Perma-Column Design and Use Guide

(PC6300, PC6400, PC6600, PC8300, PC8400 and PC8500 Models)

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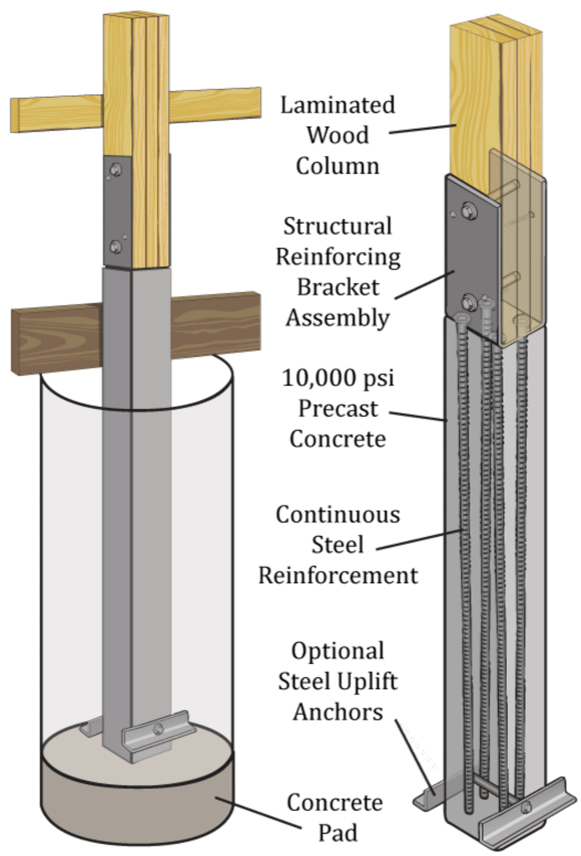
Foreword

The following *Design and Use Guide for PC6300, PC6400, PC6600, PC8300, PC8400, and PC8500 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 by post-frame building engineers and designers as a companion document to the *Engineering Design Manual for Series 6300, 6400, 8300, 8400 Perma-Columns* (herein referred to as "the ***Perma-Column*** Test Report") by David R. Bohnhoff. Each ***Perma-Column*** assembly consists of:

- A reinforced precast concrete base column component designed according to the *Building Code Requirements for Structural Concrete (ACI 318-11)* by The American Concrete Institute (ACI).
- A structural reinforcing bracket assembly designed according to the *Steel Construction Manual (14th Edition)* by The American Institute of Steel Construction (AISC).
- A laminated or solid sawn wood column component designed according to the *2012 Edition of The National Design Specification for Wood Construction (NDS)* by the American Wood Council (AWC).

Structural analysis was performed using both the load and resistance factor (LRFD) and the allowable stress/strength design (ASD) methodologies. This was done so the laboratory results from the ***Perma-Column*** Test Report, which is based on LRFD, could be used, while also expressing the expected column performance in terms of maximum allowable capacity as is customary in the wood industry.

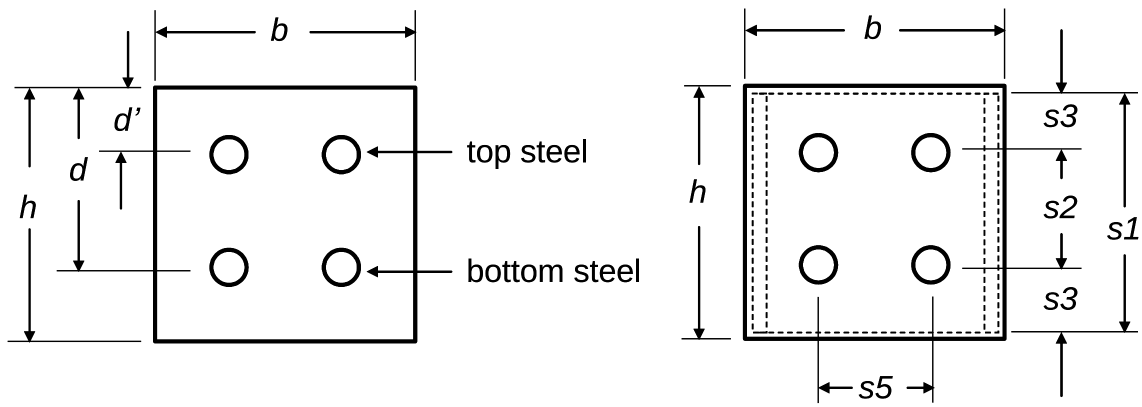
This Design and Use Guide will cover properties and design procedures for the reinforced concrete base, the structural reinforcing bracket assembly connection, and the laminated wood columns. The procedure for creating models of the ***Perma-Column*** assemblies to simulate the results of laboratory testing is discussed. A Design Chart is presented for the ***Perma-Column*** assemblies with varying heights and boundary conditions. The failure modes and design limitations on each ***Perma-Column*** assembly are listed, and an example is given showing a straight forward design approach which can be applied to all ***Perma-Column*** assemblies. Finally, wind uplift resistance is calculated for a concrete collar or a compacted fill foundation condition.

2. ***Perma-Column*** Descriptions and Properties

Dimensions and material properties for the PC6300, PC6400, PC6600, PC8300, PC8400 and PC8500 models are given in Table 2.1. Variable definitions correspond to Figure 1.1 of the ***Perma-Column*** Test Report. The PC6600 and PC8500 were not included in the laboratory testing. The PC6600 is intended for new or replacement solid-sawn 6x6 posts, and the PC8500 is to be used with a 5-ply 2x8 laminated wood column or 8x8 wood post.

Tables 2.2 through 2.4 give dimensions and section properties for several different wood column sizes and types that are included in this report: 6x6 solid-sawn; and 3-ply 2x6, 4-ply 2x6, 3-ply 2x8, 4-ply 2x8 and 5-ply 2x8 mechanically laminated and glued laminated (glulam) wood columns. The mechanically laminated group consists of #1 Southern Yellow Pine (SYP) and of #2 and better Spruce Pine Fir (SPF) lumber using standard dressed sizes (surfaced four sides (S4S)); as well as #1 SYP laminations which have been further planed for better visual appearance. The glulam group consists of SYP laminations which have been planed down as part of their standard fabrication process. ***Perma-Column*** models for use with glulam columns are identified with a "GL" at the end of the name. These models have a reduced inside dimension so as to fit tightly with the glulam products.

Table 2.1: PC6300, PC6400, PC6600, PC8300, PC8400 and PC8500 Dimensions and Properties



Variable	PC6300	PC6400	PC6600	PC8300	PC8400	PC8500
Concrete Width, *b* (in)	5.38	6.88	6.38	5.38	6.88	8.31
Concrete Depth, *h* (in)	5.44	5.44	5.44	7.19	7.19	7.19
Depth to Top Steel, *d'* (in)	1.50	1.50	1.50	1.56	1.56	1.56
Depth to Bottom Steel, *d* (in)	3.94	3.94	3.94	5.62	5.62	5.62
Width of Steel Bracket, *s1* (in)	5.00	5.00	5.00	7.00	7.00	7.00
Top & Bottom Steel Spacing, *s2* (in)	2.44	2.44	2.44	4.06	4.06	4.06
Steel Distance to Bracket Edge, *s3* (in)	1.28	1.28	1.28	1.47	1.47	1.47
Area of Top Steel, A_s' (in.^{2})	0.40	0.40	0.40	0.62	0.62	0.62
Area of Bottom Steel, A_s (in.^{2})	0.40	0.40	0.40	0.62	0.62	0.62
Steel Yield Strength, f_y (lbf/in.^{2})	60,000	60,000	60,000	60,000	60,000	60,000
Concrete Comp. Strength, f_c' (lbf/in.^{2})	10,000	10,000	10,000	10,000	10,000	10,000
Steel MOE, E_s (lbf/in.^{2})	29000000	29000000	29000000	29000000	29000000	29000000

Table 2.2: Standard S4S (Surfaced Four Sides) Wood Column Dimensions and Properties

Property	6x6	3ply x 6	3ply x 8	4ply x 6	4ply x 8	5ply x 8
Width, b (in)	5.50	4.50	4.50	6.00	6.00	7.50
Depth, d (in)	5.50	5.50	7.25	5.50	7.25	7.25
Area, A (in^2)	30.25	24.75	32.63	33.00	43.5	54.38
Section Modulus, S (in^3)	27.73	22.69	39.42	30.25	52.56	65.70
Moment of Inertia, I (in^4)	76.26	62.39	142.90	83.19	190.54	238.17

Table 2.3: Planed Wood Column Dimensions and Properties

Property	3ply x 6	3ply x 8	4ply x 6	4ply x 8	5ply x 8
Width, b (in)	4.50	4.50	6.00	6.00	7.50
Depth, d (in)	5.31	7.19	5.31	7.19	7.19
Area, A (in^2)	23.90	32.36	31.86	43.14	53.93
Section Modulus, S (in^3)	21.15	38.77	28.20	51.70	64.62
Moment of Inertia, I (in^4)	56.15	139.39	74.86	185.85	232.31

Table 2.4: Glulam Column Dimensions and Properties

Property	3ply x 6	3ply x 8	4ply x 6	4ply x 8	5ply x 8
Width, b (in)	4.063	4.063	5.375	5.375	6.72
Depth, d (in)	5.25	7.00	5.25	7.00	7.00
Area, A (in^2)	21.33	28.44	28.22	37.63	47.04
Section Modulus, S (in^3)	18.66	33.18	24.69	43.90	54.88
Moment of Inertia, I (in^4)	49.0	116.12	64.81	153.64	192.08

Figure 2.1 shows the orientation of the column laminations in relation to the load. Wind load is taken by uniaxial bending about axis Y-Y. The provisions of this design guide do not apply to columns subject to biaxial bending. Figure 2.2 is a definition sketch showing embedment depth, orientation of the column, and direction of wind loading on the assembly. The ***Perma-Column*** assembly is assumed to be braced in the out-of-plane direction by girts spaced 24 inches on center. Wind load calculations for a sample post-frame building per ASCE 7-05 as referenced in IBC 2009 and ASCE 7-10 as referenced in IBC 2012 are included in Appendix 1.

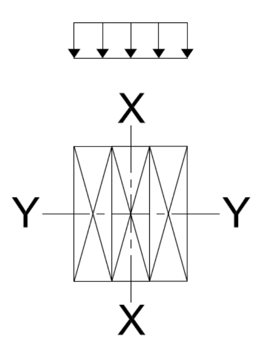


Figure 2.1 Wood Column Orientation

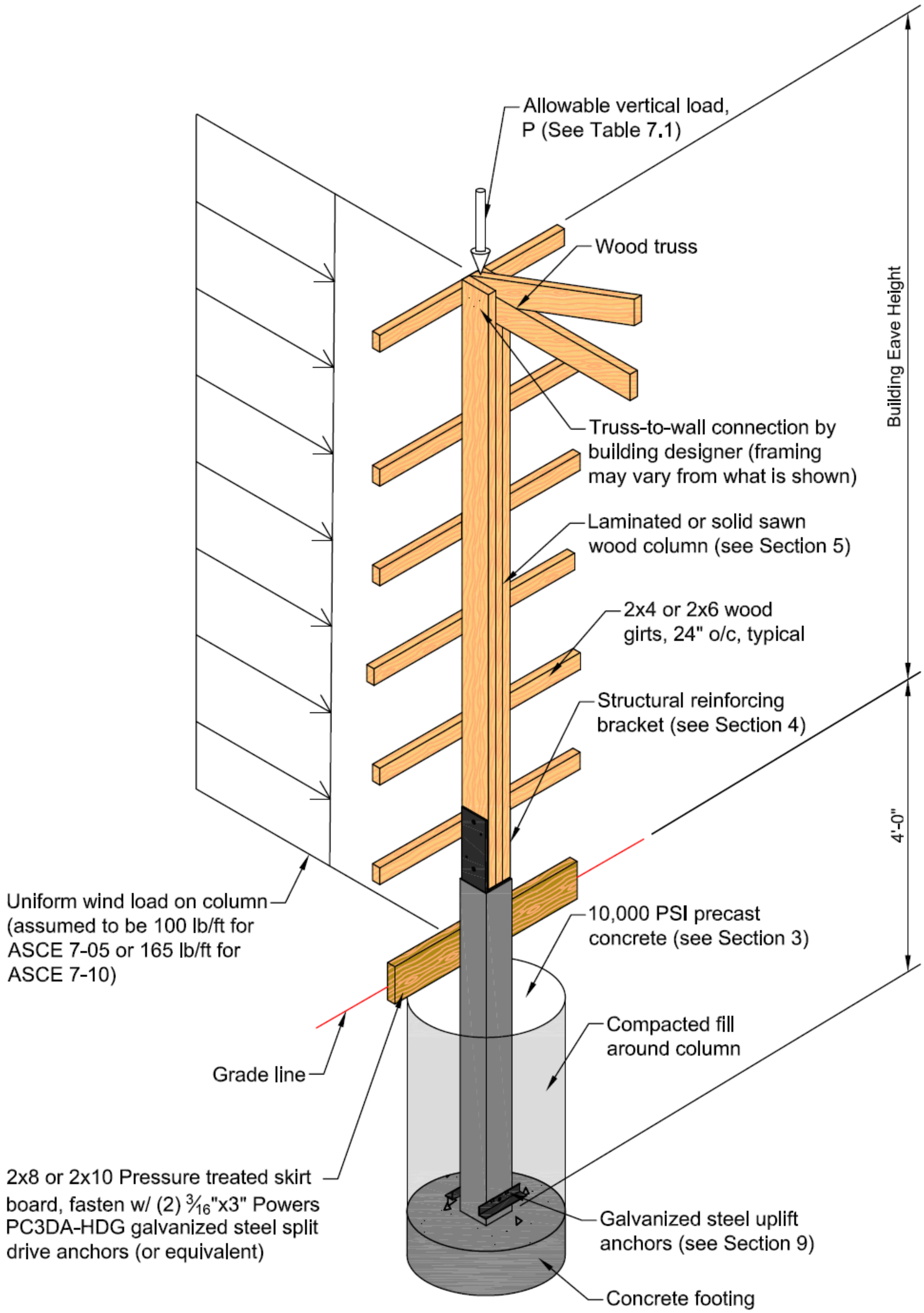


Figure 2.2 ***Perma-Column*** **load definition sketch**

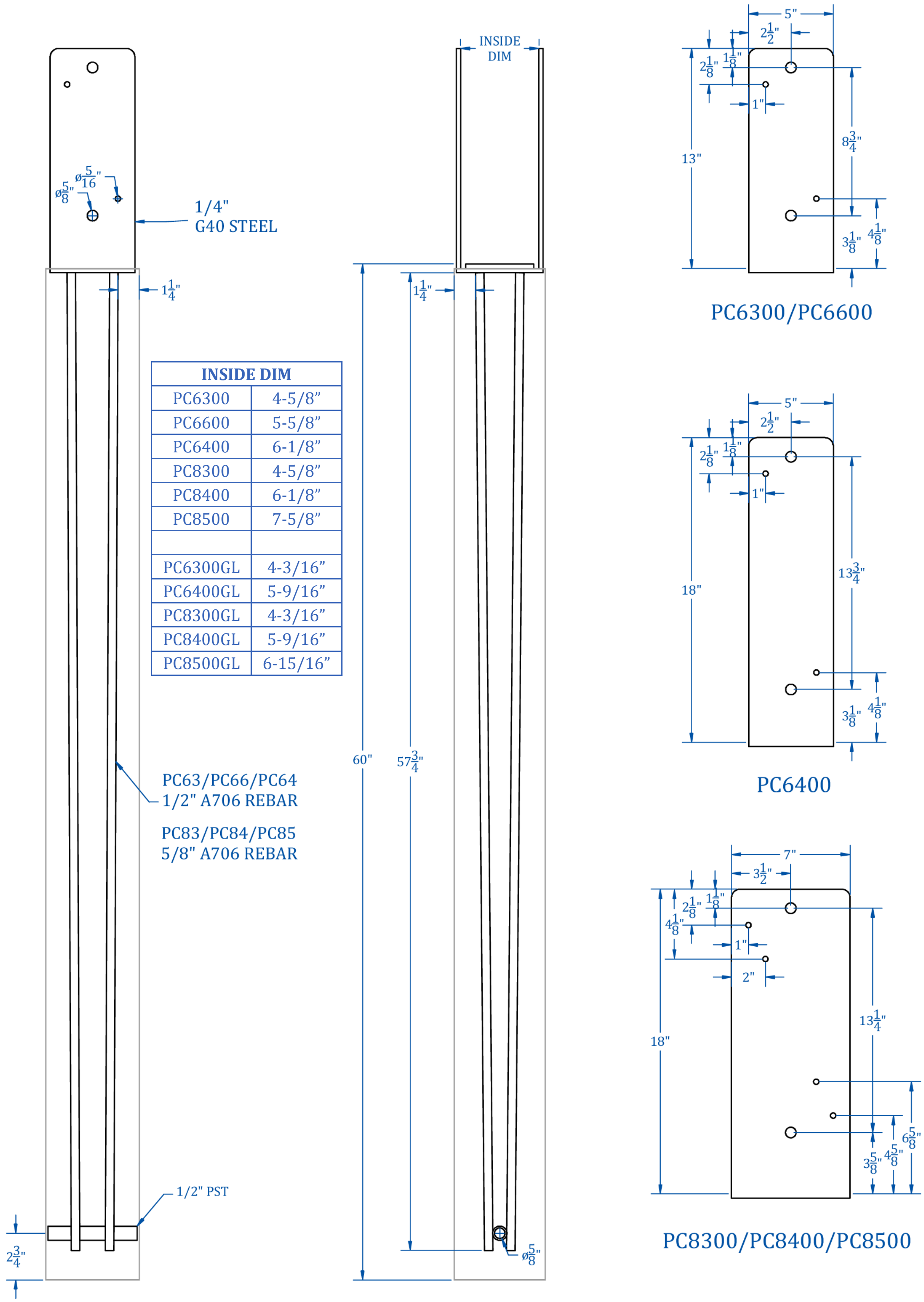
3. Reinforced Concrete Base Column Design

The reinforced concrete component is manufactured with 10,000 psi (nominal) precast 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, PC8400, and PC8500 models. The required concrete cover for reinforcing bars in precast concrete is less than cast-in-place concrete because of better placement accuracy during the manufacturing process. Each of the ***Perma-Column*** models meet the minimum concrete cover of 1.25 inches required for precast concrete components that are exposed to earth or weather. The high concrete strength and quality is achieved by adding superplasticizer, which increases 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.

4. Structural Reinforcing Bracket Assembly Design

Figure 4.1 shows dimensions for the different structural reinforcing bracket assemblies that are used with the ***Perma-Column*** assemblies. The brackets consist of ¼" structural grade 40 steel (F_y = 40 ksi) 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. Typically, one screw is installed from each side of the bracket at each bolt, except the PC8300, PC8400 and PC8500 have two screws on each side at each bolt. These screws help prevent stress concentrations around the bolt which would cause splitting of the wood members. The wood column bears directly on a ¼" steel seat plate which helps to transfer axial loads 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.

The internal forces of shear and bending moment are transferred from the wood column through the steel bracket into the concrete base. Section 4.2 of the ***Perma-Column*** Test Report states that as long as the reinforced concrete portion of the assembly is shown to adequately handle shear forces, there is no need to check the shear capacity of the bracket. This is because all shear related failures observed in laboratory testing occurred in the reinforced concrete base and not in the bracket. However, it is important to check the bending moment capacity and rotational stiffness of the bracket.



INSIDE DIM	
PC6300	4-5/8"
PC6600	5-5/8"
PC6400	6-1/8"
PC8300	4-5/8"
PC8400	6-1/8"
PC8500	7-5/8"
PC6300GL	4-3/16"
PC6400GL	5-9/16"
PC8300GL	4-3/16"
PC8400GL	5-9/16"
PC8500GL	6-15/16"

Figure 4.1 Structural Reinforcing Bracket Assemblies

4.1 Bracket Moment Capacity

This joint has significant bending moment capacity and does not need to be modeled as a pin. The laboratory testing for this joint was completed using a 1/3 point loading arrangement that isolates the shear strength and bending strength of the joint as described in Appendix B of the ***Perma-Column*** Test Report. It is important to note that testing was performed for strong axis bending only. The bending strength of both the concrete-to-steel bracket and the steel bracket-to-wood column connections must be evaluated in order to determine the overall 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 ***Perma-Column*** Test Report). Nomenclature for this section is taken from AISC. The nominal bending strength, M_n, of the concrete-to-steel bracket connection is given in Table 4.1a, and is defined in the following expression: $M_n = M_{0.03}$, where $M_{0.03}$ is the bending moment corresponding to 0.03 radians rotation in Table B.2 of the ***Perma-Column*** Test Report. 0.03 radians was chosen as a limit state as recommended in Section 4.1 of the ***Perma-Column*** Test Report. For Strength Design (LRFD), the *design flexural strength* of the concrete-to-steel bracket connection is expressed as: $\varphi_b M_n$, where $\varphi_b = 0.90$; and for Allowable Stress Design (ASD), the *allowable flexural strength* is expressed as: $M_n/\mathbf{\Omega}_b$, where $\mathbf{\Omega}_b = 1.67$.

The bolts and screws transfer shear and moment between the steel bracket and wood column. The shear strength of the fasteners themselves, not the steel bracket, control the bending strength of this joint. The bolt and screw design, as well as the nomenclature for moment capacity using LRFD and ASD, is done according to the 2012 NDS. Forces from the moment couple are applied to the upper and lower fastener groups and are distributed to the bolt and screws according to their respective stiffness as described in Section 6.1 of the ***Perma-Column*** Test Report. The unadjusted moment capacity, M, of the steel bracket-to-wood column connection is given in Table 4.1b and is determined by calculation (See Appendix 2 for calculations). The bolt yield strength was taken as the proof stress for an A325 bolt. For Strength Design (LRFD), the *design moment capacity* of the steel bracket-to-wood column connection is expressed as: $M' = M(K_F)(\phi_z)(\lambda)$, where $K_F = 3.32$, $\varphi_z = 0.65$ and $\lambda = 1.0$; and for Allowable Stress Design (ASD), the *adjusted allowable moment capacity* is expressed as: as: $M' = M(C_D)$, where $C_D = 1.6$ for wind loading.

Table 4.1a: Concrete-to-Steel Bracket Connection Bending Strength (in-kip)

Series	**Nominal Bending Strength,** M_n[1]	**Design Flexural Strength (LRFD)**[2] $\varphi_b M_n$	**Allowable Flexural Strength (ASD)**[3] $M_n/\mathbf{\Omega}_b$
PC6300	59.4	53.5	35.6
PC6400	59.0	53.1	35.3
PC6600	59.0	53.1	35.3
PC8300	103.7	93.3	62.1
PC8400	111.8	100.6	67.0
PC8500	111.8	100.6	67.0

Notes:
1. *Interpolated from Table B.2 in the* ***Perma-Column*** *Test Report at $\theta = 0.03$*
2. *$\varphi_b = 0.90$ (from AISC)*
3. *$\Omega_b = 1.67$ (from AISC)*

Table 4.1b: Steel Bracket-to-Wood Column Connection Bending Strength (in-kip)[4]

Series	Unadjusted Moment Capacity[1] M	Design Moment Capacity (LRFD)[2] $M' = M(K_F)(\phi_z)(\lambda)$	Adjusted Allowable Moment Capacity (ASD)[3] $M' = M(C_D)$
PC6300	18.2	39.3	29.1
PC6400	29.3	63.3	46.9
PC6600	18.2	39.3	29.1
PC8300	35.4	76.4	56.6
PC8400	35.4	76.4	56.6
PC8500	35.4	76.4	56.6

Notes:
1. *From Calculations in Appendix 2*
2. $K_F = 3.32$; $\varphi_z = 0.65$; $\lambda=1.0$ *(all other factors are 1.0 per NDS)*
3. $C_D = 1.6$ *(all other factors are 1.0 per NDS)*
4. *For Southern Pine lumber or timber*

Comparing the corresponding columns of Tables 4.1a and 4.1b, it is clear that the steel bracket-to-wood column connection controls the bending strength of the joint in most cases. The design flexural strength (LRFD) of the PC6400 concrete-to-steel bracket connection is 53.1 in-kip from Table 4.1a, and the design moment capacity (LRFD) is 63.3 in-kip from Table 4.1b. 53.1 < 63.3 so the Table 4.1a value controls. For all other cases, the bending strength given in Table 4.1b for the steel-to-wood connection controls.

4.2 Bracket 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 ***Perma-Column*** Test Report (also in Appendix 3 of this document) show joint rotation versus bending moment data for the steel bracket-to-concrete connection. As indicated in Section 4.1, the steel bracket-to-wood connection controls the allowable flexural strength of the bracket for all cases except the PC6400. From Figure B.4 of the ***Perma-Column*** Test Report, the allowable flexural strength of the steel-to-wood column connection occurs approximately at 0.01 radians rotation. Therefore, the rotation, θ, of 0.01 radians, and the corresponding bending moments, M, should be used to establish the rotational stiffness of each bracket defined as M/θ. This point on the joint rotation versus bending moment curve (See Appendix 3) best represents the stiffness the bracket will have when loaded. Table 4.2 shows the calculated stiffness values for the concrete-to-steel and the steel-to-wood joints. These stiffness values are needed in order to create a model per 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 ***Perma-Column*** Test Report. The slip modulus should be assigned to the fastener group by taking the sum of the values of the individual fasteners in the group. The slip modulus for the ½" bolt in double shear is 85.5 kips per inch, and for each screw is 28.7 kips per inch. The actual distance between the centroids of the fastener groups can be calculated based upon the stiffness and location of each fastener. This value is given in Table 4.2 for each model and is also included on a sketch in Appendix 3.

Table 4.2: Steel Bracket Connection Stiffness Values

Series	Concrete-to-Steel Bracket Stiffness, $M_{0.01}/\theta$ (in-kip/rad)[1]	Steel Bracket-to-Wood Column Stiffness (in-kip/rad)[2]	Distance Between Fastener Groups Based Upon Stiffness (in)[2]
PC6300	2570	4570	8.25
PC6400	2990	12075	13.25
PC6600	2990	4570	8.25
PC8300	5770	12120	11.6
PC8400	6470	12120	11.6
PC8500	6470	12120	11.6

Notes:

1. *Calculated from Table B.2 in the **Perma-Column** Test Report*
2. *From Calculations in Appendix 3*

4.3 Friction

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

5. Laminated Wood Column Design

The wood portion of the ***Perma-Column*** assembly is designed using the ASD methodology in accordance with the *NDS by AWC* as discussed in Section 1 of this guide. The wood design values used in this work are shown in Tables 5.1 through 5.3. Southern Pine design values for mechanically laminated columns are taken from the Southern Pine Inspection Bureau (SPIB) Supplement Number 13 which went into effect June 1, 2013. Design procedures were taken from ASAE EP559.1 *Design Requirements and Bending Properties for Mechanically-Laminated Wood Assemblies*. Design procedures for glulams were taken from the *NDS by AWC.* Table 5.4 contains adjustment factors to be applied to the wood design values.

Table 5.1: #1 Southern Pine Standard S4S (Surfaced Four Sides) Wood Column Design Values

Property	6x6	3ply x 6	3ply x 8	4ply x 6	4ply x 8	5ply x 8
Flexure, F_b (psi)[1]	1350	1350	1250	1350	1250	1250
Shear, F_v (psi)	165	175	175	175	175	175
Axial Compression, F_c (psi)	990	1550	1500	1550	1500	1500
Modulus of Elasticity, E ($\times 10^6$ psi)	1.3	1.6	1.6	1.6	1.6	1.6
Minimum MOE, E_{min} ($\times 10^6$ psi)	0.47	0.58	0.58	0.58	0.58	0.58

Table 5.2: #1 Southern Pine Planed Wood Column Design Values

Property	3ply x 6	3ply x 8	4ply x 6	4ply x 8	5ply x 8
Flexure, F_b (psi)[1]	1350	1250	1350	1250	1250
Shear, F_v (psi)	175	175	175	175	175
Axial Compression, F_c (psi)	1550	1500	1550	1500	1500
Modulus of Elasticity, E ($\times 10^6$ psi)	1.6	1.6	1.6	1.6	1.6
Minimum MOE, E_{min} ($\times 10^6$ psi)	0.58	0.58	0.58	0.58	0.58

1. *Bending design values have been adjusted for size (C_F)*

Table 5.3: Glulam Column Design Values[1]

Property	3ply x 6	3ply x 8	4ply x 6	4ply x 8	5ply x 8
Flexure, F_b (psi)	2050	1900	2350	2350	2350
Shear, F_v (psi)	300	300	300	300	300
Axial Compression, F_c (psi)	2150	2150	2150	2150	2150
Modulus of Elasticity, E ($x10^6$ psi)	1.7	1.7	1.7	1.7	1.7
Minimum MOE, E_{min} ($x10^6$ psi)	0.88	0.88	0.88	0.88	0.88

1. *Design values from published values of a representative manufacturer*

Table 5.4: Adjustment Factors for Design Values

Variable		Load Duration Factor	Wet Service Factor		Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Repetitive Member Factor (not used for Glulam Columns)		Column Stability Factor
		C_D (NDS)	C_M (NDS)		C_t (NDS)	C_L (NDS)	C_F (NDS)	C_{fu} (NDS)	C_r (VG, ASAE EP559.1)		C_P (NDS)
			Wet	Dry					3-ply	4-ply & 5-ply	
$F_b' = F_b$	x	1.60	0.85	1.00	1.00	1.00	1.00	1.00	1.35	1.40	-
$F_t' = F_t$	x	The columns in this analysis are not loaded in tension, this section does not apply									
$F_v' = F_v$	x	1.60	0.97	1.00	1.00	-	-	-	-	-	-
$F_{cp}' = F_{cp}$	x	-	0.67	1.00	1.00	-	-	-	-	-	-
$F_c' = F_c$	x	1.15	0.80	1.00	1.00	-	1.00	-	-	-	Varies
$E' = E$	x	-	0.90	1.00	1.00	-	-	-	-	-	-
$E_{min}' = E_{min}$	x	-	0.90	1.00	1.00	-	-	-	-	-	-

No wet service reductions have been used for the *Perma-Column* assemblies since the wood portion is not in contact with the soil or concrete and is assumed to be used within an enclosed building. The wet service reduction factor is shown in Table 5.4 and it is different for each variable. 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. Structural analyses were performed using #1 Southern Yellow Pine (SYP) and #2 and better Spruce Pine Fir (SPF) with planed or standard dressed mechanically laminated lumber.

The column laminations for the mechanically laminated columns are assumed to be fastened together with two rows of 0.131" minimum diameter nail or wire fasteners using a 9 inch maximum on center spacing and a nailing pattern with near side and far side installation as shown in Figure 5.1. The minimum fastener length is 3.75" for 3 ply and 4 ply columns, and 4.5" for 5 ply columns. Fastener calculations are given in Appendix 2. Alternatively, it is acceptable for the fasteners to penetrate all the way, or nearly all the way, through the column width.

Construction structural adhesive may be applied between the plies on each face, but it was not included in the calculations for the mechanically laminated columns. The mechanical fasteners alone are adequate to resist the Minimum Required Interlayer Shear as listed in Table 4 of ASAE EP559.1.

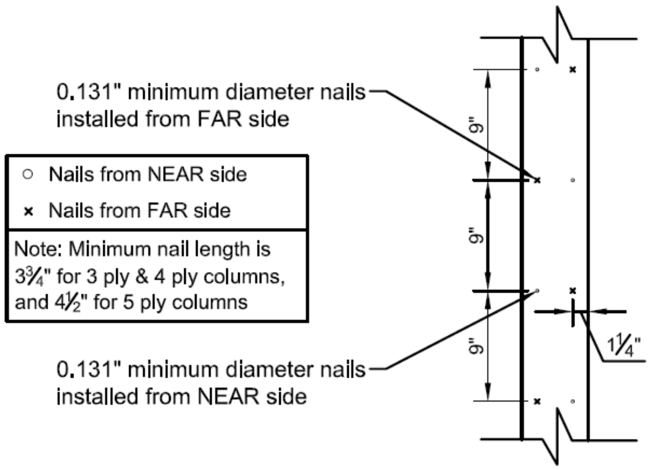


Figure 5.1 Wood Column Fastening Pattern

Glulam columns are also included in Table 7.1 in Section 7. Glulam columns have no mechanical fasteners and they depend heavily on the glue applied between the laminations.

6. Modeling

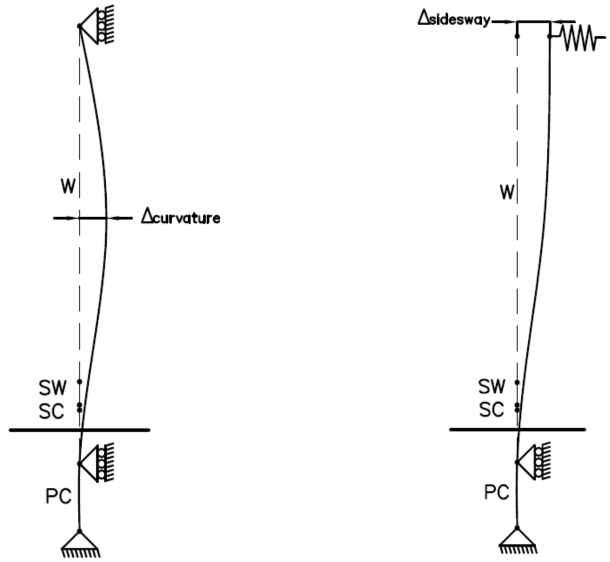
Figure 6.1 shows an example of the structural analogs that were used to check each ***Perma-Column*** assembly. The structural analysis was performed using Visual Analysis by Integrated Engineering Software. The structural analog was created with element stiffness values that closely simulate laboratory test results. These structural analogs can be used to predict ***Perma-Column*** assembly behavior under many different load conditions. Three post models were analyzed for each height to simulate different boundary conditions at the eave.

- **Eave Condition I** assumes a very rigid diaphragm which allows no horizontal movement at the eave. The deflection limit for this condition is L/120 for walls without brittle finishes and is controlled by curvature of the post.
- **Eave Condition II** allows horizontal movement corresponding to the eave height divided by 240 (L/240). The deflection limit for this condition is L/240 for walls with brittle finishes and is controlled by sidesway at the eave or, on taller columns, at the location of additional curvature of the column.
- **Eave Condition III** allows horizontal movement corresponding to the eave height divided by 120 (L/120). The deflection limit for this condition is L/120 for walls without brittle finishes and is controlled by sidesway at the eave or, on taller columns, at the location of additional curvature of the column.

Column deflection should be checked using service loads and deflection limits are taken from *IBC 2012 Table 1604.3* for exterior walls with brittle or flexible finishes, L/240 and L/120, respectively. These eave displacements were evaluated using ASD load combinations, mainly dead plus wind (D+W), and the larger of sidesway or curvature was taken as the controlling value. Horizontal movement is created in the models for these two eave conditions by using a spring support in place of a roller support with the spring constant adjusted to allow the appropriate amount of deflection.

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 Appendix 3 (see also Table 5.2.1 in the ***Perma-Column*** Test Report). I_e for the PC6600 was matched with I_e for PC6400 for modeling purposes. Similarly, I_e for the PC8500 was matched with I_e for PC8400. Bending, axial and shear strength properties of the reinforced concrete are summarized in Appendix 3 of this document. For a detailed discussion, see Section 3 of the ***Perma-Column*** Test Report. Element "PC" of the analogs shown in Figure 6.1 represents the reinforced concrete base.

Element "SC" in the analog represents the concrete-to-steel bracket connection. The purpose of this element is to model the bending flexibility of the steel bracket where it attaches to the concrete. This element was assigned a modulus of elasticity, E_s, of 29 million psi, an effective length, L_e, of 2.5 inches, and an effective moment of inertia, I, equal to $S(L_e)/E_s$ where S is the rotational stiffness from Table 4.2. Table 6.1 summarizes the moment of inertia used for the steel bracket elements.



Eave Condition I Analog Eave Condition II & III Analog

Figure 6.1 Structural Analogs for a Column with Pin or Spring at Top

Table 6.1: Moment of Inertia, I, For Steel Bracket Elements (in^4)[1]

Series	Concrete-to-Steel Bracket Element "SC"	Steel Bracket-to-Wood Column Element "SW"
PC6300	0.2462	1.2615
PC6400	0.2515	5.4130
PC6600	0.2515	1.2615
PC8300	0.4769	4.5965
PC8400	0.4711	4.5965
PC8500	0.4711	4.5965

Note:

1. *From Calculations in Appendix 3*

Element "SW" in the analog is used to model the steel bracket-to-wood column connection. This element extends between the fastener groups and models the rotation between steel and wood. The length varies depending upon the distance between the centroids of the two fastener groups. In the structural models, 8 inches was used for PC6300 and PC6600, 13 inches for PC6400, and 11 inches for PC8300, PC8400, and PC8500 (see Figure 4.1).

Element “W” in the analog represents the laminated or solid sawn wood column with an E value of 1.6 million psi for # 1 SYP, and 1.4 million psi for #2 and better SPF.

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. After the structural analog was created and the loading applied, a P-delta analysis was performed for columns with eave conditions II and III to account for increased section forces induced by column deflection.

7. *Perma-Column* Design Chart

Table 7.1 shows the allowable vertical load, P, for ***Perma-Column*** assemblies under a uniform wind load of 100 pounds per linear foot (plf) for wind loads calculated per ASCE 7-05 and 165 plf if wind loads are calculated per ASCE 7-10. The post heights evaluated range from 10’-0” up to 22’-0” in two foot increments. Blank boxes in the chart indicate the column fails in deflection due to the uniform wind load. The failure modes checked are as follows:

1. Deflection Due to Service Loads
2. Wood Elements
    a. Axial load
    b. Combined axial and bending moment
    c. Shear
3. Steel Bracket-to-Wood Column Connection Element
    a. Maximum bending moment in element
4. Concrete-to-Steel Bracket Connection Element
    a. Maximum bending moment in element
5. Concrete Elements
    a. Factored bending moment and axial force compared to Interaction Diagram in Appendix 3 (see also Figure 3.3.2 in the ***Perma-Column*** Test Report)
    b. Factored shear forces compared to design shear strength in Appendix 3 (see also Table 3.4.1 in the ***Perma-Column*** Test Report)

The notes at the bottom of Table 7.1 describe the assumptions and conditions to which these allowable vertical loads apply. The effective buckling length factor, K_e, was taken as 0.8 for columns fixed at the base and pinned at the top (Eave Condition I in Table 7.1). The exact buckling mode case for columns fixed at the base and pinned at the top with a small amount of translation allowed (Eave Conditions II and III in Table 7.1) is not given in classical tables for K_e. Because these columns are part of a diaphragm assembly where the horizontal movement is small compared to the height of the column, K_e is assumed to be 1.0. It is 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. Additional wind bracing or knee braces may be needed in the building design if no diaphragm resistance is present. This is important to remember when using the PC6600 as a replacement post as well. The overall building design should be evaluated to verify that the replacement post is adequate.

The main controlling factors in the calculations behind this chart are the imposed deflection limits, and the strength of the wood portion of the column. A 6x6 #2 pressure-treated column, a 3 ply 2x6 #1 SYP pressure-treated non-spliced column, and a 4 ply 2x8 #1 SYP column with structural finger joints using the same wind load and boundary conditions are included in Table 7.1 for comparison purposes. These comparison columns are designed as traditional embedded post-frame foundations with treated wood in the ground. It is important to note that the 4 ply 2x8 embedded post has treated laminations in the ground and non-treated above grade. The joint between these laminations is a certified structural finger joint as defined in ASAE EP 559.1. **The PC6300 and PC8400 perform significantly better than their 3 ply 2x6 and 4 ply 2x8 counterparts mainly because they have no wet service reduction, and the maximum bending moment is resisted by the concrete component below grade**.

In some circumstances, the calculated loads may exceed the capacity of a single ***Perma-Column***. A typical example of this is on either side of large door openings. Figure 7.1 shows an installation detail for a double ***Perma-Column***. The uniform wind load and the allowable vertical load can both be doubled for ***Perma-Columns*** installed as per this detail, if header framing and wall girts are provided to allow both columns to share the load equally. In addition, the footings on either side of large door openings need to be sized appropriately for the increased vertical and lateral loads they will carry.

Table 7.1 Perma-Column Design Chart

		Allowable vertical load, P (lbs), for Perma-Column assemblies under constant wind load																				
	Building Eave Height (ft)	10			12			14			16			18			20			22		
	Eave Conditions	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
	Max Deflection (in)	1	0.5	1	1.2	0.6	1.2	1.4	0.7	1.4	1.6	0.8	1.6	1.8	0.9	1.8	2	1	2	2.2	1.1	2.2
#1 SYP Std. S4S	**PC6600**	**6x6 #1 SYP**	24400	21600	21700	**21900**	17700	17700	17400	12000	12500	12000		8200	8200								
	PC6300	**3 ply x 6**	31600	24400	24200	**24500**	16600	17100	16400	10800	11100	11100		7400	7600								
	PC6400	4 ply x 6	42000	32600	32100	33500	23800	23600	24200	16100	16500	17000	11000	11300	12100		7900	8700					
	PC8300	3 ply x 8	48000	42600	42400	43200	35000	34700	37300	27600	27200	30500	20700	20700	22500	15000	15600	17000	11000	11600	12800		8600
	PC8400	**4 ply x 8**	64000	56900	48000	57600	46700	46300	**49600**	37200	36800	41600	29600	28700	32800	22000	**22500**	25100	18200	17000	19500	13700	13000
	PC8500	5 ply x 8	79600	71000	48400	71600	58300	52000	61600	46400	46400	51700	37000	36600	42800	28700	29000	33000	24100	22300	25900	18400	17000
#1 SYP planed	PC6300	3 ply x 6	29400	22400	22400	21900	14800	15200	14500	9500	9900	9700		6500	6600								
	PC6400	4 ply x 6	39000	29600	29600	30600	21500	21600	21600	14300	14700	15000	9700	10000	10700		6900	7600					
	PC8300	3 ply x 8	47900	42000	40500	42800	34000	34000	36400	27200	25900	28900	19700	19800	21300	14100	14700	15900	10300	10800	11900		7900
	PC8400	4 ply x 8	63200	55200	41100	56800	46000	46000	48800	36000	36000	40400	28300	27600	31200	20800	21300	23800	15600	16100	18200	11800	12200
	PC8500	5 ply x 8	79600	70000	48300	71400	57200	56800	61200	45600	45600	51200	36000	35800	40800	27400	28000	31400	22900	21300	24400	17400	15900
Glulam	**PC6300**	**3 ply x 6**	41200	31600	31200	**30000**	19800	20400	19600	12700	13200	13200		8700	8900		5700						
	PC6400	4 ply x 6	54400	41800	37200	43200	29300	29400	29600	19500	19900	20700		13600	14800		9600						
	PC8300	3 ply x 8	64000	57600	42800	58400	47600	42700	50800	36000	32800	37600	25200	24900	27400	17900	18800	20200	13000	13700	15000		10000
	PC8400	**4 ply x 8**	84000	76000	47200	78000	62400	52800	**66800**	48500	46000	55200	37300	35600	41600	27500	**27700**	31800	22800	21200	24600		16300
	PC8500	5 ply x 8	106000	81300	47700	96800	78000	56000	84000	61200	60600	69600	47600	46600	54200	35900	36200	41700	30100	27800	32600	23000	22600
#2 SPF	PC6300	3 ply x 6	26600	20900	20900	20400	14000	14300	13500	8900	9200	8900		6000	6000								
	PC6400	4 ply x 6	35400	27700	27800	28400	20400	20700	20300	13500	13900	14100	9100	9300	9900		6400	4500					

Colored boxes are for easy strength comparison

	Comparison to traditional embedded, pressure-treated wood columns - allowable vertical load, P (lbs)																				
Building Eave Height (ft)	10			12			14			16			18			20			22		
Eave Conditions	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
Max Deflection (in)	0	0.5	1	0	0.6	1.2	0	0.7	1.4	0	0.8	1.6	0	0.9	1.8	0	1	2	0	1.1	2.2
6x6 #2 trd SYP	15600	13800	12200	**13500**	9900	8100	9100	9400													
3 ply x 6 trd. #1 SYP*	24000	17400	15400	**17400**	11600	10200	11800	7900	6900	8100		4700	5600								
4 ply x 8 #1 SYP**	59200	51200	50400	54000	41200	36800	**45400**	31200	27500	38000	23300	21000	29700	17700	**16000**	22900	13800	12300	17700		9500
3 ply x 6 Glulam	36800	24600	22000	**24600**	16400	14600	16600	10900	9900	11500		7000	8000								
4 ply x 8 Glulam	67600	67600	67600	67600	54800	49100	**60800**	40000	35600	48800	29400	26800	37200	22400	**20500**	28800	17400	15700	22200		12300

* Standard S4S mechanically laminated non-spliced fully treated column

** Standard SFS mechanically laminated column with certified structural finger joints between treated and untreated laminations

Chart Assumptions:

1) This chart is for Perma-Columns used in a normal post-frame building (enclosed all four sides) where the columns are supported at the top by diaphragm action of the building
2) Design conforms with IBC 2009 and IBC 2012. ASCE 7 Wind design criteria: Building (Risk) Category II, Wind Exposure C, Enclosed Building, 28ft max midheight, 3.5:12 roof pitch, ASCE 7-05 wind speed 90 mph, ASCE 7-10 wind speed 115 mph, ASCE 7-05 Wind Load = 100 lb/ft for 8 ft column spacings; ASCE 7-10 Wind Load = 165 lb/ft for 8ft column spacings.
3) Southern Pine design values are per Southern Pine Inspection Bureau (SPIB) Supplement No. 13. Glulam design values per Table 5.3
4) IBC 2009 (ASCE 7-05) Load combinations used are: 1) Dead+Snow 2) Dead+.75(Wind+Snow) 3) Dead+Wind
5) IBC 2012 (ASCE 7-10) Load combinations used are: 1) Dead+Snow 2) Dead+.75(0.6Wind+Snow) 3) Dead+0.6Wind
6) See Tables 2.2, 2.3 and 2.4 for member dimensions and properties
7) Dead load to total load ratio = 0.25
8) Effective length factor, Ke, is 0.8 for Condition I, and 1.0 for Conditions II and III
9) P-delta analysis used for eave conditions II and III to account for forces induced by deflection
10) Eave Condition I is a propped cantilever with no horizontal movement at the eave, horizontal deflection limit of L/120 for walls without brittle finishes
11) Eave Condition II allows L/240 horizontal movement. Actual deflections based on larger of sidesway or curvature, horizontal deflection limit of L/240 for walls with brittle finishes
12) Eave Condition III allows L/120 horizontal movement. Actual deflections based on larger of sidesway or curvature, horizontal deflection limit of L/120 for walls without brittle finishes
13) Repetitive member factor for 3-ply column is 1.35, and for 4-ply and 5-ply columns is 1.40 per ASAE EP559.1
14) Dry use factor applied to wood portion in Perma-Column assembly; wet use factor applied to the entirety of the 6x6 #2 trd SYP, 3-ply x6 trd #1 SYP, and embeded portion of 4-ply x8 #1 SYP, 3-ply x6 Glulam, and 4-ply x8 Glulam columns w/o Perma-Columns
15) Exterior sidewall post with lateral loading from wind only
16) Full lateral bracing and major axis bending only; no loads acting on weak axis; no kneebraces; no splices in laminated wood portion of Perma-Column assembly
17) Non-constrained post foundation designed per ASAE EP 486.1 with 4'-0" embedment depth
18) Blank in chart represents deflection controls design
19) Final column design should include a complete building analysis by a Design Professional

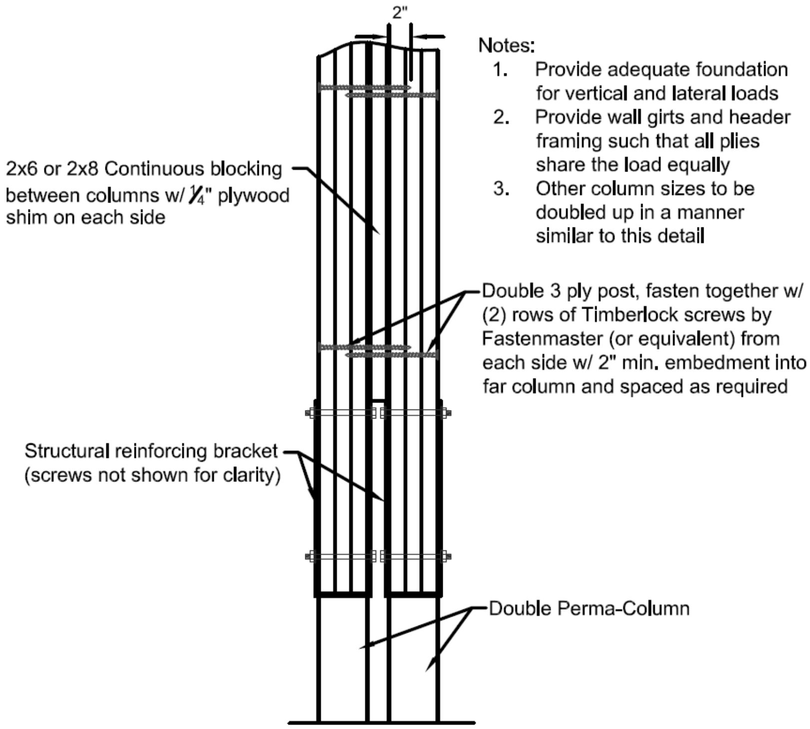


Figure 7.1 Double *Perma-Column* installation detail

8. Design Example

This design example is for a PC8300 with a 3 ply 2x8 #1 Southern Pine planed, mechanically laminated wood column. The column is 16' high and the eave is allowed to deflect horizontally 1.6 inches (L/120). The vertical load is 4.5 kips dead load and 13.5 kips snow load. The horizontal loading is 100 plf due to wind loading calculated per ASCE 7-05. 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 Appendix 4 of this document.

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

A. ASCE 7-05 ASD (Wood column and steel bracket)
   1) D + W
   2) D + S
   3) D + 0.75S + 0.75W

B. ASCE 7-05 LRFD (Concrete column and steel bracket)
   1) 1.2D + 1.6S + 0.8W
   2) 1.2D + 0.5S + 1.6W
   3) 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 Actual deflections (D+W) are within allowable of 16(12)/120 = 1.6 [OK]

8.2.2 The maximum internal forces in the wood elements are:

a. Load Combination A1: M_{max} = 20.7 inch-kips and P_{max} = 4.6 kips
b. Load Combination A2: M_{max} = 0 inch-kips and P_{max} = 18.1 kips
c. Load Combination A3: M_{max} = 16.8 inch-kips and P_{max} = 14.7 kips

8.2.3 The interaction value in the combined axial force and bending moment check is 0.83 and 0.82 for load combinations A2 and A3 respectively, < 1.0 [OK]

8.2.4 The allowable shear capacity of a 3 ply 2x8 #1 SYP member is 280 psi (40,320 psf). The maximum shear is 38 psi (5438 psf). [OK]

Shear Diagram (Comb A1 and B3) Moment Diagram (Comb A1 and B3)

Figure 8.1 Shear and Moment diagram for PC8300, 16' high with 1.6" maximum deflection under load combination A1

8.2.5 Steel Bracket-to-Wood Connection Element SW

8.2.5.1 The bending moment on the connection produces an equal and opposite force on the top and bottom fastener groups. The maximum moment in bracket-to-wood element is 29.3 inch-kips (ASD). The resultant load on each fastener group is 2.66 kips assuming a distance of 11 inches between the centroid of each group. The force on one bolt and four screws is 1.08 kips and 1.45 kips respectively. The allowable capacity of (1) ½" bolt with steel side plates loaded in double shear and of (4) ¼" SDS screws with steel side plates loaded in single shear is 2.19 kips and 2.69 kips respectively. [OK]

8.2.6 Concrete-to-Steel Bracket Connection Element SC

8.2.6.1 The maximum moment in the steel bracket element is 31.6 inch-kips (ASD); the allowable flexural strength is M_n/Ω_b = 103.7/1.67 = 62.1 inch-kips as discussed in Section 4.1 of this guide. [OK]

8.2.6.2 Alternatively, the maximum factored bending moment in the steel bracket element is 54.0 inch-kips (LRFD); the design flexural strength is φM_n = 0.90(103.7) = 93.3 inch-kips as discussed in Section 4.1 of this guide. [OK]

8.2.7 Concrete Element PC

8.2.7.1 The maximum factored bending moment and axial forces in the concrete column element are listed below and are well within the envelope of the design bending and axial strength interaction diagram for the PC8300 shown in Appendix 3.

a. Load Combination B1: M_u = 47.6 inch-kips and P_u = 27.4 kips
b. Load Combination B2: M_u = 92.3 inch-kips and P_u = 12.5 kips
c. Load Combination B3: M_u = 90.4 inch-kips and P_u = 4.3 kips

8.2.7.2 The minimum design shear strength of the PC8300 as given in Appendix 3 is 4.5 kips. The factored shear in this example problem is 2.9 kips. [OK]

This column is adequate for the design loading.

9. Wind Uplift Resistance

Figure 9.1 shows three foundation conditions that may be used with a ***Perma-Column***: the standard design, concrete collar or PC Extender. The wind uplift resistance can be evaluated for each foundation condition using the procedure described in ASAE EP486.1 *Shallow Post Foundation Design*. The uplift calculations in this section follow the ASD equations of *EP486.1*, and therefore should be compared with net uplift loads from ASD load combinations in the International Building Code (IBC) 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 concrete weight (PC Extender option also includes weight of footing), can be calculated from the following equation:

$$U = \alpha G[0.33\pi\{[(d-t)+0.5w/\tan\theta]^3(\tan\theta)^2 - 0.125w^3/\tan\theta\} - A_p(d-t)] + 0.25C\pi w^2 tG + 0.25CA_p 5$$

Source: ANSI/ASAE EP486.1: 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, (9.81N/kg) 1 lbf/lbm
d = embedment depth, (m) 4 ft
t = collar thickness, (m) 1 ft
w = collar width, (m) ft

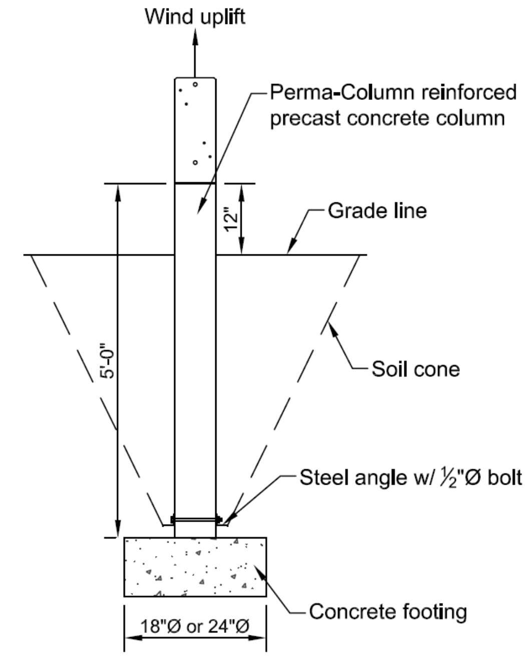
For rectangular footings and collars:
Steel uplift angles are fastened to the post displacing a round corner, truncated prismatic wedge of soil radiating above the angles. The uplift resistance from the mass of the truncated prismatic volume is calculated by the following equation:

$$U = \alpha G[(wl - A_p)(d-t) + (w+l)(d-t)^2\tan\theta + 0.33\pi(d-t)^3\tan^2\theta] + 0.25CA_p 5$$

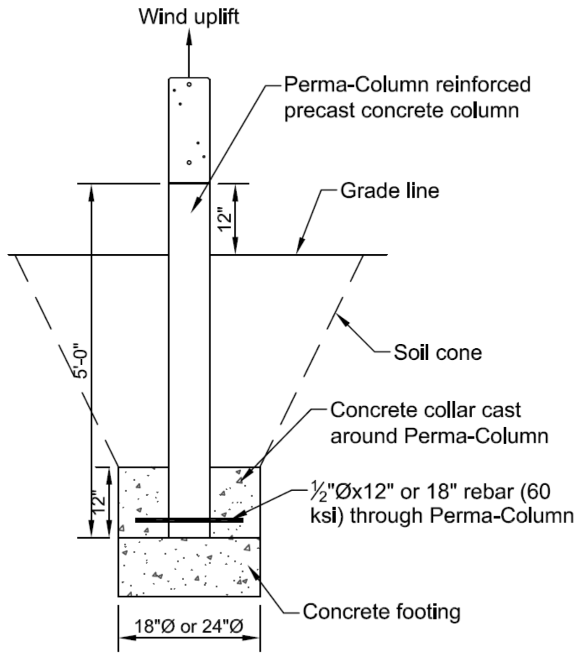
Source: ANSI/ASAE EP486.1: Shallow Post Foundation Design

where:
U = soil uplift resistance, (kN) lbf
α = soil density, (kg/m^3) lb/ft^3
G = gravitational constant, (9.81N/kg) 1 lbf/lbm
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 degrees

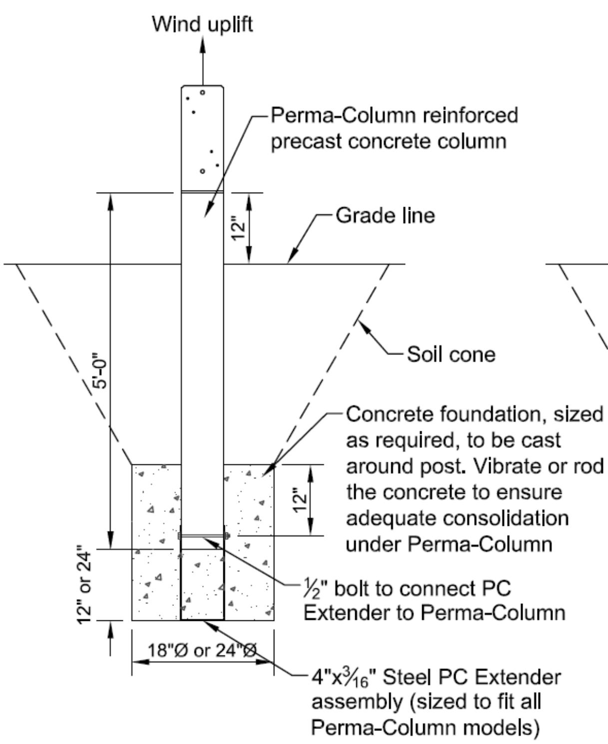
Both of the equations above from EP486.1 have been adjusted to include 25% of the self-weight of the 5 foot tall concrete ***Perma-Column.***



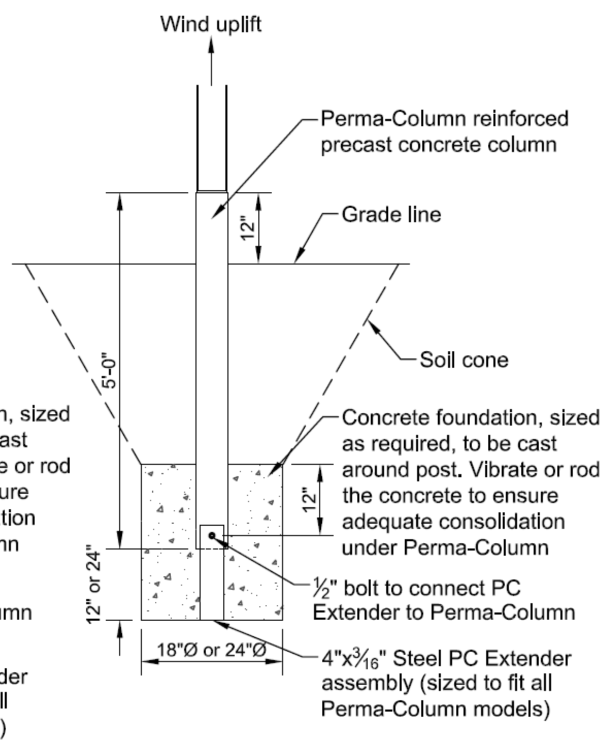
<u>Standard Design</u>



<u>Concrete Collar</u>



<u>PC Extender View 1</u>



<u>PC Extender View 2</u>

Figure 9.1 *Perma-Column* foundation options

The standard foundation installation as shown in Figure 9.1 has a concrete footing to support the ***Perma-Column*** for gravity loads and two 50 ksi yield strength galvanized steel angles that provide uplift resistance. The concrete collar option has a concrete footing for gravity loads, and a separate concrete collar for uplift resistance, as well as increased moment resistance for the embedded column. A ½" diameter reinforcing bar is used as a positive attachment to engage the collar. The PC Extender is a steel assembly bolted to the bottom of the ***Perma-Column.*** Some of the benefits of this foundation option include deeper embedment depths, a monolithically poured concrete footing and collar, and slightly higher uplift resistance values because the footing weight can be included.

Table 9.1 shows the allowable wind uplift resistance in pounds for these foundation conditions and variations:

1. 2x2x8 ½ x 0.134" galvanized steel anchor with compacted fill around posts
2. 2x2x12 x 0.134" galvanized steel anchor with compacted fill around posts
3. 18" diameter concrete collar with ½"x12" reinforcing bar through ***Perma-Column***
4. 24" diameter concrete collar with ½"x18" reinforcing bar through ***Perma-Column***
5. 18" diameter concrete footing with 12" PC Extender
6. 24" diameter concrete footing with 12" PC Extender
7. 18" diameter concrete footing with 24" PC Extender
8. 24" diameter concrete footing with 24" PC Extender

Table 9.1: Allowable Perma-Column Wind Uplift Resistance (lbs)[1]

Type	**Standard**		**Concrete Collar**		**12" PC Extender**[2]		**24" PC Extender**[2]	
Series	**2x2x8 ½ Angles**	**2x2x12 Angles**	**18" Collar**	**24" Collar**	**18" Collar**	**24" Collar**	**18" Collar**	**24" Collar**
PC6300	2090	2490	2130	2980	2400	3450	2660	3920
PC6400	2110	2490	2130	2970	2400	3440	2660	3920
PC6600	2100	2490	2130	2980	2400	3450	2660	3920
PC8300	2110	2480	2130	2970	2400	3440	2660	3920
PC8400	2120	2470	2120	2970	2390	3440	2650	3910
PC8500	2130	2460	2120	2960	2390	3430	2650	3910

Notes:

1. *These values to be compared with calculated net wind uplift from ASD load combinations in IBC*
2. *The weight of the collar and footing has been added to the uplift resistance calculation*

Sample calculations for the different foundation options included in Table 9.1 can be found in Appendix 5. The failure modes that were checked to determine the wind uplift resistance of the standard foundation are uplift resistance of the soil cone, bolt shear capacity, shear rupture at the bolt hole, and steel angle bending capacity. In determining the steel angle bending capacity, it was assumed that the horizontal leg would act as a cantilever, the reaction from the soil on the horizontal leg would be located 1/3 of the distance away from the vertical leg because of the angle stiffness, and the nominal moment strength, $M_n = M_p,= F_yZ$ </= $1.6F_yS$ (per AISC Section F11). Plastic moment behavior will occur near the bolt location first and then move out toward each end of the angle. Since the angles are fastened with only one bolt, it is not realistic to assume that the angles will become fully plastic over their entire length. For calculation purposes, the effective bending length is assumed to be 60% of the total angle length.

10. Summary and Conclusion

The ***Perma-Column*** assembly is designed to be the main structural column in a post-frame building and can be used as an alternative to embedded wood posts. Self Compacting Concrete (SCC) technology makes it possible to manufacture a high quality pre-cast concrete product with a low water-to-cement ratio. The 10,000 psi (nominal) compressive strength protects the reinforcing bars by limiting chips and cracks during handling; reduces the effect of freeze-thaw cycles; and provides a smooth, attractive finish. The structural reinforcing bracket assembly can be designed as a moment connection that is capable of resisting loads for most post-frame building applications, and it allows for the use of non-treated wood by keeping the laminations above grade. The wood portion of the ***Perma-Column*** assemblies can be any grade or species of lumber, and can be used with different types of wood shapes. This guide contains #1 Southern Pine and #2 and better Spruce-Pine-Fir lumber using selected sizes of solid sawn, mechanically laminated and glulam shapes. According to data in Table 7.1, The ***Perma-Column*** assembly will enhance the structural performance of the wood counterpart it replaces for decades.

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*** performance in post-frame buildings of various spans and heights. This guide contains the necessary tools and assumptions needed to create a structural model. The calculations used to produce the design chart indicate that the ***Perma-Column*** assemblies are limited primarily by overall deflection, and by the strength of the laminated wood members. There are several foundation detail options including concrete collars, steel uplift angles, and foundation extenders that can be used with a ***Perma-Column*** to achieve adequate resistance for lateral, gravity and uplift loads for most applications. **The *Perma-Column* assemblies perform significantly better than standard mechanically laminated wood columns under the same boundary conditions primarily because no wet service reduction is required for the concrete component, and the maximum bending moment is resisted by the concrete portion below grade.** The ***Perma-Column*** is a permanent foundation solution for the post-frame building market.

"THE PERMANENT SOLUTION"