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

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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.

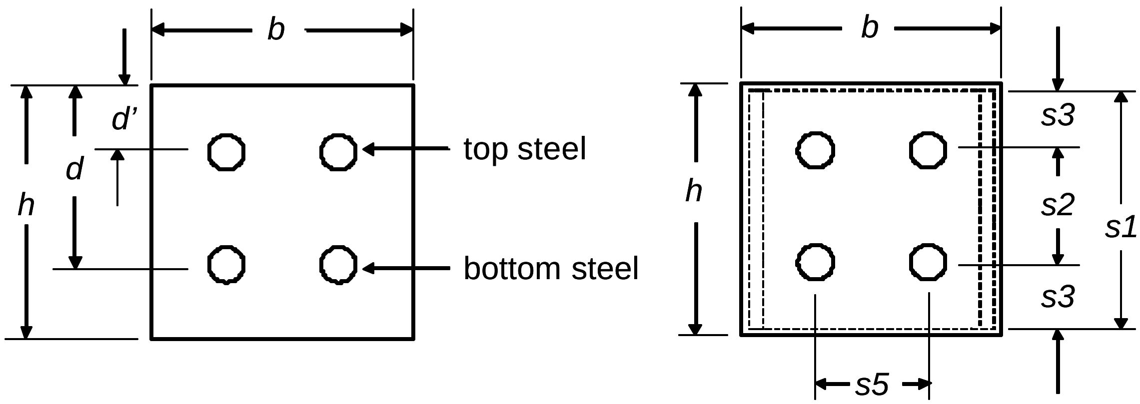

Figure 1.1. Variable definitions for Perma-columns.

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

Variable	Symbol	Units	PC6300	PC6400	PC8300	PC8400
Overall Concrete Width	$\boldsymbol{b}$	in.	5.38	6.88	5.38	6.88
Overall Concrete Depth	$\boldsymbol{h}$	in.	5.44	5.44	7.19	7.19
Depth to Top Steel	$\boldsymbol{d'}$	in.	1.50	1.50	1.56	1.56
Depth to Bottom Steel	$\boldsymbol{d}$	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	$\boldsymbol{A_s'}$	$in.^2$	0.40	0.40	0.62	0.62
Area of Bottom Steel	$\boldsymbol{A_s}$	$in.^2$	0.40	0.40	0.62	0.62
Steel Yield Strength	$\boldsymbol{f_y}$	$lbf/in.^2$	60000	60000	60000	60000
Concrete Compressive Strength	$\boldsymbol{f_c'}$	$lbf/in.^2$	10000	10000	10000	10000
Steel Modulus of Elasticity	$\boldsymbol{E_s}$	$lbf/in.^2$	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:

$$\boldsymbol{P_u} \leq f\ \boldsymbol{P_n} \tag{2.2.1}$$

$$\boldsymbol{M_u} \leq f\ \boldsymbol{M_n} \tag{2.2.2}$$

$$\boldsymbol{V_u} \leq f\ \boldsymbol{V_n} \tag{2.2.3}$$

where:

$\boldsymbol{P_u}$	=	Required axial force (axial force due to factored loads)
$\boldsymbol{M_u}$	=	Required bending moment (bending moment due to factored loads)
$\boldsymbol{V_u}$	=	Required shear force (shear force due to factored loads)
$\boldsymbol{P_n}$	=	Nominal axial strength
$\boldsymbol{M_n}$	=	Nominal moment strength
$\boldsymbol{V_n}$	=	Nominal shear strength
f	=	Resistance factor
$f\ \boldsymbol{P_n}$	=	Design (or useable) axial strength
$f\ \boldsymbol{M_n}$	=	Design (or useable) moment strength
$f\ \boldsymbol{V_n}$	=	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 $f\,\boldsymbol{P_n}$, $f\,\boldsymbol{M_n}$, and $f\,\boldsymbol{V_n}$ 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., $\boldsymbol{V_u}$, $\boldsymbol{M_u}$, $\boldsymbol{P_u}$). To use the resistance factors outlined in following sections for strength design will require use of the following ANSI/ASCE 7 load combinations.

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

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

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

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

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

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

D = Dead Load
E = Earthquake Load
F = Fluid Load
H = Soil Load
L = Live Load
$\boldsymbol{L_r}$ = Roof Live Load
R = Rain Load
S = Snow Load
T = Self-Straining Load
W = Wind Load
x = 1.0 for garages, areas of public occupancy, and values of ***L*** greater than 100 lbf/ft^2. When ***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	f 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, f 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 e_{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 $a = b_1 c$ 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 b_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 b_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 (e_{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:

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

$$C = b_1\, c\, b\, 0.85\, f_c' \qquad (3.1.2)$$

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

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

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

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;

$$b_1\, c\, b\, 0.85\, f_c' = A_s'\, E_s\, 0.003\,(d' - c)/c + A_s\, f_y \qquad (3.1.6)$$

which can be rewritten as:

$$c^2[b_1\, b\, 0.85\, f_c'/(A_s'\, f_y)] + c\,[(0.003\, E_s/f_y) - A_s/A_s'] - 0.003\, d'\, E_s/f_y = 0 \qquad (3.1.7)$$

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.

$$M_o = T_{(bottom)}\,(d - b_1 c/2) + T_{(top)}\,(d' - b_1 c/2) \tag{3.1.8}$$

or

$$M_o = A_s\, f_y\,(d - b_1 c/2) + A_s'\, E_s\, 0.003\,(d' - c)(d'/c - b_1/2) \tag{3.1.9}$$

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

Variable	Symbol	Units	PC6300	PC6400	PC8300	PC8400
Stress Block Depth Factor	b_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	$e_{s(top)}$	in./in.	0.00089	0.00133	0.00028	0.00065
Strain in Bottom Steel	$e_{s(bot)}$	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)*	$f\,M_o$	lbf - in.	77700	83600	158000	165400

* f = 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 $b_1 c$ 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:

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

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:

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

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 0.1***h*** 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*	$f\ P_{n(max)}$	Lbf	119500	148200	161800	199700

* f = 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.

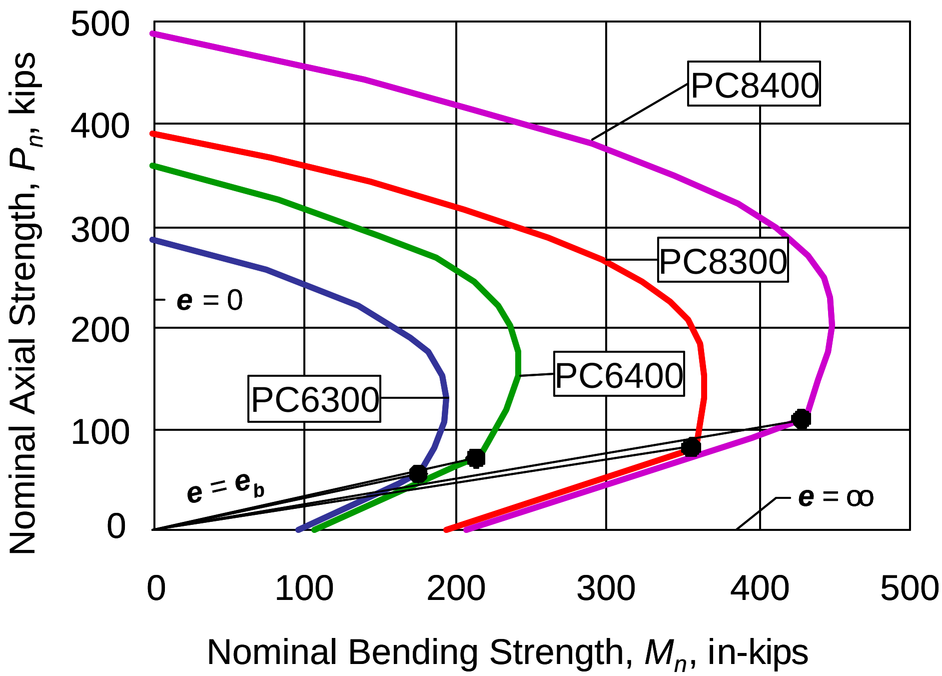

Figure 3.3.1. Strength interaction diagrams for axial compression and bending moment about the major axis of ***Perma-Columns***.

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.

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

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

$$C = b_1\, c\, b\, 0.85\, f_c' \quad (3.3.2)$$

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

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

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

$$M_b = C\,(h - b_1 c)/2 - T_{(top)}\,(h/2 - d') + T_{(bottom)}\,(d - h/2) \quad (3.3.6)$$

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 $\boldsymbol{P_n}$ from the plastic centroid of the columns. It follows, as shown in figure 3.3.1, that the vertical axis represent $\boldsymbol{e} = 0$ and the horizontal axis represents $\boldsymbol{e} = \infty$.

Table 3.3.1 Axial Compression and Bending Strength Interaction Values

Nominal Axial	Nominal Bending Strength, $\boldsymbol{M_n}$, kips-inches			
Strength, $\boldsymbol{P_n}$, 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 $f\ \boldsymbol{P_n}$ decreases from $\boldsymbol{0.1\ f_c'\ A_g}$ to zero.

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

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	$\phi_b M_n$ 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

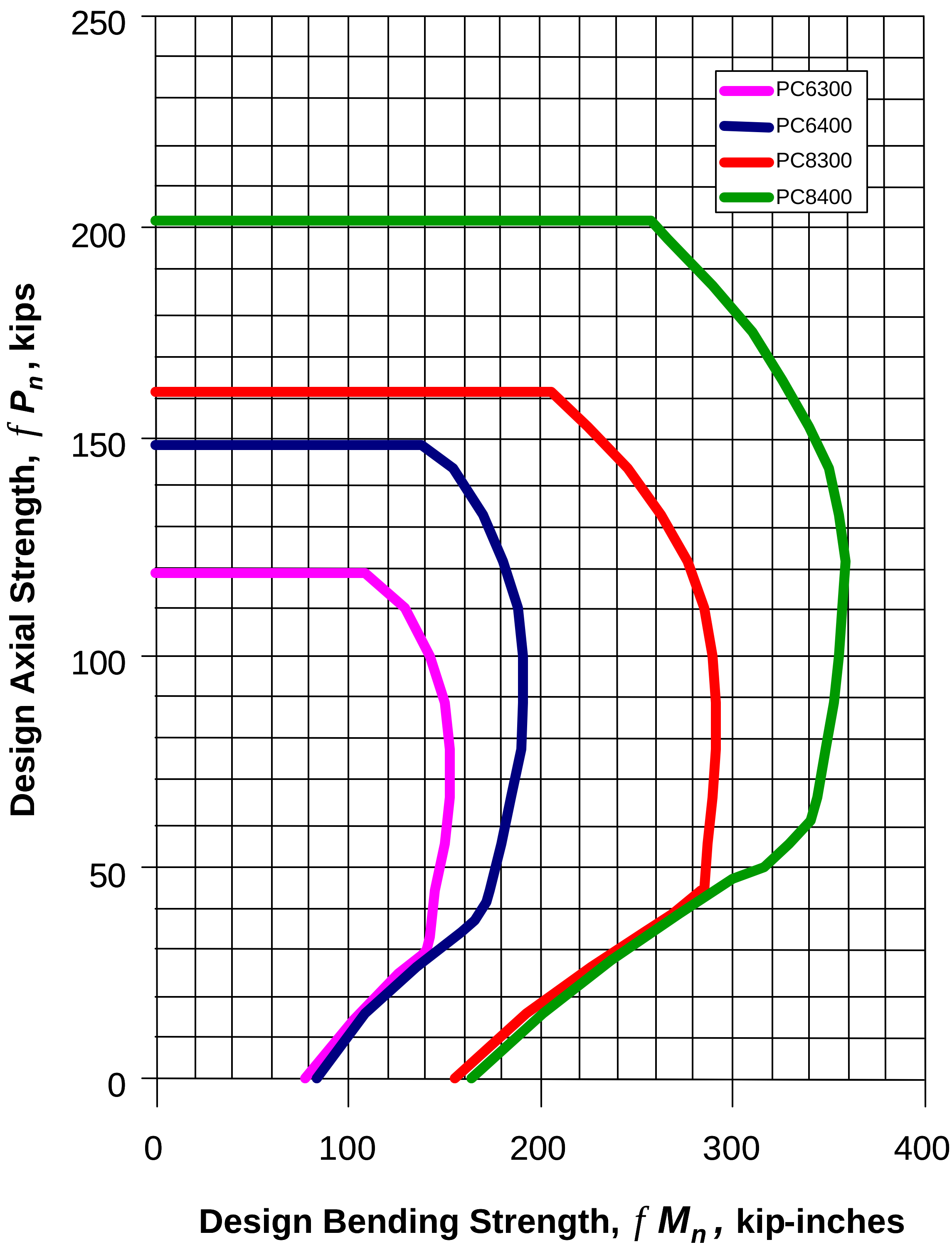

Figure 3.3.2. Design bending and axial strength interaction values for ***Perma-Columns***.

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:

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

or

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

but not greater than:

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

or

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

Where $(\boldsymbol{f_c'})^{1/2}$ 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 ($f\ \boldsymbol{V_n}$) 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**	
		$\boldsymbol{V_n=V_c}$ lbf	$f\ \boldsymbol{V_n}$* lbf	$\boldsymbol{V_n=V_c}$ lbf	$f\ \boldsymbol{V_n}$* lbf	$\boldsymbol{V_n=V_c}$ lbf	$f\ \boldsymbol{V_n}$* lbf	$\boldsymbol{V_n=V_c}$ lbf	$f\ \boldsymbol{V_n}$* 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	$\boldsymbol{M_u/V_u}$, 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	**	**

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

Although the design shear strength of a component $f\ \boldsymbol{V_n}$ decreases as the ratio of bending moment to shear load induced by the factored loads ($\boldsymbol{M_u/V_u}$) 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:

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

However, $\boldsymbol{V_c}$ can not be greater than:

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

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

Since $\boldsymbol{N_u}$ cannot exceed $f_a\,\boldsymbol{P_n}$, the maximum $\boldsymbol{N_u}$ 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 $f\ \boldsymbol{V_n} = f\ \boldsymbol{V_c}$: 168.4, 212.5, 242.8 and 305.8 kips for PC6300, PC6400, PC8300 and PC8400, respectively.

Design shear strengths $f\ \boldsymbol{V_n}$ for PC6300 columns, obtained by multiplying $\boldsymbol{V_c}$ 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 $\boldsymbol{M_u/V_u}$ and $\boldsymbol{N_u/V_u}$ combinations.

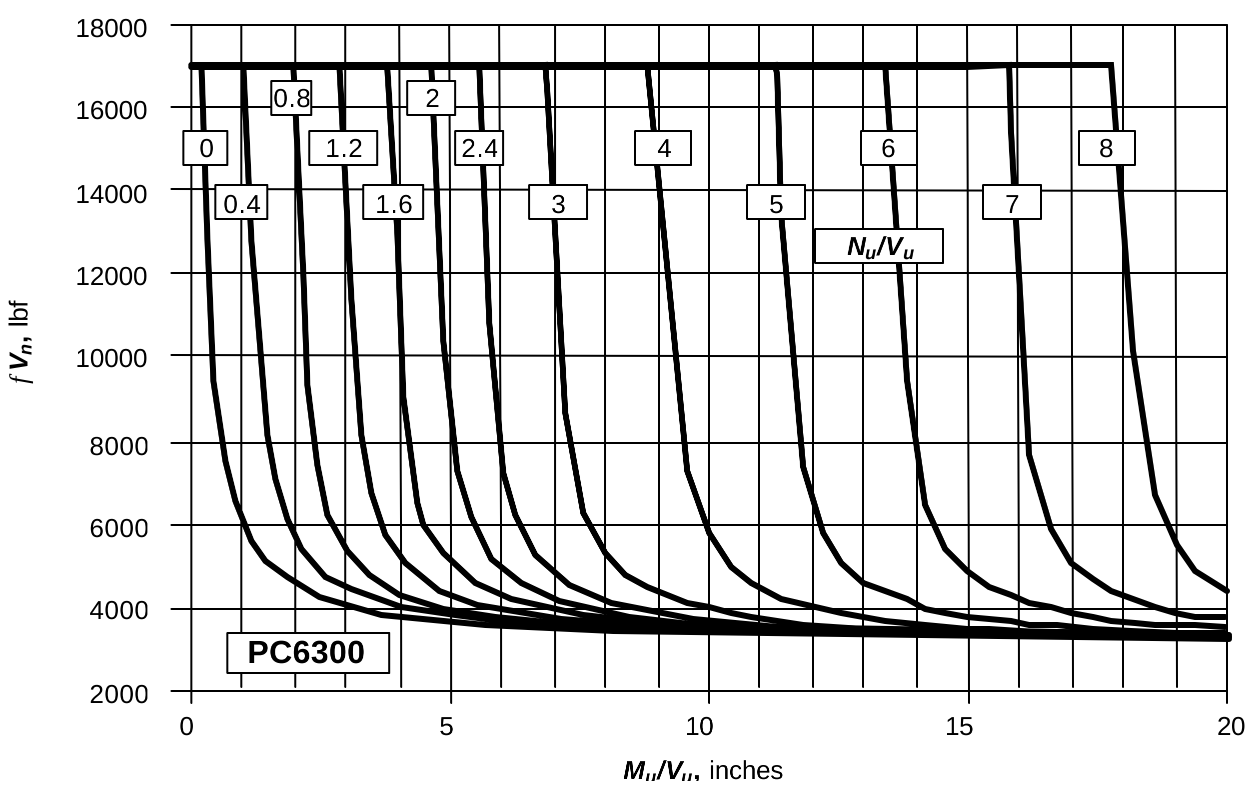

Figure 3.4.1. Design shear strengths $f\ \boldsymbol{V_n}$ for PC6300 columns as a function of $\boldsymbol{M_u/V_u}$ and $\boldsymbol{N_u/V_u}$

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 $f\boldsymbol{P_n}$ replaced by $\boldsymbol{P_u}$ (i.e., $\boldsymbol{P_u} \leq f\ \boldsymbol{P_n}$) and $f\boldsymbol{M_n}$ replaced by $\boldsymbol{M_u}$. The relationships between $\boldsymbol{P_u}$ and $\boldsymbol{M_u}$ for the wood posts were calculated using the following design equation for wood members under combined bending and axial compressive loads.

$$\left[\frac{\boldsymbol{P_u}}{l\ f_c \boldsymbol{P'}}\right]^2 + \frac{\boldsymbol{M_m}}{l\ f_b \boldsymbol{M'}} \leq 1.0 \qquad (3.5.1)$$

where:

$\boldsymbol{P_u}$	=	Axial compressive force due to factored loads
$l\ f_c \boldsymbol{P'}$	=	Design resistance for axial compression from Table 3.5.1
$\boldsymbol{M_m}$	=	Factored moment, including any magnification for second-order effects
	=	$\boldsymbol{M_u}$ for short columns
$l\ f_b \boldsymbol{M'}$	=	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	$\boldsymbol{F_v}$	kips/in.^{2}	0.26	0.26	0.26	0.26
Flexure	$\boldsymbol{F_b}$	kips/in.^{2}	4.19	4.19	3.81	3.81
Axial Compression	$\boldsymbol{F_c}$	kips/in.^{2}	4.20	4.20	3.96	3.96
Applicable Adjustment Factors						
Wet Service Factor - Shear	$\boldsymbol{C_M}$		0.97	0.97	0.97	0.97
Wet Service Factor - Flexure	$\boldsymbol{C_M}$		0.85	0.85	0.85	0.85
Wet Service Factor - Axial Comp.	$\boldsymbol{C_M}$		0.80	0.80	0.80	0.80
Shear Stress Factor**	$\boldsymbol{C_H}$		1.95	1.95	1.95	1.95
Load Sharing Factor***	$\boldsymbol{C_r}$		1.35	1.35	1.40	1.40
Adjusted Reference Strength Values						
Shear ($\boldsymbol{F_v C_M C_H}$)	$\boldsymbol{F_v}$’	kips/in.^{2}	0.49	0.49	0.49	0.49
Flexure ($\boldsymbol{F_b C_M C_r}$)	$\boldsymbol{F_b}$’	kips/in.^{2}	4.81	4.81	4.53	4.53
Axial Compression ($\boldsymbol{F_b C_M}$)	$\boldsymbol{F_c}$’	kips/in.^{2}	3.36	3.36	3.17	3.17
Adjusted Resistance Values						
Shear ($\boldsymbol{F_v}$’ $\boldsymbol{A}$/1.5)	***V’***	kips	8.11	10.82	10.70	14.26
Moment ($\boldsymbol{F_b}$’ $\boldsymbol{S}$)	***M’***	kips-in.	109.1	145.4	178.7	238.3
Axial Compression ($\boldsymbol{F_c}$’ $\boldsymbol{A}$)	***P’***	kips	83.2	110.9	103.4	137.8
Resistance and Time Effect Factors						
Resistance Factor – Shear	f_v		0.75	0.75	0.75	0.75
Resistance Factor – Bending	f_b		0.85	0.85	0.85	0.85
Resistance Factor – Axial Comp.	f_c		0.90	0.90	0.90	0.90
Time Effect Factor – Wind Load	l_w		1.0	1.0	1.0	1.0
Time Effect Factor – Snow Load	l_s		0.8	0.8	0.8	0.8
Design Resistance Values Under Wind Loading						
Shear	$l_w f_v$ ***V’***	kips	6.09	8.11	8.02	10.70
Moment	$l_w f_b$ ***M’***	kips-in.	92.7	123.6	151.9	202.6
Axial Compression	$l_w f_c$ ***P’***	kips	74.8	99.8	93.0	124.0
Design Resistance Values Under Snow Loading						
Shear	$l_s f_v$ ***V’***	kips	4.87	6.49	6.42	8.56
Moment	$l_s f_b$ ***M’***	kips-in.	74.2	98.9	121.5	162.1
Axial Compression	$l_s f_c$ ***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

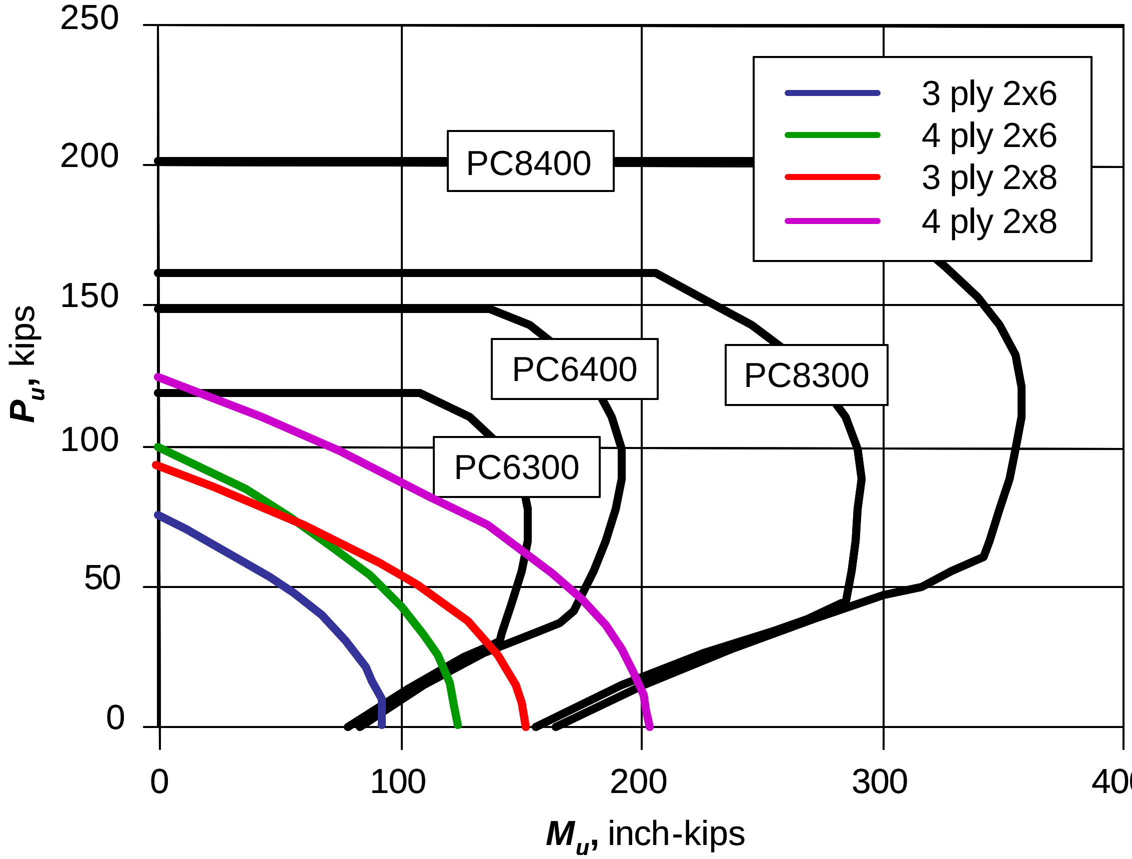

Figure 3.5.1. Interaction diagrams for maximum allowed axial and bending moment induced by wind loads for ***Perma-Columns*** and No.1 Southern Yellow Pine posts.

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

Tests have not yet been conducted to isolate the shear strength of the steel bracket-to-concrete connection.

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.

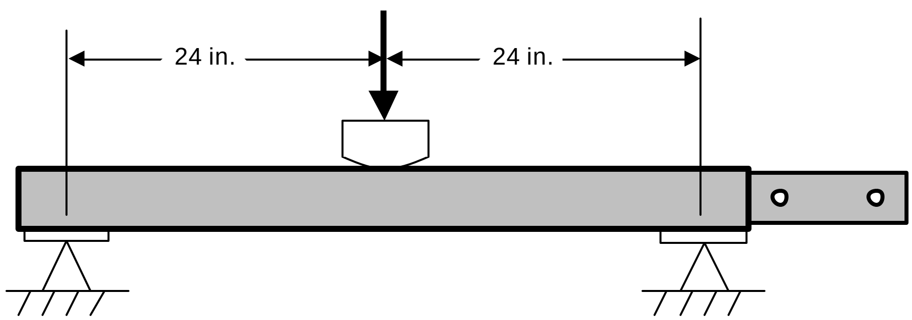

Figure A.1. Bending test set-up for ***Perma-Columns***.

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.

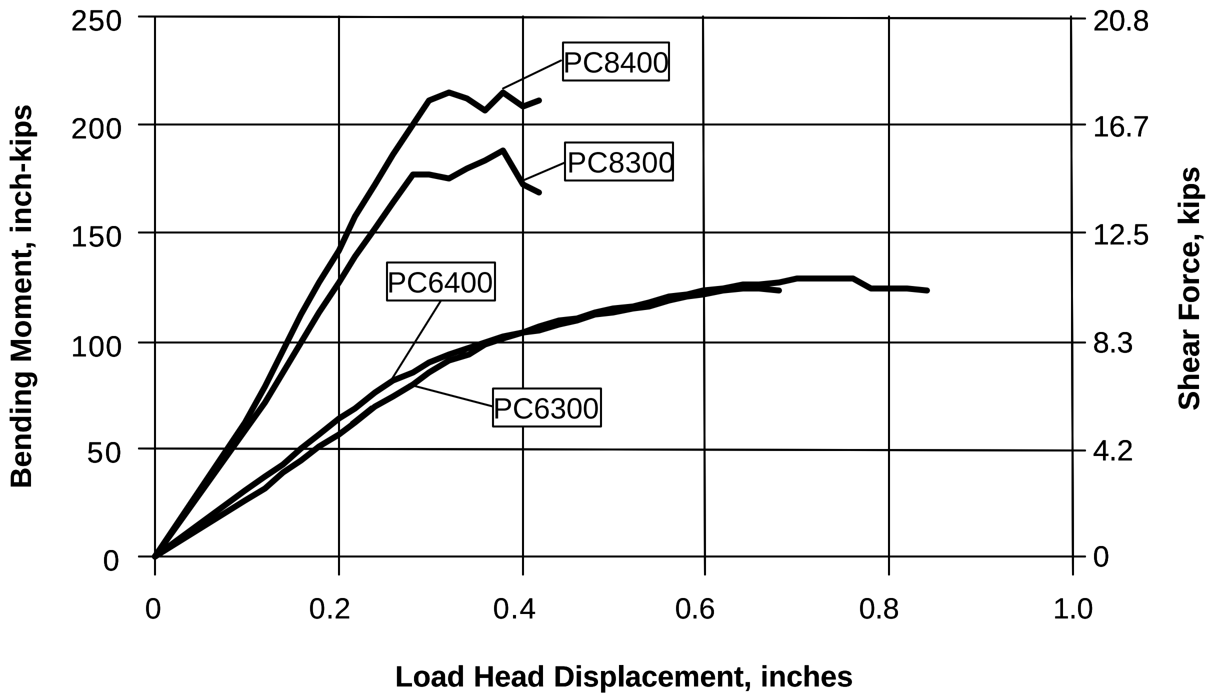

Figure A.2. Average load-head displacement versus shear force/midspan bending moment.

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

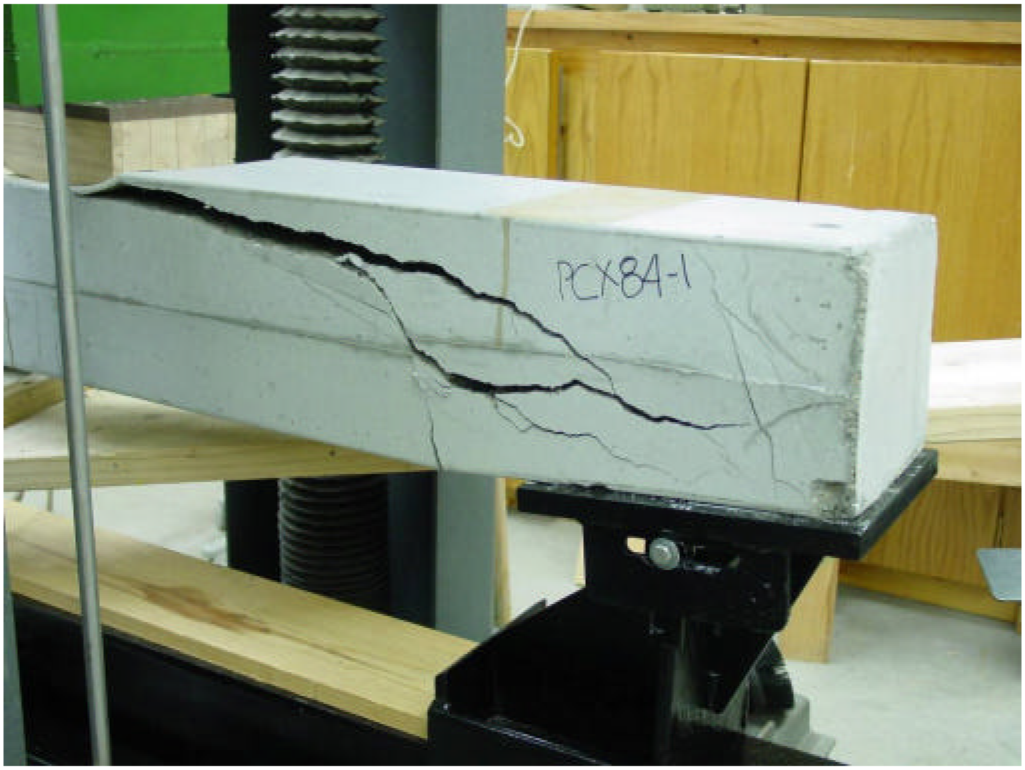

Figure A.4. Shear failure mode characteristic of all PC8300 and PC8400 series columns.

A.3 Comparisons

Table A.2 compares bending test results with ACI nominal shear and bending moment strength values ($\boldsymbol{V_n}$ and $\boldsymbol{M_n}$ values, respectively).

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

Perma-Column Series	Replicate Number	Average Test Maximum	ACI Nominal Strength	Ratio, Test/ACI	Test Underestimates Maximum Strength?
PC6300	Shear, kips	5.28	4.35*	1.21	Yes
	Bending Moment, inch-kips	126.6	97.2	1.30	No
PC6400	Shear, kips	5.54	5.48*	1.01	Yes
	Bending Moment, inch-kips	132.9	104.5	1.27	No
PC8300	Shear, kips	8.18	6.46*	1.27	No
	Bending Moment, inch-kips	196.4	197.4	1.00	Yes
PC8400	Shear, kips	9.51	8.07*	1.18	No
	Bending Moment, inch-kips	228.2	206.8	1.10	Yes

* From Table 3.4.1 $\boldsymbol{M_u/V_u}$ = 12 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).

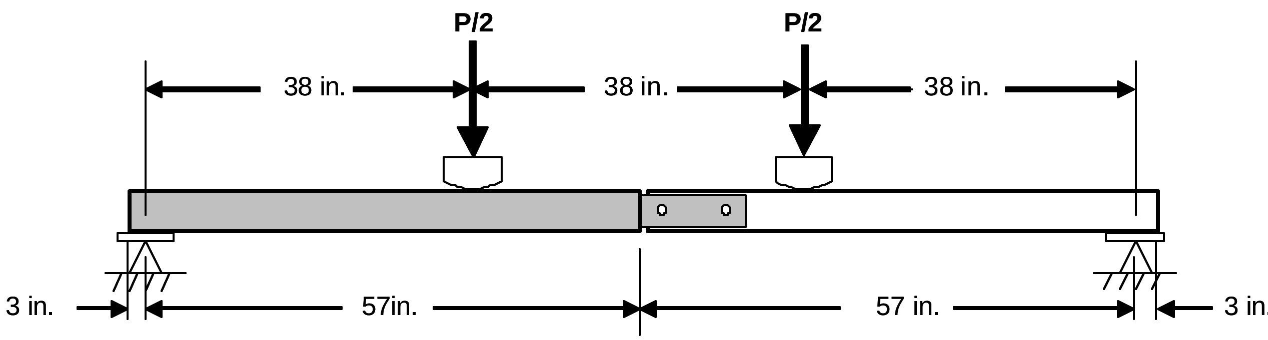

Figure B.1. Bending test set-up for ***Perma-Column*** steel bracket to concrete column connection.

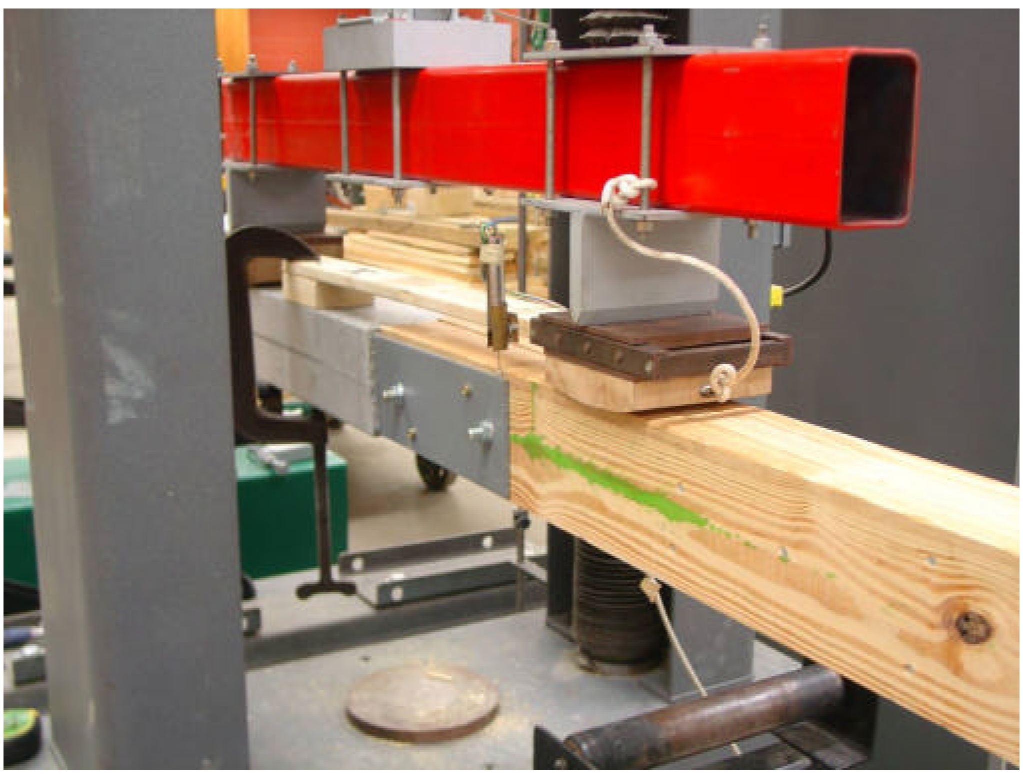

Figure B.2. Bending test set-up for ***Perma-Column*** steel bracket to concrete column connection showing LVDT used to measure joint rotation.

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 where 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 concrete-to-steel bracket 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.

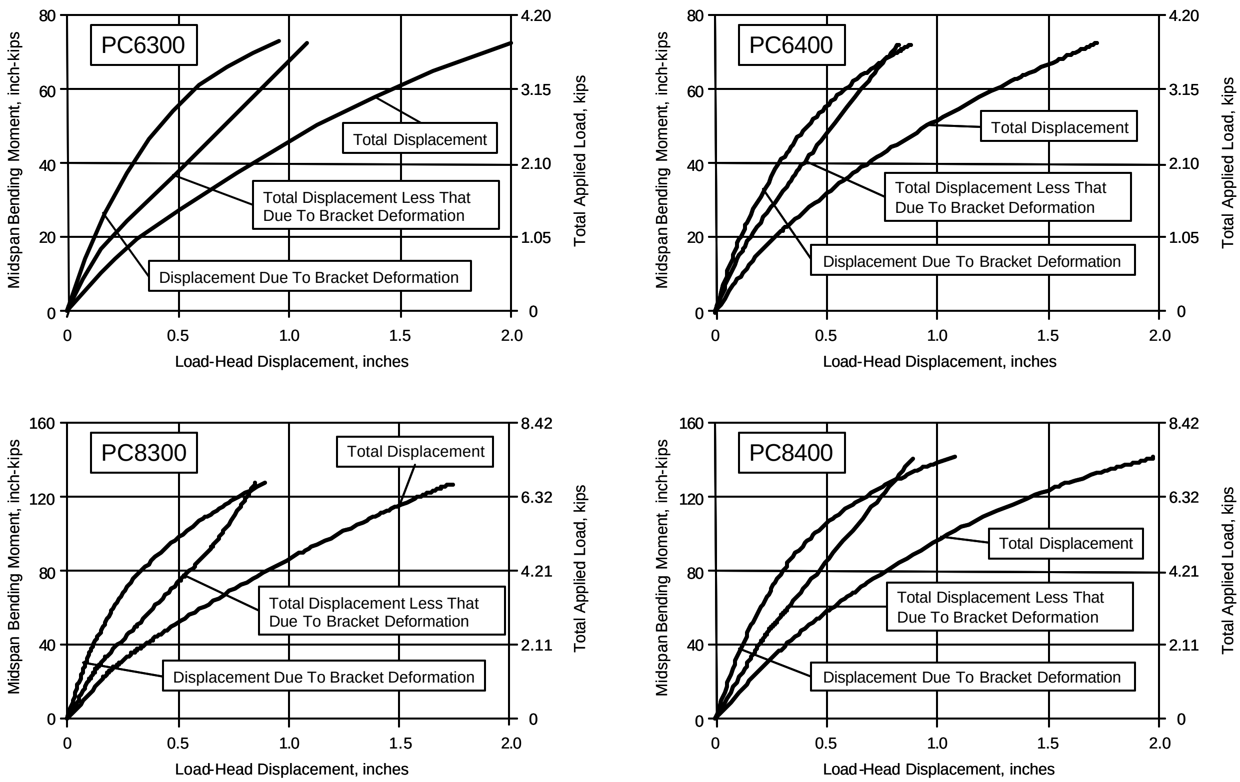

Figure B.3. Relative portion of total load-head displacement due to rotation of concrete-to-steel bracket connection.

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.

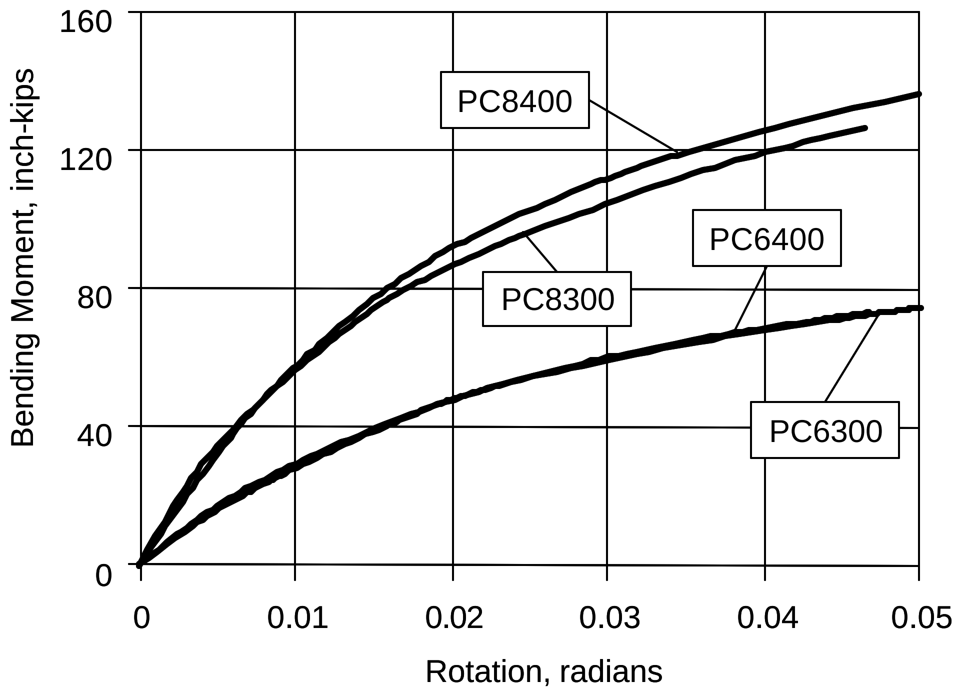

Figure B.4. Joint rotation versus bending moment for concrete column-to-steel bracket connections. Plotted from data compiled in Table B.2.

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

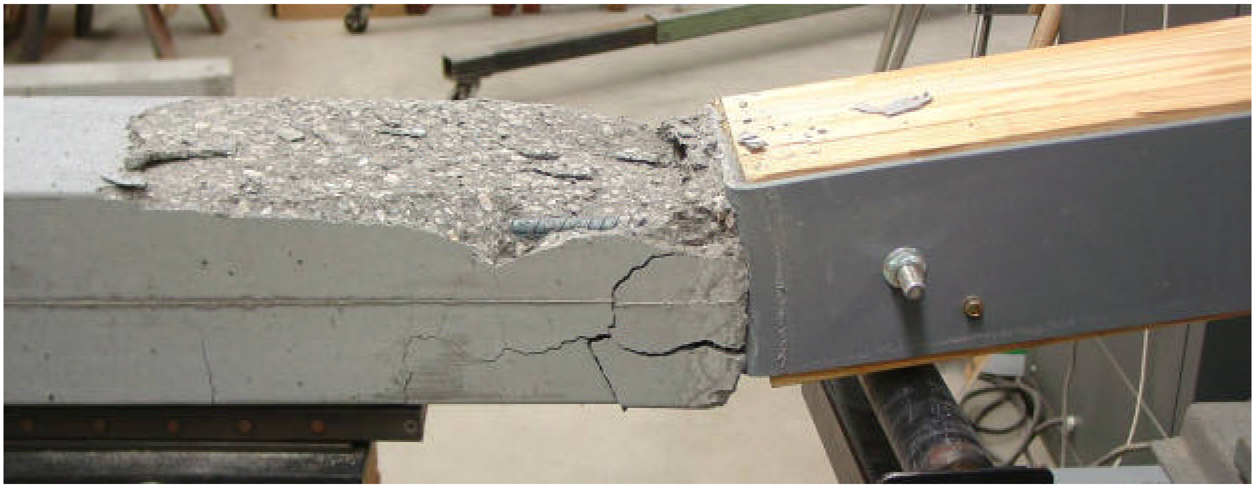

Figure B.5. Typical bending failure mode for concrete-to-steel bracket connection.

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.

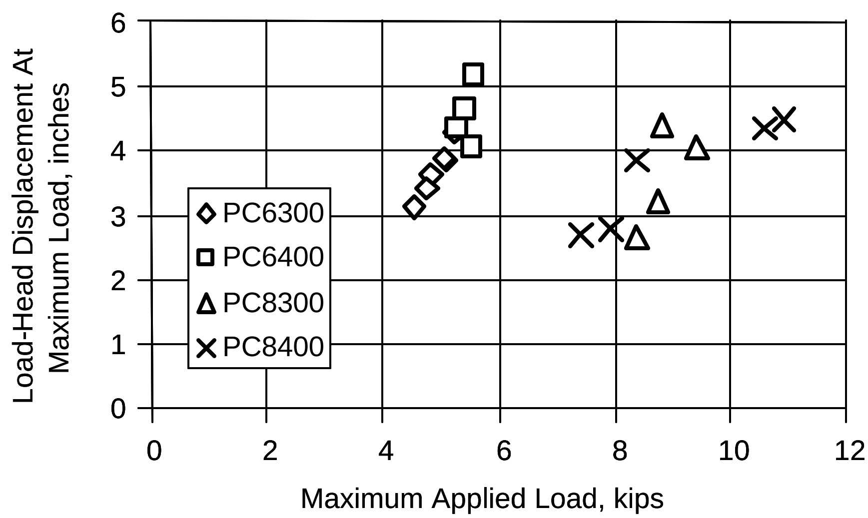

Figure B.7. Relationship between maximum applied load and load-head displacement.

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.