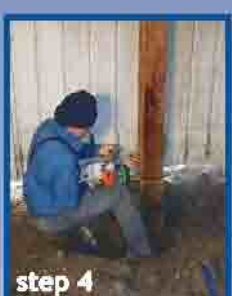
11. Reattach siding and skirt board.

*Building renovation is hazardous and should only be performed by a construction professional. This guide in its entirety can be found on the web at www.permacolumn.com


step 1


step 2


step 3


step 4


step 7


step 8


step 9


step 10

TESTIMONIALS

"When we first saw the Perma-Columns, we knew that was our answer."

"We have a 22 year old pole barn and had been watching the condition of the poles for roughly a year. Nothing appeared abnormal above ground, but when we dug the wooden poles out we found that nearly all of them were well over **50% gone.** This was an accident waiting to happen.

When we first saw the Perma-Columns, we knew that was our answer. They proved to be a cost and time effective means of solving our problem. Today we have a building, that in essence, has a concrete foundation with no wood in the ground."

Steve Cramer
Findlay, OH

"That much rot on treated wood was surprising..."

"We found that all the posts on our 100' x 110' show arena were nearly or completely rotted off below the ground. That much rot on treated wood was surprising in a building that is just 20 years old. After we did the work, it's like we have a new building."

Tom Fulton
Huntington County Fair Board
Huntington, IN

5 Models To Meet Your Needs

PC6300 - 3 ply 2" X 6" laminated wood column

PC6400 - 4 ply 2" X 6" laminated wood column

PC6600 - 6" X 6" wood post and replacement

PC8300 - 3 ply 2" X 8" laminated wood column

PC8400 - 4 ply 2" X 8" laminated wood column

*Consult a design professional for appropriate model.

PERMA COLUMN

PERMA-COLUMN®

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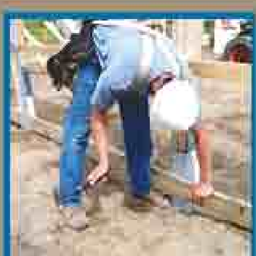
Deliver

Place

Level



Attach Skirt

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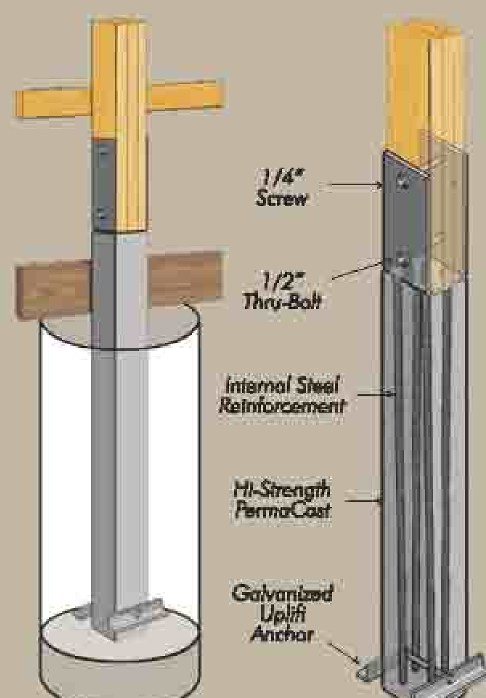


PERMA COLUMN

phone: 1-800-622-7190
sales@permacolumn.com

I. REVOLUTIONARY DESIGN

A. PERMA-COLUMNS are the first product to combine the economy of post-frame construction with the durability of a concrete foundation.

B. The exclusive design of the PERMA-COLUMN is patented.

II. TECHNICAL SPECIFICATIONS

A. The latest in SCC pre-casting technology creates a product with three times the strength of regular concrete.

B. Polymer fiber reinforcement and premium grade steel reinforcement dramatically enhance flexural strength.

C. Corrosion inhibitors protect internal steel reinforcement.

D. Air entraining admixtures insure resistance to harsh freeze-thaw cycles.

III. INVESTMENT SECURITY

A. PERMA-COLUMNS keep wood out of the ground ensuring that your building's foundation will never rot.

B. PERMA-COLUMNS protect your building investment by securing its value for a lifetime.

IV. ENVIRONMENTAL SAFETY

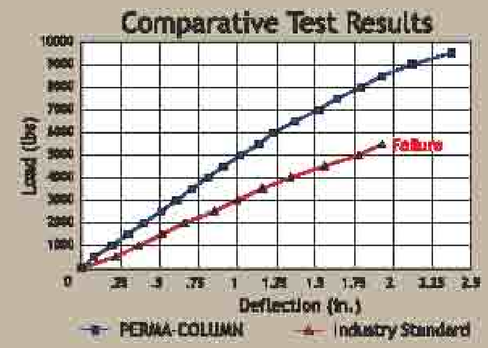
A. PERMA-COLUMNS do not use the harsh chemicals found in treated lumber, which are being phased out by the EPA.

B. Using an environmentally friendly product promotes peace of mind.

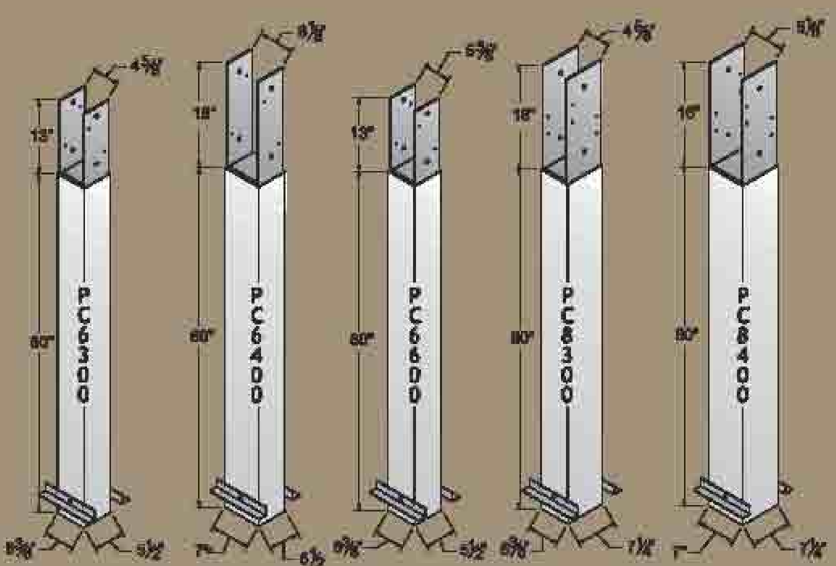
V. STRENGTH DATA

A. PERMA-COLUMNS have been tested by The University of Wisconsin. Design manuals are available by request.

B. In comparative strength tests performed by Purdue University, PERMA-COLUMNS have proven to outperform the industry standard.



5 MODELS TO MEET YOUR NEEDS



PC6300 - for a 3 ply 2" X 6" laminated wood column

PC6400 - for a 4 ply 2" X 6" laminated wood column

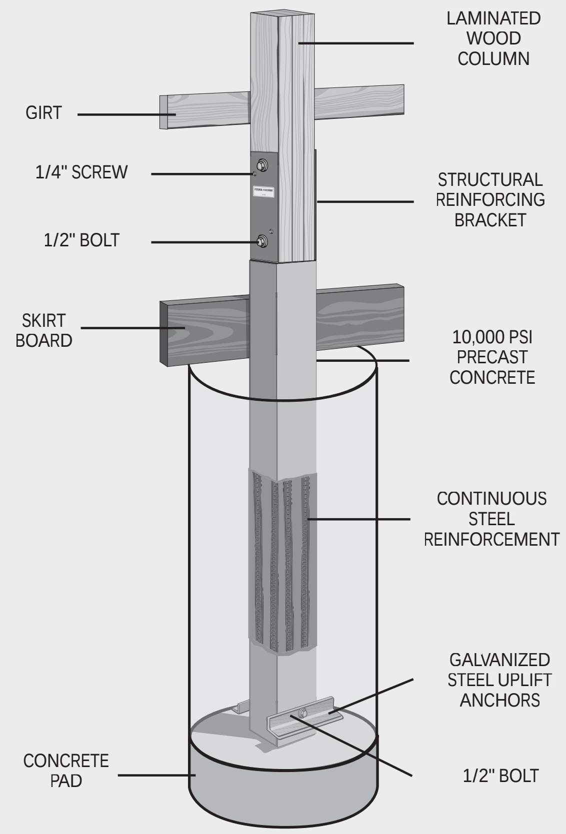
PC6600 - for a 6" X 6" wood post replacement

PC8300 - for a 3 ply 2" X 8" laminated wood column

PC8400 - for a 4 ply 2" X 8" laminated wood column

*Consult a design professional for appropriate model.

Installation Manual



THE PERMANENT SOLUTION

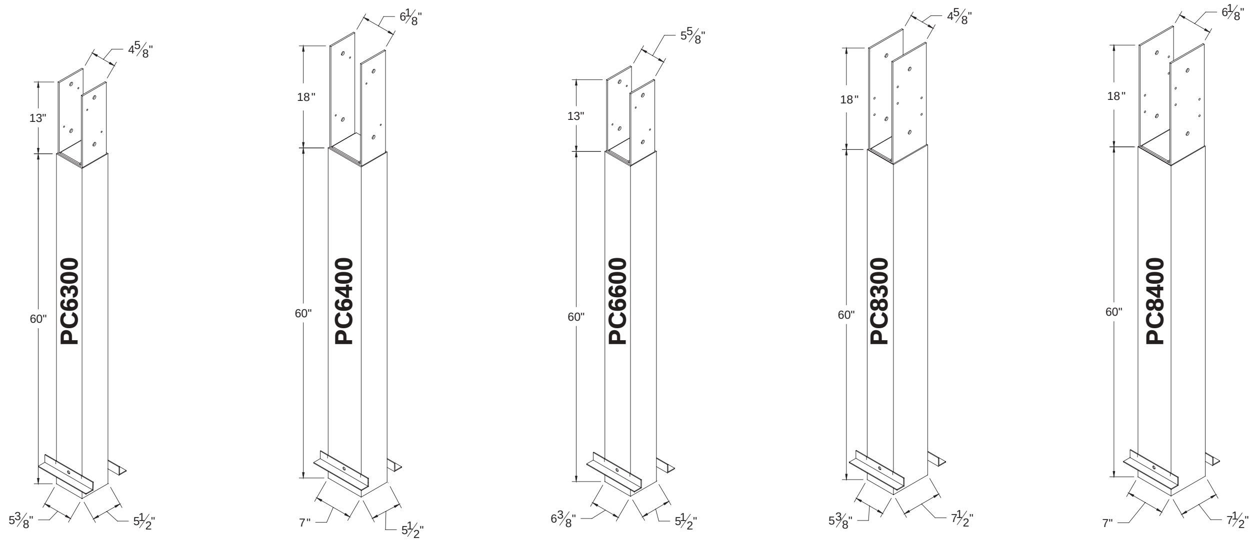
Extreme Strength

Lasting Longevity

Environmentally Friendly

PERMA COLUMN

Five Models



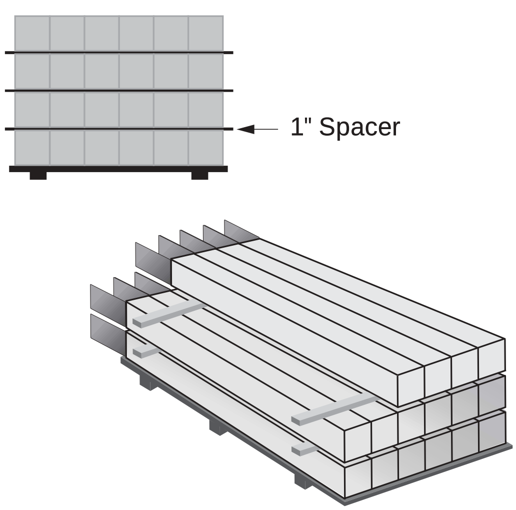
FIVE MODELS TO MEET YOUR DESIGN REQUIREMENTS.

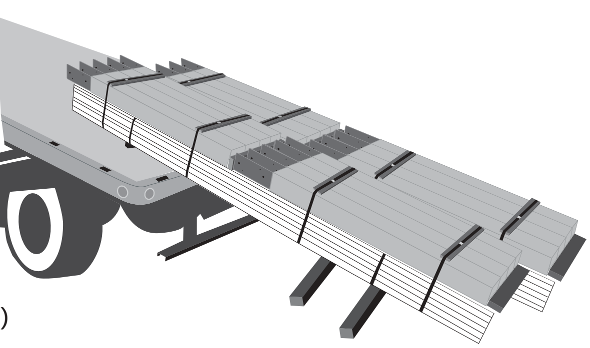
PC6300 — designed for a 3 ply 2" X 6" laminated wood column (5½" X 4½")
PC6400 — designed for a 4 ply 2" X 6" laminated wood column (5½" X 6")
PC6600 — designed for a 6" X 6" wood post and replacement (5½" X 5½")
PC8300 — designed for a 3 ply 2" X 8" laminated wood column (7¼" X 4½")
PC8400 — designed for a 4 ply 2" X 8" laminated wood column (7¼" X 6")
Consult a design professional for appropriate model.

Handling & Storage

- Avoid direct impact of steel forks with concrete column to reduce surface chipping.
- Minor surface chipping can be repaired with premixed concrete patch available at your local hardware store.
- Place a wood spacer between every row of columns to avoid concrete to concrete contact. (A)
- Do not stack columns more than four rows high on a pallet. (A)
- Keep columns covered during storage and shipping to preserve appearance.
- PCs* can be dump unloaded by banding securely to wood bundle. (B)

(A)



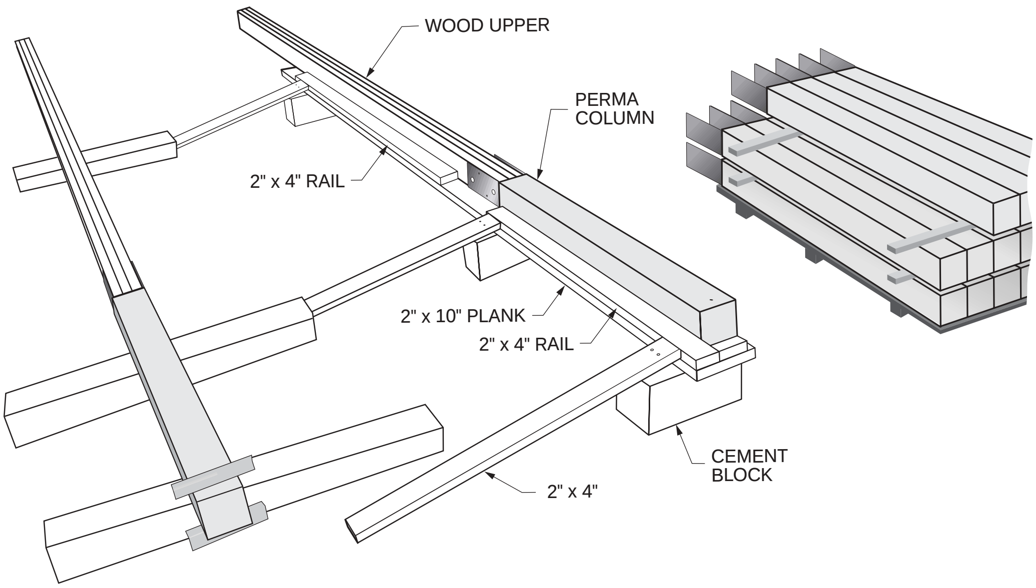


(B)

**PERMA-COLUMN to be hereafter referred to as PC.*

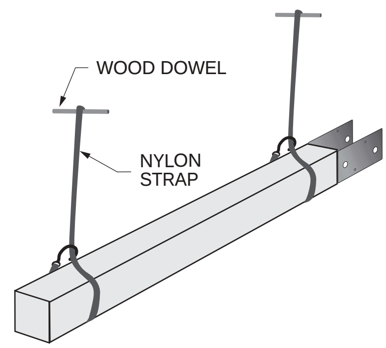
Option 1: Preassembled Column

1. Place unassembled PC's close to the assembly table.
2. Assembly table to be level and no more that 12" off the ground to facilitate manual lifting. (C) If a hoist is available, the assembly table works best at around 30" high.
3. A 2" x 10" plank with a 2" x 4" back rail setting on cement blocks will suffice. (C)



(C)

4. Lift PC using a nylon "choke" strap around each end. (D)
5. Place PC on assembly table and insert wood column in steel bracket. Wood column must sit tight against the steel seat. Bottom of wood column may need to be square cut to insure tight fit. (E)
6. Clamp wood column to table before drilling or inserting screws to hold joint tight during assembly.
7. Install 1/4" x 3" Simpson™ SDS screws (or equal) before drilling 1/2" bolt holes through wood column.

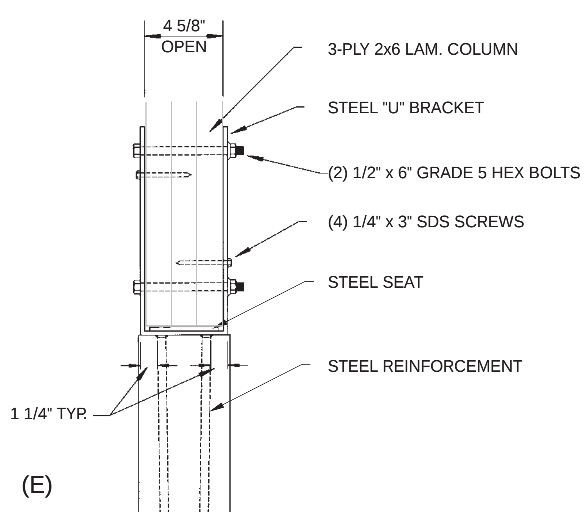


(D)

Fastener Requirements*

(4) 1/4" x 3" Simpson™ SDS screws (or equal) required for PC6300, PC 6400 and PC6600.
(8) 1/4" x 3" Simpson™ SDS screws (or equal) required for PC8300 and PC8400.
(2) 1/2" x 6" Grade #5 HHCS bolt, nut and washer required for PC6300 and PC8300.
(2) 1/2" x 7" Grade #5 HHCS bolt, nut and washer required for PC6600.
(2) 1/2" x 8" Grade #5 HHCS bolt, nut and washer required for PC6400 and PC8400.
*Fasteners are not provided.

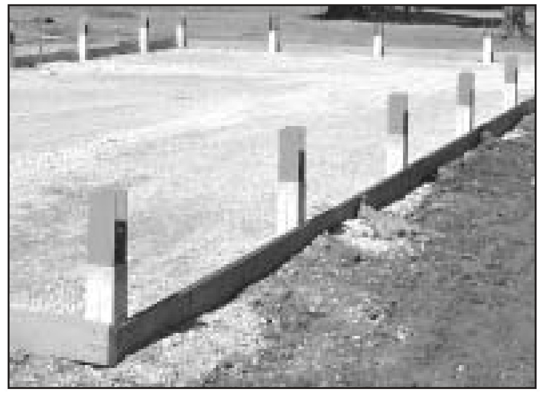
8. Insert 1/2" grade 5 bolts in drilled holes and tighten nuts to approximately 110 foot pounds of torque.
9. Roll assembled column off the back side of the table onto wood stickers for skid loader pick-up.



(E)

Option 2: Unassembled Column

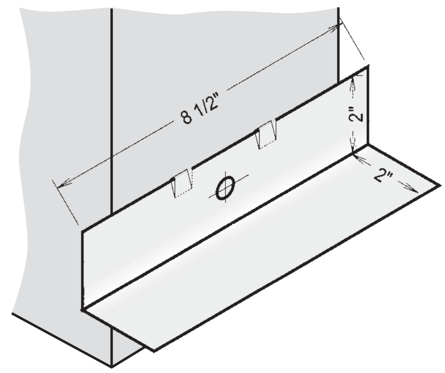
- PC's can be installed without the wood upper, using the same procedures as the "Post Hole Digging Requirements" below.
- This will allow for better access to place the concrete floor. (F)
- Using this option allows for building to be started in the fall and completed in the winter.



(F)

Attaching Uplift Anchors

20AU UPLIFT ANCHOR



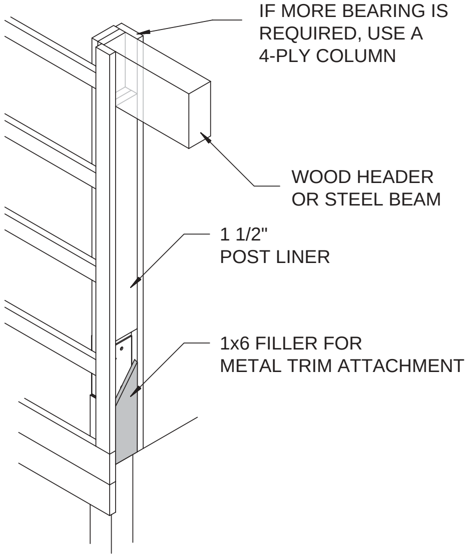
1. Consult a design professional for appropriate sized uplift anchor.
   PC20UA Uplift Anchor 2" x 2" x 8.5" Galvanized
   PC30UA Uplift Anchor 3" x 3" x 12" Galvanized

2. Attach uplift anchor with 1/2" bolt. Tighten nut firmly until uplift anchor does not rotate.
   1/2" x 7" bolt and nut required for PC6300, PC6400 and PC6600.
   1/2" x 9" bolt and nut required for PC8300 and PC8400.

Post Hole Digging Requirements

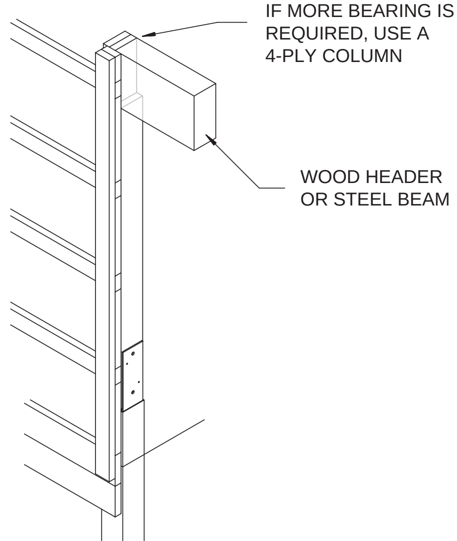
- For required PC embedment depth, consult a design a professional.
- Concrete portion of PC's to be flush with the sidewall girt line.
- **Overhead door openings to be 3" wider than desired finished opening to accommodate 1 1/2" PC bracket trim-out. (G)**
- **Slide door openings to be the same as the desired finished opening.**
  PC brackets intrude into the slide door opening by 3/8". (H)
- Dig post hole depth so all PC brackets are at a uniform height. If adjustment is necessary, use tamped gravel.
- If bedrock is contacted at a post hole location, the bottom of the PC may be cut off using a masonry saw. The uplift anchor hole will need to be redrilled using a 9/16" masonry drill bit.

OHD DOOR JAMB



(G)

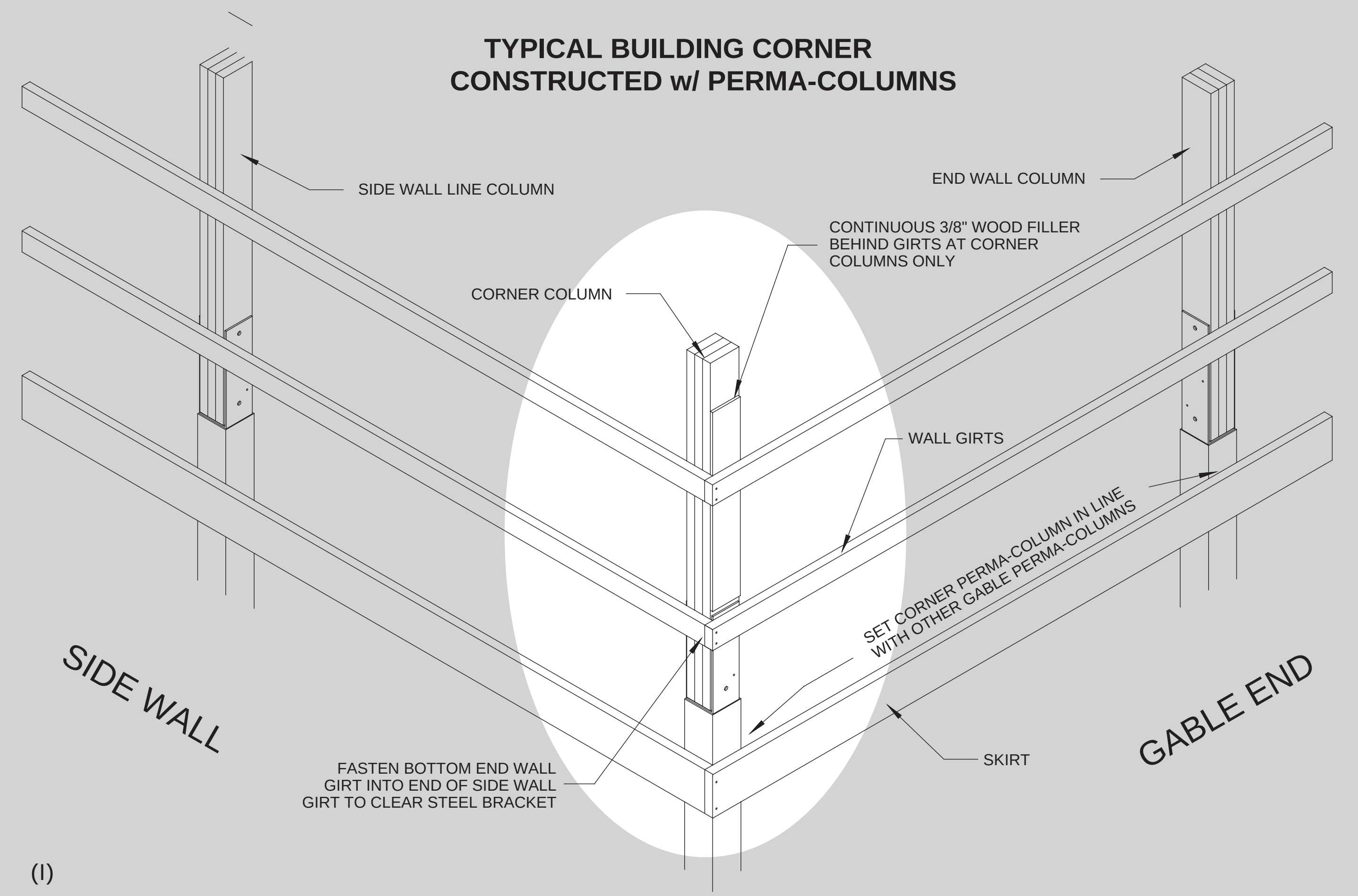
SLIDE DOOR JAMB



(H)

Column Placement and Leveling

1. Attach a 3/8" wood filler to the (4) corner columns to make girts and skirt flush. (I)
2. Lift multiple PC assemblies with a skid loader and drive along the post hole line.
3. Place concrete pad in bottom of hole prior to setting PC. Consult design professional to determine thickness of concrete pad.
4. Tilt PC assemblies off skid loader forks into post hole. (J)
5. Plumb the PC columns using standard leveling procedures. (K)
6. Backfill post holes with appropriate materials, tamping 6" layers until hole is filled.



(I)


(J)

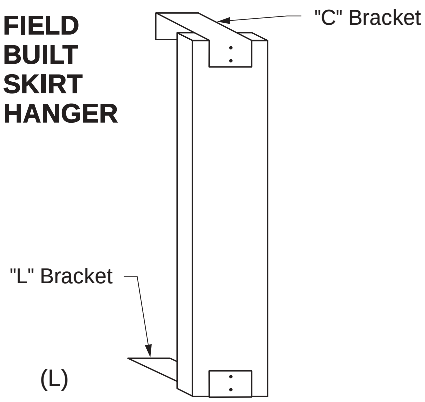

(K)

Skirt Board Attachment

1. Install first row of sidewall girts before attaching the skirtboard. The skirtboard will be hung from this first row of girts.

   Make two skirt hangers to allow for hands free skirt placement. (L)
   - Using a 2" x 4" board, cut the board to the proper length to hang the skirtboard.
   - Attach a metal "C" bracket (1 1/2" pocket) to this board to hang over the bottom girt row.
   - Attach a metal "L" bracket (1 1/2" seat) to this board to hang the skirtboard in place.

2. Hang the skirtboard from the first girt row using skirt hanger while drilling and attaching.
3. Drill a 3/16" hole through the skirt board and 2" into the concrete post using a hammer drill. (M)
   NOTE: Angle the drill toward the center of the concrete post to avoid hitting the interior rebar. (E page 2)
4. Drive a 3/16" x 3" Powers stainless steel split drive anchor (PC3DA-SS) into the post until the skirt is secure. (N)



(L)

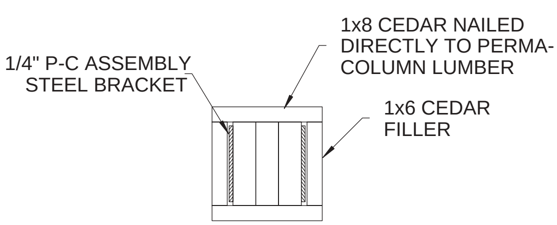
(M)

(N)

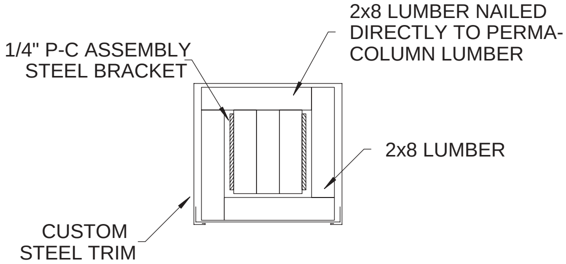
Porch Post Trim Detail

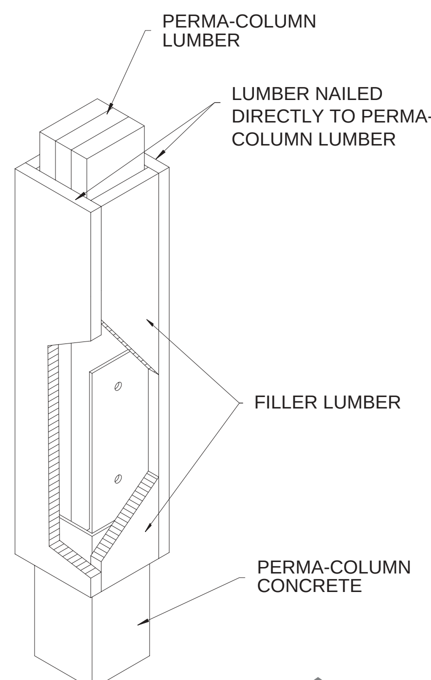
1. Porch posts can be trimmed-out using the following methods.

METHOD 1



METHOD 2





Perma-Column Design and Use Guide

for

PC6300, PC6400, PC6600,
PC8300, and PC8400 Models

by
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Timber Tech Engineering, Inc
E-Mail: bl@timbertecheng.com

April 18, 2005

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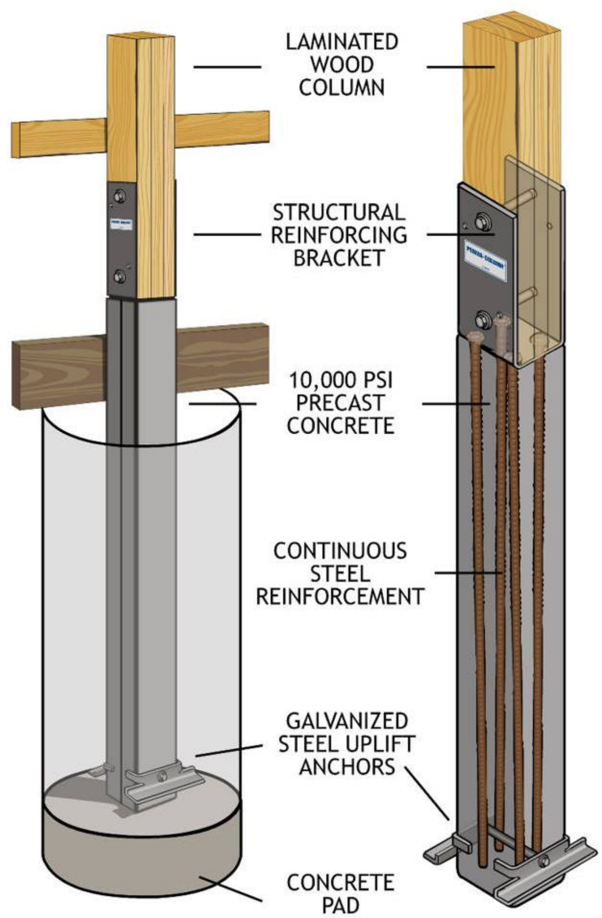
Table of Contents

Foreword

The following *Design and Use Guide for PC6300, PC6400, PC6600, PC8300, and PC8400 Models* has been written by Brent Leatherman to help engineers apply information appearing in the *Engineering Design Manual for Series 6300, 6400, 8300, 8400 Perma-Columns*. I wrote the latter document after conducting a number of tests on Perma-Columns in my laboratory at the University of Wisconsin-Madison. In addition to a summary of UW-Madison test results, the *Engineering Design Manual* contains details on how to calculate design properties for Perma-Columns. The *Design and Use Guide* reviews these procedures, and contains sample calculations and comparisons that you will not find in the *Engineering Design Manual.*

Perma-Columns are a more environmentally-friendly alternative to preservative-treated wood, and quite likely, a more durable alternative. Using the information contained in this document and in the *Engineering Design Manual*, an engineer can better ensure the structural integrity and safety of buildings incorporating Perma-Columns.

David R. Bohnhoff, Ph.D., P.E.



1. Design Overview

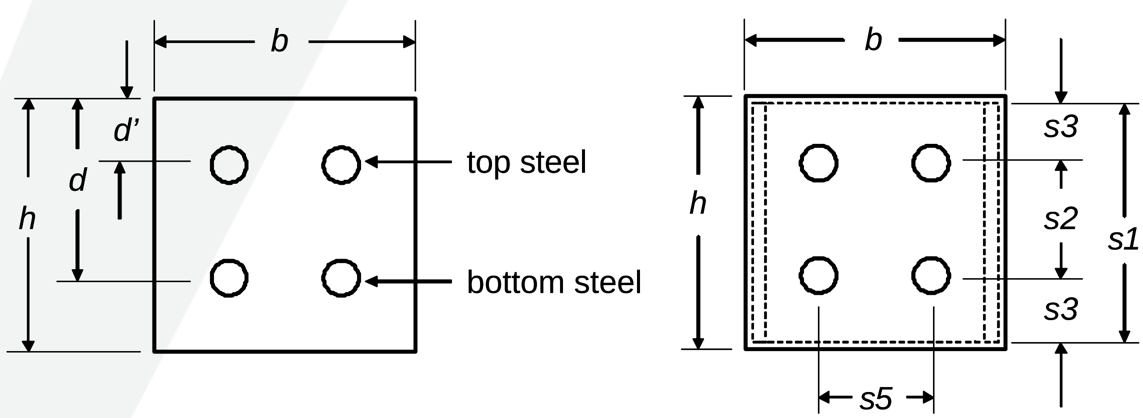
This guide is intended to be used as a companion document to the *Engineering Design Manual for Series 6300, 6400, 8300, 8400 Perma-Columns* (herein referred to as "the Manual") by David R. Bohnhoff. Each ***Perma-Column*** assembly consists of a reinforced concrete base designed according to The American Concrete Institute (ACI), a steel bracket designed according to The American Institute of Steel Construction (AISC), and a mechanically laminated wood column designed according to The American Forest and Paper Association (AF&PA) specifications. The structural analysis for each of these components was performed using a load and resistance factor (LRFD) design methodology. This was done to allow use of one set of load combinations for the entire assembly, and to provide an accurate look at column failure modes. The deflection limits used in this design were taken from *IBC 2003 Table 1604.3* for exterior walls with brittle or flexible finishes, and are L/240 and L/120, respectively. The overall column deflection is to be checked using service (unfactored) loads.

This guide will cover properties, and design issues for the reinforced concrete base, the steel bracket connection, and the mechanically laminated wood columns. We will look at creating models of the ***Perma-Column*** assemblies to simulate the results of laboratory testing. Design charts will be presented for all the ***Perma-Column*** assemblies with varying heights, and boundary conditions. The failure modes and design limitations on each ***Perma-Column*** assembly will be discussed, and we will give an example showing a straight forward design approach which can be applied to all ***Perma-Column*** assemblies. Finally, we will look at wind uplift capacity for a concrete collar or a packed fill foundation condition.

2. *Perma-Column* Descriptions

Dimensions and material properties for the PC6300, PC6400, PC6600, PC8300, and PC8400 models are given in Table 2.1. The PC6600 model is intended for new or replacement solid-sawn 6x6 posts and was not included in the laboratory testing. Variable definitions correspond to Figure 1.1 of the Manual. Section properties for 3-ply 2x6, 4-ply 2x6, 3-ply 2x8, and 4-ply 2x8 mechanically laminated wood columns are given in Table 3.5.1 of the Manual.

Table 2.1: PC6300, PC6400, PC6600, PC8300, and PC8400 Dimensions and Properties



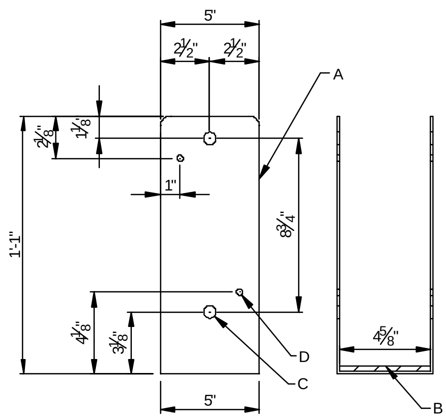
Variable	Symbol	Units	PC6300	PC6400	PC8300	PC8400	PC6600
Overall Concrete Width	b	in.	5.38	6.88	5.38	6.88	6.38
Overall Concrete Depth	h	in.	5.44	5.44	7.19	7.19	5.44
Depth to Top Steel	d'	in.	1.50	1.50	1.56	1.56	1.50
Depth to Bottom Steel	d	in.	3.94	3.94	5.62	5.62	3.94
Width of Steel Bracket	$s1$	in.	5.00	5.00	7.00	7.00	5.00
Top & Bottom Steel Spacing	$s2$	in.	2.44	2.44	4.06	4.06	2.44
Steel Distance to Bracket Edge	$s3$	in.	1.28	1.28	1.47	1.47	1.28
Area of Top Steel	A_s'	in.^{2}	0.40	0.40	0.62	0.62	0.40
Area of Bottom Steel	A_s	in.^{2}	0.40	0.40	0.62	0.62	0.40
Steel Yield Strength	f_y	lbf/in.^{2}	60,000	60,000	60,000	60,000	60,000
Concrete Compressive Strength (nominal)	f_c'	lbf/in.^{2}	10,000	10,000	10,000	10,000	10,000
Steel Modulus of Elasticity	E_s	lbf/in.^{2}	29000000	29000000	29000000	29000000	29000000

3. Reinforced Concrete Design

The reinforced concrete component is manufactured with 10,000 psi (nominal) pre-cast concrete and four (4) 60,000 psi vertical reinforcing bars. Number 4 bars are used for the PC6300, PC6400, and PC6600, while number 5 bars are used for the PC8300, and PC8400 models. The required concrete cover for reinforcing bars in pre-cast concrete is lower because of better placement accuracy during the manufacturing process. The high concrete strength and quality is achieved by adding superplasticizers which increase strength by allowing a low water-to-cement ratio. Fiber reinforcers are added to reduce shrinkage, increase impact resistance, and increase flexural strength. Other admixtures are included in the concrete mix to increase freeze/thaw resistance, protect the steel reinforcement from rusting, increase flexural and compressive strength, and optimize the hydration process. Bending, axial and shear strength properties of the reinforced concrete are discussed in Section 3 of the Manual.

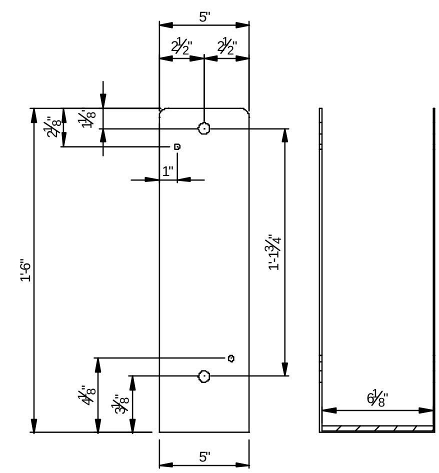
4. Steel Bracket Design

Figure 4.1 shows dimensions for the different steel brackets that are used with the ***Perma-Column*** assemblies. The brackets consist of ¼” A36 steel with 5/8” diameter holes for the bolts, and 5/16” diameter holes for screws. The bracket connection utilizes ½” diameter A325 bolts in double shear with hex nuts torqued to 110 ft-lbs, and ¼”x3” strong drive screws (SDS) by Simpson Strong-Tie or equivalent in single shear installed from each side. Typically, one screw is installed from each side of the bracket at each bolt, except the PC8300 and the PC8400 have two screws on each side at each bolt. Screws help prevent stress concentrations around the bolt which would cause splitting of the wood members. The wood columns bear directly on a ¼” steel seat plate which helps to transfer axial loads directly into the concrete base. Four A706 weldable reinforcing bars are inserted in holes in the bottom of the bracket and fillet welded, connecting the bracket to the concrete base.

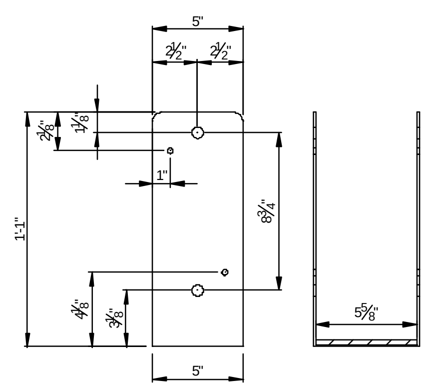


Parts Notes:
A = 1/4" steel bracket
B = 1/4" steel seat plate
C = 5/8"Ø hole for bolt
D = 5/16"Ø hole for screw

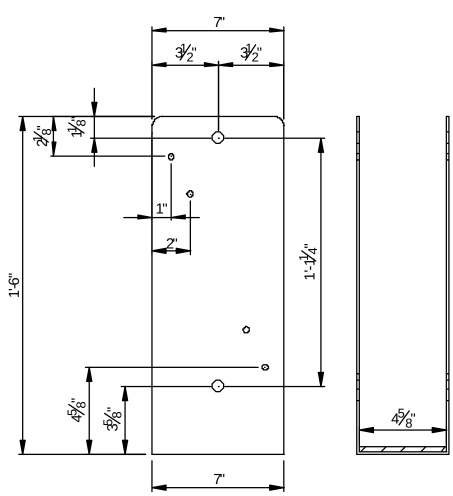
PC6300



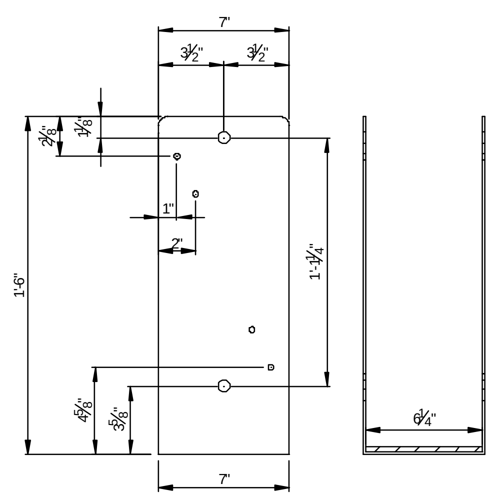
PC6400



PC6600



PC8300



PC8400

Figure 4.1 Steel Bracket Assemblies

4.1 Bracket Moment Capacity

This joint has significant moment capacity and does not need to be modeled as a pin. The strength of the concrete-to-steel bracket, and the steel bracket-to-wood post connections needs to be evaluated in order to determine the moment capacity of the joint. The reinforcing bars transfer shear and moment between the concrete base and the steel bracket. The failure modes observed in the laboratory testing are 1.) concrete crushing and 2.) tension steel fracture (see Figure B.5 and B.6 in the Manual). The bolts and screws transfer shear and moment between the steel bracket and wood column. The fasteners themselves, and not the steel bracket, control the strength of this joint. The bolt and screw design should be performed according to the 1996 edition of the *LFRD Manual for Engineered Wood Construction* by The American Forest and Paper Association (AF&PA).

4.2 Rotational Stiffness

The rotational stiffness of the steel bracket connection depends upon both concrete-to-steel, and steel-to-wood movement. Table B.2 and Figure B.4 in the Manual show joint rotation versus bending moment data for the steel bracket-to-concrete connection. The moment capacity of each steel bracket was chosen as 60% of the maximum tabulated value shown in Table B.2 of the Manual. Table 4.1 shows the calculated stiffness values for the concrete-to-steel joint as discussed in Section 6.2 of the Manual. A linear assumption between joint rotation and bending moment was used to determine the rotational stiffness of each bracket. This stiffness value is needed in order to create a model as discussed in Section 6 of this Guide.

The stiffness of the steel-to-wood connection is controlled by the slip modulus for the bolts and screws, and is discussed in Section 6 of the Manual. The slip modulus should be assigned to the fastener group by summing the values of the individual fasteners in the group. The slip modulus for the ½" bolt in double shear is 85.5 k/in, and for the screws is 28.7 kips per inch.

Table 4.1 Element Strength and Stiffness Values

Series	Concrete-to-Steel Stiffness (k-in/rad)	Ultimate Strength (in-kip)
PC6300	2500	70.4
PC6400	2523	70.9
PC6600	2503	70.4
PC8300	4815	123.9
PC8400	4874	138.9

4.3 Friction

The moment capacity and rotational stiffness of the steel-to-wood joint is enhanced by friction produced when the bracket tends to pinch together as the bending moment increases. No increases were taken in the Manual for this phenomenon, but it remains as an additional safety factor against steel bracket-to-wood connection failure.

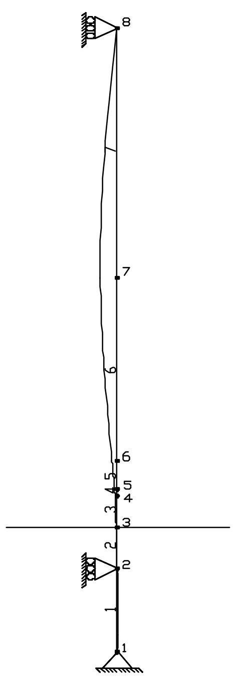
5. Mechanically Laminated Wood Column Design

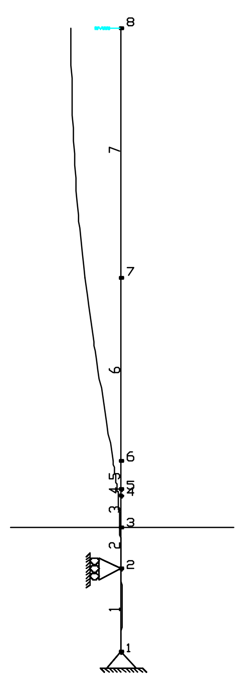
The wood portion of a ***Perma-Column*** assembly is designed using LRFD because that is the preferred method of design for the steel and reinforced concrete components. Reference strengths for wood member sizing, and the factored resistance values for connection detailing are taken from the 1996 edition of the *LFRD Manual for Engineered Wood Construction* by AF&PA. Design procedures were taken from ASAE EP559 *Design Requirements and Bending Properties for Mechanically Laminated Columns* and from *The LFRD Manual*. **No wet service reductions have been used since the wood portion is not in contact with the soil or concrete**, and it is assumed to be used in an enclosed building. There are no splices in the wood laminations. Axial load is assumed to be transferred by direct bearing on the seat plate, and not through bolts or screws. Buckling length for bending about the strong axis is one foot less than the overall column height because the concrete portion extends one foot above grade. The corresponding effective buckling length factor, Ke, was conservatively taken as 1.2 for columns fixed at the base, with horizontal movement allowed at the top; and 0.8 for columns pinned at the top. Structural analyses were performed using #1 Southern Yellow Pine (SYP), and #2 Spruce Pine Fir (SPF). The #1 SYP Nail-Lam "Plus" column as manufactured by Ohio Timberland Products, Inc was also included. More wood species and column assemblies will be checked in the future.

6. Modeling

Figure 6.1 shows an example of the structural analogs that were used to check each ***Perma-Column*** assembly. The structural analysis was performed using *Frame Analysis and Design* by Digital Canal, Inc. The structural analog was created with element stiffness values that closely simulate laboratory test results. These structural analogs can be used to predict ***Perma-Column*** assembly behavior under many different load conditions. The concrete element for each ***Perma-Column*** model was created using a concrete modulus of elasticity E_c, of 5.7 million psi, and an effective moment of inertia, I_e, as given in Table 5.2.1 in the Manual. I_e for the PC6600 was taken as 30 in^4 for modeling purposes. Elements 1, 2, and 3 of the analogs shown in Figure 6.1 represent the reinforced concrete base.

Element 4 in the analog represents the steel bracket. The purpose of this element is to model the bending flexibility of the steel bracket where it attached to the concrete. This element was assigned a modulus of elasticity, Es of 29 million psi, an effective length Le of 2.5 inches, and an effective moment of inertia I equal to S(Le)/Es where S is the rotational stiffness from Table 4.1. Table 6.1 summarizes the moment of inertia used for the concrete and steel bracket elements.





<u>8300-16-0 Analog</u> <u>8300-16-120 Analog</u>

Figure 6.1 Structural analogs for a column with pin or spring at top

Table 6.1 Element Moment of Inertia Values (in^4)			
Series	Concrete Element (Ie)	Concrete-to-Steel Element (I)	Steel-to-Wood Element (I)
PC6300	25.4	0.215	1.26
PC6400	38.7	0.218	5.41
PC6600	30	0.216	1.26
PC8300	72.2	0.415	4.6
PC8400	88.3	0.420	4.6

Element 5 in the analog is used to model the bracket-to-wood connection. This element extends between the fastener groups, and models the rotation between steel and wood. The length varies depending on the centroid of the fastener group. 8 inches was used for the PC6300 and PC6600, 13 inches for PC6400, and 11 inches for PC8300 and PC8400 (see Figure 4.1).

Elements 6 and 7 in the analog represent the laminated wood column with an E value of 1.7 million psi for # 1 SYP, and 1.4 million psi for #2 SPF.

After the structural analog was created and the loading applied, a P-delta analysis was performed to account for increased section forces induced by column deflection. Three post models were analyzed for each height to simulate different boundary conditions at the eave. The first assumes a very rigid diaphragm which allows no horizontal movement at the eave. The second and third models allow a horizontal movement corresponding to L/240 and L/120 respectively. These eave displacements were evaluated using service loads, and the larger of sidesway or curvature was taken as the controlling value. Horizontal movement was created in the model by using a spring support in place of a roller support. The post foundation was modeled assuming a 4'-0" embedment depth. A pin was used at the bottom, and a vertical roller at 1/3 the embedment depth to simulate a non-constrained post foundation.

7. Perma-Column Design Charts

Table 7.1 shows the maximum factored vertical load, $P_{u,}$ for ***Perma-Column*** assemblies under a constant wind load of 120 pounds per lineal foot. The post heights evaluated range from 8'-0" up to 20'-0" in two foot increments. Blank boxes in the chart indicate the column fails in deflection due to the constant wind load. Gray shaded numbers indicate ***Perma-Column*** assemblies limited by the steel bracket capacity. The failure modes checked are as follows:

1. Deflection Due to Service Loads
2. Wood Elements
    a. Combined axial and bending moment
    b. Shear
3. Steel Bracket Element
    a. Bending moment at base
4. Bracket-to-Wood Connection Element
    a. Combined shear and bending moment on steel-to-wood connection
5. Concrete Elements
    a. Bending moment and axial force compared to Interaction Diagram in Figure 3.3.2 in the Manual
    b. Shear

Table 7.1 Perma-Column Design Chart

Maximum factored vertical load, Pu (kips), for Perma-Column assemblies under constant wind load

#1 SYP		Column Height (ft) 8			10			12			14			16			18			20		
	Eave Condition	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**
	Eave Deflection (in)	**0**	**0.8**	**0.4**	**0**	**1**	**0.5**	**0**	**1.2**	**0.6**	**0**	**1.4**	**0.7**	**0**	**1.6**	**0.8**	**0**	**1.8**	**0.9**	**0**	**2**	**1**
PC6600	**6x6**	38	33	33	32	24.6		24	16.2		17											
PC6300	**3 ply x 6**	56	33.6	33.6	42	23.4	23.4	29	16.8	16.8	19	10.8		13	5.4							
PC6400	**4 ply x 6**	75	45	45	58	31.8	31.8	42	22.8	22.8	29	16.8		20	12		15					
PC8300	**3 ply x 8**	80	63	63	70	48	48	60	36	36	41	27.6	27.6	34	21.6	21.6	26	15	15	19		
PC8400	**4 ply x 8**	100	63	63	90	63	63	80	48	48	64	36	36	49	29.4	29.4	38	23.4	23.4	29	9	
Ohio Timberland Nail-Lam "Plus"																						
PC6300	**3 ply x 6**	57	35	35	41.5	20.5	20.5	27.5	12.8	12.8	20.3	9.3		12	5.4							
PC6400	**4 ply x 6**	78.8	49	49	58.6	29	29	40	18.6	18.6	27.2	12.4		18.7	8.4		13.8					
PC8300	**3 ply x 8**	84	72.6	72.6	79	54.5	54.5	63.9	34.6	36.6	43.3	23.6	23.6	35.4	16.4	16.4	27.2	12.4	12.4	20		
PC8400	**4 ply x 8**	114	98.6	98.6	104	71	71	89	48.7	48.7	69.2	33.8	33.8	51.5	24	24	39.6	18	18	30.4	9	

Comparison to typical pressure treated wood columns	Column Height (ft) 8			10			12			14			16			18			20		
Eave Condition	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**
Eave Deflection (in)	**0**	**0.8**	**0.4**	**0**	**1**	**0.5**	**0**	**1.2**	**0.6**	**0**	**1.4**	**0.7**	**0**	**1.6**	**0.8**	**0**	**1.8**	**0.9**	**0**	**2**	**1**
6x6 #2 trd SYP	21	21	21	16	10.8	10.8	11														
3 ply x 6 trd. #1 SYP*	41	30	30	38	20.4	20.4	19	12	12	13	5.7		8								

* Non-spliced

#2 SPF		Column Height (ft) 8			10			12			14			16			18			20		
	Eave Condition	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**
	Eave Deflection (in)	**0**	**0.8**	**0.4**	**0**	**1**	**0.5**	**0**	**1.2**	**0.6**	**0**	**1.4**	**0.7**	**0**	**1.6**	**0.8**	**0**	**1.8**	**0.9**	**0**	**2**	**1**
PC6300	**3 ply x 6**	48	26.4	26.4	27	17.4	17.4	18	8.4	8.4	11	1.5										
PC6400	**4 ply x 6**	49	33	33	38	24.6	24.6	27	16.8	16.8	18	7.8		11	1.8							

Chart Assumptions:

- All members and connections designed using Load and Resistance Factor Design (LRFD) with P-delta analysis to account for forces induced by deflection
- Constant wind load of 120 pounds per lineal foot on each post based on 90 mph wind speed
- All posts pin supported at top to simulate resistance from diaphragm action
- Eave Condition I allows no horizontal movement at eave, Condition II allows L/120, and Condition III allows L/240 horizontal movement.
- Maximum deflection limit under service loads of L/120, actual deflections based on larger of sidesway or curvature
- Effective length factor, Ke, is 0.8 for Condition I, and 1.2 for Conditions II and III
- Non-constrained post foundation with 4'-0" embedment depth
- Full lateral bracing and major axis bending only; no loads acting on weak axis
- Dry use for laminated wood portion in Perma column assembly
- No splices in laminated wood portion
- Exterior sidewall post with lateral loading from wind only
- Laminated wood portion transfers axial loads through direct bearing on steel seat plate
- Blank in chart represents deflection controls design, gray box indicates wood connection at steel bracket controls
- Final column design should include a complete building analysis by a Design Professional

The notes at the bottom of the chart describe the assumptions and conditions to which these maximum vertical loads apply. This chart assumes columns with full lateral bracing and only major axis bending. It is also important to note that the structural analogs used to create these charts have a support at the top of the post to simulate resistance to horizontal loads due to diaphragm action. The ***Perma-Column*** assemblies are not designed for “flagpole” situations where no support at the top of the posts can be expected. Additional wind bracing or kneebraces may need to be added to the overall building design if no diaphragm resistance is present. This is especially important to keep in mind when using the PC6600 as a replacement post. The overall building design should be evaluated to verify that the replacement post is adequate.

The chart shows that the main controlling factors in the design are the imposed deflection limits, and the strength of the wood portion of the column. The bracket connection to the wood post controls the design for the 20’ high, condition 2 only**. A comparison to a 6x6 #2 treated column, and a 3 ply 2x6 #1 treated non-spliced column using the same wind load and same boundary conditions is shown. The PC6300 performs significantly better than its 3 ply 2x6 treated counterpart mainly because it has no wet service reduction, and the maximum bending moment is resisted by the concrete component below grade.** Updating this chart with an effective buckling length factor, Ke, of 1.0 for conditions II and III may better represent the actual behavior of the columns in the field, and would give better performance overall.

8. Design Example

This design example is for a PC8300 with a 3 ply 2x6 #1 Southern Pine laminated wood column. The column is 16’ high and the eave is allowed to deflect horizontally 1.6” (L/120). The vertical load is 2.8 kips dead load, and 8.4 kips snow load. The horizontal loading is 120 pounds per foot due to wind load. All assumptions listed in the chart apply to this example, as does the structural analog with a spring shown in Figure 6.1. This is a summary of the design process; the detailed calculations are available in the Appendices to this document.

8.1 The controlling load combinations for the given dead, snow, and wind loading are as follows

1) D + W (Service loads for deflection check)
2) 1.2D + 1.6S + 0.8W
3) 1.2D + 0.5S + 1.6W
4) 0.9D + 1.6W

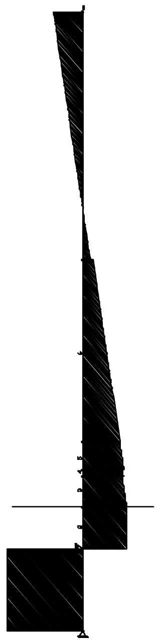
8.2 The column is analyzed for the given loading and the failure modes checked as outlined in Section 7 above.

8.2.1 Deflection due to service loads
Actual deflections are within allowable of 16(12)/120 = 1.6 OK

8.2.2 The factored internal forces in the wood elements are M_{ux} = 25 inch-kips and Pu = 16.8 kips for load combination 2, and M_{ux} = 60 inch-kips and Pu = 6.5 kips for load combination 3.

8.2.2.1 The interaction value in the combined axial force and bending moment check is .96 for load combination 2, and 0.49 for load combination 3. 0.91 < 1.0 OK

8.2.2.2 The design shear strength of a 3 ply 2x8 SYP member is 8.5 kips. The factored shear is 1.5 kips. OK



Shear Diagram
(Comb 3)

Moment Diagram
(Comb 3)

Figure 8.1 Shear and Moment diagram for PC8300, 16’ high with 1.6” maximum deflection under load combination 3

8.2.3 Steel Bracket Element

8.2.3.1 The maximum factored bending moment at the bottom of the steel bracket is 37.7 inch-kips compared to the chosen allowable moment of 74.34 inch-kips from Table B.2 in the Manual OK

8.2.4 Bracket-to-Wood Connection Element

8.2.4.1 The combined shear and bending moment on the connection produce an equal and opposite force on the top and bottom fastener groups. The factored shear is 1.5 kips, and the average factored moment is 28.85 inch-kips. These combine to produce a resultant load of 4.2 kips on each fastener group assuming a distance of 11 inches between the centroid of each group. The maximum allowed connection force due to factored loads is 4.7 kips. OK

8.2.5 Concrete Elements

8.2.5.1 The maximum factored bending moment below grade under load combination 3 is 92 inch-kips along with a factored axial force of 10 kips. These are well within the allowable envelope for the PC8300 when compared to the Interaction Diagram in Figure 3.3.2 in the Manual. OK

8.2.5.2 The minimum design shear strength of the PC8300 as given in Table 3.4.1 of the Manual is 4.5 kips. The factored shear in this example problem is 3.1 kips. OK

This column is adequate for the design loading.

9. Wind Uplift Capacity

Figure 9.1 shows two foundation conditions that may be used with a ***Perma-Column***. The wind uplift capacity can be evaluated for each foundation condition using the procedure described in *ANSI/ASAE EP486.1 October 2000 Shallow Post Foundation Design*. The uplift calculations in this section follow the allowable stress design equations of *EP486.1*, and therefore are unfactored capacities. They should be compared with unfactored net uplift values to determine adequacy for a particular situation. Upward movement of a ***Perma-Column*** post foundation cannot occur without displacing a cone of soil as defined below.

For circular footings and collars:
Circular cast-in place concrete collars displace a conically shaped wedge of soil. The potential resistance of a circular collar, including soil and attached concrete, can be calculated from the following equation:

$$U = aG[0.33p\{[(d-t)+0.5w/\tan q]^3(\tan q)^2 - 0.125w^3/\tan q\} - A_p(d-t)] + 0.25Cpw^2tG$$

Source: ANSI/ASAE EP486.1 OCT00: Shallow Post Foundation Design

where:
U = soil and foundation uplift resistance, kN (lbf)
α = soil density, kg/m^3 (85 lb/ft^3)
C = presumed concrete density, 90 kg/m^3 (150 lb/ft^3)
G = gravitational constant, 1 lbf/lbm (9.81N/kg)
d = embedment depth, m (4 ft)
t = collar thickness, m (1 ft)
w = collar width, m (ft)

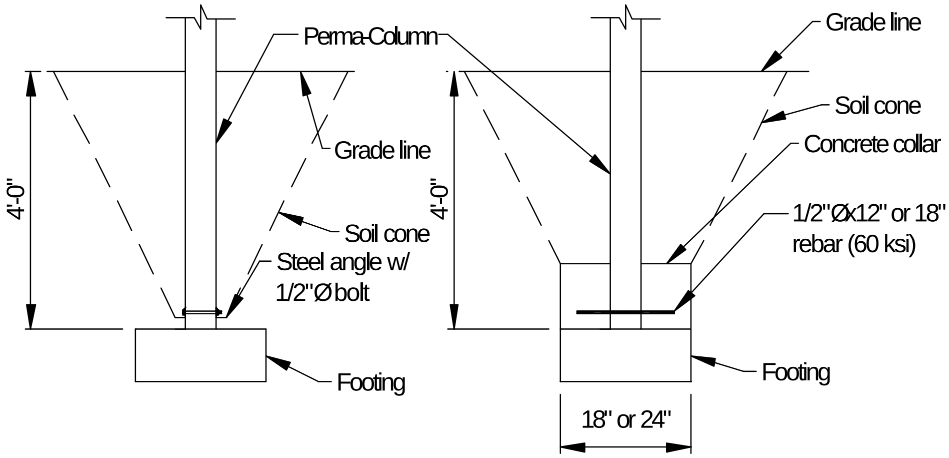
For rectangular footings and collars:

Angle plates are fastened to the post displacing a round corner, truncated prismatic wedge of soil radiating above the angle plates. The uplift resistance from the mass of the truncated prismatic volume is calculated by the following equation:

$$U = aG[(wl - A_p)(d-t) + (w+l)(d-t)^2\tan q + 0.33p(d-t)^2\tan^2 q]$$

Source: ANSI/ASAE EP486.1 OCT00: Shallow Post Foundation Design

where:
U = soil uplift resistance, kN (lbf)
α = soil density, kg/m^3 (lb/ft^3)
G = gravitational constant, 1 lbf/lbm (9.81N/kg)
d = embedment depth, m (ft)
t = steel collar thickness, m (ft)
w = width of collar, m (ft)
l = length of collar, m (ft)
A_p = post cross sectional area, m^2 (ft^2)
θ = soil friction angle, 26 deg



Standard Design   Alternative Design

Figure 9.1 Foundation Details

Table 9.1 shows the wind uplift capacity in pounds for these foundation conditions:

1. 18” diameter concrete collar with ½”x12” reinforcing bar through ***Perma-Column***
2. 24” diameter concrete collar with ½”x18” reinforcing through ***Perma-Column***
3. 2x2x8 ½ x 0.134” galvanized steel anchor with packed fill around posts
4. 2x2x12 x 0.134” galvanized steel anchor with packed fill around posts

Table 9.1 Allowable Unfactored Uplift*

PermaColumn	**Concrete Collar**		**Uplift Angle**	
	18”	**24”**	**2x2x8 ½**	**2x2x12**
PC6300	2272	3253	1908	2693
PC6400	2257	3238	2004	2789
PC6600	2262	3243	1972	2757
PC8300	2255	3236	2005	2790
PC8400	2236	3217	2102	2887

* Units are in pounds (lb)

The values in the chart are all limited by the weight of the soil cone. The shear strength of a 1/2” Grade 2 bolt (ASTM A307 bolt) is 10.0 ksi as published by the *AISC Ninth Edition ASD Construction Manual Table J3.2*. A ½” bolt has a cross sectional area of 0.196 in^2, thus a Grade 2 bolt in double shear will resist 3.92 kips (3920 pounds). The uplift angles are analyzed as a cantilever with a unit load at the midspan. The maximum uplift is calculated by the equation: $P_{allow} = (S_xF_b)/(L/2)$ (See calculation and Fig 9.2 below).

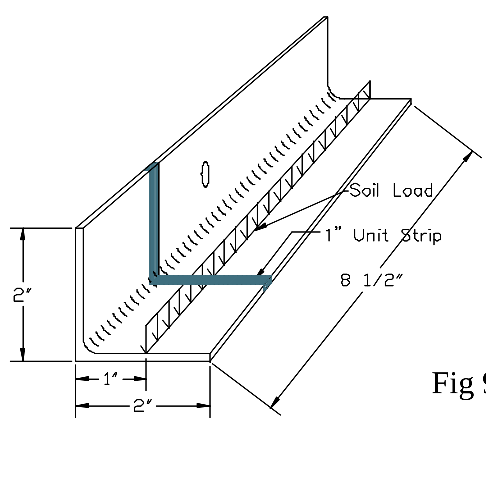
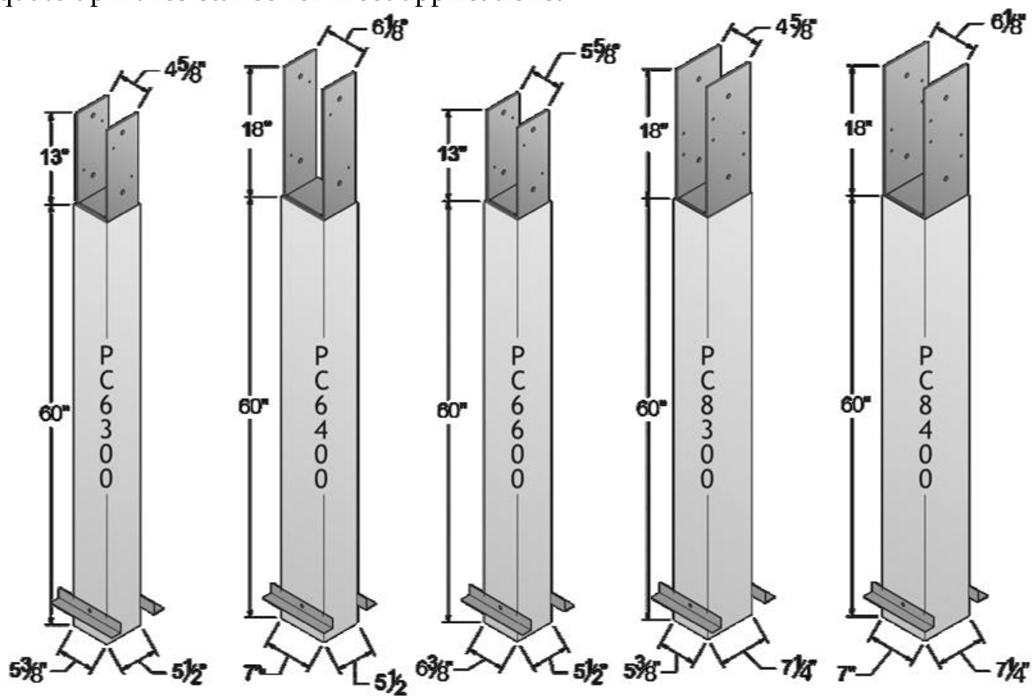


Fig 9.2

10. Summary and Conclusion

Perma-Column assemblies consist of wood, steel, and reinforced concrete elements, and should be designed using LRFD. The PC6600 is not included in the Manual, but can be expected to perform between the PC6300 and the PC6400 models. New technologies with Self Compacting Concrete (SCC) make it possible to manufacture a high quality product through the use of superplasticizers and a low water to cement ratio. The 10,000 psi (nominal) compressive strength protects the reinforcing bars by limiting chips and cracks during handling, and also by reducing the effect of freeze-thaw cycles. Steel bracket can be designed as a moment connection if the structural analog accurately models the rotational stiffness from the laboratory testing for the concrete-to-steel and steel-to-wood connections. The wood portion of the ***Perma-Column*** assemblies can be any grade or species of lumber; however, this Guide only deals with #1 SYP, the #1 SYP Nail-Lam "Plus" column as manufactured by Ohio Timberland Products, Inc, and #2 SPF. The effective length factor, Ke, for buckling was conservatively taken as 1.2 for columns fixed at the base, with horizontal movement allowed at the top (conditions II and III); and 0.8 for columns pinned at the top (condition I). Using an effective buckling length factor, Ke, of 1.0 for conditions II and III may better represent the actual behavior of the columns in the field, and would give better performance overall.

Each ***Perma-Column*** component can be modeled using a structural analog with properties corresponding to the results of the laboratory testing, and can be used to simulate the ***Perma-Column*** behavior under many other load conditions. The design charts in this Guide show that the ***Perma-Column*** assemblies are limited primarily by overall deflection, and by strength of the laminated wood members. The steel bracket connection to the wood component may be a limiting factor for tall columns under high wind loads. **The Perma-Column assemblies perform significantly better than typical mechanically laminated wood columns under the same boundary conditions mainly because they have no wet service reduction, and the maximum bending moment is resisted by the concrete component below grade. The steel bracket and the reinforced concrete base did not control under the design conditions presented in this Guide.** There are several foundation detail options including concrete collars and steel uplift angles that can be used with a ***Perma-Column*** to achieve adequate uplift resistance for most applications.



"The Permanent Solution"

PERMA COLUMN POST SIZE CHART

TO BE USED FOR ESTIMATING PURPOSES ONLY — ENGINEER OF RECORD REQUIRED

CONDITIONS: DL INCR 1.2, LL INCR 1.6, 90 MPH WL, BLDG LENGTH NOT TO EXCEED 2 1/2 TIMES WIDTH, 4' 0" POST DEPTH

POST SPACE FT.	BLDG WIDTH FT.	TTL RF DL	TTL RF LL		TOTAL ROOF LOAD 35	10' UNDER TRUSS	12' UNDER TRUSS	14' UNDER TRUSS	16' UNDER TRUSS	18' UNDER TRUSS	20' UNDER TRUSS
8	24	7	28	=	5.11 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	26	7	28	=	5.53 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	28	7	28	=	5.96 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	30	7	28	=	6.38 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	32	7	28	=	6.81 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	34	7	28	=	7.24 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	36	7	28	=	7.66 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	38	7	28	=	8.09 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	40	7	28	=	8.51 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	42	7	28	=	8.94 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	44	7	28	=	9.36 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	46	7	28	=	9.79 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	48	7	28	=	10.21 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	50	7	28	=	10.64 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	52	7	28	=	11.07 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	54	7	28	=	11.49 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	56	7	28	=	11.92 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	58	7	28	=	12.34 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	60	7	28	=	12.77 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	62	7	28	=	13.19 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	64	7	28	=	13.62 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	66	7	28	=	14.04 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	68	7	28	=	14.47 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	70	7	28	=	14.90 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	72	7	28	=	15.32 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	74	7	28	=	15.75 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	76	7	28	=	16.17 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	78	7	28	=	16.60 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	80	7	28	=	17.02 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	82	7	28	=	17.45 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	84	7	28	=	17.88 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	86	7	28	=	18.30 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	88	7	28	=	18.73 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	90	7	28	=	19.15 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	92	7	28	=	19.58 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	94	7	28	=	20.00 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	96	7	28	=	20.43 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	98	7	28	=	20.85 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'

3 PLY X 6 # 1 SYP = 4 PLY X 6 # 2 SPF MAY ALSO BE UTILIZED

August 25, 2005

PERMA COLUMN POST SIZE CHART

TO BE USED FOR ESTIMATING PURPOSES ONLY ENGINEER OF RECORD REQUIRED

CONDITIONS: DL INCR 1.2, LL INCR 1.6, 90 MPH WL, BLDG LENGTH NOT TO EXCEED 2 1/2 TIMES WIDTH, 4' 0" POST DEPTH

POST SPACE FT.	BLDG WIDTH FT.	TTL RF DL	TTL RF LL		TOTAL ROOF LOAD 40	10' UNDER TRUSS	12' UNDER TRUSS	14' UNDER TRUSS	16' UNDER TRUSS	18' UNDER TRUSS	20' UNDER TRUSS
8	24	10	30	=	5.76 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	26	10	30	=	6.24 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	28	10	30	=	6.72 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	30	10	30	=	7.20 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	32	10	30	=	7.68 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	34	10	30	=	8.16 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	36	10	30	=	8.64 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	38	10	30	=	9.12 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	40	10	30	=	9.60 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	42	10	30	=	10.08 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	44	10	30	=	10.56 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	46	10	30	=	11.04 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	48	10	30	=	11.52 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	50	10	30	=	12.00 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	52	10	30	=	12.48 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	54	10	30	=	12.96 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	56	10	30	=	13.44 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	58	10	30	=	13.92 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	60	10	30	=	14.40 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	62	10	30	=	14.88 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	64	10	30	=	15.36 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	66	10	30	=	15.84 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	68	10	30	=	16.32 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	70	10	30	=	16.80 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	72	10	30	=	17.28 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	74	10	30	=	17.76 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	76	10	30	=	18.24 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	78	10	30	=	18.72 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	80	10	30	=	19.20 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	82	10	30	=	19.68 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	84	10	30	=	20.16 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	86	10	30	=	20.64 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	88	10	30	=	21.12 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	90	10	30	=	21.60 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	92	10	30	=	22.08 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	94	10	30	=	22.56 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	96	10	30	=	23.04 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	98	10	30	=	23.52 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'

3 PLY X 6 # 1 SYP = 4 PLY X 6 # 2 SPF MAY ALSO BE UTILIZED

August 25, 2005

PERMA COLUMN POST SIZE CHART

TO BE USED FOR ESTIMATING PURPOSES ONLY   ENGINEER OF RECORD REQUIRED

CONDITIONS: DL INCR 1.2, LL INCR 1.6, 90 MPH WL, BLDG LENGTH NOT TO EXCEED 2 1/2 TIMES WIDTH, 4' 0" POST DEPTH

POST SPACE FT.	BLDG WIDTH FT.	TTL RF DL	TTL RF LL		TOTAL ROOF LOAD 50	10' UNDER TRUSS	12' UNDER TRUSS	14' UNDER TRUSS	16' UNDER TRUSS	18' UNDER TRUSS	20' UNDER TRUSS
8	24	10	40	=	7.30 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	26	10	40	=	7.90 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	28	10	40	=	8.51 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SP 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	30	10	40	=	9.12 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	32	10	40	=	9.73 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	34	10	40	=	10.34 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	36	10	40	=	10.94 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 2 SP 14'	3 PLY X 6 # 1 SYP 16'	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	38	10	40	=	11.55 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	40	10	40	=	12.16 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	42	10	40	=	12.77 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	3 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	44	10	40	=	13.38 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	46	10	40	=	13.98 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	48	10	40	=	14.59 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	4 PLY X 6 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	50	10	40	=	15.20 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	52	10	40	=	15.81 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	54	10	40	=	16.42 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	56	10	40	=	17.02 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	58	10	40	=	17.63 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 2 SPF 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	60	10	40	=	18.24 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	62	10	40	=	18.85 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	3 PLY X 8 # 1 SYP 20'
8	64	10	40	=	19.46 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 16'.	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	66	10	40	=	20.06 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	68	10	40	=	20.67 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	70	10	40	=	21.28 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	72	10	40	=	21.89 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	74	10	40	=	22.50 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	76	10	40	=	23.10 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	78	10	40	=	23.71 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	80	10	40	=	24.32 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	82	10	40	=	24.93 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	84	10	40	=	25.54 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 18'	4 PLY X 8 # 1 SYP 20'
8	86	10	40	=	26.14 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	4 PLY X 8 # 1 SYP 20'	4 PLY X 8 # 1 SYP 20'
8	88	10	40	=	26.75 KIPS	3 PLY X 6 # 2 SPF 10'	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	4 PLY X 8 # 1 SYP 20'	4 PLY X 8 # 1 SYP 20'
8	90	10	40	=	27.36 KIPS	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	4 PLY X 8 # 1 SYP 20'	4 PLY X 8 # 1 SYP 20'
8	92	10	40	=	27.97 KIPS	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	4 PLY X 8 # 1 SYP 20'	4 PLY X 8 # 1 SYP 20'
8	94	10	40	=	28.58 KIPS	3 PLY X 6 # 1 SYP 12'	3 PLY X 6 # 1 SYP 14'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	4 PLY X 8 # 1 SYP 20'	4 PLY X 8 # 1 SYP 20'
8	96	10	40	=	29.18 KIPS	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 16'	4 PLY X 8 # 1 SYP 20'	
8	98	10	40	=	29.79 KIPS	3 PLY X 6 # 1 SYP 12'	4 PLY X 6 # 1 SYP 14'	3 PLY X 8 # 1 SYP 16'	3 PLY X 8 # 1 SYP 16'	4 PLY X 8 # 1 SYP 20'	

3 PLY X 6 # 1 SYP = 4 PLY X 6 # 2 SPF MAY ALSO BE UTILIZED

August 25, 2005

Engineering Design Manual
for
Series 6300, 6400, 8300, 8400 Perma-Columns

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Ver101404

Table of Contents

Abstract

Elements of a ***Perma-Column***, like elements of any structural component or system, must be checked to ensure that loads applied to the building do not overload the column. In the case of a ***Perma-Column***, three items must be checked for structural adequacy: the reinforced concrete portion, the steel bracket, and the connection between the steel bracket and a wood post. In the following document, design values for the reinforced concrete portion and steel bracket are presented.

Disclaimer

Engineers using the ***Perma-Columns*** design values presented in this document must make sure that they are applicable to the condition in question. This document does not address load cases involving tension, bi-axial bending, torsion, or magnified moments as defined in ACI 318.

1. Perma Column Dimensions and Material Properties

Perma-column dimensions are graphical defined in Figure 1.1 and numerically compiled in Table 1.1. All dimensions identified in figure 1.1 except ***s5*** remain fixed along the entire length of a Perma-column. This change does not impact axial, shear, or strong axis bending properties.

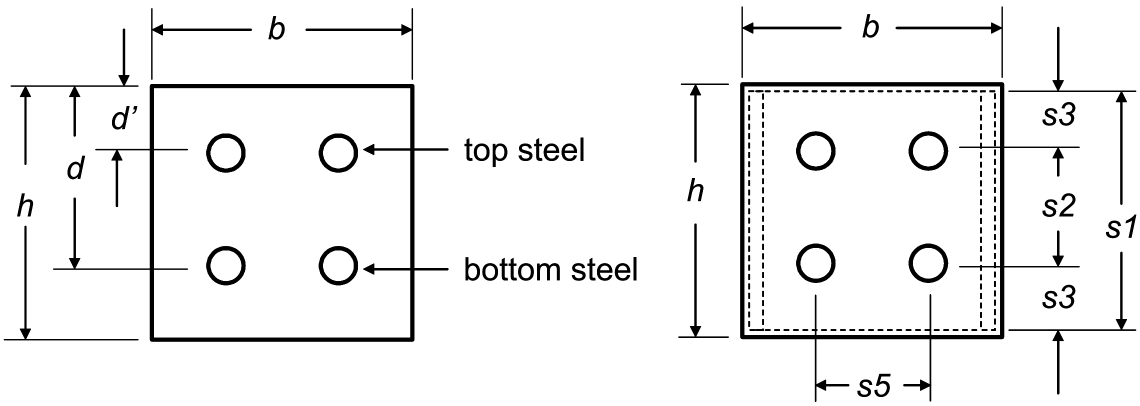

Figure 1.1. Variable definitions for Perma-columns.

Table 1.1: Perma Column Cross-Sectional Dimensions and Material Properties

Variable	Symbol	Units	PC6300	PC6400	PC8300	PC8400
Overall Concrete Width	b	in.	5.38	6.88	5.38	6.88
Overall Concrete Depth	h	in.	5.44	5.44	7.19	7.19
Depth to Top Steel	d'	in.	1.50	1.50	1.56	1.56
Depth to Bottom Steel	d	in.	3.94	3.94	5.62	5.62
Width of Steel Bracket	$s1$	in.	5.00	5.00	7.00	7.00
Top & Bottom Steel Spacing	$s2$	in.	2.44	2.44	4.06	4.06
Steel Distance to Bracket Edge	$s3$	in.	1.28	1.28	1.47	1.47
Area of Top Steel	A_s'	$in.^2$	0.40	0.40	0.62	0.62
Area of Bottom Steel	A_s	$in.^2$	0.40	0.40	0.62	0.62
Steel Yield Strength	f_y	$lbf/in.^2$	60000	60000	60000	60000
Concrete Compressive Strength	f_c'	$lbf/in.^2$	10000	10000	10000	10000
Steel Modulus of Elasticity	E_s	$lbf/in.^2$	29000000	29000000	29000000	29000000

American Concrete Institute (ACI) specifications for concrete protection of steel reinforcement require that where precast concrete components will be exposed to earth or weather, a minimum concrete cover of 1.25 inches is required on all steel reinforcement (ACI 318 Section 7.7.2). Nominal concrete cover on steel in all ***Perma-Columns*** is 1.25 inches.

2. Design Overview

One of the primary tasks of the design engineer is to ensure that structural components are not overloaded. This is a fairly systematic process that involves three major steps: establishment of design loads, structural analysis and component selection.

During the first step – establishment of design loads – the engineer must estimate the actual maximum loads (a.k.a. extreme loads) that could be applied to the structure during its design life. These estimated loads are more generally known as *nominal loads*, but may also be referred to as *service loads*, *working loads* or *unfactored loads*. Common load categories include: dead, earthquake, fluid, soil, live, roof live, rain, snow, self-straining and wind. Note that in addition to estimating these loads, an engineer must select the various combination(s) of these loads that will be applied to the structure during the structural analysis phase.

Structural analysis, which is the second step in a design process, involves determining the forces induced in structural components when loads are applied to the structure. The simplicity of many structures enables engineers to limit most analyses to what is occurring within a given plane of the structure. Referred to as a two-dimensional analysis, such an investigation typically provides the engineer with the axial force, bending moment and shear force at every point of each component lying within the plane of interest.

The third and final step in the design process is to check that the axial force, bending moment and shear forces induced in each component do not exceed allowable values. Where the allowable strength of a component is exceeded, the component must be replaced with one that is more substantial, or other changes must be made to the structure to reduce the forces induced in the component. Regardless of which route is taken, the structure must typically be reanalyzed once changes are made to one or more components.

Critically important in the design process is to ensure that there is a sufficient margin of safety built into the design. Exactly how safety is "built" into the design process depends on the overall design philosophy utilized (strength design versus allowable stress design) as discussed in the following section.

During the design process a ***Perma-Column*** would be analyzed as three separate components: a reinforced concrete base, a steel attaching bracket, and a steel-to-wood post connection system. Because each of these three elements involves a different combination of materials, three different design specifications actually apply. Specifically, American Concrete Institute (ACI) specifications control the design on the reinforced concrete base, both ACI and American Institute of Steel Construction (AISC) apply to the steel bracket design, and American Wood and Paper Association (AWPA) specifications apply to the steel-to-wood post connection system.

2.1 Design Philosophies: Strength Design vs. Allowable Stress Design

Two fundamentally different design procedures are available to determine if a structural component is strong enough to withstand the loads to which it will be subjected: allowable stress design and strength design. As previously mentioned, these two design philosophies primarily differ in how they account for uncertainties in design. In other words, they differ in how they include a factor of safety in structural calculations.

From a safety perspective, it is important to understand that many uncertainties surround structural design. For example, in practice the magnitude of extreme loads may vary from predicted values and simplifying assumptions made during structural analyses may be highly inaccurate. Additionally, the same analyses may have ignored complex and/or critical interactions between components/systems as well as critical loads and/or load combinations to which the structure is subjected. Also, actual material strengths and dimensions of components used in construction may differ measurably from those assumed during design, and construction oversights may have resulted in critical components being omitted and/or incorrectly installed.

In *allowable stress design* (ASD), stresses induced in members by nominal (a.k.a. unfactored) loads must not exceed published allowable stresses for the component in question. Safety is accounted for by publishing allowable stress values that are a fraction of the stresses that would result in a failure of the component. In *strength design*, nominal loads are increased by load factors, and the forces induced in structural components by these factored loads can not exceed published ultimate component strengths that have been reduced by resistance factors. It follows that safety in strength design is accounted for in the load factors and in the resistance factors. To this end, strength design is also referred to as *load and resistance factor design* (LRFD).

Up until the later 1950's, all wood, steel, and reinforced concrete components were designed using ASD. Traditionally, this design philosophy was referred to as *working stress design* by the American Concrete Institute (ACI). During the late 1950's, ACI introduced the strength design method as an alternative to their working stress design. During the 1980's, the steel industry followed the concrete industry and developed a strength design methodology which was referred to as *LRFD for steel construction*. The wood industry followed suit, developing an LRFD or strength design procedure for wood construction in the mid 1990s. However, unlike the concrete and steel industries, the wood industry has been relatively slow to embrace LRFD; that is, the size of virtually all wood members is still determined used ASD procedures.

2.2 Governing Equations

The three main equations for strength design can be written as:

$$P_u \leq \phi P_n \qquad (2.2.1)$$

$$M_u \leq \phi M_n \qquad (2.2.2)$$

$$V_u \leq \phi V_n \qquad (2.2.3)$$

where:

P_u	=	Required axial force (axial force due to factored loads)
M_u	=	Required bending moment (bending moment due to factored loads)
V_u	=	Required shear force (shear force due to factored loads)
P_n	=	Nominal axial strength
M_n	=	Nominal moment strength
V_n	=	Nominal shear strength
ϕ	=	Resistance factor
ϕP_n	=	Design (or useable) axial strength
ϕM_n	=	Design (or useable) moment strength
ϕV_n	=	Design (or useable) shear strength

Strength design is recommended when checking the adequacy of the reinforced concrete section and steel attachment bracket of a ***Perma-Column***. Note that in order to do this, design strength values ϕP_n, ϕM_n, and ϕV_n must be established for the reinforced concrete section and the steel attachment bracket of each ***Perma-Column*** series. This is done in Sections 3 and 4, respectively.

2.2 Load Combinations and Load Factors

The resistance factors used in strength design depend on and/or dictate the load factors and corresponding load combinations used during structural analysis to obtain the required strength values (e.g., V_u, M_u, P_u). To use the resistance factors outlined in following sections for strength design will require use of the following ANSI/ASCE 7 load combinations.

$$1.4\cdot(\boldsymbol{D} + \boldsymbol{F}) \tag{2.2.1}$$

$$1.2\cdot(\boldsymbol{D} + \boldsymbol{F} + \boldsymbol{T}) + 1.6\cdot(\boldsymbol{L} + \boldsymbol{H}) + 0.5\cdot(\boldsymbol{L_r} \text{ or } \boldsymbol{S} \text{ or } \boldsymbol{R}) \tag{2.2.2}$$

$$1.2\cdot\boldsymbol{D} + 1.6\cdot(\boldsymbol{L_r} \text{ or } \boldsymbol{S} \text{ or } \boldsymbol{R}) + (\text{x}\cdot\boldsymbol{L} \text{ or } 0.8\cdot\boldsymbol{W}) \tag{2.2.3}$$

$$1.2\cdot\boldsymbol{D} + 1.6\cdot\boldsymbol{W} + \text{x}\cdot\boldsymbol{L} + 0.5\cdot(\boldsymbol{L_r} \text{ or } \boldsymbol{S} \text{ or } \boldsymbol{R}) \tag{2.2.4}$$

$$1.2\cdot\boldsymbol{D} + 1.0\cdot\boldsymbol{E} + \text{x}\cdot\boldsymbol{L} + 0.2\cdot\boldsymbol{S} \tag{2.2.5}$$

$$0.9\cdot\boldsymbol{D} + (1.0\cdot\boldsymbol{E} \text{ or } 1.6\cdot\boldsymbol{W}) + 1.6\boldsymbol{H} \tag{2.2.6}$$

$\boldsymbol{D}$ = Dead Load
$\boldsymbol{E}$ = Earthquake Load
$\boldsymbol{F}$ = Fluid Load
$\boldsymbol{H}$ = Soil Load
$\boldsymbol{L}$ = Live Load
$\boldsymbol{L_r}$ = Roof Live Load
$\boldsymbol{R}$ = Rain Load
$\boldsymbol{S}$ = Snow Load
$\boldsymbol{T}$ = Self-Straining Load
$\boldsymbol{W}$ = Wind Load
x = 1.0 for garages, areas of public occupancy, and values of $\boldsymbol{L}$ greater than 100 lbf/ft^2. When $\boldsymbol{L}$ is less than or equal to 100 lbf/ft^2, set x equal to 0.5.

3. Strength Properties of the Reinforced Concrete

Perma-Columns can be characterized as reinforced concrete members without lateral steel reinforcement. Lateral reinforcement functions as shear reinforcement when the column is subjected to bending loads, and as tie reinforcement when the column is subjected to axial compressive forces. Tie reinforcement is not a necessity in ***Perma-Columns*** because of the relatively low axial forces to which the columns are subjected. Such lateral reinforcement would also increase column size. Note that to meet ACI 318 Section 7.10 requirements for tie reinforcement requires a minimum No. 3 size bar spaced no further apart than least dimension of the column. To wrap a No.3 bar around longitudinal reinforcement and still meet ACI cover requirements would increase member width and thickness by 0.75 inches.

Design (or useable) strength values for the reinforced concrete portion of ***Perma-Columns*** are developed in the following sections. Design values are obtained by multiplying nominal strengths by the resistance factors in Table 3.0. These factors are from ACI 318 Appendix C and are only applicable when used in combination with the ASCE/ANSI 7 load factors and combinations in Section 2.2

Table 3.0 – Resistance Factors for *Perma-Column* Design

Application	ϕ Value
Flexure, without axial load	0.80
Flexure, with axial tension	0.80
Axial compression	0.55
Flexure with axial compression*	0.55 to 0.80
Axial tension	0.80
Shear and torsion	0.75

* For low values of axial compression, ϕ may be increased towards the value for flexure, 0.80 , according to equations in ACI 318. See Section 3.2.

A resistance factor of 0.55 is listed in Table 3.0 for axial compression and axial compression with flexure. This value is recommended because ***Perma-Columns*** do not contain lateral reinforcement. With spiral reinforcement, an axial compression resistance factor of 0.70 would be used. With tie reinforcement, the axial compression resistance factor would be 0.65. The selected resistance factor of 0.55 is the ACI 318 resistance factor for plain (i. e., non-reinforced) concrete. Note that if ***Perma-Columns*** were only subjected to concentrically applied axial loads, it is quite likely that they would not even need longitudinal reinforcement. This is because an upright compression member whose height does not exceed three times its least lateral dimension, is considered a pedestal under ACI 318 Section 22.8 and does not require steel reinforcement. According to the ACI Commentary (ACI 318 Section R22.8), the 3-to-1 limitation on height to thickness ratio does not apply for portions of pedestals embedded in soil capable of providing lateral restraint (Note: In application, ***Perma-Columns*** are unlikely to extend more than three times their thickness above grade).

According to ACI 318 Section 10.2, the strength design of members for flexure and axial loads shall be based on the following assumptions:

1. Member strength is based on satisfying applicable conditions of equilibrium and compatibility of strains.
2. Strain in reinforcement and concrete is directly proportional to the distance from the neutral axis.
3. Maximum usable strain ε_{cu} at extreme concrete compression fiber is equal to 0.003
4. Stress in reinforcement below specified yield strength f_y is equal to E_s times the steel strain.
5. Tensile strength of concrete shall be neglected in axial and flexural calculations.
6. The relationship between concrete compressive stress distribution and concrete strain when nominal strength is reached may be taken as an equivalent rectangular stress distribution, wherein a concrete stress intensity of 0.85 f_c' is assumed to be uniformly distributed over an equivalent compressive zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis is measured in a direction perpendicular to that axis. The value of β_1 is 0.85 for f_c' values less than or equal to 4000 psi and 0.65 for f_c' values greater than or equal to 8000 psi. Linearly interpolation is used to obtain β_1 for f_c' values between 4000 and 8000 psi.

3.1 Bending Strength Under Just Flexural Load (No Axially Applied Loads)

Based on the previous assumptions, the following equations can be written for conditions at failure (ε_{cu} = 0.003 and bottom steel yielding) of a ***Perma-Column*** subjected to flexure alone. Variables and designated values for each ***Perma-Column*** series are given in Tables 1.1 and 3.1:

$$\varepsilon_{s(top)} = 0.003\,(d' - c)/c \qquad (3.1.1)$$

$$C = \beta_1\, c\, b\, 0.85\, f_c' \qquad (3.1.2)$$

$$T_{(top)} = A_s'\, E_s\, \varepsilon_{s(top)} \quad \text{but no greater than } f_y A_s' \qquad (3.1.3)$$

$$T_{(bottom)} = A_s\, f_y \qquad (3.1.4)$$

$$C = T_{(top)} + T_{(bottom)} \qquad (3.1.5)$$

Equation 3.1.1 for strain in the top steel returns a negative value when the top steel is located above the neutral axis (i.e., $d' \leq c$). When this is the case, $T_{(top)}$ will have a negative value in all remaining calculations.

Substituting equation 3.1.1 into equation 3.1.3, and then substituting equations 3.1.2, 3.1.3 and 3.1.4 into equation 3.1.5 yields the following equation;

$$\beta_1\, c\, b\, 0.85\, f_c' = A_s'\, E_s\, 0.003\,(d' - c)/c + A_s\, f_y \qquad (3.1.6)$$

which can be rewritten as:

$$c^2[\beta_1\, b\, 0.85\, f_c'/(A_s'\, f_y)] + c\,[(0.003\, E_s/f_y) - A_s/A_s'] - 0.003\, d'\, E_s/f_y = 0 \qquad (3.1.7)$$

With the exception of the distance to the neutral axis c, all variables in equations 3.1.6 and 3.1.7 are known. Consequently, c can be determined directly using the quadratic equation. Once this is done, the nominal moment strength M_n is calculated using the following equation. Note that M_n = M_o when the member is subjected to flexure alone.

$$M_o = T_{(bottom)}\ (d - \beta_1 c/2) + T_{(top)}\ (d' - \beta_1 c/2) \qquad (3.1.8)$$

or

$$M_o = A_s\, f_y\ (d - \beta_1 c/2) + A_s'\, E_s\ 0.003\ (d' - c)(d'/c - \beta_1/2) \qquad (3.1.9)$$

Table 3.1. ***Perma-Column*** Flexural Strength Characteristics (Under Flexure Alone)

Variable	Symbol	Units	PC6300	PC6400	PC8300	PC8400
Stress Block Depth Factor	β_1		0.65	0.65	0.65	0.65
Distance to Neutral Axis	c	in.	1.156	1.038	1.425	1.284
Strain in Top Steel	$\varepsilon_{s(top)}$	in./in.	0.00089	0.00133	0.00028	0.00065
Strain in Bottom Steel	$\varepsilon_{s(bot)}$	in./in.	0.00722	0.00838	0.00828	0.01013
Conc. Compressive Force	C	lbf	34341	39471	42315	48802
Net Force in Bottom Steel	$T_{(bottom)}$	lbf	24000	24000	37200	37200
Net Force in Top Steel	$T_{(top)}$	lbf	10341	15471	5115	11602
Nominal Moment Strength (flexure alone)	M_o	lbf -in.	97200	104500	197400	206800
Design (Useable) Strength (flexure alone)*	ϕM_o	lbf -in.	77700	83600	158000	165400

* $\phi = 0.80$

Since in all cases, c is less than d', the top steel is not located in the compression region. In other words, all steel is tension steel when the nominal moment strength, M_o is reached. Table 3.1 values for strain in the top steel are all less than 0.00207 in./in., thus indicating that the top steel does not yield before a compressive strain (in the extreme concrete fiber in compression) of 0.003 in./in. is reached. If the top steel were within $\beta_1 c$ of the top of the beam, the area of concrete in compression would have to be reduced by the cross sectional area of top steel A_s'.

3.2 Axial Load Strength Under Zero Eccentricity (No Flexural Load)

Under typical installation ***Perma-Columns*** would be classified as short columns and the nominal axial strength under zero eccentricity would be calculated as:

$$P_o = 0.85\, f_c'\ (A_g - A_{st}) + f_y\, A_{st} \qquad (3.2.1)$$

Truly concentrically loaded columns (i.e., columns with zero eccentricity) are nonexistent as eccentricity will occur due to slight changes in end conditions, inaccuracy of manufacture, and variations in steel and concrete properties even when the load is theoretically concentric. To account for this eccentricity, ACI 318 Section 10.3.5 requires that the maximum nominal axial strength $P_{n(max)}$ not exceed 0.80 P_o for tied columns and 0.85 P_o for spiral reinforced columns. Because ***Perma-Columns*** do not contain any lateral reinforcement, it is recommended that the maximum nominal axial strength $P_{n(max)}$ be limited to 0.75 P_o, that is:

$$P_{n(max)} = 0.75\, P_o = 0.75\, [0.85\, f_c'\ (A_g - A_{st}) + f_y\, A_{st}] \qquad (3.2.2)$$

The 0.75 P_o limit on $P_{n(max)}$ is equivalent to an eccentricity of 0.120 h for a typical ***Perma-Column***. A 0.80 P_o limit on $P_{n(max)}$ would be equivalent to an eccentricity of around 0.093 h for a typical ***Perma-Column***. As a rule of thumb, it is good to assume an eccentricity of at least 0.1h when designing columns similar in size to ***Perma-Columns***.

$P_{n(max)}$ values for Perma-columns, as calculated using equation 3.2.2 are tabulated in Table 3.2

Table 3.2. Perma Column Axial Strength Characteristics (Under Pure Axial Loads)

Variable	Symbol	Units	PC6300	PC6400	PC8300	PC8400
Total Steel Area	A_{st}	sq inch	0.80	0.80	1.24	1.24
Gross Cross-Section Area	A_g	sq inch	29.23	37.41	38.63	49.45
Steel Yield Strength	f_y	lbf/in.^{2}	60000	60000	60000	60000
Concrete Comp. Strength	f_c'	lbf/in.^{2}	10000	10000	10000	10000
Nominal Axial Load Strength at Zero Eccentricity	P_o	Lbf	289600	359200	392200	484200
Maximum Nominal Axial Load Strength	$P_{n(max)}$	Lbf	217200	269400	294200	363100
Maximum Design (Useable) Axial Load Strength*	$\phi P_{n(max)}$	Lbf	119500	148200	161800	199700

* $\phi = 0.55$

3.3. Strength Under Combined Bending & Axial Compressive Loads

Seldom, if ever, will a Perma-Column be subjected to a pure axial or a pure bending load. Consequently, allowable axial load/bending moment interactions outlined in this section will typically apply.

The addition of an axial compressive load to a reinforced concrete component that is under a pure bending load will, up to a certain point, increase the amount of bending load to which the concrete member can be subjected (see Figure 3.3.1). This is not the case with a wood, steel, or plain concrete member. This phenomenon results from the fact that the bending strength of a reinforced concrete component under a pure bending load is limited by yielding of tension steel, and the addition of a compressive load: (1) reduces the tensile strain in the tension steel, and (2) increases the area of concrete in compression.

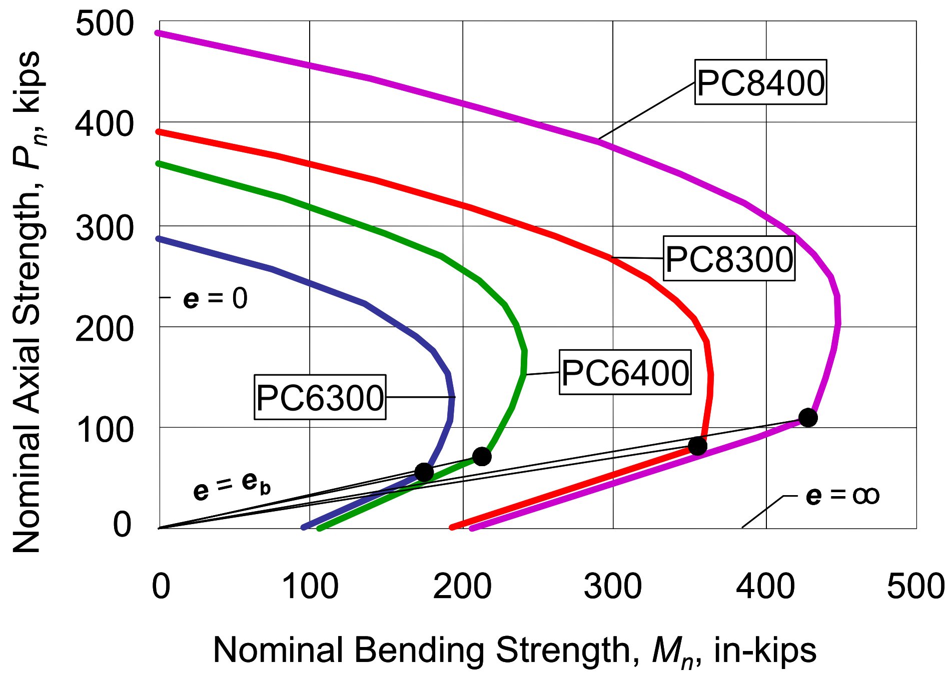

Figure 3.3.1. Strength interaction diagrams for axial compression and bending moment about the major axis of ***Perma-Columns***.

The dots on the diagram in Figure 3.3.1 represent the balanced strain condition ($P_n = P_b$, $M_n = M_b$) which is the point at which the tension steel just begins to yield when the maximum concrete strain just reaches 0.003. There is only one combination of P_b and M_b under which these two strain states can simultaneously exist. They can be calculated using the following equations.

$$c = 0.003\, d/(f_y/E_s + 0.003) \quad (3.3.1)$$

$$\varepsilon_{s(top)} = 0.003\,(d' - c)/c \quad (3.3.1)$$

$$C = \beta_1\, c\, b\, 0.85\, f_c' \quad (3.3.2)$$

$$T_{(top)} = A_s'\, E_s\, \varepsilon_{s(top)} \quad \text{but no greater than } f_y A_s' \quad (3.3.3)$$

$$T_{(bottom)} = A_s\, f_y \quad (3.3.4)$$

$$P_b = C - T_{(top)} - T_{(bottom)} \quad (3.3.5)$$

$$M_b = C\,(h - \beta_1 c)/2 - T_{(top)}\,(h/2 - d') + T_{(bottom)}\,(d - h/2) \quad (3.3.6)$$

M_b and P_b are the moment and axial force that produce the same internal affects as C, $T_{(top)}$, and $T_{(bottom)}$. When calculating M_b, axial force P_b is placed at the plastic centroid of the column. Because of their symmetry, the plastic centroid of ***Perma-Columns*** is at the geometric center of the members (i.e., at $h/2$).

All other points that make up the plots in Figure 3.3.1 were obtained in a fashion similar to that use to determine M_b and P_b. Specifically, a strain value was selected for the bottom steel, and the maximum strain in the concrete was fixed at 0.003. From these two values, the location of the neutral axis and strain in the top steel were calculated. Forces C, $T_{(top)}$, $T_{(bottom)}$ were then calculated and substituted in equations 3.3.5 and 3.3.6 to obtain P_n and M_n, respectively A few of these P_n-M_n interaction values have been compiled in Table 3.3.1. Radial lines extending from the origin in Figure 3.3.1 represent constant ratios of M_n to P_n, that is they represent eccentricities e of the load P_n from the plastic centroid of the columns. It follows, as shown in figure 3.3.1, that the vertical axis represent $e = 0$ and the horizontal axis represents $e = \infty$.

Table 3.3.1 Axial Compression and Bending Strength Interaction Values

Nominal Axial	Nominal Bending Strength, M_n, kips-inches			
Strength, P_n, kips	**PC6300**	**PC6400**	**PC8300**	**PC8400**
0	97.2	104.4	197.5	206.8
20	127.9	136.2	243.1	253.6
40	157.2	167.3	287.3	299.5
60	175.3	196.3	324.4	343.6
80	181.0	214.1	360.7	380.6
100	186.0	221.7	370.0	417.4
120	189.3	228.5	371.6	439.9
140	189.8	233.7	372.8	444.4
160	186.4	236.6	372.7	448.5
180	177.1	236.4	369.7	451.6
200	160.9	232.5	361.3	453.2
220	131.4	223.5	349.2	452.3
240	101.1	208.6	332.7	446.4
260	63.3	188.3	311.0	437.2
280	17.8	155.6	283.6	424.3
300		123.3	250.0	407.2
320		85.1	194.7	385.5
340		40.7	146.5	359.0
360			91.2	327.3
380			28.9	288.4
400				232.2
440				128.2
480				9.1

Design strength values are compiled in Table 3.3.2 and graphically displayed in Figure 3.3.2. These values were obtained by reducing nominal strength values in Table 3.3.1 by appropriate resistance factors. While a resistance factor of 0.80 is applicable to all bending values, the axial resistance factor of 0.55 can be increased to 0.80 (i.e., the resistance factor for bending) as ϕP_n decreases from $0.1 f_c' A_g$ to zero.

To check the adequacy of a design, first divide the moment due to factored load M_u, by the compression axial force due to factored load, P_u, to obtain the eccentricity due to factored loads. Next, find the eccentricity (in the far right column of Table 3.3.2) and associated $\phi_a P_n$ and $\phi_b M_n$ values that correspond to the calculated eccentricity. The design in question is adequate in compression and bending as long as the $\phi_a P_n$ value exceeds P_u and the $\phi_b M_n$ value exceeds M_u.

Table 3.3.2 Axial Compression and Bending Strength Design Values

Notable Points	P_n kips	ϕ_a	$\phi_a P_n$ kips	M_n kips-inch	$\phi_b M_n$ kips-inches	Eccentricity, e, inches
			PC6300			
	0.0	0.80	**0.0**	97.2	**77.8**	∞
	20.0	0.71	**14.1**	127.9	**102.3**	7.25
	40.0	0.61	**24.5**	157.2	**125.8**	5.14
$\phi_a P_n = 0.1 f_c' A_g$ ➔	53.1	0.55	**29.2**	172.1	**137.7**	4.71
Balanced Condition ➔	54.5	0.55	**30.0**	173.6	**138.9**	4.64
	60.0	0.55	**33.0**	175.3	**140.3**	4.25
	80.0	0.55	**44.0**	181.0	**144.8**	3.29
	100.0	0.55	**55.0**	186.0	**148.8**	2.71
	120.0	0.55	**66.0**	189.3	**151.4**	2.29
	140.0	0.55	**77.0**	189.8	**151.8**	1.97
	160.0	0.55	**88.0**	186.4	**149.1**	1.69
	180.0	0.55	**99.0**	177.1	**141.7**	1.43
	200.0	0.55	**110.0**	160.9	**128.7**	1.17
Max. Axial Strength ➔	217.2	0.55	**119.5**	135.5	**108.4**	0.91
			PC6400			
	0.0	0.80	**0.0**	104.4	**83.6**	∞
	20.0	0.73	**14.5**	136.2	**108.9**	7.50
	40.0	0.65	**26.1**	167.3	**133.8**	5.12
	60.0	0.58	**34.8**	196.3	**157.0**	4.52
$\phi_a P_n = 0.1 f_c' A_g$ ➔	68.0	0.55	**37.4**	205.1	**164.1**	4.39
Balanced Condition ➔	73.8	0.55	**40.6**	211.5	**169.2**	4.17
	80.0	0.55	**44.0**	214.1	**171.3**	3.89
	100.0	0.55	**55.0**	221.7	**177.3**	3.22
	120.0	0.55	**66.0**	228.5	**182.8**	2.77
	140.0	0.55	**77.0**	233.7	**187.0**	2.43
	160.0	0.55	**88.0**	236.6	**189.3**	2.15
	180.0	0.55	**99.0**	236.4	**189.2**	1.91
	200.0	0.55	**110.0**	232.5	**186.0**	1.69
	220.0	0.55	**121.0**	223.5	**178.8**	1.48
	240.0	0.55	**132.0**	208.6	**166.9**	1.26
	260.0	0.55	**143.0**	188.3	**150.7**	1.05
Max. Axial Strength ➔	269.4	0.55	**148.2**	173.0	**138.4**	0.93

Table 3.3.2 Axial Compression and Bending Strength Design Values, cont.

Notable Points	P_n kips	ϕ_a	$\phi_a P_n$ kips	M_n kips-inch	ϕ_b M_n kips-inches	Eccentricity, e, inches
			PC8300			
	0.0	0.80	**0.0**	197.5	**158.0**	
	20.0	0.73	**14.6**	243.1	**194.5**	13.34
	40.0	0.66	**26.3**	287.3	**229.8**	8.74
	60.0	0.59	**35.2**	324.4	**259.5**	7.37
$\phi_a P_n = 0.1 f_c' A_g$ ➔	70.2	0.55	**38.6**	342.9	**274.4**	7.10
	80.0	0.55	**44.0**	360.7	**288.5**	6.56
Balanced Condition ➔	85.3	0.55	**46.9**	368.5	**294.8**	6.29
	100.0	0.55	**55.0**	370.0	**296.0**	5.38
	120.0	0.55	**66.0**	371.6	**297.3**	4.50
	140.0	0.55	**77.0**	372.8	**298.2**	3.87
	160.0	0.55	**88.0**	372.7	**298.1**	3.39
	180.0	0.55	**99.0**	369.7	**295.7**	2.99
	200.0	0.55	**110.0**	361.3	**289.0**	2.63
	220.0	0.55	**121.0**	349.2	**279.4**	2.31
	240.0	0.55	**132.0**	332.7	**266.1**	2.02
	260.0	0.55	**143.0**	311.0	**248.8**	1.74
	280.0	0.55	**154.0**	283.6	**226.9**	1.47
Max. Axial Strength ➔	294.2	0.55	**161.8**	259.8	**207.8**	1.28
			PC8400			
	0.0	0.80	**0.0**	206.8	**165.4**	
	20.0	0.74	**14.9**	253.6	**202.9**	13.63
	40.0	0.69	**27.6**	299.5	**239.6**	8.70
	60.0	0.63	**38.0**	343.6	**274.9**	7.24
	80.0	0.58	**46.2**	380.6	**304.5**	6.59
$\phi_a P_n = 0.1 f_c' A_g$ ➔	89.9	0.55	**49.5**	398.8	**319.1**	6.45
	100.0	0.55	**55.0**	417.4	**333.9**	6.07
Balanced Condition ➔	112.8	0.55	**62.0**	437.8	**350.2**	5.64
	120.0	0.55	**66.0**	439.9	**351.9**	5.33
	140.0	0.55	**77.0**	444.4	**355.5**	4.62
	160.0	0.55	**88.0**	448.5	**358.8**	4.08
	180.0	0.55	**99.0**	451.6	**361.3**	3.65
	200.0	0.55	**110.0**	453.2	**362.6**	3.30
	220.0	0.55	**121.0**	452.3	**361.8**	2.99
	240.0	0.55	**132.0**	446.4	**357.1**	2.71
	260.0	0.55	**143.0**	437.2	**349.8**	2.45
	280.0	0.55	**154.0**	424.3	**339.5**	2.20
	300.0	0.55	**165.0**	407.2	**325.7**	1.97
	320.0	0.55	**176.0**	385.5	**308.4**	1.75
	340.0	0.55	**187.0**	359.0	**287.2**	1.54
	360.0	0.55	**198.0**	327.3	**261.8**	1.32
Max. Axial Strength ➔	363.1	0.55	**199.7**	321.2	**256.9**	1.29

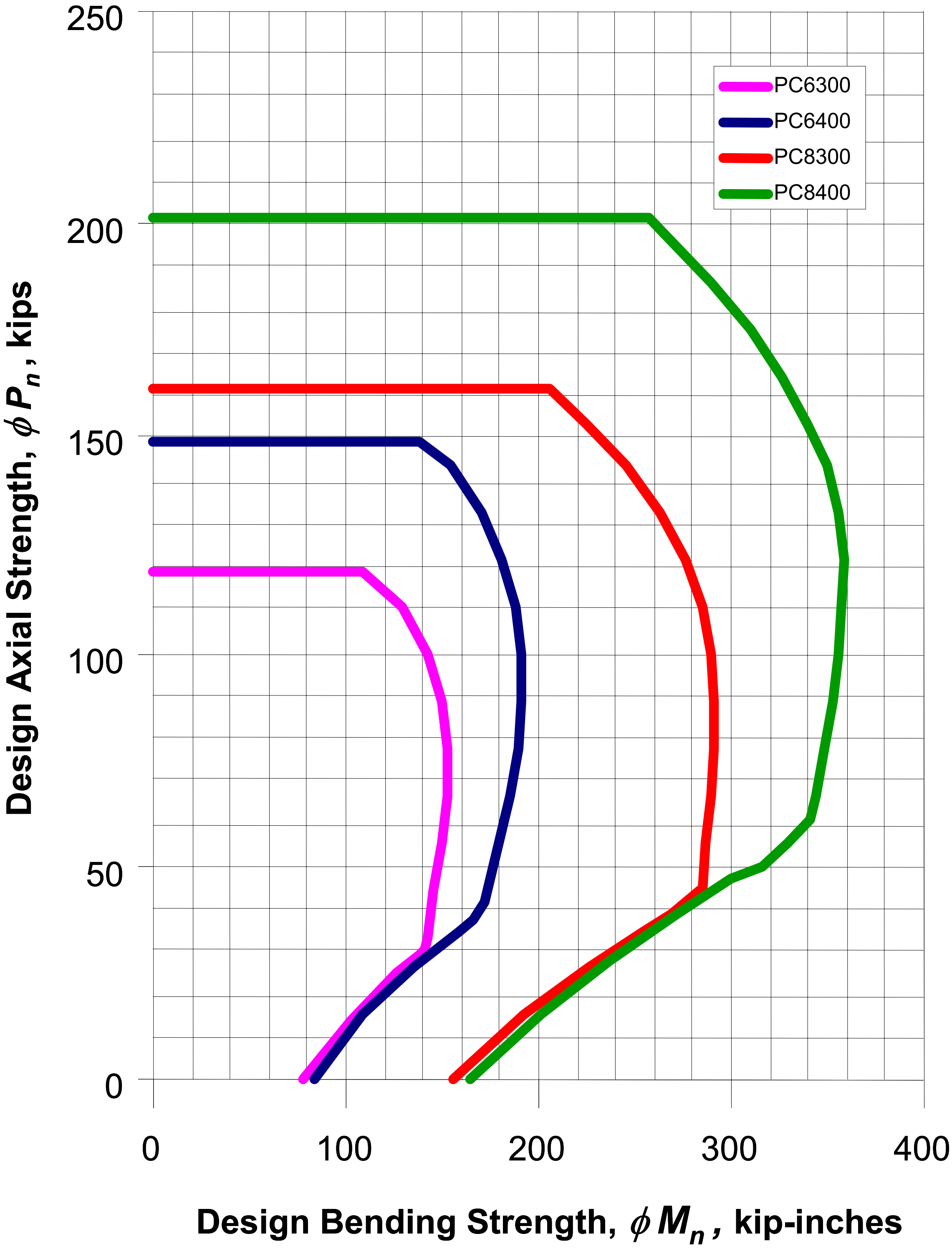

Figure 3.3.2. Design bending and axial strength interaction values for ***Perma-Columns***.

3.4 Shear Strength

The nominal shear strength of a reinforced concrete component, $\boldsymbol{V_n}$, is equal to the sum of the shear strength provided by the concrete, $\boldsymbol{V_c}$, and the shear strength provided by shear reinforcement, $\boldsymbol{V_s}$, that is, $\boldsymbol{V_n = V_c + V_s}$. Because they do not contain shear reinforcement, $\boldsymbol{V_n = V_c}$ for a ***Perma-Column***. It should be noted that ACI 318 Section 11.5.5.1 restricts $\boldsymbol{V_n}$ to $\boldsymbol{V_c/2}$ for most components that do not contain reinforcement. The increase from $\boldsymbol{V_n = V_c/2}$ to $\boldsymbol{V_n = V_c}$ is allowed for ***Perma-Columns*** because their overall depth is less than 10 inches.

For members subjected to shear and flexure only, $\boldsymbol{V_c}$ can be taken as the greater of:

$$V_c = 2.0\, b\, d\, (f_c')^{1/2} \tag{3.4.1}$$

or

$$V_c = 1.9\, b\, d\, (f_c')^{1/2} + 2500\, A_s\, d\, V_u/M_u \tag{3.4.2}$$

but not greater than:

$$V_c = 3.5\, b\, d\, (f_c')^{1/2} \tag{3.4.3}$$

or

$$V_c = 1.9\, b\, d\, (f_c')^{1/2} + 2500\, A_s \tag{3.4.4}$$

Where $(f_c')^{1/2}$ shall not be taken to be greater than 100 lbf/in^2. Note that equation 3.4.4 limits the ratio of $\boldsymbol{d\, V_u/M_u}$ in equation 3.4.2 to unity. Perma-Column design shear strength values ($\phi\, V_n$) calculated using equations 3.4.1 through 3.4.4 are compiled in Table 3.4.1. Note that in all cases, equation 3.3.4 and not equation 3.4.3 controls the maximum design shear strength.

Table 3.4.1 Shear Strength Design Values
(Without Increases From Axial Compressive Forces)

		PC6300		PC6400		PC8300		PC8400	
		V_n=V_c lbf	$\phi\, V_n$* lbf	V_n=V_c lbf	$\phi\, V_n$* lbf	V_n=V_c lbf	$\phi\, V_n$* lbf	V_n=V_c lbf	$\phi\, V_n$* lbf
Minimum from Equation 3.4.1		4236	3177	5421	4066	6042	4531	7733	5800
Maximum from Equation 3.4.4		5024	3768	6150	4613	7289	5467	8896	6672
Value from Equation 3.4.2	M_u/V_u, in.								
	4	5009	3757	6135	4602	7917	5938	9524	7143
	5	4812	3609	5938	4454	7482	5611	9089	6816
	6	4680	3510	5807	4355	7191	5393	8798	6599
	8	4516	3387	5643	4232	6828	5121	8435	6327
	10	4418	3313	5544	4158	6611	4958	8218	6163
	12	4352	3264	5479	4109	6465	4849	8072	6054
	14	4305	3229	5432	4074	6362	4771	7969	5977
	16	4270	3202	**	**	6284	4713	7891	5918
	20	**	**	**	**	6175	4631	7782	5837
	24	**	**	**	**	6102	4577	**	**
	28	**	**	**	**	6051	4538	**	**

*ϕ = 0.75 for shear
** Minimum value from equation 3.4.1 is greater

Although the design shear strength of a component ϕV_n decreases as the ratio of bending moment to shear load induced by the factored loads (M_u/V_u) increases, any axial compressive load induced by the factored loads will increase design shear strength. To this end, ACI allows use of the following equations for members subjected to axial compression in addition to bending:

$$V_c = 1.9\, b\, d\, (f_c')^{1/2} + 2500\, A_s\, d\, V_u/\{M_u - N_u\,(4h - d)/8\} \quad (3.4.5)$$

However, V_c can not be greater than:

$$V_c = 3.5\, b\, d\, (f_c')^{1/2}\{1 + N_u/(500\, A_g)\}^{1/2} \quad (3.4.6)$$

Where N_u is the axial force in lbf due to factored loads (positive for compression and negative for tension). When the quantity $\{M_u - N_u\,(4h - d)/8\}$ in equation 3.4.5 is negative, V_c shall be calculated using equation 3.4.6.

Since N_u cannot exceed $\phi_a P_n$, the maximum N_u values for PC6300, PC6400, PC8300 and PC8400 are 119.5, 148.2, 161.8 and 199.7 kips, respectively (see Table 3.3.2). Substituting these values into equation 3.4.6 and multiplying by a resistance factor of 0.75 produces the following maximums for $\phi V_n = \phi V_c$: 168.4, 212.5, 242.8 and 305.8 kips for PC6300, PC6400, PC8300 and PC8400, respectively.

Design shear strengths ϕV_n for PC6300 columns, obtained by multiplying V_c values from Equation 3.4.5 by a shear resistance factor of 0.75, are shown in Figure 3.4.1 for a variety of M_u/V_u and N_u/V_u combinations.

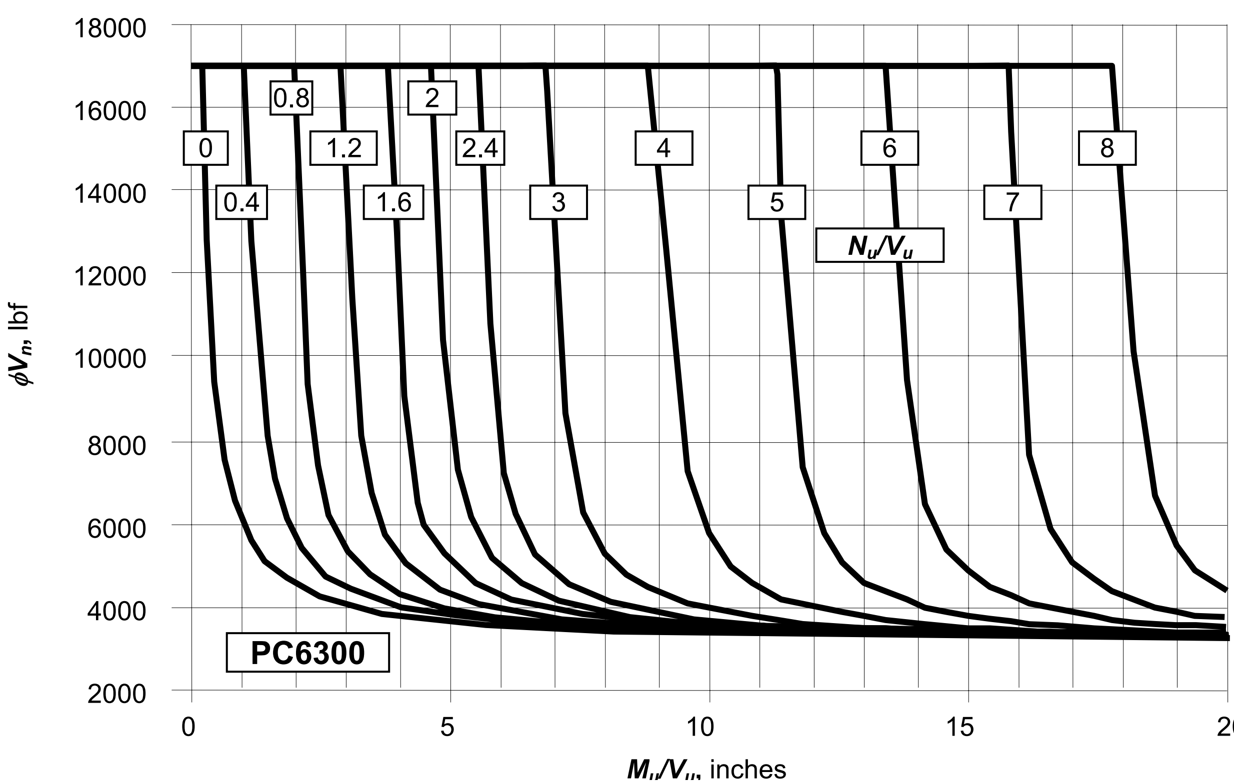

Figure 3.4.1. Design shear strengths ϕV_n for PC6300 columns as a function of M_u/V_u and N_u/V_u

3.5 Comparison With Wood Strength Values

A common question is how do ***Perma-Column*** design strength values compare with those of the laminated wood posts they replace or to which they are attached? Prior to making some of these comparisons, the following points should be understood.

1. Wood strengths are dependent upon the duration of the applied load. The amount of load a wood component can sustain decreases the longer the load acts upon the structure. There is no time dependent reduction in the amount of load a reinforced concrete component can sustain.
2. Magnitude of bending moment applicable to a reinforced concrete component increases as axial compressive load is applied to the component. Conversely, the magnitude of bending moment that can be applied to a wood member decreases as an axial compressive load is applied to the component.
3. The magnitude of shear force to which a reinforced concrete component can be subjected increases as an axial compressive load is applied to the component, but decreases as the bending moment in the member increases. The design shear strength of a wood member is not measurably affected by the axial or bending forces acting on the member.
4. Wood design values must be reduced when wood is used in a moist environment. After initial curing, concrete design strengths are not affected by changes in the moisture content of the surrounding environment.

Table 3.5.1 contains load and resistance factor design (LRFD) values for mechanically-laminated posts fabricated from No. 1 Southern Yellow Pine and used where the wood moisture content will exceed 19% for extended time periods. No. 1 Southern Yellow Pine is a common visual grade for laminated posts. Note that any post that would be used in place of a ***Perma-Column*** would have to be designed for higher moisture contents. Also, like the concrete design strengths established in previous sections, the LRFD values for wood in Table 3.5.1 must be used in conjunction with the load combinations and load factors in Section 2.2.

Figure 3.5.1 contains a graphical comparison of maximum allowed axial and bending moment induced by wind loads for ***Perma-Columns*** and No.1 Southern Yellow Pine posts. Values for the ***Perma-Columns*** are the same as those shown in Figure 3.3.2. Note that the axes have been retitled with ϕP_n replaced by P_u (i.e., $P_u \le \phi P_n$) and ϕM_n replaced by M_u. The relationships between P_u and M_u for the wood posts were calculated using the following design equation for wood members under combined bending and axial compressive loads.

$$\left[\frac{P_u}{\lambda\, \phi_c\, P'}\right]^2 + \frac{M_m}{\lambda\, \phi_b\, M'} \le 1.0 \qquad (3.5.1)$$

where:

P_u	=	Axial compressive force due to factored loads
$\lambda\, \phi_c\, P'$	=	Design resistance for axial compression from Table 3.5.1
M_m	=	Factored moment, including any magnification for second-order effects
	=	M_u for short columns
$\lambda\, \phi_b\, M'$	=	Adjusted moment resistance from Table 3.5.1

Table 3.5.1 Properties of Mechanically-Laminated No. 1 Southern Yellow Pine Columns*

Variable Description	Symbol	Unit				
Physical Characteristics						
Number of Plys			3	4	3	4
Nominal Ply Size		in. x in.	2x6	2x6	2x8	2x8
Cross-Sectional Area	$\boldsymbol{A}$	in.^{2}	24.75	33.00	32.63	43.50
Section Modulus	$\boldsymbol{S}$	in.^{3}	22.69	30.25	39.42	52.56
Tabulated Reference Strength Values						
Shear	F_v	kips/in.^{2}	0.26	0.26	0.26	0.26
Flexure	F_b	kips/in.^{2}	4.19	4.19	3.81	3.81
Axial Compression	F_c	kips/in.^{2}	4.20	4.20	3.96	3.96
Applicable Adjustment Factors						
Wet Service Factor - Shear	C_M		0.97	0.97	0.97	0.97
Wet Service Factor - Flexure	C_M		0.85	0.85	0.85	0.85
Wet Service Factor - Axial Comp.	C_M		0.80	0.80	0.80	0.80
Shear Stress Factor**	C_H		1.95	1.95	1.95	1.95
Load Sharing Factor***	C_r		1.35	1.35	1.40	1.40
Adjusted Reference Strength Values						
Shear ($F_v C_M C_H$)	F_v'	kips/in.^{2}	0.49	0.49	0.49	0.49
Flexure ($F_b C_M C_r$)	F_b'	kips/in.^{2}	4.81	4.81	4.53	4.53
Axial Compression ($F_b C_M$)	F_c'	kips/in.^{2}	3.36	3.36	3.17	3.17
Adjusted Resistance Values						
Shear ($F_v' A/1.5$)	V'	kips	8.11	10.82	10.70	14.26
Moment ($F_b' S$)	M'	kips-in.	109.1	145.4	178.7	238.3
Axial Compression ($F_c' A$)	P'	kips	83.2	110.9	103.4	137.8
Resistance and Time Effect Factors						
Resistance Factor – Shear	ϕ_v		0.75	0.75	0.75	0.75
Resistance Factor – Bending	ϕ_b		0.85	0.85	0.85	0.85
Resistance Factor – Axial Comp.	ϕ_c		0.90	0.90	0.90	0.90
Time Effect Factor – Wind Load	λ_w		1.0	1.0	1.0	1.0
Time Effect Factor – Snow Load	λ_s		0.8	0.8	0.8	0.8
Design Resistance Values Under Wind Loading						
Shear	$\lambda_w \phi_v V'$	kips	6.09	8.11	8.02	10.70
Moment	$\lambda_w \phi_b M'$	kips-in.	92.7	123.6	151.9	202.6
Axial Compression	$\lambda_w \phi_c P'$	kips	74.8	99.8	93.0	124.0
Design Resistance Values Under Snow Loading						
Shear	$\lambda_s \phi_v V'$	kips	4.87	6.49	6.42	8.56
Moment	$\lambda_s \phi_b M'$	kips-in.	74.2	98.9	121.5	162.1
Axial Compression	$\lambda_s \phi_c P'$	kips	59.9	79.8	74.4	99.2

* From AF&PA 1996 Edition of Load and Resistance Factor Design Manual For Engineered Wood Construction. **Assumes columns with full lateral bracing and only major axis bending**

** Based on changes incorporated in 2001 Edition of Allowable Stress Design Manual for Engineering Wood Construction (AF&PA, 2001).

*** From ASAE EP559

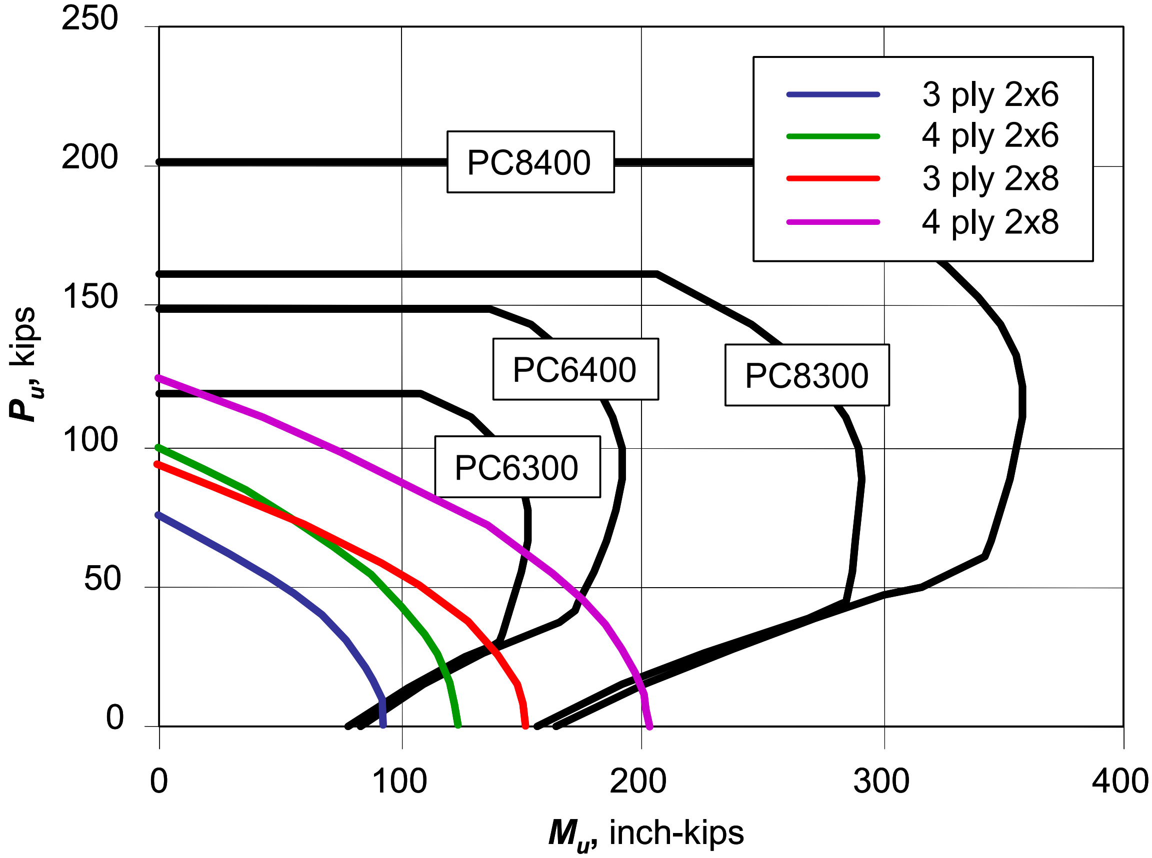

Figure 3.5.1. Interaction diagrams for maximum allowed axial and bending moment induced by wind loads for ***Perma-Columns*** and No.1 Southern Yellow Pine posts.

Table 3.5.2 Comparison of Perma-Column and Wood Post Shear Strengths

Variable Description	Units	3-ply 2x6	4-ply 2x6	3-ply 2x8	4-ply 2x8
LRFD Design Shear Strength for No.1 SP (Wind Load)	kips	6.09	8.11	8.02	10.70
		PC6300	**PC6400**	**PC8300**	**PC8400**
ACI Design Shear Strength (No Bending)	kips	3.77	4.61	5.47	6.67
Average Maximum Test Shear	kips	10.55	11.08	16.36	19.02
Average Maximum Test Shear x 0.55	kips	5.80	6.09	9.00	10.46

Table 3.5.2 contains ***Perma-Column*** and No.1 Southern Yellow Pine design shear strength values. As evidenced from the table, the American Forest & Paper Association (AF&PA) LRFD values for the wood posts exceed the ACI design shear values for the ***Perma-Columns*** by an average 55%. However, as evidenced from actual bending test data (Appendix C), the ***Perma-Column*** design shear values are extremely conservative. When a conservative resistance factor of 0.55 is applied to the ***Perma-Column*** test values, there is essentially no difference between the design shear strengths of the No.1 SP posts and the ***Perma-Columns***.

4. Strength Properties of the Steel Bracket

Using published procedures to determine design values for ***Perma-Column*** components is a straight forward process with two exceptions, that being the determination of design values for the shear and bending strength of the steel bracket-to-concrete connection. Because of the complex geometry and interaction between steel and concrete at the steel bracket-to-concrete connection, neither ACI or AISC (American Institute of Steel Construction) design procedures can be applied. To this end, engineers are required to rely on laboratory tests that have been specially designed to isolate the shear strength and bending strength of the joint.

4.1 Bending Strength

Bending strength of the steel bracket, or more specifically, the steel bracket-to-concrete connection, was determined by laboratory testing. Test procedures, equipment and results are presented in Appendix B along with a brief discussion.

Translating test values to design strengths requires (1) a selection of a nominal bending strength, M_n, and (2) application of a resistance factor for bending ϕ_b. Selection of a nominal bending strength is complicated issue that requires additional investigation. Because of the ductility of the connection, one may want to assign a limit state of 0.02 or 0.03 radians to the connection. Nominal bending strength would then be the bending strength associated with the selected rotation. This is not an uncommon approach for steel connections characterized by larger deformations. With a nominal bending strength so defined, a bending resistance factor of 0.90 would be appropriate.

4.2 Shear Strength

All shear related failures associated with the shear tests described in Appendix C occurred on "non-bracketed" ends of test specimens. This means that the shear strength of a steel bracket-to-concrete connection is at least as great as that for the reinforced concrete base. This is not surprising since the steel bracket functions much like shear reinforcing in that it ties longitudinal reinforcing together. Consequently, as long as the reinforced concrete portion of a ***Perma-Column*** is shown to adequately handle applied shear forces, there is no need to check the shear capacity of the steel bracket-to-concrete column connection.

5. Stiffness of the Reinforced Concrete

Calculation of a structural component's bending deflections requires knowledge of the flexural rigidity of the component. For components manufactured from a single, homogenous material (e.g. steel) and possessing a constant cross-section, flexural rigidity is simply equal to the product of the modulus of elasticity of the material and the moment of inertia associated with the component's cross-section.

Calculation of the flexural rigidity of a reinforced concrete component is not as straight forward as that for most wood or steel components for two primary reasons. First, a reinforced concrete member consists of two materials (concrete and steel) each with a different modulus of elasticity. Second, the cross-sectional moment of inertia of a reinforced concrete component changes rapidly once a tension crack forms in the concrete. Because tension cracking is directly related to the applied bending moment, and applied bending moment generally varies significantly along a component, cracking (and hence effective moment of inertia) will vary along the length of the component.

To simplify flexural rigidity calculations for reinforced concrete components, the component is treated as a solid "non-reinforced" concrete component. For cracked cross-sections, this requires transformation of the cross-sectional area of reinforcing steel to an equivalent cross-sectional area of concrete. When reinforced concrete components are treated as solid concrete components, the modulus of elasticity assigned to the member is the modulus of elasticity for the concrete.

5.1 Concrete Modulus of Elasticity

ACI 8.5.1 states that the modulus of elasticity for concrete, E_c, for normal weight concrete shall be calculated as $57{,}000(f_c')^{0.5}$ where f_c' is the specified concrete strength in lbf/in^2. Given an f_c' of 10,000 lbf/in^2 this equates to a E_c for ***Perma-Columns*** of 5,700,000 lbf/in^2.

5.2 Reinforced Concrete Moment of Inertia

ACI 9.5.2.3 states that immediate deflections shall be calculated using E_c and an effective moment of inertia, I_e, that is calculated using the following formula, but is not greater than I_g.

$$I_e = (M_{cr}/M_a)^3\, I_g + [\, 1 - (M_{cr}/M_a)^3\,]\, I_{cr} \qquad (5.2.1)$$

where:

I_e = Effective moment of inertia for deflection calculations, in^4
I_{cr} = Moment of inertia of cracked section transformed to concrete, in^4
I_g = Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in^4
M_a = Maximum moment in member at stage deflection is computed, in-lbf
M_{cr} = Cracking moment, in-lbf
= $f_r\, I_g / y_t$
f_r = Modulus of rupture of concrete, lbf/in^2
= $7.5(fc')^{0.5}$ for normal weight concrete
= 750 lbf/in^2 for ***Perma-Columns***
y_t = Distance from centroidal axis of gross section neglecting reinforcement, to extreme fiber in tension, inches

Effective moment of inertia values calculated using equation 5.2.1 are compiled in Table 5.1 and have been plotted in Figure 5.2.1.

Table 5.2.1: Perma Column Moment of Inertia Values

Variable	Symbol	Units	PC6300	PC6400	PC8300	PC8400
Overall Concrete Width	b	in.	5.38	6.88	5.38	6.88
Overall Concrete Depth	h	in.	5.44	5.44	7.19	7.19
Gross Moment of Inertia	I_g	$in.^4$	72.2	92.3	166.6	213.1
Depth to Centroidal Axis	y_t	in.	2.72	2.72	3.60	3.60
Cracking Moment	M_{cr}	in-lbf	19900	25450	34770	44460
Ratio of Steel to Concrete MOE	E_s/E_c		5.09	5.09	5.09	5.09
Distance to Neutral Axis	c	in.	1.156	1.038	1.425	1.284
Cracked Moment of Inertia	I_c	$in.^4$	18.8	20.1	59.0	62.5

	Actual Bending Moment, M_a, in-lbf	PC6300	PC6400	PC8300	PC8400
Effective Bending Moment, I_e, $in.^4$	0	72.2	92.3	166.6	213.1
	15000	72.2	92.3	166.6	213.1
	20000	71.4	92.3	166.6	213.1
	25000	45.7	92.3	166.6	213.1
	30000	34.4	64.2	166.6	213.1
	35000	28.6	47.9	164.5	213.1
	40000	25.4	38.7	129.7	213.1
	45000	23.4	33.2	108.6	207.7
	50000	22.2	29.6	95.2	168.4
	60000	20.7	25.6	79.9	123.8
	70000	20.0	23.6	72.2	101.1
	80000	19.6	22.4	67.8	88.3
	90000	19.4	21.7	65.2	80.7
	100000		21.3	63.5	75.7
	105000		21.1	62.9	73.9
	110000			62.4	72.4
	120000			61.6	70.2
	130000			61.1	68.5
	140000			60.6	67.3
	150000			60.3	66.4
	160000			60.1	65.7
	170000			59.9	65.2
	180000			59.8	64.8
	190000			59.7	64.4
	200000				64.2

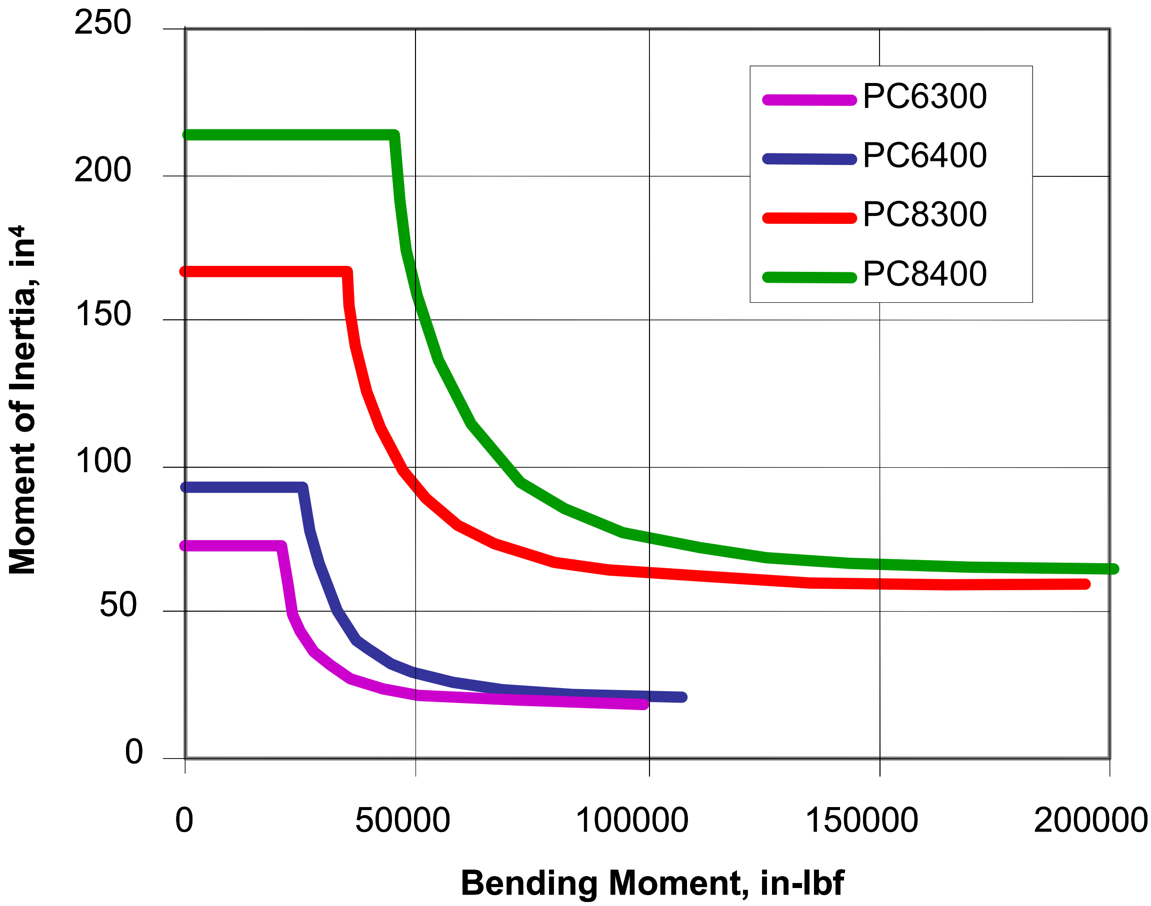

Figure 5.2.1. Effective moment of inertia for strong axis bending of ***Perma-Columns*** as a function of actual bending moment.

Although the actual bending moment (and hence effective moment of inertia) varies along the length of a member, Equation 5.2.1 assumes that the effective moment of inertia used in modeling will be fixed for the entire length (i.e., span) of the member. The specific I_e to use for such modeling should be that calculated using Equation 5.2.1 with M_a, set equal to the maximum actual bending moment for the entire span. ACI 9.5.2.4 recommends using the bending moment located at the midspan of simple and continuous spans, and the bending moment located at the support for cantilevers.

Equation 5.2.1 is based on the work of Dan E. Branson ("Instantaneous and Time-Dependent Deflections of Simple and Continuous Reinforced Beams," Part 1, Report No. 7 Alabama Highway Research Report, Bureau of Public Roads, August 1963). For engineers wishing to recognize the continuous variation of the moment of inertia with span, Branson proposed using the fourth power instead of the third power in Equation 5.2.1. In this case, M_{cr} and M_a would be the cracking and applied moment, respectively, at each segment along the span.

6. Rotational Stiffness of Steel Bracket-to-Wood Connection

The moment-rotational relationship (a.k.a. rotational stiffness) of the steel bracket-to-wood connection can be estimated once the shear load-slip relationship (a.k.a. the slip modulus) of the individual fasteners has been established and the geometry of the fastener pattern has been specified.

6.1 Fastener Slip Modulus

The slip modulus of a fastener is the ratio of (1) the shear load transferred by the fastener, and (2) the slip between the two components being connected by the fastener (a.k.a. interlayer slip). While the relationship between fastener shear load and interlayer slip is non-linear (especially near ultimate load), it is sufficiently linear at lower loads to justify use of a fixed value for most structural analyses.

Based on ***Perma-Column*** steel bracket-to-wood connection testing reported in Appendix D, a slip modulus of 85,500 lbf/inch is recommended for a 0.5-inch diameter SAE grade 5 bolt in double shear. Similarly, a slip modulus of 28,700 lbf/inch is recommended for a 0.25-inch diameter Simpson Strong-Drive® wood screw in single shear. These values should be relatively accurate for any steel bracket-to-wood connection featuring lumber with a specific gravity near 0.50.

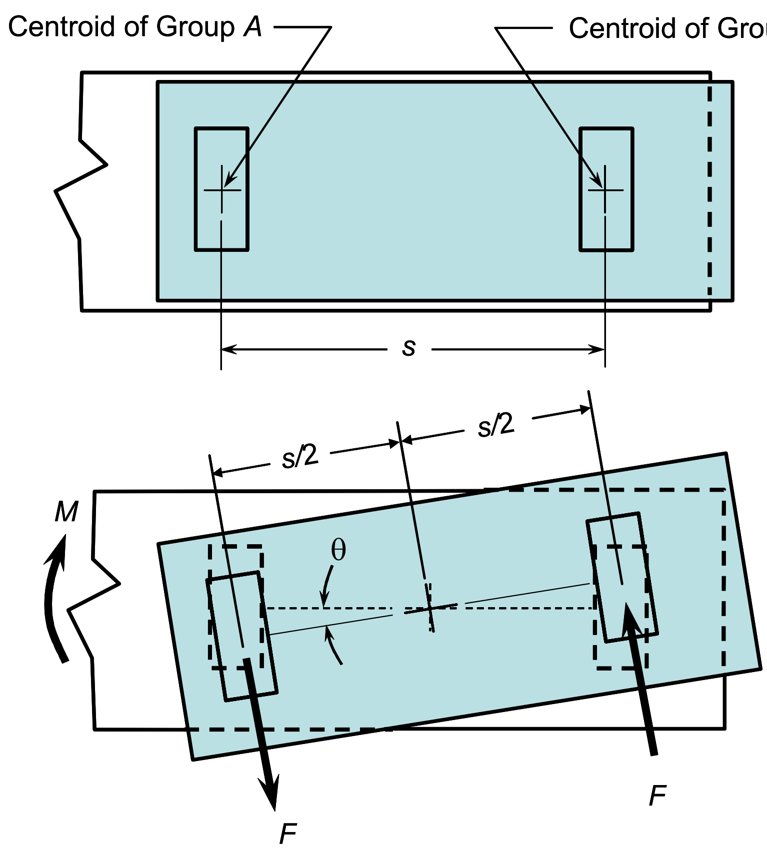

Figure 6.1.1. Rotation between wood post and steel bracket under a pure bending moment when two identical fastener groups are spaced a distance, ***s***.

Figure 6.1.1 illustrates the rotation between a ***Perma-Column*** steel bracket and post under a pure bending moment, ***M.*** Each of the small boxes in Figure 6.1.1 represents an individual fastener or a small group of closely spaced fasteners. When there is a small group of fasteners, it is advantageous to assign a slip modulus to the entire group. This is simply accomplished by summing the slip modulus values of the individual fasteners comprising the group. For example, if the group contains one 0.5-inch diameter bolt in double shear and two SDS screws in single shear, the slip modulus for the group would equal 85,500 plus 2 x 287,000 or 142,900 lbf/inch.

6.2. Rotational Stiffness

The rotational stiffness of a connection is the ratio of applied moment, $\boldsymbol{M}$, to rotation, $\boldsymbol{\theta}$, as defined in Figure 6.1.1. To obtain a relationship between $\boldsymbol{M}$ and $\boldsymbol{\theta}$, use is made of the fact that the distance from the center of joint rotation to a fastener or fastener group, $\boldsymbol{s}/2$, when multiplied by rotation, $\boldsymbol{\theta}$, is equal to the interlayer slip of the fastener or fastener group, $\boldsymbol{\Delta}$. Interlayer slip is in turn equal to the shear force applied to the fastener or fastener group, $\boldsymbol{F}$, divided by the slip modulus, $\boldsymbol{k}$. Finally, shear force, $\boldsymbol{F}$, when multiplied by the fastener spacing, s, is equal to the applied bending moment, $\boldsymbol{M}$. In equation form

$$\Delta = \theta s/2 \qquad (6.2.1)$$

$$k = F/\Delta \qquad (6.2.2)$$

$$M = F s \qquad (6.2.3)$$

From these equations the following relationship for rotation stiffness can be obtained.

$$M/\theta = k s^2/2 \qquad (6.2.4)$$

Where:

- $\boldsymbol{M}$ = Applied bending moment
- $\boldsymbol{\theta}$ = Joint rotation
- $\boldsymbol{k}$ = Slip modulus for a fastener or fastener group
- $\boldsymbol{s}$ = Spacing between fasteners or fastener groups

6.3 Connection Modeling

Adequate assessment of the impact of joint behavior on structural load distribution requires proper modeling of the moment-rotational relationship. This a very straight forward process when the structural analysis program incorporates special joint elements that enable designers to simply enter the rotational stiffness as calculated using equation 6.2.4.

When special joint elements are not available, a designer can model the joint using a simple beam or frame element. The flexural rigidity, $\boldsymbol{EI}$, that must be assigned to this element is given as:

$$EI = k s^2 L_e / 2 \qquad (6.3.1)$$

Where:

- $\boldsymbol{EI}$ = Product of the modulus of elasticity and moment of inertia assigned to the modeling element
- $\boldsymbol{k}$ = Slip modulus for a fastener or fastener group
- $\boldsymbol{s}$ = Spacing between fasteners or fastener groups (see Figure 6.1.1)
- $\boldsymbol{L_e}$ = Length of the modeling element

Output from the structural analyses will include the bending moment, $\boldsymbol{M}$. By dividing this bending moment by the spacing between the fastener groups, $\boldsymbol{s}$, the force $\boldsymbol{F}$ on each group is obtained. If there is more than one fastener in the group, it is important to realize that this force $\boldsymbol{F}$ is divided among the individual fasteners in accordance with their individual slip modulus. In equation form:

$$F_i = M\,k_i\,/\,(s\,k) \tag{6.3.2}$$

$$F_i = F\,k_i\,/\,k \tag{6.3.3}$$

Where:

F_i	=	Shear force on fastener $\boldsymbol{i}$ where $\boldsymbol{i}$ is one fastener within a group of fasteners
F	=	Total shear force acting on fastener group
k_i	=	Slip modulus of fastener $\boldsymbol{i}$
k	=	Slip modulus for fastener group

Appendix A: Column Bending Tests

A.1 Introduction, Test Methods and Equipment

Several ***Perma-Columns*** where loaded to failure in bending to validate ACI design values. These ***Perma-Columns*** were supported and loaded as shown in Figure A.1. The spacing of 48 inches between supports was selected to provide sufficient bearing at supports while also providing a shear span to effective beam depth ratio that would push the bending strength limits of the columns during test. In this particular case, the shear span is equal to 24 inches – the distance between a support and the load point. With an effective depth of 3.97 inches, the shear span to depth ratio *(a/d* ratio) for PC6300 and PC6400 series columns is 6.04. The effective depth of 5.53 inches for PC8300 and PC8400 columns results in an *a/d* ratio of 4.34.

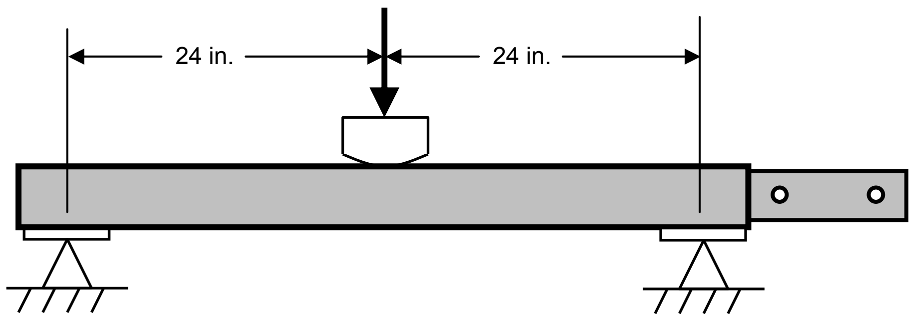

Figure A.1. Bending test set-up for ***Perma-Columns***.

Beams with an *a/d* ratio greater than 6 fall under the general category of *long* beams. Beams with an *a/d* ratio between about 2.5 and 6 fall under the general category of *intermediate length* beams. Long beams typically fail in flexure. Failure begins with yielding of the tension reinforcement and ends by crushing of the concrete at the point of maximum bending moment. In addition to nearly vertical flexural cracks near the point of maximum bending moment, prior to failure slightly inclined cracks may be present between the support and region of maximum bending moment (Wang and Salmon, 1985). For intermediate length beams, vertical cracks form first, followed by inclined flexure-shear cracks. At the sudden occurrence of a flexure-shear crack, a beam is not able to redistribute load and additional load can generally not be sustained. The load corresponding to the point at which the flexure-shear crack forms represents the shear strength of the beam (Wang and Salmon, 1985).

Columns were loaded using a Tinius Olson Universal Compression-Tension Testing Machine. Load-head rate was fixed at 0.2 inches/minute. Applied load and load-head movement were recorded at 0.5 second intervals using a Campbell Scientific CR23X datalogger.

A.2 Results

Table A.1 contains the load-head displacement, maximum applied load, maximum shear load (i.e., ½ total applied load) and corresponding maximum bending moment at the point of maximum load for each test specimen. Figure A.2 contains a plot of midspan displacement versus shear/bending moment for each ***Perma-Column*** series. Data for each curve in this figure was obtained by averaging load-displacement data for all specimens tested of that particular series.

Table A.1 Perma-Column Bending Test Results

Perma-Column Series	Replicate Number	Load-Head Disp. at Max. Load, inches	Maximum Applied Load, kips	Maximum Shear Force, kips	Max. Bending Moment, inch-kips
PC6300	1	0.71	10.5	5.23	125.5
	3	0.73	10.4	5.19	124.7
	4	0.65	10.5	5.26	126.2
	5	*	10.6	5.31	127.5
	6	0.68	10.8	5.39	129.2
	Average	0.69	10.6	5.28	126.6
	COV**	5.0 %	1.4 %	1.4 %	1.4 %
PC6400	1	0.71	10.8	5.42	130.1
	2	0.78	10.6	5.29	126.9
	3	*	11.6	5.78	138.7
	4	0.83	10.5	5.23	125.4
	5	0.88	12.0	5.98	143.4
	Average	0.80	11.1	5.54	132.9
	COV**	9.1%	5.9 %	5.9 %	5.9 %
PC8300	1	0.41	17.4	8.71	209.1
	2	0.38	15.7	7.85	188.5
	3	0.36	16.0	7.98	191.4
	Average	0.38	16.4	8.18	196.4
	COV**	6.6 %	5.7 %	5.7 %	5.7 %
PC8400	1	0.42	19.7	9.86	236.6
	2	0.32	18.6	9.32	223.6
	3	0.35	18.1	9.03	216.6
	4	0.44	19.7	9.84	236.1
	Average	0.38	19.0	9.51	228.2
	COV**	14.8 %	4.3 %	4.3 %	4.3 %

* Not recorded
** Coefficient of Variation = Standard Deviation x 100 /Average

Perma-Columns failed as expected given their *a/d* ratios. With their long beam classification in this test, PC6300 and PC6400 series columns were expected to exhibit a pure bending failure and did. A typical PC6300/PC6400 failure, which is shown in Figure A.3, was characterized by formation of vertical tension cracks, followed by tension steel yielding and eventual concrete crushing at midspan. A typical PC8300/PC8400 failure is shown in Figure A.4. Unlike their shallower counterparts, failure of these columns was controlled by their shear strength as is evidenced by the flexure-shear crack in Figure A.4.

The difference in failure modes between PC6300/PC6400 series columns and PC8300/PC8400 series columns is reflected in the load-displacement plots in Figure A.2. Curves associated with bending failures are smooth as tension steel continues to yield. Failures associated with shear are abrupt as flexure-shear cracks suddenly form. The slight difference between PC6300 and PC6400 at low loads in attributable to the greater width, and hence greater uncracked moment of inertia of the PC6400 series. At high loads there is no difference between PC6300 and PC6400 columns as their behavior near failure is due to the relative location and cross-sectional area of tension steel which is identical in both series. In the case of PC8300 and PC8400 columns, concrete cross-sectional area (which is greater in PC8400) controls strength and stiffness right up to failure.

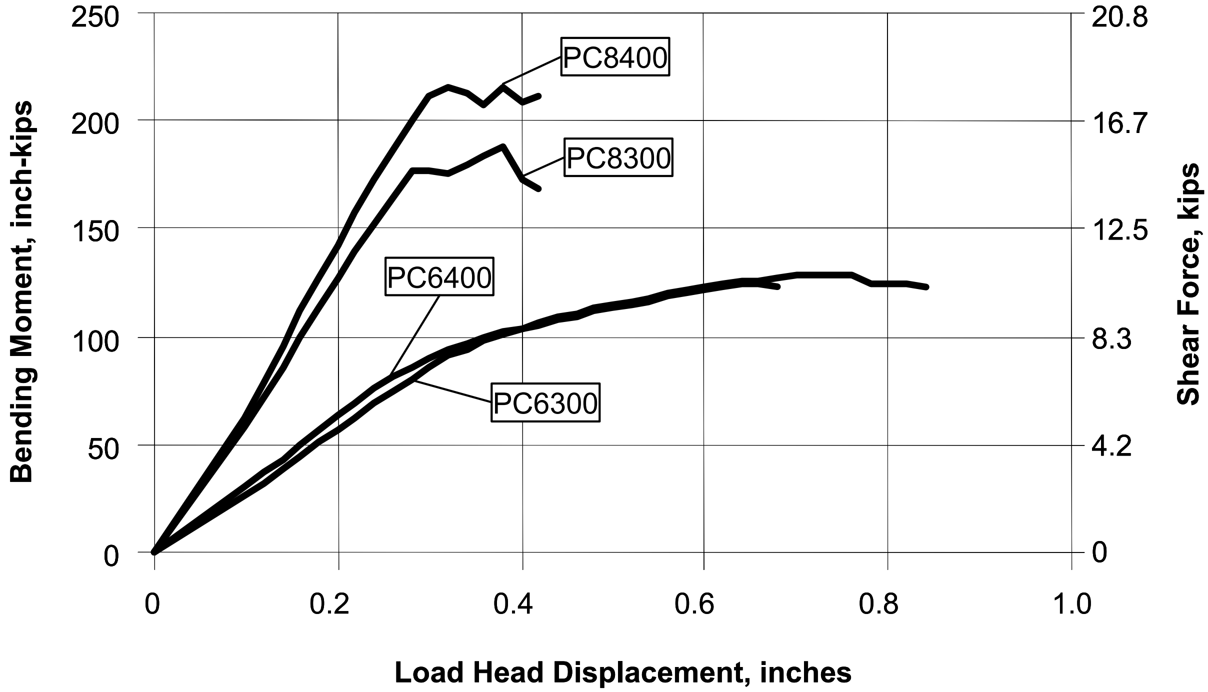

Figure A.2. Average load-head displacement versus shear force/midspan bending moment.

Figure A.3. Bending failure mode characteristic of all PC6300 and PC6400 series columns.

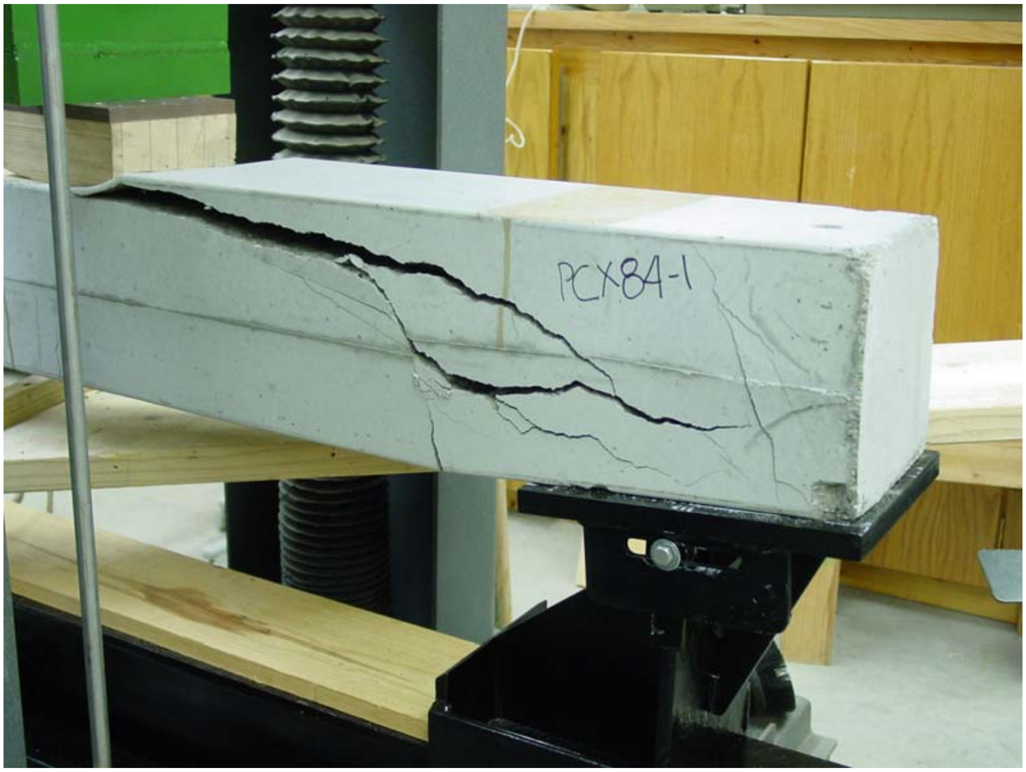

Figure A.4. Shear failure mode characteristic of all PC8300 and PC8400 series columns.

A.3 Comparisons

Table A.2 compares bending test results with ACI nominal shear and bending moment strength values (***V**$_n$* and ***M**$_n$* values, respectively).

Table A.2 Comparison of Test Results With ACI Nominal Strengths

Perma-Column Series	Property	Average Test Maximum	ACI Nominal Strength	Ratio, Test/ACI	Test Underestimates Maximum Strength?
PC6300	Shear, kips	5.28	4.24*	1.25	Yes
	Bending Moment, inch-kips	126.6	97.2	1.30	No
PC6400	Shear, kips	5.54	5.42*	1.02	Yes
	Bending Moment, inch-kips	132.9	104.5	1.27	No
PC8300	Shear, kips	8.18	6.10*	1.34	No
	Bending Moment, inch-kips	196.4	197.4	1.00	Yes
PC8400	Shear, kips	9.51	7.73*	1.23	No
	Bending Moment, inch-kips	228.2	206.8	1.10	Yes

* From Table 3.4.1 M_u/V_u = 24 inches

In all cases, actual assembly strength surpassed ACI nominal values, even in those cases where the average test maximum was not associated with the cause of failure. For example, tests of PC8300 and PC8400 columns ended when the shear capacity of the assemblies was reached; consequently, the associated bending moments listed in Tables A.1 and A.2 are less than the actual bending capacity of the assemblies.

It is clearly evident from Table A.2 that ACI nominal shear strengths underestimate the actual shear strength of the assemblies by at least 20%. This can be partly attributed to the quality of the concrete used in the manufacture of ***Perma-Columns***.

A.4 References

Wang and Salmon. 1985. Reinforced Concrete Design. 4th Edition. Harper & Row Publishers. New York, New York.

Appendix B: Bracket Bending Tests

B.1 Test Methods and Equipment

Several ***Perma-Columns*** where loaded to failure in bending to determine the flexural strength and stiffness of the steel bracket-to-concrete column connection. The 1/3 point loading arrangement shown in Figures B.1 and B.2 was used with the interface between the steel bracket and the concrete located in the center of the shear-free, constant-moment region (i.e., the region between the two load points).

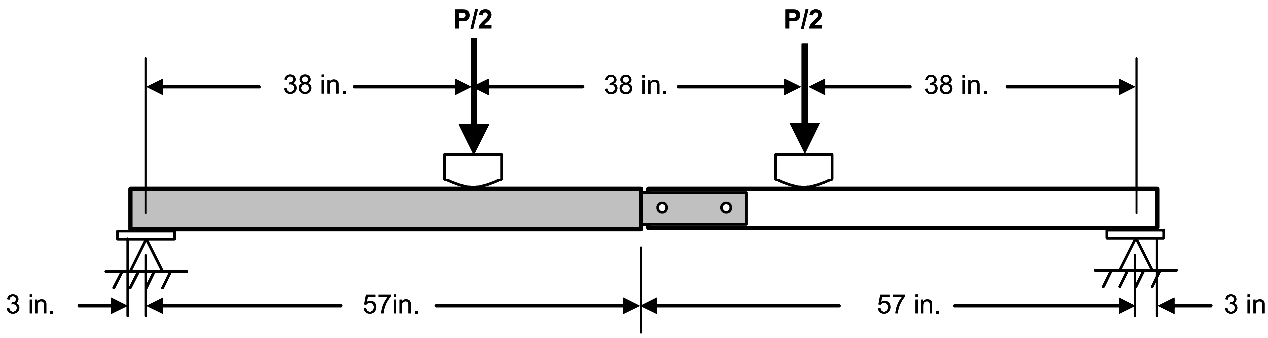

Figure B.1. Bending test set-up for ***Perma-Column*** steel bracket-to-concrete column connection.

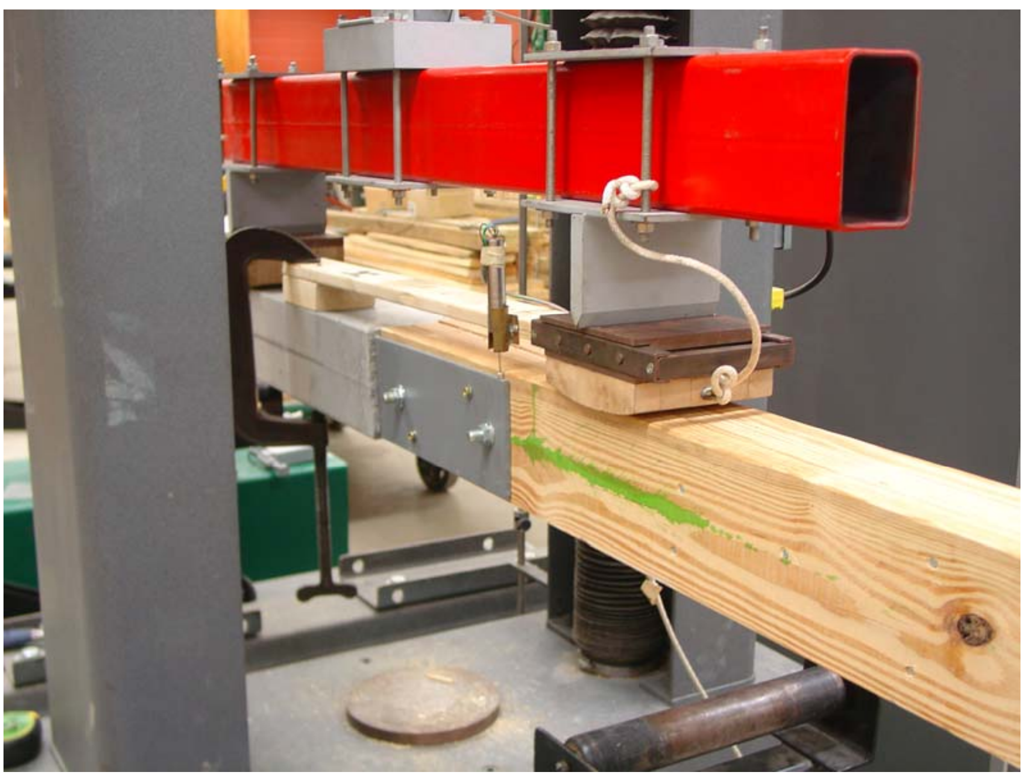

Figure B.2. Bending test set-up for ***Perma-Column*** steel bracket-to-concrete column connection showing LVDT used to measure joint rotation.

Loads were applied using a Tinius Olson Universal Compression-Tension Testing Machine as shown in Figure B.2. Load-head rate was fixed at 0.6 inches/minute. A linear variable differential transformer (LVDT) attached as shown in Figure B.2 was used to measure bending rotation between the concrete column and steel bracket. The core of the LVDT was placed on the edge of the bracket at a location 12 inches from concrete-to-steel bracket interface. Applied load, load-head movement, and displacements measured with the LVDT were recorded at 0.5 second intervals using a Campbell Scientific CR23X datalogger.

Six specimens each of Series PC6300 and PC8400, and four specimens each of PC6400 and PC8300 were tested. Replicates 5 and 6 of PC8400 where fabricated with #4 rebar instead of the standard #5 rebar used in these assemblies.

B.2 Results

Table B.1 contains the maximum applied load and corresponding load-head displacement, end shear (i.e., ½ total applied load) and bending moment for each test specimen.

Table B.1 Perma-Column Bracket Bending Test Results

	PC6300		**PC6400**		**PC8300**		**PC8400***	
Rep.	Max. Load	Load Head Disp.	Max. Load	Load Head Disp.	Max. Load	Load Head Disp.	Max. Load	Load Head Disp.
	kips	inches	kips	inches	kips	inches	kips	inches
1	5.26	4.27	5.44	4.64	8.74	3.20	10.94	4.46
2	4.83	3.61	5.28	4.35	8.37	2.65	10.61	4.33
3	4.76	3.42	5.57	5.17	9.41	4.05	7.94	2.79
4	4.56	3.12	5.53	4.06	8.84	4.36	7.40	2.68
5	5.10	3.86					8.79	
6	5.04	3.89					8.38	3.85
Rep.	Max. End Shear	Max. Bending Moment	Max. End Shear	Max. Bending Moment	Max. End Shear	Max. Bending Moment	Max. End Shear	Max. Bending Moment
	kips	in.-kips	kips	in.-kips	kips	in.-kips	kips	in.-kips
1	2.63	99.8	2.72	103.3	4.37	166.1	5.47	207.9
2	2.42	91.8	2.64	100.4	4.18	158.9	5.31	201.6
3	2.38	90.5	2.79	105.9	4.71	178.8	3.97	150.9
4	2.28	86.6	2.77	105.1	4.42	168.0	3.70	140.7
5	2.55	97.0					4.40	167.0
6	2.52	95.8					4.19	159.3
Average	2.46	93.58	2.73	103.66	4.42	167.96	4.51	171.22
Std Dev	0.127	4.844	0.064	2.436	0.217	8.228	0.723	27.468
COV	0.052	0.052	0.024	0.024	0.049	0.049	0.160	0.160
5% E.L.**		85.6		99.6		154.4		126.0

* Specimens 5 and 6 fabricated with #4 rebar.

** 5% E. L. = 5% Exclusion limit assuming normal distribution = Mean – Std Dev (1.645)

The LVDT was used to measure rotation of the steel bracket-to-concrete connection because previous testing showed this to be a major component of total bending deformation. In this study, load head displacement provides a relative measure of column bending deformation. That said, it should be noted that load head displacement, as recorded in this study, was not just due to flexure of the column assembly, but also included a relatively small amount of deformation associated with strain in assembly supports, load blocks and the load distributing beam.

That portion of the total load head displacement that is directly attributable to deformation (i.e., rotation) of the steel bracket-to-concrete connection was obtained by dividing connection rotation (in radians) by two and multiplying by 38 inches (i.e., the distance between a support and adjacent load point). A comparison of this displacement with total load head displacement is graphically illustrated for each ***Perma-Column*** series in Figure B.3.

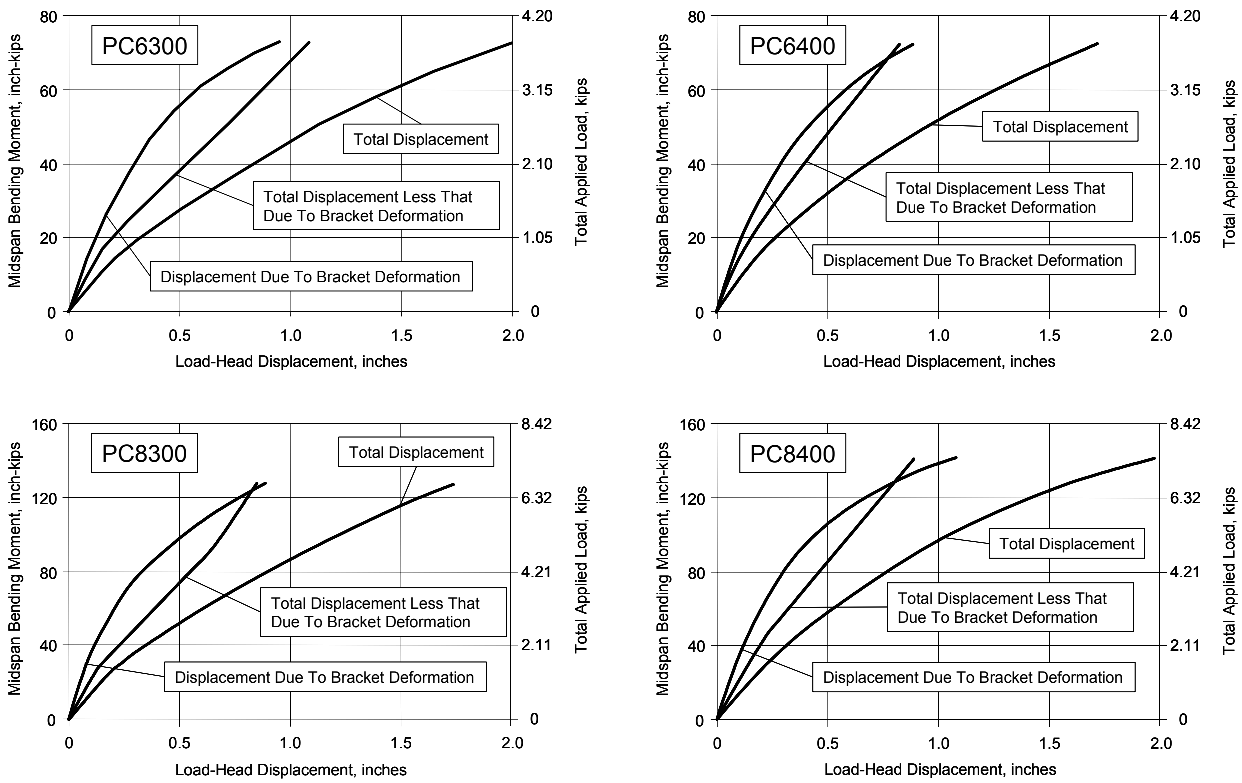

Figure B.3. Relative portion of total load-head displacement due to rotation of the steel bracket-to-concrete column connection.

Figure B.4 contains a plot of midspan bending moment versus connection rotation for each of the columns series. Data used to generate these plots is compiled in Table B.2. These values represent the average of data from the various specimen tests. It is important to note that there was essentially no difference between load-displacement curves of individual replications of a particular column series. Likewise, there was little difference between the moment-rotation curves of replicates of a particular column series.

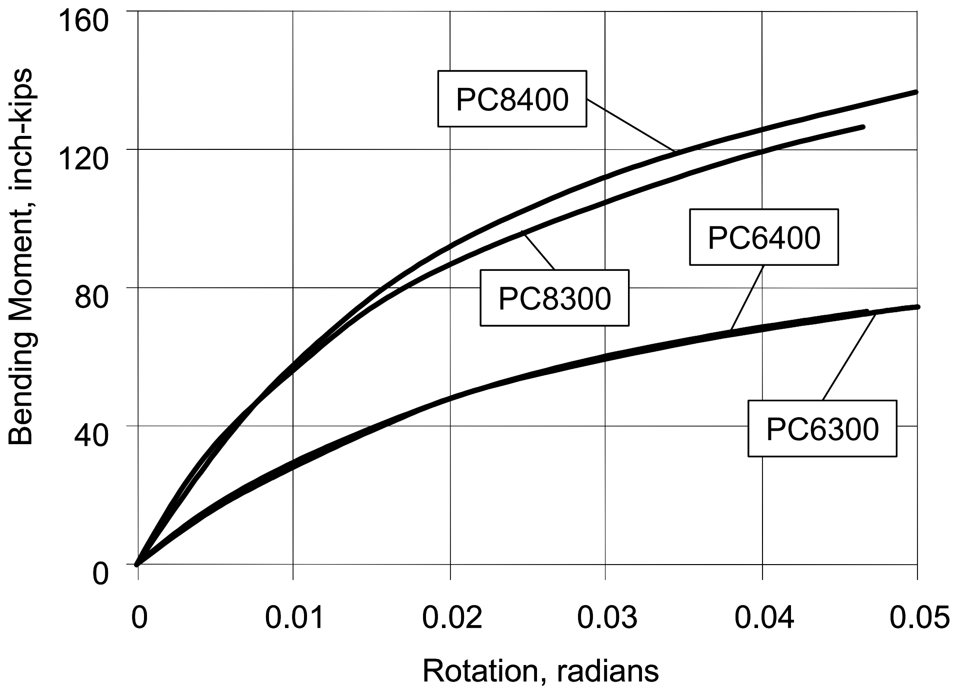

Figure B.4. Joint rotation versus bending moment for steel bracket-to-concrete connection. Plotted from data compiled in Table B.2.

Table B.2. Joint Rotation Vs. Bending Moment for Steel Bracket-to-Concrete Connections

PC6300		PC6400		PC8300		PC8400	
Rotation radians	Moment in.-kips	Rotation radians	Moment in.-kips	Rotation radians	Moment in.-kips	Rotation radians	Moment in.-kips
0.00203	7.0	0.00233	8.1	0.00185	12.9	0.00198	12.2
0.00465	15.0	0.00508	17.2	0.00400	28.9	0.00446	28.5
0.00700	20.9	0.00780	23.8	0.00610	39.6	0.00694	42.0
0.00900	25.7	0.01025	29.9	0.00810	48.7	0.00916	54.1
0.01115	30.3	0.01278	35.3	0.01043	57.7	0.01184	64.7
0.01320	35.0	0.01550	40.5	0.01295	67.1	0.01420	74.0
0.01540	39.4	0.01850	45.0	0.01573	75.6	0.01712	83.5
0.01778	43.9	0.02153	49.6	0.01873	83.2	0.02030	92.2
0.02040	48.1	0.02478	52.8	0.02238	90.3	0.02374	99.8
0.02305	51.6	0.02833	57.2	0.02638	97.2	0.02732	107.3
0.02613	55.5	0.03195	61.1	0.03048	104.6	0.03122	113.8
0.02948	58.9	0.03590	64.7	0.03475	111.5	0.03526	119.4
0.03303	62.1	0.04015	67.9	0.03920	117.9	0.03954	125.3
0.03683	65.2	0.04470	70.9	0.04370	123.9	0.04408	130.4
0.04073	67.8					0.04874	135.1
0.04523	70.4					0.05364	138.9

PC6300 and PC6400 assemblies failed due to concrete crushing that was preceded by significant yielding of tension side steel (see Figure B.5). While most PC8300 and PC8400 failed in a similar fashion, a couple failed (e.g., PC8400 replicates 3 and 4) when high stress concentrations due to excessive deformation resulted in a fracture of the tension steel-to-steel bracket interface as shown in Figure B.6.

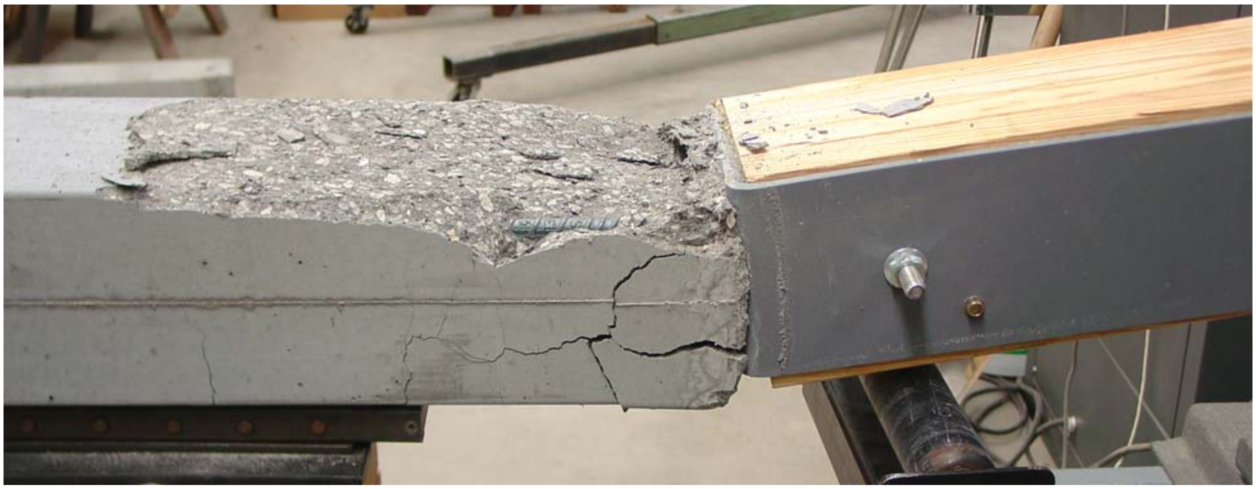

Figure B.5. Typical bending failure mode for steel bracket-to-concrete connection.

Figure B.6. Fracture of tension steel rebar connection in PC8400 replicate 4.

B.3 Discussion

Table B.3 contains a comparison of mean bracket bending strengths to (1) bending strength values for the reinforced concrete sections from test (Appendix A), and (2) calculated ACI nominal bending strength values from Section 3.1. The more realistic comparisons are those for series PC6300 and PC6400 because the failure mode associated with maximum bracket bending strength was the same as that for the reinforced column tests (Appendix A), that is, failure in both cases resulted from concrete crushing after significant yielding of tension steel. As the footnote in Table B.3 points out, comparisons of maximum bracket bending strengths for PC8300 and PC8400 with bending strengths from Appendix A are not as meaningful as strengths reported in Appendix A for those series was controlled by shear (and not bending) strength.

Table B.3. Comparison of Bracket Bending Strength to Column Bending Strength Values

Variable	PC6300	PC6400	PC8300	PC8400
Mean Bending Strength, inch-kips	93.6	103.7	168.0	171.2
Ratio of Mean Bending Strength to Reinforced Concrete Bending Strength As Determined By Test	73.9%	78.0%	85.5%*	75.0%*
Ratio of Mean Bending Strength to ACI Reinforced Concrete Nominal Bending Strength	96.3%	99.2%	85.1%	82.8%

* Percentages inflated because bending strength values from Appendix A for PC8300 and PC8400 are not maximum bending strengths as maximum load was limited by shear strength.

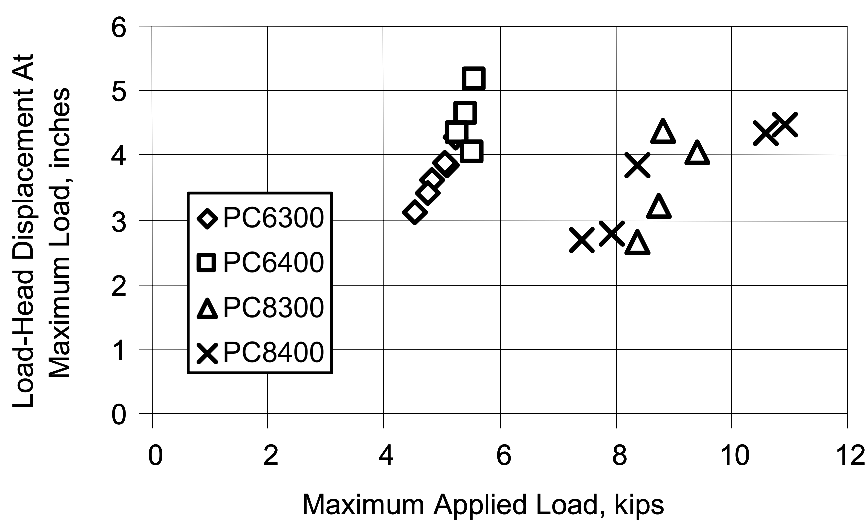

Figure B.7. Relationship between maximum applied load and load-head displacement.

The maximum load values and associated load-head displacements listed in Table B.1 have been plotted in Figure B.7. This figure shows that higher maximum loads are associated with higher displacements at failure. This is logical since load continues to increase as tension steel yields and displacement increases.

The plots in Figure B.3 illustrate that deformation in the joint area accounts for an increasing percentage of total load-head displacement as load increases. At approximately 75% of maximum load, joint rotation accounts for the bulk of the load-head displacement. This is due to the formation of a plastic hinge in the joint region.

The switch from #5 to a #4 rebar in replicates 5 and 6 of the PC8400 series appears to have had a significant affect on joint bending strength when these two replicates are compared to PC8400 replicates 1 and 2 which had similar failure modes. This is expected as joint bending strength is directly related to tension steel cross-sectional area.

Appendix C: Shear Tests

C.1 Test Methods and Equipment

Twelve ***Perma-Columns*** where loaded to failure to determine the shear strength of the assemblies. The 1/3 point loading arrangement shown in Figure C.1 was used. Loads were applied using the Tinius Olson Universal Compression-Tension Testing Machine shown in Figure C.2. Load-head rate was fixed at 0.2 inches/minute. Three specimens of each ***Perma-Column*** Series were tested.

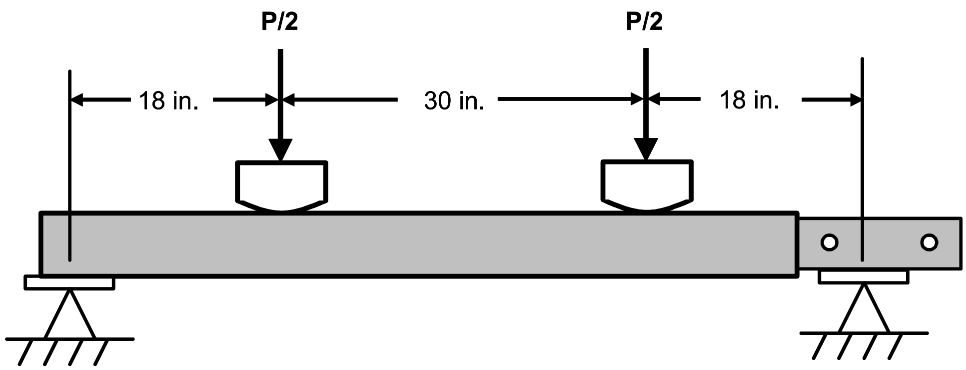

Figure C.1. Test set-up for determining ***Perma-Column*** shear strength.

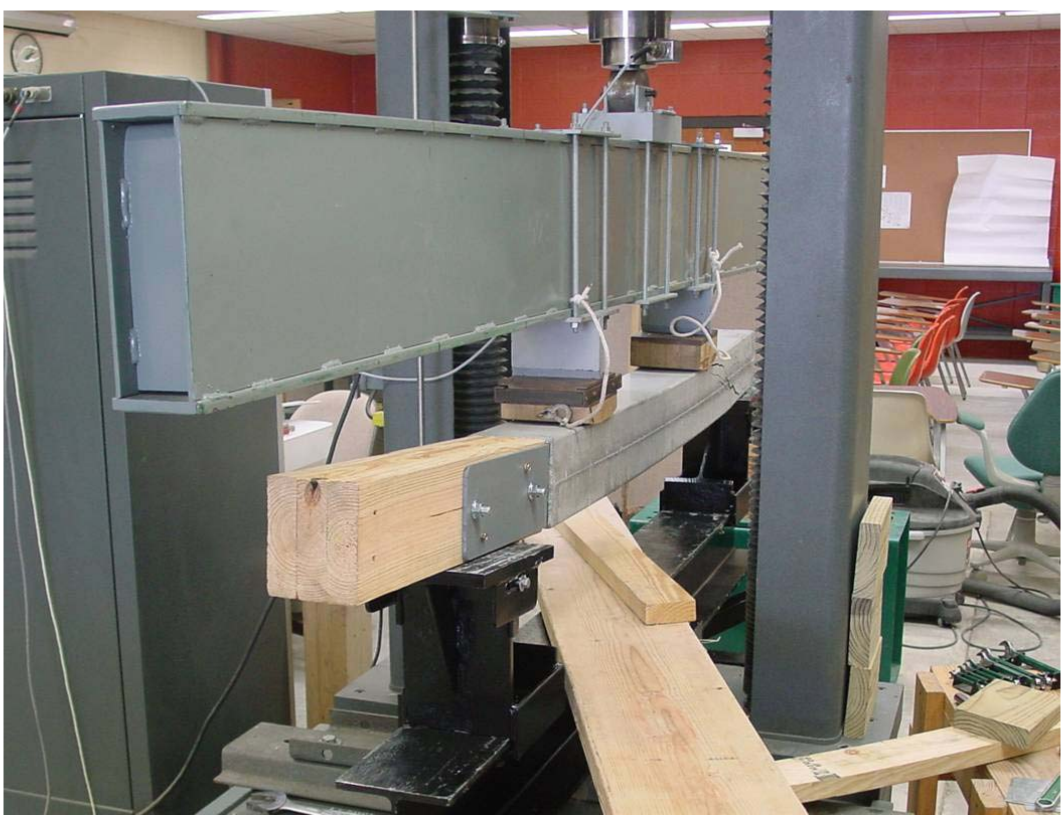

Figure C.2. Load application with a Tinius Olson Universal Compression-Tension Testing Machine.

The 18 inch distance between beam support and beam load point results in a *a/d* ratio of 4.6 for the PC6300 and PC6400 ***Perma-Columns***, and *a/d* ratio of 3.2 for the PC8300 and PC8400 ***Perma-Columns***. As noted in Appendix A, beams with an *a/d* ratio between about 2.5 and 6 fall under the general category of *intermediate length* beams. For intermediate length beams, vertical cracks form first, followed by inclined flexure-shear cracks. At the sudden occurrence of a flexure-shear crack, a beam is not able to redistribute load and additional load can generally not be sustained. The load corresponding to the point at which the flexure-shear crack forms represents the shear strength of the beam (Wang and Salmon, 1985).

C.2 Results

Table C.1 contains the maximum end shear (i.e., ½ total applied load) and the maximum bending moment for each test specimen.

Table C.1 Perma-Column Shear Test Results

	PC6300		PC6400		PC8300		PC8400	
Rep.	Max. Shear Force	Max. Bending Moment	Max. Shear Force	Max. Bending Moment	Max. Shear Force	Max. Bending Moment	Max. Shear Force	Max. Bending Moment
	kips	in.-kips	kips	in.-kips	kips	in.-kips	kips	in.-kips
1	6.14	110.5	5.86	105.6	12.58	226.4	13.74	247.3
2	5.65	101.8	6.43*	115.7*	13.93	250.8	13.93*	250.7*
3	6.12	110.1	6.01	108.2	12.97	233.5	13.59*	244.6*
Average	5.97	107.5	6.10	109.8	13.16	236.9	13.75	247.5
Std Dev	0.27	4.9	0.29	5.2	0.70	12.6	0.17	3.0
COV, %	4.59		4.78		5.30		1.23	

* Bending related failure at bracket end of specimen

There were two distinctly different failure modes associated with the tests. Nine of the twelve specimens exhibited a shear failure on the "non-bracketed" end of the specimen as shown in the top and middle pictures in Figure C.3. Maximum load in the other three specimens (one PC8300 and two PC8400) was limited by a bending related failure of the steel bracket-to-concrete column connection. This failure mode is shown in the bottom photograph in Figure C.3.

C.3 Comparisons

Table C.2 contains a comparison of test results with ACI nominal strengths. Because this test was structured to determine the *shear* strength of the assemblies, the tests should not underestimate maximum *shear* strengths (as was the case with the bending tests reported in Appendix A). However, because of the two bending related failures associated with PC8400 specimens, it could be argued that the test results still underestimate the average maximum shear strength of the PC8400 assemblies. It is for this reason that the word "maybe" appears in the far right column of the Table C.2.

The test-to-ACI shear strength ratios of 2.12 and 1.76 in Table C.2 demonstrate that the average maximum shear strength values for the PC8300 and PC8400 Series Perma-Columns are significantly greater than the ACI nominal strengths. These ratios are also measurably greater than the test-to-ACI shear strength ratios of 1.34 and 1.23 reported for the same column series in Appendix A.

Figure C.3. Inclined cracking associated with shear failure of PC6300 (top) and PC8300 (middle) specimens. Bending failure at the steel bracket-to-concrete column connection (bottom) in a PC8400 specimen.

Table C.2 Comparison of Test Results With ACI Nominal Strengths

Perma-Column Series	Property	Average Test Maximum	ACI Nominal Strength	Ratio, Test/ACI	Test Underestimates Maximum Strength?
PC6300	Shear, kips	5.97	4.24*	1.41	No
	Bending Moment, inch-kips	107.5	97.2	1.11	Yes
PC6400	Shear, kips	6.10	5.42*	1.13	No
	Bending Moment, inch-kips	109.8	104.5	1.05	Yes
PC8300	Shear, kips	13.16	6.22*	2.12	No
	Bending Moment, inch-kips	236.9	197.4	1.20	Yes
PC8400	Shear, kips	13.75	7.83*	1.76	Maybe
	Bending Moment, inch-kips	247.5	206.8	1.20	Yes

* From Table 3.4.1 M_u/V_u = 18 inches

The three bending failures were a direct result of the relatively high loads that the assemblies were able to sustain. Because the shear strengths were higher than originally predicted (based on Appendix A data), bending moments in the vicinity of the steel bracket-to-concrete column connection reached failure levels (see Appendix B) prior to the assembly failing in shear.

C.4 Concrete Compressive Strength

Five 3- by 6-inch concrete cylinders that were cast along with the Perma-Column shear tests specimens were tested 28 days after fabrication (Figure C.4). The resulting compressive strengths were 9656, 11302, 11309, 11116 and 11492 lbf per square inch for an average compressive strength of 10975 psi.

Figure C.4. Compressive strength testing of concrete used in shear test specimens.

Appendix D: Steel Bracket-to-Wood Connection Tests

D.1 Overview

A series of tests were conducted to examine the performance of three different steel bracket-to-wood connections. These tests were conducted for two primary purposes. First, there was interest in determining the relative strength of (1) connections made with two 0.5- by 5.0-inch SAE grade 5 bolts, (2) connections made with twelve 0.25- by 3.0-inch Simpson Strong-Drive® wood screws (a.k.a. SDS screws), and (3) connections featuring two 0.5- by 5.0-inch bolts and four of the 0.25-inch diameter SDS screws. These three connection types are herein referred to as the bolt, screw, and combo connections, respectively. The second objective of these tests was to determine bending moment-rotation relationships for the connections.

The three different connections were made using the steel bracket design shown in Figure D.1. This bracket was designed so that (1) two identical bent plates could be welded together to form a single bracket, and (2) bolt holes, but not screw holes, would line up after the bent plates were welded together. There were six 0.25-inch diameter and two 0.50-inch diameter holes in each plate. The relative arrangement of 0.25-inch diameter holes A, B and C was identical on both ends of the plate. The white dots in Figure D.1 show the location of holes on the backside of the bracket after two identical bent plates were welded together.

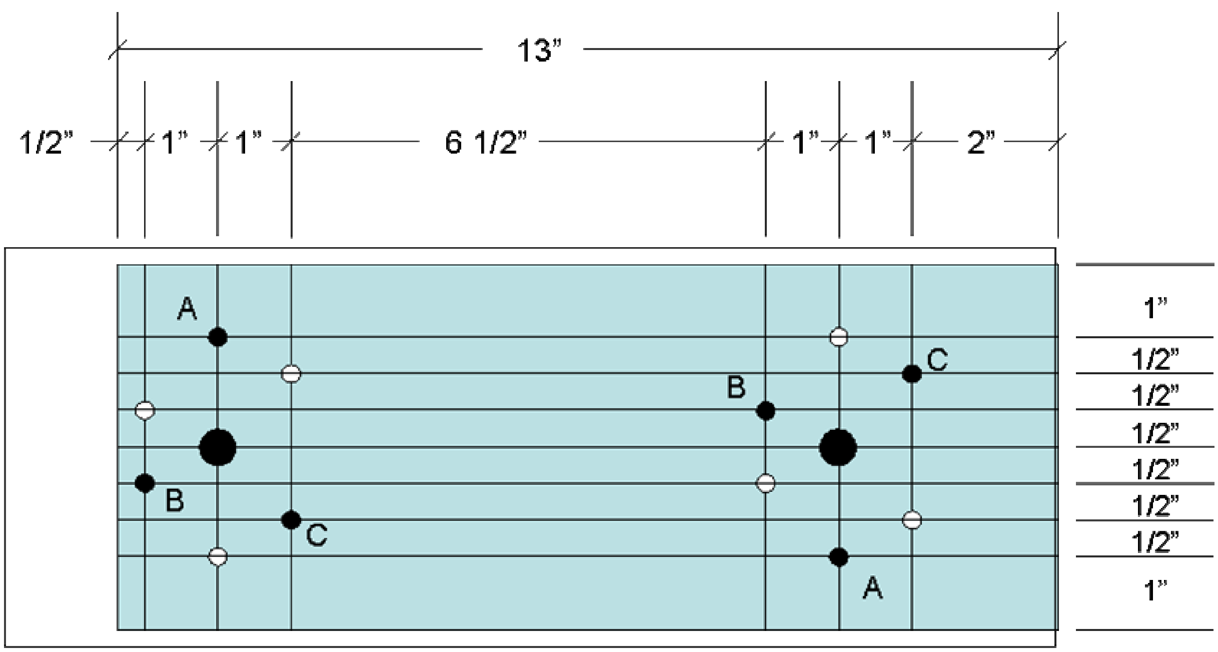

Figure D.1. Dimensions and hole pattern for steel bracket used in connection tests.

The four SDS screws used in the “combo” connection tests were located in holes labeled with an “A” in Figure D.1.

D.2 Wood Properties

Wood properties can significantly affect the strength and stiffness of mechanical connections. Generally, the higher the specific gravity of the lumber (i.e. the denser the lumber), the stiffer and stronger the mechanical connections. For this reason, three different grade/species of lumber were used in connection tests: 1950f-1.7E Southern Pine, No.2 Southern Pine, and No. 2 Spruce-Pine-Fir.

Nine 14-foot long nominal 2- by 6-inch pieces of each lumber grade/species were obtained for this study. The nine pieces were nail-laminated to form three 14-foot long, three-ply assemblies of each lumber grade/species. Each of these assemblies was then loaded to a total load, ***P***, of 3000 lbf using the 1/3 point loading arrangement shown in Figures D.2 and D.3.

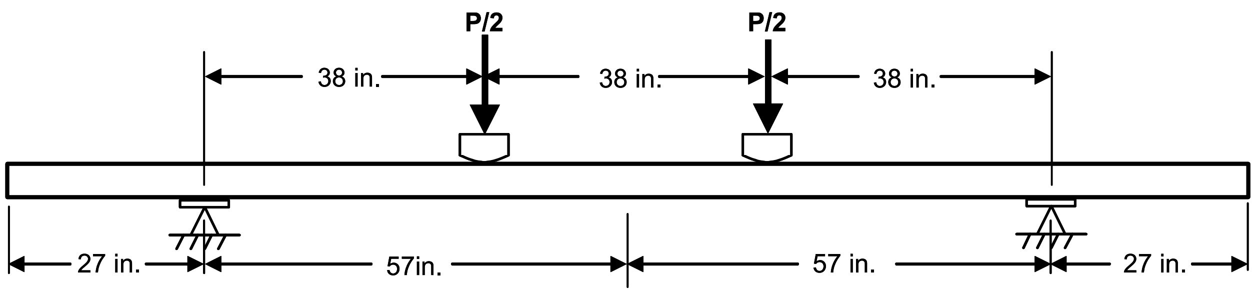

Figure D.2. Load configuration for tests used to determine lumber modulus of elasticity.

Figure D.3. Test set-up for determination of lumber modulus of elasticity.

Load-head rate was fixed at 0.2 inches/minute with midspan displacements measured using a wire deflectometer. The wire deflectometer consisted of two spring-tensioned piano wires (one on each side of the assembly) whose ends were anchored on the neutral axis of the assembly at points directly above the support reactions. The vertical movement of the assembly relative to each wire was measured with a linear variable differential transformer (LVDT).

Ratios of applied load, $\boldsymbol{P}$, to midspan deflection, Δ, were determined by linear regression of all data collected between 300 and 3000 lbf of applied force. These ratios were then substituted into the following equation to determine the apparent modulus of elasticity, $\boldsymbol{E_l}$ of the lumber:

$$\boldsymbol{E_l} = (\boldsymbol{P}/\Delta)(3\boldsymbol{aL}^2 - 4\boldsymbol{a}^3)/(4\boldsymbol{bh}^3) \quad \text{(D.1)}$$

Where:

$\boldsymbol{E_l}$	=	Apparent modulus of elasticity of lumber
$\boldsymbol{P}$	=	Total load applied to the assembly
Δ	=	Deflection at midspan (relative to that at the supports)
$\boldsymbol{a}$	=	Distance from reaction to nearest load point
	=	38 inches
$\boldsymbol{L}$	=	Span of beam
	=	114 inches
$\boldsymbol{b}$	=	Assembly width
	=	4.5 inches
$\boldsymbol{h}$	=	Assembly depth
	=	5.5 inches

Resulting apparent modulus of elasticity values are tabulated in the right column of Table D.1. This table also contains moisture content, bulk density and specific gravity values which were calculated from measurements taken immediately after connection tests. Specifically, each three-ply assembly was weighed and measured to determine its green mass and volume, respectively. Samples were then cut from each ply and oven-dried to determine their moisture content.

Table D.1 Properties Of Lumber Used In Connection Tests

Lumber Species and Grade	3-Ply Assembly Number	Average Moisture Content, percent dry basis	Wet Bulk Density*, lbm/ft^3	Dry Bulk Density*, lbm/ft^3	Specific Gravity, based on oven dry weight and green volume	Apparent Modulus of Elasticity, lbf/in^2
Spruce-Pine-Fir No. 2	1	10.35	33.4	30.2	0.484	1.91E+06
	2	10.31	28.6	25.9	0.415	1.72E+06
	3	10.62	30.3	27.4	0.439	1.89E+06
Southern Pine No.2	1	5.71	31.3	29.6	0.475	1.64E+06
	2	5.93	30.9	29.2	0.467	1.55E+06
	3	6.71	32.2	30.1	0.483	1.74E+06
Southern Pine 1950f 1.7E	1	9.72	38.3	34.9	0.560	2.42E+06
	2	10.2	36.9	33.5	0.537	1.85E+06
	3	11.29	34.2	30.7	0.492	1.47E+06

* Based on green volume

D.3 Connection Test Methods and Equipment

Each 3-ply assembly was cut at midspan and then spliced back together with a pair of steel brackets that had been welded together using special steel splice plates. These specimens were then loaded to failure using the 1/3 point loading arrangement shown in Figures D.4 and D.5. Load-head rate was fixed at 0.2 inches/minute. The same wire deflectometers used to measure the midspan deflection in the MOE tests (see Figure D.3) were used to monitor the vertical displacement of the center of the steel bracket.

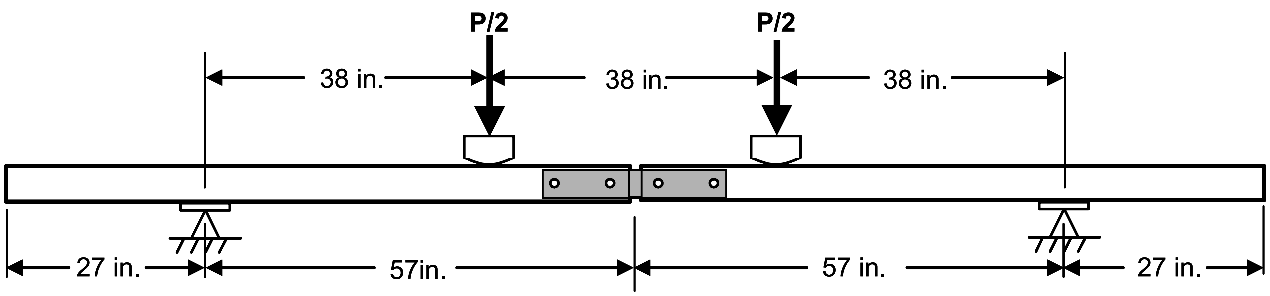

Figure D.4. Load configuration for tests used to determine lumber modulus of elasticity.

Figure D.5. Bolted connection prior to testing showing placement of wire deflectometers (left) and combo connection (connection with bolts and SDS screws) at maximum load (right).

Each 3-ply nail-laminated assembly was used for two connection tests. This was accomplished by removing the fasteners and steel bracket after the first test, and joining together (with a new steel bracket and fasteners) what were the two extreme ends of the bolted assembly during the first test. Using the same 3-ply nail-laminated assembly in two different connection tests resulted in better accounting of differences in connection behavior resulting from changes in lumber properties. The experimental design for these connection tests is summarized in Table D.2. This table identifies the two connection test specimens made from each 3-ply assembly. Note that each connection test specimen is denoted with a three character ID. The first character is a "B", "S" or "C" and identifies the connection as either a bolted, screwed or combination bolt/screw connection, respectively. The second character is an "L", "M" or "H" and designates low, medium or high specific gravity lumber, respectively, which in turns denotes No. 2 SPF, No.2 SP, and 1950f 1.7E SP, respectively. The last character (a 1, 2 or 3) is the number of the 3-ply lumber assembly used to fabricate the connection specimen.

Table D.2 Experimental Design for Connection Tests

Lumber Species and Grade	3-Ply Assembly Number	Specimen Identification		
		Bolted Connection	Screwed Connection	Combo Connection
Spruce-Pine-Fir No. 2	1	BL1	SL1	
	2	BL2		CL2
	3		SL3	CL3
Southern Pine No.2	1		SM1	CM1
	2	BM2		CM2
	3	BM3	SM3	
Southern Pine 1950f 1.7E	1	BH1	SH1	
	2	BH2		CH2
	3		SH3	CH3

D.4 Results

The maximum bending moment sustained by each connection (i.e., midspan bending moment at failure) is given in Table D.3. These values were obtained by multiplying ½ the maximum applied load by the distance between a support and nearest load point (38 inches). Figure D.6 contains a plot of total applied load versus midspan deflection for each of the 18 assembly tests.

Table D.3 Midspan Bending Moment at Failure

Lumber Species and Grade	3-Ply Assembly Number	Midspan Bending Moment, inch-lbf			
		Bolted Connection	Screwed Connection	Combination Connection	Average*
Spruce-Pine-Fir No. 2	1	51870	77650		59590 (11650)
	2	43050		63310	
	3		59770	61900	
Southern Pine No.2	1		67410	71440	58600 (11780)
	2	54055		58520	
	3	38130	62050		
Southern Pine 1950f 1.7E	1	64160	69480		59670 (8740)
	2	45030		62320	
	3		53960	63040	
Average*		49380 (9300)	65060 (8290)	63420 (4290)	59290 (10170)

* Values in parentheses are sample standard deviation values

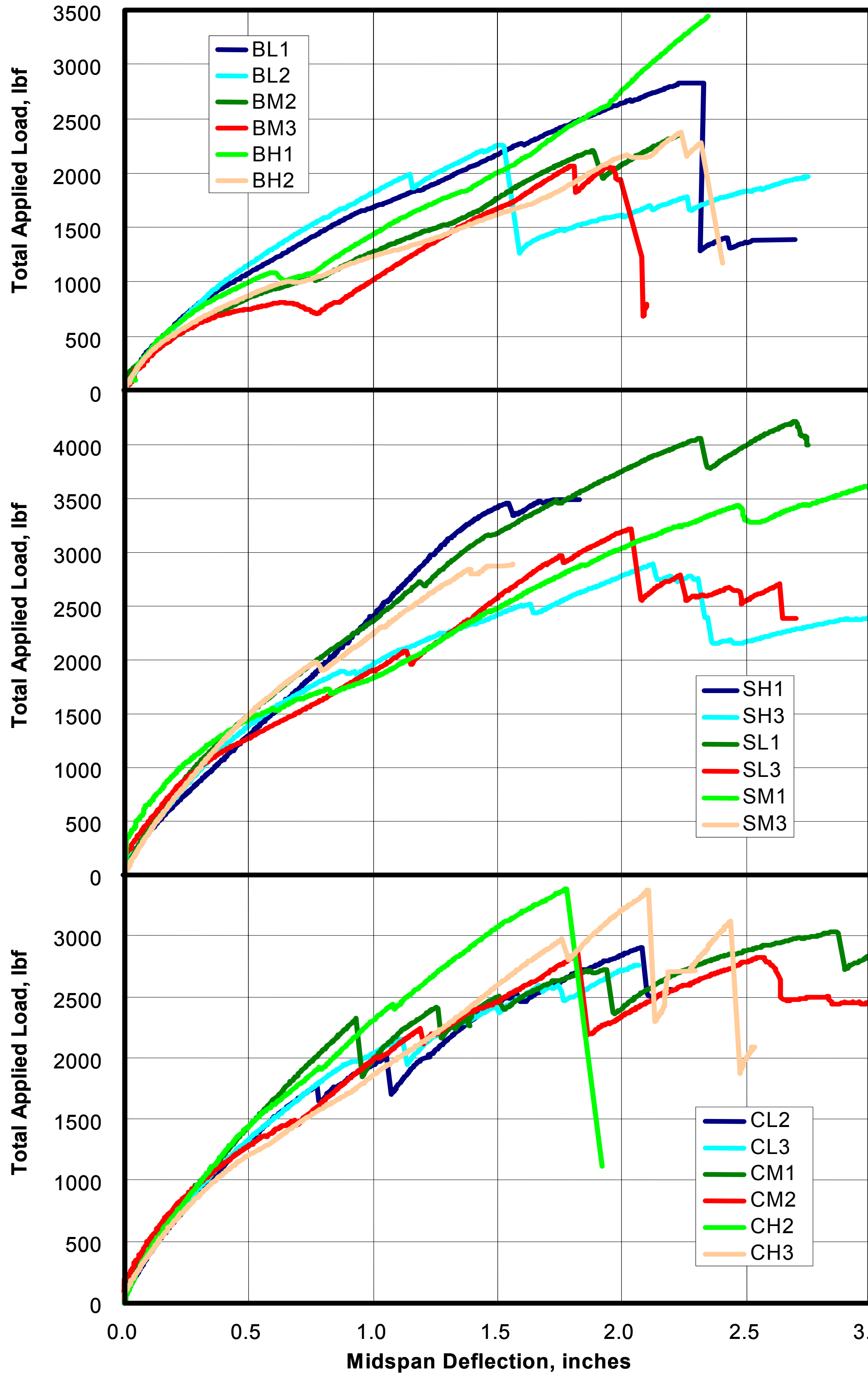

Figure D.6. Midspan deflection as a function of total applied load for each of the 18 connection test specimens.

One of the primary purposes for conducting the connection tests was to determine the relationship between the bending moment applied to the connection and the corresponding rotation between the bracket and wood post. Moment-rotation relationships can be obtained from the applied load versus midspan deflection plots using the lumber modulus of elasticity values in Table D.1 and by assuming (1) no deformation of the steel bracket, and (2) that both halves of each connection behave identically during testing. The assumption of zero bracket deformation is sound because of the manner in which the brackets were fabricated for this series of tests, and because of the negligible strain in the steel under the loads to which the brackets were subjected. The assumption that both halves of each connection behave identically is very reasonable at low load levels and generally good up until the point of a wood or fastener failure. Once a failure occurs, there is likely to be considerably more deformation (i.e., rotation) associated with that half of the connection in which the failure occurred. The second photograph in Figure D.5 illustrates the asymmetrical joint rotation that occurs once a wood or fastener failure occurs.

With the preceding assumptions, the total midspan deflection measured during the tests can be broken into two components, that due to rotation at the connections and that due to bending of the lumber. These two components of deflection are illustrated and identified in Figure D.7 as Δ_r and Δ_w, respectively.

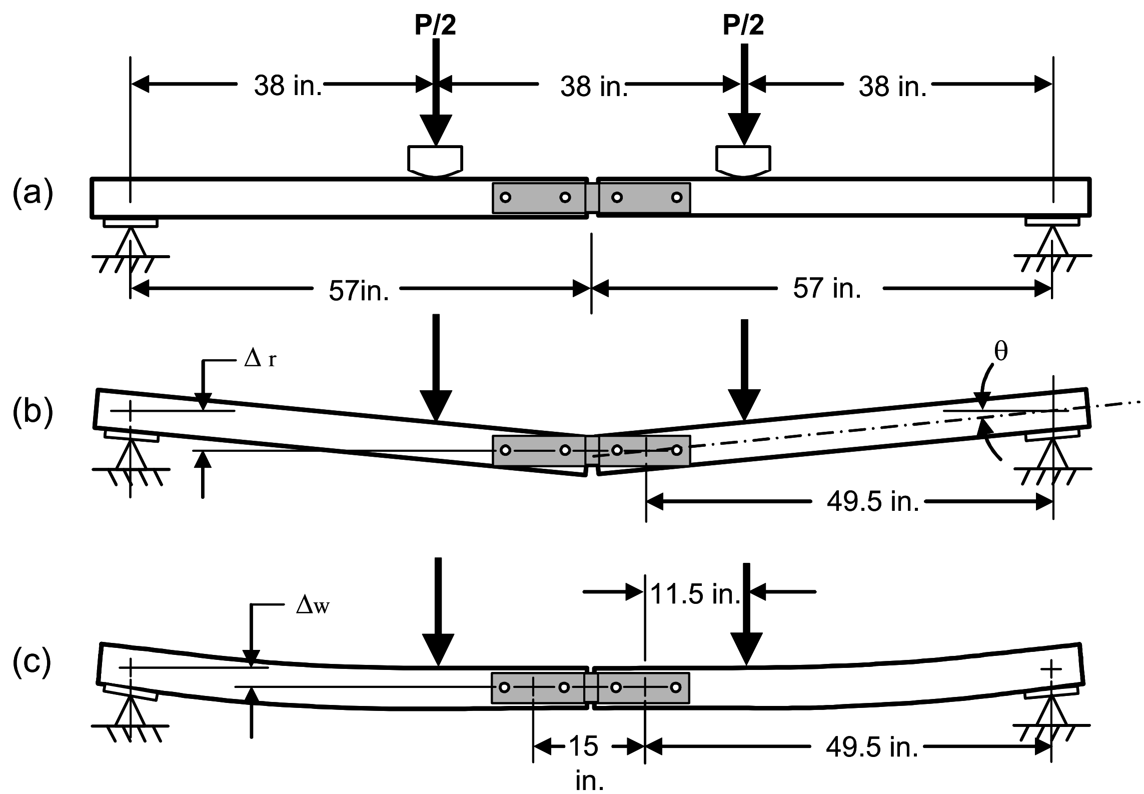

Figure D.7. Components of total midspan deflection include (b) that due to rotation at the connection, Δ_r, and (c) that due to bending strain in the lumber, Δ_w.

The two components of total midspan displacement are calculated as:

$$\Delta_r = \boldsymbol{\theta} \boldsymbol{D} \tag{D.2}$$

$$\Delta_w = (\boldsymbol{P}/\boldsymbol{E_l})(3\boldsymbol{aL}_e^2 - 4\boldsymbol{a}^3)/(4\boldsymbol{bh}^3) \tag{D.3}$$

Where:

Δ_r = Midpsan deflection due to joint rotation
$\boldsymbol{\theta}$ = Joint rotation
$\boldsymbol{D}$ = Distance from support to center of joint rotation
= 49.5 inches
Δ_w = Midspan deflection due to bending of wood posts
$\boldsymbol{E_l}$ = Apparent modulus of elasticity of lumber
$\boldsymbol{P}$ = Total load applied to the assembly
$\boldsymbol{a}$ = Distance from reaction to nearest load point
= 38 inches
$\boldsymbol{L_e}$ = Effective bending span of 3-ply assembly (total assembly length minus effective length of steel bracket)
= 99 inches (114 inches – 15 inches)
$\boldsymbol{b}$ = Assembly width
= 4.5 inches
$\boldsymbol{h}$ = Assembly depth
= 5.5 inches

Because the joint region is in a shear-free region, the center of rotation between each half of the steel bracket and the corresponding section of the 3-ply assembly will be located at the centroid of the fastener pattern. These centroids are 7.5 inches from the midspan of the test specimens, or distance $\boldsymbol{D}$ of 49.5 inches from each support.

Joint rotation, $\boldsymbol{\theta}$, can be calculated be rearranging equation D.2 and equating Δ_r to the difference between total midspan deflection, Δ_t, and Δ_w. In equation form:

$$\boldsymbol{\theta} = [\Delta_t - (\boldsymbol{P}/\boldsymbol{E_l})(3\boldsymbol{aL}_e^2 - 4\boldsymbol{a}^3)/(4\boldsymbol{bh}^3)]/\boldsymbol{D} \tag{D.4}$$

Equation D.4 was used to convert total applied load versus total midspan deflection data to bending moment versus joint rotation data. A portion of this data is compiled in Table D.4 for each test specimen. Average values for each of the three connection types are also compile in Table D.4 and have been plotted in Figure D.8.

Lumber specific gravity values are also tabulated in Table D.4. To determine if specific gravity had a measurable impact on joint stiffness, the specific gravity values were plotted against the bending moment at a rotation of 0.002 (Figure D.9). This plot does not show a strong relationship between specific gravity and joint stiffness for any of the connection types. This was expected given (1) the small range in specific gravity values, and (2) the high variability in joint stiffness (as indicated by the relatively high COV values in Table D.4).

Table D.4 Joint Bending Moment-Rotation Data

Specimen ID	Lumber Specific Gravity	Bending Moment, inch-lbf for Rotation in Radians of 0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.010	0.020
SL1	0.484	8416	14992	19381	22759	26081	29052	31639	34026	38654	60083
SL3	0.439	6026	11581	15925	18662	20867	22566	23849	25291	28042	41689
SM1	0.475	12405	17590	20746	23136	25508	27130	28384	29231	31162	43050
SM3	0.483	7782	13953	18936	23672	27464	30597	33194	35692	36320	54621
SH1	0.560	3879	7923	11272	14102	16941	19726	22524	25224	31090	60535
SH3	0.492	7496	14128	19287	22849	26038	28353	30457	32141	35251	44749
Average	0.489	7667	13361	17591	20863	23817	26237	28341	30268	33420	50788
COV (%)*	8.1	36.9	24.6	19.8	18.1	17.0	16.0	15.2	14.7	11.9	17.1
BL1	0.484	4089	8338	11783	15392	18285	20495	22398	24444	28039	39222
BL2	0.415	3532	7065	10233	13401	15820	17613	19152	20764	23759	35702
BM2	0.467	2327	4653	6980	9241	11024	12606	13937	15269	17348	27172
BM3	0.483	2370	4739	7109	9567	11131	12341	13147	13784	14657	22535
BH1	0.560	3948	7825	10803	12920	14659	16264	17456	18677	20647	31153
BH2	0.537	2629	5257	8005	9990	11710	13231	14544	15630	17889	24888
Average	0.491	3149	6313	9152	11752	13772	15425	16772	18095	20390	30112
COV (%)*	10.5	25.5	25.8	22.4	21.4	21.6	21.2	21.3	22.2	23.9	21.5
CL2	0.415	5752	10965	15049	18534	22002	24945	27455	29914	33499	42313
CL3	0.439	6210	11732	15523	18638	21519	24248	26623	28580	32679	42823
CM1	0.475	5079	10405	14857	19140	23073	26904	30359	33572	39541	42506
CM2	0.467	8363	13995	17333	19855	22059	24041	25692	26904	29049	45205
CH2	0.537	7931	13098	17224	21350	25014	27937	30271	32252	36478	57196
CH3	0.492	6261	11966	15807	18774	21225	22965	24347	26095	29691	48319
Average	0.471	6599	12027	15966	19382	22482	25173	27458	29553	33490	46394
COV (%)*	9.0	19.4	11.1	6.7	5.6	6.2	7.5	8.9	10.0	12.0	12.4

COV (coefficient of variation) in % = 100(Standard Deviation of Values)/(Average of Values)

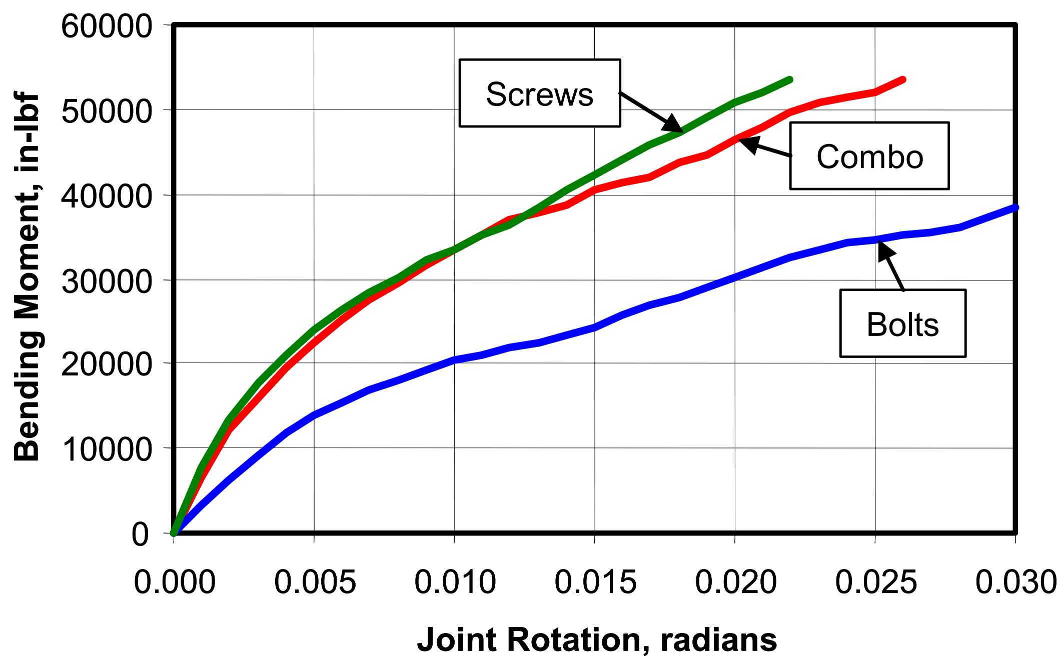

Figure D.8. Average joint bending moment-rotation curves for the three connection types.

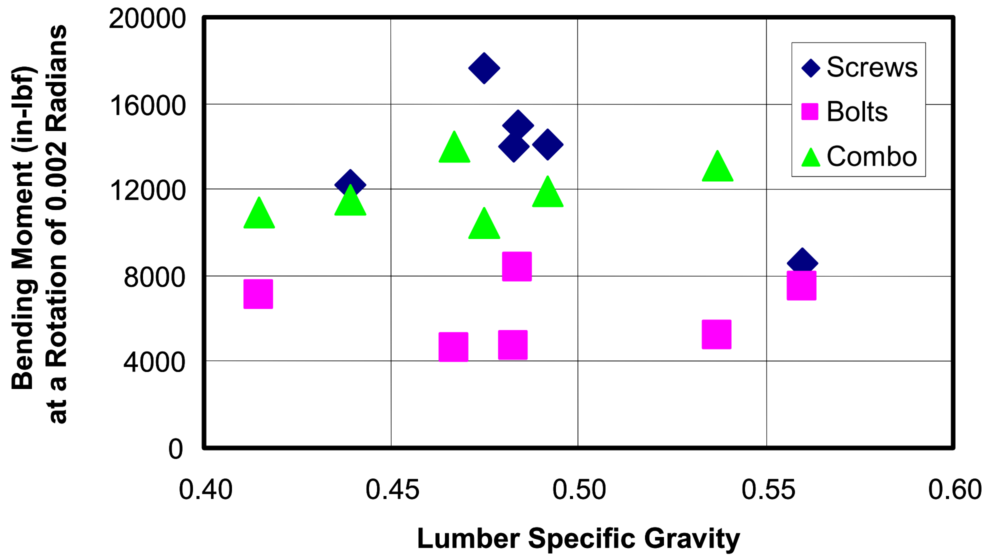

Figure D.9. Relationships between lumber specific gravity values and test specimen bending moments at a joint rotation of 0.002.

D.5 Fastener Shear Strength and Load-Slip Relationships

A joint located in a zone of constant bending moment and no shear will rotate about the centroid of the fastener pattern. For the joints in this study, this point is located midway between the centroid of the two fastener patterns as shown in Figure D.10. From the free body diagram in Figure D.10, it is evident that the shear force, ***F***, in each fastener pattern in numerically equal to the bending moment divided by 8.50 inches. It is also evident from Figure D.10 that the average interlayer slip (i.e., movement between the wood and steel bracket) at the center of each fastener pattern is equal to the product of the rotation and 4.25 inches.

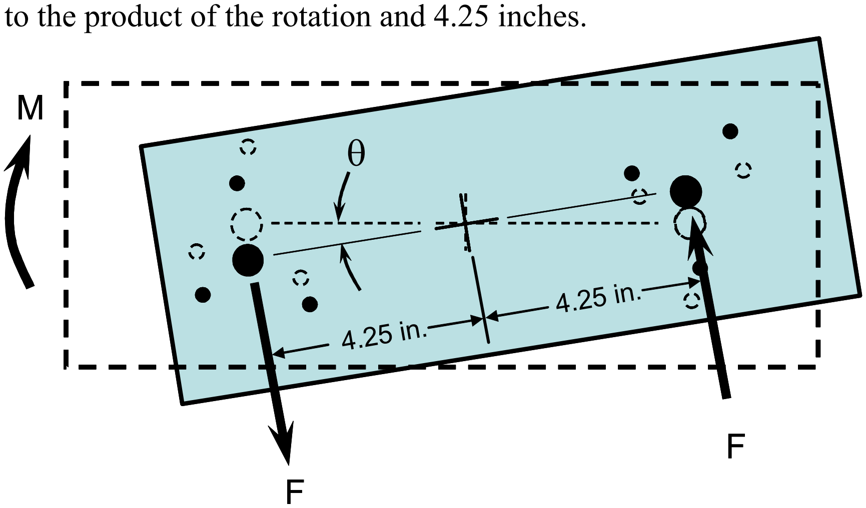

Figure D.10. Average shear force, ***F***, acting on each fastener pattern is numerically equal to the applied bending moment, ***M***, divided by 8.50 inches.

Dividing bending moments by 8.5 inches and multiplying rotations by 4.25 inches yields the interlayer slips and shear forces appearing in Table D.5. Note that shear forces in columns 1, 2 and 3 columns are calculated from the average bending moment in the bolt, screw, and combination connections, respectively. Values in column 4 were obtained by divided values in column 2 by six. Values in column 5 were obtained by doubling the values in column 4 and adding the sum to the values in column 1. Table D.5 data is graphically illustrated in Figure D.11.

Table D.5 Fastener Shear Force As A Function of Interlayer Slip

Interlayer Slip, inches	Shear Force, lbf				
	Single Bolt in Double Shear (1)*	Six Screws in Single Shear (2)*	Single Bolt in Double Shear plus Two Screws in Single Shear (3)*	One Screw in Single Shear (4)*	Single Bolt in Double Shear plus Two Screws in Single Shear (5)*
0.0043	370	902	776	150	671
0.0085	743	1572	1415	262	1267
0.0128	1077	2070	1878	345	1767
0.0170	1383	2455	2280	409	2201
0.0213	1620	2802	2645	467	2554
0.0255	1815	3087	2962	514	2844
0.0298	1973	3334	3230	556	3085
0.0340	2129	3561	3477	593	3316
0.0383	2270	3778	3719	630	3530
0.0425	2399	3932	3940	655	3709
0.0468	2475	4127	4124	688	3851
0.0510	2561	4287	4333	714	3990
0.0553	2642	4511	4445	752	4146
0.0595	2735	4748	4543	791	4318
0.0638	2866	4974	4765	829	4523
0.0680	3023	5172	4869	862	4747
0.0723	3162	5406	4949	901	4964
0.0765	3256	5580	5132	930	5116
0.0808	3407	5773	5261	962	5331
0.0850	3543	5975	5458	996	5534
0.0893	3679	6128	5641	1021	5722
0.0935	3813	6289	5835	1048	5909
0.0978	3941		5965		
0.1020	4028		6052		
0.1063	4085		6133		
0.1105	4143		6280		
Maximum Shear Force	5810	7650	7460	1275	8360

* Values in columns 1, 2 and 3 are for bolt, screw, and combination connections, respectively. Values in column 4 were obtained by divided values in column 2 by six. Values in column 5 were obtained by doubling column 4 values and adding the sum to the values in column 1.

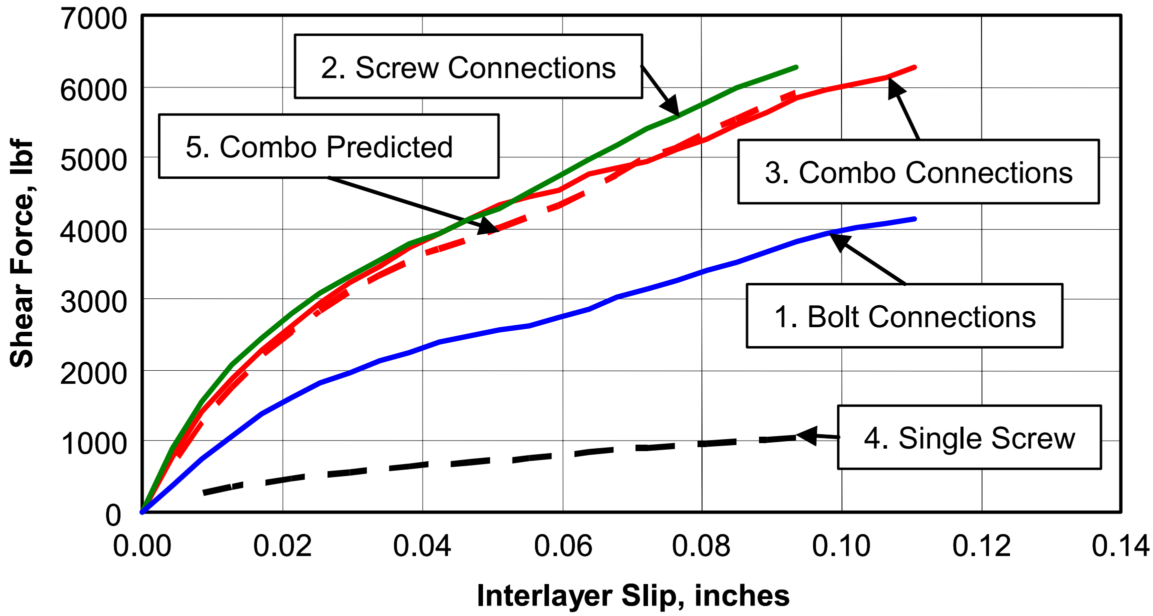

Figure D.11. Graphically display of shear force – interlayer slip data from Table D.5. Numbers in label boxes correspond to Table D.5 column numbers.

The data in Table D.5 and plot in Figure D.11 demonstrate that the stiffness of a connection comprised of a combination of bolts and screws can be predicted using stiffness data for a single bolt plus stiffness data for a single screw.

Since being able to predict connection stiffness is fundamental to modeling connections, least-square linear regression was used to determine the initial slope of the shear-load versus interlayer slip relationship for a single bolt in double shear (column 1 of Table D.5) and for a single screw in single shear (column 4 of Table D.5). Only data points up to a slip of 0.013 inches were used in the regressions. These analyses provided the following interlayer shear stiffness values:

<u>Interlayer Shear Stiffness Values</u>

0.5-inch diameter SAE grade 5 bolt in double shear = 85,500 lbf/inch

0.25- by 3.0-inch Simpson Strong-Drive® wood screw in single shear = 28,700 lbf/inch

The Wood Handbook (FPL, 1999) recommends an interlayer shear stiffness in lbf/inch for bolts or lag screws in wood-to-metal connections of 270,000 $\boldsymbol{D}^{1.5}$ where $\boldsymbol{D}$ is the fastener diameter in inches. This equation yields 95,460 lbf/inch for a 0.5-inch diameter fastener and 33,750 lbf/inch for a 0.25 inch diameter fastener. These values are surprisingly close to those determined via regression analysis given the fact that such values are highly influenced by wood and fastener material properties, and in this study, by friction between the wood and steel brackets.

Table D.5 shows average ultimate shear strengths of 5810 lbf for the 0.5-inch diameter SAE grade 5 bolt in double shear, and 1275 lbf for the 0.25- by 3.0-inch Simpson Strong-Drive® wood screw in single shear. These values compare with calculated 10-minute ($\boldsymbol{C_D}$ = 1.6) NDS allowable stress design (ASD) values of approximately 1950 lbf (1220 lbf x $\boldsymbol{C_D}$ = 1.6) for the SAE grade 5 bolt and approximately 450 lbf (280 lbf x C_D = 1.6) for the SDS screw. The calculated NDS values are approximately 35% of the average ultimate shear strengths.

The 10-minute NDS ASD value of 450 calculated for the SDS screw is based on a shank diameter of 0.242 inches and a bending yield strength of 164,000 lbf/in^2. This 10-minute value is less than the normal load duration (C_D = 1.0) value of 470 lbf published by Simpson for the SDS screw when it is used to connect minimum 14 gage steel to Southern Pine or Douglas Fir. According to Mr. Aram Khachadourian of Simpson Strong-Tie Company, Inc., Simpson arrived at their higher design value by laboratory testing specimens in accordance with ASTM D1761. They then determined connection yield strengths by the 5% diameter offset method (using a root diameter of 0.185 inches), and divided this yield strength by a $K_D = 10D + 0.5 = 2.35$ to obtain the normal load duration (i.e., 10 year) design value (in this case, the K_D value includes the safety factor and adjustment for load duration). This value was lower than the alternative method of dividing the mean ultimate load by a factor of safety of 2.0 and a load duration factor of 1.6. Using the same 0.185 inch diameter as Simpson produces a 5% diameter offset value of 545 lbf (Figure D.12) and a corresponding normal load duration value of 230 lbf for the SDS screw fasteners in this study. This value is less than half of that reported by Simpson. Note that dividing the mean ultimate load of 1275 lbf by a safety factor of 2.0 and load duration factor of 1.6 produces a normal load duration design value of 400 lbf which is much closer to Simpson's design value of 470 lbf.

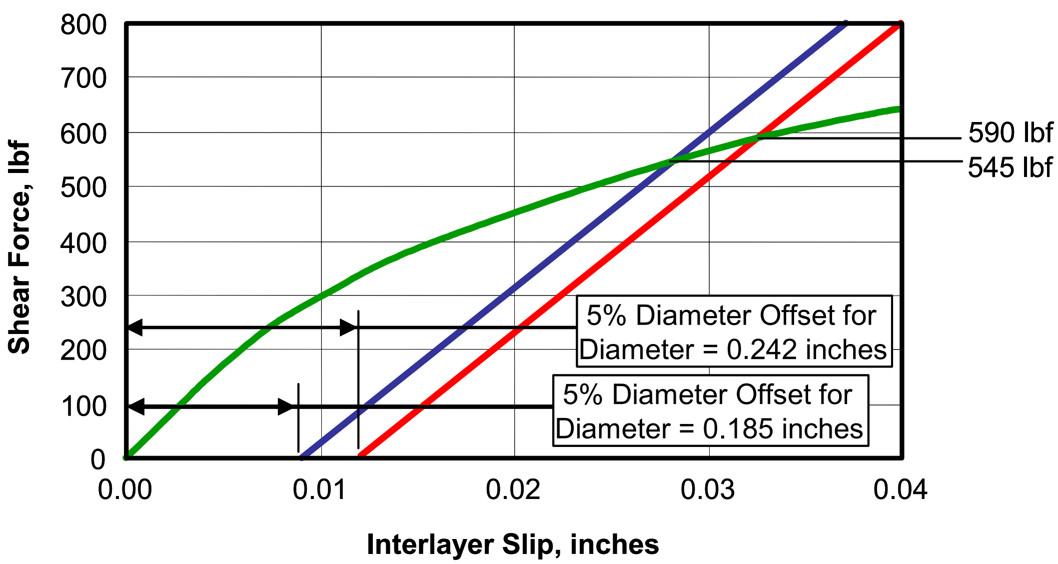

Figure D.12. Determination of the 5% diameter offset values for an SDS screw.

D.6 References

Forest Products Laboratory. 1999. Wood handbook--Wood as an engineering material. Gen. Tech. Rep. FPL-GTR-113. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 463 p.

I. REVOLUTIONARY DESIGN

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B. The exclusive design of the PERMA-COLUMN is patented.

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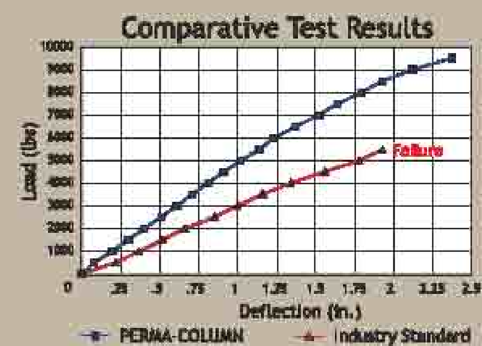
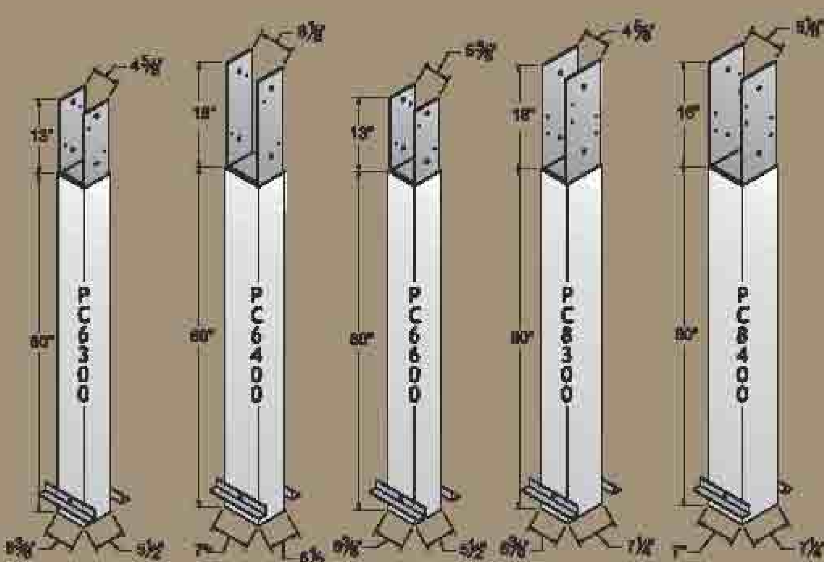


5 MODELS TO MEET YOUR NEEDS



PC6300 - for a 3 ply 2" X 6" laminated wood column

PC6400 - for a 4 ply 2" X 6" laminated wood column

PC6600 - for a 6" X 6" wood post replacement

PC8300 - for a 3 ply 2" X 8" laminated wood column

PC8400 - for a 4 ply 2" X 8" laminated wood column

*Consult a design professional for appropriate model.

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