DESIGN GUIDE APPENDICES

For

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APPENDIX 1
(Wind Load Calculations)

ASCE 7-05 Wind Load Calculations

Project Name: Sample 80 ft Wide x 22 ft Tall Post-Frame Building

WIND LOAD: Method 2 - Analytical Procedure - Low-Rise

Building Inputs:

Length Parallel to Ridge, B (length, typ)	n/a	ft
Length Normal to Ridge, L (width, typ)	80	ft
Wall Height, z	22	ft, max
Building Midheight, h	28	ft
Roof Pitch (rise per 12 units of run)	3.5	12
Building Category	II	
Exposure Category	C	

Calculation Inputs:

Basic Wind Speed, V	90	mph
Topographic Factor, K_{zt}	1.00	
Envelope:	Enclosed Building	
Wind Directionality Factor	0.85	
Roof Overhang (ft)	1	

Definitions:

Case A - Wind Direction Normal to Roof Ridge, Pressure Coefficients Vary With Roof Angle.
Case B - Wind Direction Parallel to Ridge, Pressure Coefficients are Constant for all Roof Angles.
Interior Zones - Zones 1 - 6 Below
Edge Zones - Zones 1E - 6E Below

Intermediate Calculations:

Importance Factor, I	1.00	*Table 6-1*
Nom. Height of Atmospheric Boundary (z_g)	900	
Vel. Press. Exp. Coefficient, K_z	0.97	*Table 6-3, Note 2*
3-s Gust Speed Power Law Exponent (α)	9.5	

Calculated Roof Angle	16.3	deg
Internal Press. Coefficient, Gc_{pi}	0.18	-0.18
Velocity Pressure, q_h	17.0	psf

Wind Load Results:

A. Main Wind Force Resisting System: ASCE7-05, Figure 6-10 **Equation: $p = q_h[(GC_{pf}-(Gc_{pi})]$**

	Case A				Case B			
		P (psf)				P (psf)		
	Gcpf	I*	II**	III~	Gcpf	I*	II**	III~
Zone 1: Windward Side Wall	0.50	5.4	**11.5**	8.5	0.40	3.7	9.8842	6.82
Zone 2: Windward Roof	-0.69	-14.8	-8.7	-11.8	-0.69	-14.8	-8.691	-11.76
Zone 3: Leeward Roof	-0.45	-10.8	-4.6	-7.7	-0.37	-9.4	-3.238	-6.31
Zone 4: Leeward Side Wall	-0.40	-9.8	-3.7	-6.7	-0.29	-8.0	-1.875	-4.94
Zone 5: Windward Gable Wall	-				-0.45	-10.7	-4.601	-7.67
Zone 6: Leeward Gable Wall	-				-0.45	-10.7	-4.601	-7.67
Zone 1E: Windward Side Wall Edge	0.75	9.8	**15.9**	12.8	0.61	7.3	13.463	10.40
Zone 2E: Windward Roof Edge	-1.07	-21.3	-15.2	-18.2	-1.07	-21.3	-15.17	-18.23
Zone 3E: Leeward Roof Edge	-0.65	-14.1	-8.0	-11.1	-0.53	-12.1	-5.965	-9.03
Zone 4E: Leeward Side Wall Edge	-0.59	-13.1	-6.9	-10.0	-0.43	-10.4	-4.26	-7.33

* Internal Pressure Positive ** Internal Pressure Negative ~ Internal Pressure Zero

Column wind load calculated using a weighted average of pressures on first column in from corner = **12.6** **psf**

(Assumes that 25% of column tributary area falls under Windward Side Wall Edge pressure)

VERTICAL LOAD

Roof Dead Load = 25% of Total Load
Roof Snow Load = 75% of Total Load

ASCE 7-10 Wind Load Calculations

Project Name: Sample 80 ft Wide x 22 ft Tall Post-Frame Building

Envelope Procedure for Low-Rise Buildings

Building Inputs:

Length Parallel to Ridge, B (length, typ)	n/a	ft
Length Normal to Ridge, L (width, typ)	80	ft
Wall Height, z	22	ft
Building Midheight, h	28	ft
Roof Pitch (rise per 12 units of run)	3.5	12
Risk Category	II	
Exposure Category	C	

Calculation Inputs:

Basic Wind Speed, V	115	mph
Topographic Factor, K_{zt}	1.00	
Wind Directionality Factor, K_d	0.85	
Enclosure Classification:	Enclosed Building	
Roof Overhang (ft)	1	

Definitions:

Case A - Wind Direction Normal to Roof Ridge, Pressure Coefficients Vary With Roof Angle.
Case B - Wind Direction Parallel to Ridge, Pressure Coefficients are Constant for all Roof Angles.
Interior Zones - Zones 1 - 6 Below
Edge Zones - Zones 1E - 6E Below

Intermediate Calculations:

			Calculated Roof Angle	16.3	deg
Nom. Height of Atmospheric Boundary (z_g)	900	*(Table 26.9-1)*	Internal Press. Coefficient, Gc_{pi}	0.18	-0.18
Vel. Press. Exp. Coefficient, K_z	0.97	*(Table 28.3-1)*			
3-s Gust Speed Power Law Exponent (α)	9.5	*(Table 26.9-1)*	Velocity Pressure, q_h	27.8	psf

Wind Load Results:

A. Main Wind Force Resisting System: ASCE7-10, Figure 28.4-1 **Equation: p = qh[(GCpf - (GCpi)]**

	Case A				Case B			
		P (psf)				P (psf)		
	Gcpf	I*	II**	III~	Gcpf	I*	II**	III~
Zone 1: Windward Side Wall	0.50	8.8	**18.9**	13.8	-0.45	-17.5	-7.5	-12.5
Zone 2: Windward Roof	-0.69	-24.2	-14.2	-19.2	-0.69	-24.2	-14.2	-19.2
Zone 3: Leeward Roof	-0.45	-17.6	-7.6	-12.6	-0.37	-15.3	-5.3	-10.3
Zone 4: Leeward Side Wall	-0.40	-16.0	-6.0	-11.0	-0.45	-17.5	-7.5	-12.5
Zone 5: Windward Gable Wall					0.40	6.1	16.1	11.1
Zone 6: Leeward Gable Wall					-0.29	-13.1	-3.1	-8.1
Zone 1E: Windward Side Wall Edge	0.75	15.9	**25.9**	20.9	-0.48	-18.4	-8.3	-13.4
Zone 2E: Windward Roof Edge	-1.07	-34.8	-24.8	-29.8	-1.07	-34.8	-24.8	-29.8
Zone 3E: Leeward Roof Edge	-0.65	-23.1	-13.1	-18.1	-0.53	-19.8	-9.7	-14.7
Zone 4E: Leeward Side Wall Edge	-0.59	-21.4	-11.3	-16.4	-0.48	-18.4	-8.3	-13.4
Zone 5E: Windward Gable Wall Edge					0.61	12.0	22.0	17.0
Zone 6E: Leeward Gable Wall Edge					-0.43	-17.0	-7.0	-12.0

* Internal Pressure Positive ** Internal Pressure Negative ~ Internal Pressure Zero

Column wind load calculated using a weighted average of pressures on first column in from corner = **20.6 psf**

(Assumes that 25% of column tributary area falls under Windward Side Wall Edge pressure)

VERTICAL LOAD

Roof Dead Load = 25% of Total Load
Roof Snow Load = 75% of Total Load

APPENDIX 2
(Column Connection Calculations)

STEEL BRACKET CONNECTION DESIGN FOR PC6300 (STEEL BRACKET TO #1 SYP WOOD COLUMN)

Maximum Moment in Bracket "Element", in-lb	M_{max}	19054	(ASD)
Spacing Between Fastener Groups, in	s	8.25	← Actual centroidal spacing based on fastener stiffnesses
Load on Fastener Group, lbf	R	2310	
Slip Modulus for 1/2" Bolt, lbf/in	k_b	85500	
Slip Modulus for 1/4" SDS Screw, lbf/in	k_{SDS}	28700	
Number of SDS Screws per Fastener Group	N_{SDS}	2	(one from each side, near bolt)
Load on Bolt, lbf	P_b	1382	$P_b = Rk_b / (k_b + N_{SDS}k_{SDS})$
Load on SDS screws, lbf	P_{SDS}	928	$P_{SDS} = RN_{SDS}k_{SDS} / (k_b + N_{SDS}k_{SDS})$

BOLTS - WOOD TO STEEL PLATES - DOUBLE SHEAR - NDS 2005, ASD

Number of bolts	N	1		$F_{em, par}$	6160
Bolt Diameter (in)	D	0.5		$F_{em, perp}$	3626
Main Member Thickness (in)	t_m	4.5		F_{em}	3626
Side Member Thickness (in)	t_s	0.25		R_e	0.042
Dowel Bearing Strength (psi)	F_{es}	87000		K_θ	1.250
Bolt Yield Strength (psi)	F_{yb}	85000	*Proof Stress for A325*	$1+R_e$	1.042
Max Angle Load to Grain (deg)	θ	90		$2+R_e$	2.042
Specific Gravity	G	0.55		k_3	12.328
Reference Lateral Design Value (Z)	Z	1368	← *NDS 2005, Table 11LA-11*	I_m	1631
Duration Factor	C_D	1.6		III_s	1368
Geometry Factor	C_Δ	1		IV	1755
Wet Service Factor	C_M	1			
Adjusted Lateral Design Value (lbs)	Z'	2189	$Z' = Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Total Allowable Connection Capacity (lbs)	V	2189	← V=NZ'		
Actual/Allowable =	*1382 ÷ 2189*	=	*0.63*	< *1.0*	*PASS*

SDS SIMPSON SCREWS - SHEAR - STEEL TO WOOD (ESR-2236)

Number of screws	N	2	
Screw Diameter (in)	D	0.242	
Screw Length (in)	L	3	
Thickness of Steel Plate (in)	t_s	0.25	
Thickness of Wood Member (in)	t_w	4.5	
Screw Penetration into main member (in)	p	2.75	
Specific Gravity of Wood Member		0.55	
Lateral Design Value (lbs)	Z	420	*(ESR-2236)*
Duration Factor	C_D	1.6	
End Grain Factor	C_{eg}	1	
Toe Nail Factor	C_{tn}	1	
Geometry Factor	C_Δ	1	*(ESR-2236)*
Wet Service Factor	C_M	1	
Adjusted Lateral Design Value (lbs)	Z'	672	$Z' = Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$
Total Allowable Connection Capacity (lbs)	V	1344	V=NZ'
Actual/Allowable = *928 ÷ 1344*	=	*0.69*	< *1.0* *PASS*

ALLOWABLE MOMENT CAPACITY - PROVIDED BY BOLT AND SCREW GROUPS

Total Unadjusted Allowable Shear Capacity (lbs)	V_T	2208	$V_T = (V_{bolt} + V_{screw}) \div (C_D \times C_{eg} \times C_{tn} \times C_\Delta \times C_M)$
Total Unadjusted Allowable Moment Capacity (in-kips)	M	18.2	$M = (V_T \times s)/1000$

STEEL BRACKET CONNECTION DESIGN FOR PC6400 (STEEL BRACKET TO #1 SYP WOOD COLUMN)

Maximum Moment in Bracket "Element", in-lb	M_{max}	22559	(ASD)
Spacing Between Fastener Groups, in	s	13.25	← Actual centroidal spacing based on fastener stiffnesses
Load on Fastener Group, lbf	R	1703	
Slip Modulus for 1/2" Bolt, lbf/in	k_b	85500	
Slip Modulus for 1/4" SDS Screw, lbf/in	k_{SDS}	28700	
Number of SDS Screws per Fastener Group	N_{SDS}	2	(one from each side, near bolt)
Load on Bolt, lbf	P_b	1019	$P_b = Rk_b / (k_b + N_{SDS}k_{SDS})$
Load on SDS screws, lbf	P_{SDS}	684	$P_{SDS} = RN_{SDS}k_{SDS} / (k_b + N_{SDS}k_{SDS})$

BOLTS - WOOD TO STEEL PLATES - DOUBLE SHEAR - NDS 2005, ASD

Number of bolts	N	1		$F_{em, par}$	6160
Bolt Diameter (in)	D	0.5		$F_{em, perp}$	3626
Main Member Thickness (in)	t_m	6		F_{em}	3626
Side Member Thickness (in)	t_s	0.25		R_e	0.042
Dowel Bearing Strength (psi)	F_{es}	87000		K_θ	1.250
Bolt Yield Strength (psi)	F_{yb}	85000	*Proof Stress for A325*	$1+R_e$	1.042
Max Angle Load to Grain (deg)	θ	90		$2+R_e$	2.042
Specific Gravity	G	0.55		k_3	12.328
Reference Lateral Design Value (Z)	Z	1368	← *NDS 2005, Table 11LA-11*	I_m	2175
Duration Factor	C_D	1.6		III_s	1368
Geometry Factor	C_Δ	1		IV	1755
Wet Service Factor	C_M	1			
Adjusted Lateral Design Value (lbs)	Z'	2189	$Z'=Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Total Allowable Connection Capacity (lbs)	V	2189	← V=NZ'		
Actual/Allowable = *1019 ÷ 2189*	=	*0.47*	< *1.0* *PASS*		

SDS SIMPSON SCREWS - SHEAR - STEEL TO WOOD (ESR-2236)

Number of screws	N	2	
Screw Diameter (in)	D	0.242	
Screw Length (in)	L	3	
Thickness of Steel Plate (in)	t_s	0.25	
Thickness of Wood Member (in)	t_w	6	
Screw Penetration into main member (in)	p	2.75	
Specific Gravity of Wood Member		0.55	
Lateral Design Value (lbs)	Z	420	*(ESR-2236)*
Duration Factor	C_D	1.6	
End Grain Factor	C_{eg}	1	
Toe Nail Factor	C_{tn}	1	
Geometry Factor	C_Δ	1	*(ESR-2236)*
Wet Service Factor	C_M	1	
Adjusted Lateral Design Value (lbs)	Z'	672	$Z'=Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$
Total Allowable Connection Capacity (lbs)	V	1344	V=NZ'
Actual/Allowable = *684 ÷ 1344*	=	*0.51*	< *1.0* *PASS*

ALLOWABLE MOMENT CAPACITY - PROVIDED BY BOLT AND SCREW GROUPS

Total Unadjusted Allowable Shear Capacity (lbs)	V_T	2208	$V_T = (V_{bolt} + V_{screw}) \div (C_D \times C_{eg} \times C_{tn} \times C_\Delta \times C_M)$
Total Unadjusted Allowable Moment Capacity (in-kips)	M	29.3	$M = (V_T \times s)/1000$

STEEL BRACKET CONNECTION DESIGN FOR PC6600 (STEEL BRACKET TO #1 SYP WOOD COLUMN)

Maximum Moment in Bracket "Element", in-lb	M_{max}	20218	(ASD)
Spacing Between Fastener Groups, in	s	8.25	← Actual centroidal spacing based on fastener stiffnesses
Load on Fastener Group, lbf	R	2451	
Slip Modulus for 1/2" Bolt, lbf/in	k_b	85500	
Slip Modulus for 1/4" SDS Screw, lbf/in	k_{SDS}	28700	
Number of SDS Screws per Fastener Group	N_{SDS}	2	(one from each side, near bolt)
Load on Bolt, lbf	P_b	1466	$P_b = Rk_b / (k_b + N_{SDS}k_{SDS})$
Load on SDS screws, lbf	P_{SDS}	984	$P_{SDS} = RN_{SDS}k_{SDS} / (k_b + N_{SDS}k_{SDS})$

BOLTS - WOOD TO STEEL PLATES - DOUBLE SHEAR - NDS 2005, ASD

Number of bolts	N	1		$F_{em, par}$	6160
Bolt Diameter (in)	D	0.5		$F_{em, perp}$	3626
Main Member Thickness (in)	t_m	5.5		F_{em}	3626
Side Member Thickness (in)	t_s	0.25		R_e	0.042
Dowel Bearing Strength (psi)	F_{es}	87000		K_θ	1.250
Bolt Yield Strength (psi)	F_{yb}	85000	*Proof Stress for A325*	$1+R_e$	1.042
Max Angle Load to Grain (deg)	θ	90		$2+R_e$	2.042
Specific Gravity	G	0.55		k_3	12.328
Reference Lateral Design Value (Z)	Z	1368	← *NDS 2005, Table 11LA-11*	I_m	1994
Duration Factor	C_D	1.6		III_s	1368
Geometry Factor	C_Δ	1		IV	1755
Wet Service Factor	C_M	1			
Adjusted Lateral Design Value (lbs)	Z'	2189	$Z' = Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Total Allowable Connection Capacity (lbs)	V	2189	← V=NZ'		
Actual/Allowable = *1466 ÷ 2189*	=	*0.67*	< 1.0 PASS		

SDS SIMPSON SCREWS - SHEAR - STEEL TO WOOD (ESR-2236)

Number of screws	N	2	
Screw Diameter (in)	D	0.242	
Screw Length (in)	L	3	
Thickness of Steel Plate (in)	t_s	0.25	
Thickness of Wood Member (in)	t_w	5.5	
Screw Penetration into main member (in)	p	2.75	
Specific Gravity of Wood Member		0.55	
Lateral Design Value (lbs)	Z	420	*(ESR-2236)*
Duration Factor	C_D	1.6	
End Grain Factor	C_{eg}	1	
Toe Nail Factor	C_{tn}	1	
Geometry Factor	C_Δ	1	*(ESR-2236)*
Wet Service Factor	C_M	1	
Adjusted Lateral Design Value (lbs)	Z'	672	$Z' = Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$
Total Allowable Connection Capacity (lbs)	V	1344	V=NZ'
Actual/Allowable = *984 ÷ 1344*	=	*0.73*	< 1.0 PASS

ALLOWABLE MOMENT CAPACITY - PROVIDED BY BOLT AND SCREW GROUPS

Total Unadjusted Allowable Shear Capacity (lbs)	V_T	2208	$V_T = (V_{bolt} + V_{screw}) \div (C_D \times C_{eg} \times C_{tn} \times C_\Delta \times C_M)$
Total Unadjusted Allowable Moment Capacity (in-kips)	M	18.2	$M = (V_T \times s)/1000$

STEEL BRACKET CONNECTION DESIGN FOR PC8300 (STEEL BRACKET TO #1 SYP WOOD COLUMN)

Maximum Moment in Bracket "Element", in-lb	M_{max}	42460	(ASD)
Spacing Between Fastener Groups, in	s	11.6	← Actual centroidal spacing based on fastener stiffnesses
Load on Fastener Group, lbf	R	3660	
Slip Modulus for 1/2" Bolt, lbf/in	k_b	85500	
Slip Modulus for 1/4" SDS Screw, lbf/in	k_{SDS}	28700	
Number of SDS Screws per Fastener Group	N_{SDS}	4	(two from each side, near bolt)
Load on Bolt, lbf	P_b	1562	$P_b = Rk_b / (k_b + N_{SDS}k_{SDS})$
Load on SDS screws, lbf	P_{SDS}	2098	$P_{SDS} = RN_{SDS}k_{SDS} / (k_b + N_{SDS}k_{SDS})$

BOLTS - WOOD TO STEEL PLATES - DOUBLE SHEAR - NDS 2005, ASD

Number of bolts	N	1		$F_{em, par}$	6160
Bolt Diameter (in)	D	0.5		$F_{em, perp}$	3626
Main Member Thickness (in)	t_m	4.5		F_{em}	3626
Side Member Thickness (in)	t_s	0.25		R_e	0.042
Dowel Bearing Strength (psi)	F_{es}	87000		K_θ	1.250
Bolt Yield Strength (psi)	F_{yb}	85000	*Proof Stress for A325*	$1+R_e$	1.042
Max Angle Load to Grain (deg)	θ	90		$2+R_e$	2.042
Specific Gravity	G	0.55		k_3	12.328
Reference Lateral Design Value (Z)	Z	1368	← *NDS 2005, Table 11LA-11*	I_m	1631
Duration Factor	C_D	1.6		III_s	1368
Geometry Factor	C_Δ	1		IV	1755
Wet Service Factor	C_M	1			
Adjusted Lateral Design Value (lbs)	Z'	2189	$Z' = Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Total Allowable Connection Capacity (lbs)	V	2189	← V=NZ'		
Actual/Allowable = *1562 ÷ 2189*	=	*0.71*	< *1.0* *PASS*		

SDS SIMPSON SCREWS - SHEAR - STEEL TO WOOD (ESR-2236)

Number of screws	N	4	
Screw Diameter (in)	D	0.242	
Screw Length (in)	L	3	
Thickness of Steel Plate (in)	t_s	0.25	
Thickness of Wood Member (in)	t_w	4.5	
Screw Penetration into main member (in)	p	2.75	
Specific Gravity of Wood Member		0.55	
Lateral Design Value (lbs)	Z	420	*(ESR-2236)*
Duration Factor	C_D	1.6	
End Grain Factor	C_{eg}	1	
Toe Nail Factor	C_{tn}	1	
Geometry Factor	C_Δ	1	*(ESR-2236)*
Wet Service Factor	C_M	1	
Adjusted Lateral Design Value (lbs)	Z'	672	$Z' = Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$
Total Allowable Connection Capacity (lbs)	V	2688	V=NZ'
Actual/Allowable = *2098 ÷ 2688*	=	*0.78*	< *1.0* *PASS*

ALLOWABLE MOMENT CAPACITY - PROVIDED BY BOLT AND SCREW GROUPS

Total Unadjusted Allowable Shear Capacity (lbs)	V_T	3048	$V_T = (V_{bolt} + V_{screw}) \div (C_D \times C_{eg} \times C_{tn} \times C_\Delta \times C_M)$
Total Unadjusted Allowable Moment Capacity (in-kips)	M	35.4	$M = (V_T \times s)/1000$

STEEL BRACKET CONNECTION DESIGN FOR PC8400 (STEEL BRACKET TO #1 SYP WOOD COLUMN)

Maximum Moment in Bracket "Element", in-lb	M_{max}	43193	(ASD)
Spacing Between Fastener Groups, in	s	11.6	← Actual centroidal spacing based on fastener stiffnesses
Load on Fastener Group, lbf	R	3724	
Slip Modulus for 1/2" Bolt, lbf/in	k_b	85500	
Slip Modulus for 1/4" SDS Screw, lbf/in	k_{SDS}	28700	
Number of SDS Screws per Fastener Group	N_{SDS}	4	(two from each side, near bolt)
Load on Bolt, lbf	P_b	1589	$P_b = Rk_b / (k_b + N_{SDS}k_{SDS})$
Load on SDS screws, lbf	P_{SDS}	2134	$P_{SDS} = RN_{SDS}k_{SDS} / (k_b + N_{SDS}k_{SDS})$

BOLTS - WOOD TO STEEL PLATES - DOUBLE SHEAR - NDS 2005, ASD

Number of bolts	N	1		$F_{em, par}$	6160
Bolt Diameter (in)	D	0.5		$F_{em, perp}$	3626
Main Member Thickness (in)	t_m	6		F_{em}	3626
Side Member Thickness (in)	t_s	0.25		R_e	0.042
Dowel Bearing Strength (psi)	F_{es}	87000		K_θ	1.250
Bolt Yield Strength (psi)	F_{yb}	85000	*Proof Stress for A325*	$1+R_e$	1.042
Max Angle Load to Grain (deg)	θ	90		$2+R_e$	2.042
Specific Gravity	G	0.55		k_3	12.328
Reference Lateral Design Value (Z)	Z	1368	← *NDS 2005, Table 11LA-11*	I_m	2175
Duration Factor	C_D	1.6		III_s	1368
Geometry Factor	C_Δ	1		IV	1755
Wet Service Factor	C_M	1			
Adjusted Lateral Design Value (lbs)	Z'	2189	$Z' = Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Total Allowable Connection Capacity (lbs)	V	2189	← V=NZ'		
Actual/Allowable = *1589 ÷ 2189*	=	*0.73*	< *1.0* *PASS*		

SDS SIMPSON SCREWS - SHEAR - STEEL TO WOOD (ESR-2236)

Number of screws	N	4	
Screw Diameter (in)	D	0.242	
Screw Length (in)	L	3	
Thickness of Steel Plate (in)	t_s	0.25	
Thickness of Wood Member (in)	t_w	6	
Screw Penetration into main member (in)	p	2.75	
Specific Gravity of Wood Member		0.55	
Lateral Design Value (lbs)	Z	420	*(ESR-2236)*
Duration Factor	C_D	1.6	
End Grain Factor	C_{eg}	1	
Toe Nail Factor	C_{tn}	1	
Geometry Factor	C_Δ	1	*(ESR-2236)*
Wet Service Factor	C_M	1	
Adjusted Lateral Design Value (lbs)	Z'	672	$Z' = Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$
Total Allowable Connection Capacity (lbs)	V	2688	V=NZ'
Actual/Allowable = *2134 ÷ 2688*	=	*0.79*	< *1.0* *PASS*

ALLOWABLE MOMENT CAPACITY - PROVIDED BY BOLT AND SCREW GROUPS

Total Unadjusted Allowable Shear Capacity (lbs)	V_T	3048	$V_T = (V_{bolt} + V_{screw}) \div (C_D \times C_{eg} \times C_{tn} \times C_\Delta \times C_M)$
Total Unadjusted Allowable Moment Capacity (in-kips)	M	35.4	$M = (V_T \times s)/1000$

STEEL BRACKET CONNECTION DESIGN FOR PC8500 (STEEL BRACKET TO #1 SYP WOOD COLUMN)

Maximum Moment in Bracket "Element", in-lb	M_{max}	41730	(ASD)
Spacing Between Fastener Groups, in	s	11.6	← Actual centroidal spacing based on fastener stiffnesses
Load on Fastener Group, lbf	R	3597	
Slip Modulus for 1/2" Bolt, lbf/in	k_b	85500	
Slip Modulus for 1/4" SDS Screw, lbf/in	k_{SDS}	28700	
Number of SDS Screws per Fastener Group	N_{SDS}	4	(two from each side, near bolt)
Load on Bolt, lbf	P_b	1536	$P_b = Rk_b / (k_b + N_{SDS}k_{SDS})$
Load on SDS screws, lbf	P_{SDS}	2062	$P_{SDS} = RN_{SDS}k_{SDS} / (k_b + N_{SDS}k_{SDS})$

BOLTS - WOOD TO STEEL PLATES - DOUBLE SHEAR - NDS 2005, ASD

Number of bolts	N	1		$F_{em, par}$	6160
Bolt Diameter (in)	D	0.5		$F_{em, perp}$	3626
Main Member Thickness (in)	t_m	7.5		F_{em}	3626
Side Member Thickness (in)	t_s	0.25		R_e	0.042
Dowel Bearing Strength (psi)	F_{es}	87000		K_θ	1.250
Bolt Yield Strength (psi)	F_{yb}	85000	*Proof Stress for A325*	$1+R_e$	1.042
Max Angle Load to Grain (deg)	θ	90		$2+R_e$	2.042
Specific Gravity	G	0.55		k_3	12.328
Reference Lateral Design Value (Z)	Z	1368	← *NDS 2005, Table 11LA-11*	I_m	2719
Duration Factor	C_D	1.6		III_s	1368
Geometry Factor	C_Δ	1		IV	1755
Wet Service Factor	C_M	1			
Adjusted Lateral Design Value (lbs)	Z'	2189	$Z' = Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Total Allowable Connection Capacity (lbs)	V	2189	← V=NZ'		
Actual/Allowable = *1536 ÷ 2189*	=	*0.70*	< *1.0* *PASS*		

SDS SIMPSON SCREWS - SHEAR - STEEL TO WOOD (ESR-2236)

Number of screws	N	4	
Screw Diameter (in)	D	0.242	
Screw Length (in)	L	3	
Thickness of Steel Plate (in)	t_s	0.25	
Thickness of Wood Member (in)	t_w	7.5	
Screw Penetration into main member (in)	p	2.75	
Specific Gravity of Wood Member		0.55	
Lateral Design Value (lbs)	Z	420	*(ESR-2236)*
Duration Factor	C_D	1.6	
End Grain Factor	C_{eg}	1	
Toe Nail Factor	C_{tn}	1	
Geometry Factor	C_Δ	1	*(ESR-2236)*
Wet Service Factor	C_M	1	
Adjusted Lateral Design Value (lbs)	Z'	672	$Z' = Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$
Total Allowable Connection Capacity (lbs)	V	2688	V=NZ'
Actual/Allowable = *2062 ÷ 2688*	=	*0.77*	< *1.0* *PASS*

ALLOWABLE MOMENT CAPACITY - PROVIDED BY BOLT AND SCREW GROUPS

Total Unadjusted Allowable Shear Capacity (lbs)	V_T	3048	$V_T = (V_{bolt} + V_{screw}) \div (C_D \times C_{eg} \times C_{tn} \times C_\Delta \times C_M)$
Total Unadjusted Allowable Moment Capacity (in-kips)	M	35.4	$M = (V_T \times s)/1000$

STEEL BRACKET CONNECTION DESIGN FOR PC6300 (STEEL BRACKET TO CONCRETE)

Factored Moment in Bracket "Element", in-lb		M_U	38111	(LRFD)
Factored Shear in Bracket "Element", lbf		V_U	1566	(LRFD)
BENDING STRENGTH				
Ultimate Bending Strength, lbf-in, at	0.04523 Radians	M_{ult}	70400	(Engineering Design Manual, Table B.2)
Bending Moment at 0.03 Radians Rotation, lbf-in		$M_{0.03}$	59400	
Reduction Factor		φ	0.9	
Design Bending Moment, k-in			53460	
Actual/Allowable =	*38111 ÷ 53460*	=	*0.71*	< *1.0* *PASS*

SHEAR STRENGTH				
Tests show that shear strength of the bracket is at least as strong as the shear strength of concrete Perma-Column base, see Section 4.2 of the Perma-Column Engineering Design Manual by David R. Bohnhoff, Ph.D., P.E. The shear strength of the concrete Perma-Column base is shown below.				
Reduction Factor		φ	0.75	
Design Shear Strength, lbf		φV_n	3177	(W/O Increases from Axial Compressive Forces)
Actual/Allowable =	*1566 ÷ 3177*	=	*0.49*	< *1.0* *PASS*

STEEL BRACKET CONNECTION DESIGN FOR PC6400 (STEEL BRACKET TO CONCRETE)

Factored Moment in Bracket "Element", in-lb		M_U	48494	(LRFD)
Factored Shear in Bracket "Element", lbf		V_U	1757	(LRFD)
BENDING STRENGTH				
Ultimate Bending Strength, lbf-in, at	0.0447 Radians	M_{ult}	70900	(Engineering Design Manual, Table B.2)
Bending Moment at 0.03 Radians Rotation, lbf-in		$M_{0.03}$	59000	
Reduction Factor		φ	0.9	
Design Bending Moment, k-in			53100	
Actual/Allowable =	*48494 ÷ 53100*	=	*0.91*	< *1.0* *PASS*

SHEAR STRENGTH				
Tests show that shear strength of the bracket is at least as strong as the shear strength of concrete Perma-Column base, see Section 4.2 of the Perma-Column Engineering Design Manual by David R. Bohnhoff, Ph.D., P.E. The shear strength of the concrete Perma-Column base is shown below.				
Reduction Factor		φ	0.75	
Design Shear Strength, lbf		φV_n	4066	(W/O Increases from Axial Compressive Forces)
Actual/Allowable =	*1757 ÷ 4066*	=	*0.43*	< *1.0* *PASS*

STEEL BRACKET CONNECTION DESIGN FOR PC6600 (STEEL BRACKET TO CONCRETE)

Factored Moment in Bracket "Element", in-lb	M_U	37887	(LRFD)
Factored Shear in Bracket "Element", lbf	V_U	1546	(LRFD)

BENDING STRENGTH

Ultimate Bending Strength, lbf-in, at 0.0447 Radians	M_{ult}	70900	(Engineering Design Manual, Table B.2)
Bending Moment at 0.03 Radians Rotation, lbf-in	$M_{0.03}$	59000	(Assume the results are similar to 4-ply 2x6)
Reduction Factor	φ	0.9	
Design Bending Moment, k-in		53100	
Actual/Allowable = *37887 ÷ 53100*	=	*0.71*	*< 1.0 PASS*

SHEAR STRENGTH

Tests show that shear strength of the bracket is at least as strong as the shear strength of concrete Perma-Column base, see Section 4.2 of the Perma-Column Engineering Design Manual by David R. Bohnhoff, Ph.D., P.E. The shear strength of the concrete Perma-Column base is shown below. Assume results for PC6600 bracket are similar to PC6400 bracket.

Reduction Factor	φ	0.75	
Design Shear Strength, lbf	φV_n	4066	(W/O Increases from Axial Compressive Forces)
Actual/Allowable = *1546 ÷ 4066*	=	*0.38*	*< 1.0 PASS*

STEEL BRACKET CONNECTION DESIGN FOR PC8300 (STEEL BRACKET TO CONCRETE)

Factored Moment in Bracket "Element", in-lb	M_U	79471	(LRFD)
Factored Shear in Bracket "Element", lbf	V_U	1981	(LRFD)

BENDING STRENGTH

Ultimate Bending Strength, lbf-in, at 0.0437 Radians	M_{ult}	123900	(Engineering Design Manual, Table B.2)
Bending Moment at 0.03 Radians Rotation, lbf-in	$M_{0.03}$	103700	
Reduction Factor	φ	0.9	
Design Bending Moment, k-in		93330	
Actual/Allowable = *79471 ÷ 93330*	=	*0.85*	*< 1.0 PASS*

SHEAR STRENGTH

Tests show that shear strength of the bracket is at least as strong as the shear strength of concrete Perma-Column base, see Section 4.2 of the Perma-Column Engineering Design Manual by David R. Bohnhoff, Ph.D., P.E. The shear strength of the concrete Perma-Column base is shown below.

Reduction Factor	φ	0.75	
Design Shear Strength, lbf	φV_n	4531	(W/O Increases from Axial Compressive Forces)
Actual/Allowable = *1981 ÷ 4531*	=	*0.44*	*< 1.0 PASS*

STEEL BRACKET CONNECTION DESIGN FOR PC8400 (STEEL BRACKET TO CONCRETE)

Factored Moment in Bracket "Element", in-lb	M_U	82355	(LRFD)	
Factored Shear in Bracket "Element", lbf	V_U	1985	(LRFD)	
BENDING STRENGTH				
Ultimate Bending Strength, lbf-in, at 0.05364 Radians	M_{ult}	138900	(Engineering Design Manual, Table B.2)	
Bending Moment at 0.03 Radians Rotation, lbf-in	$M_{0.03}$	111800		
Reduction Factor	φ	0.9		
Design Bending Moment, k-in		100620		
Actual/Allowable = *82355 ÷ 100620*	=	*0.82*	< *1.0*	*PASS*

SHEAR STRENGTH				
Tests show that shear strength of the bracket is at least as strong as the shear strength of concrete Perma-Column base, see Section 4.2 of the Perma-Column Engineering Design Manual by David R. Bohnhoff, Ph.D., P.E. The shear strength of the concrete Perma-Column base is shown below.				
Reduction Factor	φ	0.75		
Design Shear Strength, lbf	$\boldsymbol{\varphi}V_n$	5800	(W/O Increases from Axial Compressive Forces)	
Actual/Allowable = *1985 ÷ 5800*	=	*0.34*	< *1.0*	*PASS*

STEEL BRACKET CONNECTION DESIGN FOR PC8500 (STEEL BRACKET TO CONCRETE)

Factored Moment in Bracket "Element", in-lb	M_U	81495	(LRFD)	
Factored Shear in Bracket "Element", lbf	V_U	1975	(LRFD)	
BENDING STRENGTH				
Ultimate Bending Strength, lbf-in, at 0.05364 Radians	M_{ult}	138900	(Engineering Design Manual, Table B.2)	
Bending Moment at 0.03 Radians Rotation, lbf-in	$M_{0.03}$	111800	(Assume the results are similar to 4-ply 2x8)	
Reduction Factor	φ	0.9		
Design Bending Moment, k-in		100620		
Actual/Allowable = *81495 ÷ 100620*	=	*0.81*	< *1.0*	*PASS*

SHEAR STRENGTH				
Tests show that shear strength of the bracket is at least as strong as the shear strength of concrete Perma-Column base, see Section 4.2 of				
Reduction Factor	φ	0.75		
Design Shear Strength, lbf	$\boldsymbol{\varphi}V_n$	5800	(W/O Increases from Axial Compressive Forces)	
Actual/Allowable = *1975 ÷ 5800*	=	*0.34*	< *1.0*	*PASS*

NAIL OR WIRE FASTENER CONNECTION BETWEEN LAMINATIONS - 2X6 LAMINATIONS

Nail Spacing (in)	s	18	(2 rows of nails, 18" spacing between nails in a row)		
Interlayer Shear Capacity (lbf/in)	ISC	12.0	(EP 559.1, Table 4)		
Total Shear Load per Interface, lbf	V	216	V = (ISC)s		
Shear Load Taken by Glue, lbf		0	0% (Percentage of Load Taken by Glue)		
Net Shear Load per Interface, lbf	V	216	Includes Effect of Glue		
NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD					
Nail Size		8d		*Intermediate Calculations*	
Number of nails per interface per splice length	N	2		K_D	2.2
Nail Diameter (in)	D	0.131		F_{yb} (psi)	100000
Nail Length (in)	L	n/a		$2+R_e$ =	3.0
Width of Side Member (in)	l_s	1.5		$1+2R_e$ =	3.0
Width of Main Member (in)	l_m	1.5		k_1 =	1.07
Nail Penetration into main member (in)	p	1.5		k_2 =	1.07
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	0.8		F_{em} =	5526
Specific Gravity of Side Member	G_s	0.55		F_{es} =	5526
Specific Gravity of Main Member	G_m	0.55		R_e =	1.0
Lateral Design Value (lbs)	Z	106	*(controlling yield value)*	p =	1.50
Duration Factor	C_D	1.6		$1+R_e$ =	2.0
Wet Service Factor	C_M	1		I_s	494
Temperature Factor	C_t	1		III_m	176
Toe Nail Factor	C_{tn}	1		III_s	176
End Grain Factor	C_{eg}	1		IV	106
Total Allowable Lateral Capacity (lbs)	Z'	339	← $Z' = N \times Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Actual/Allowable = *216 ÷ 339*	=	*0.64*	< *1.0* *PASS*		

NAIL OR WIRE FASTENER CONNECTION BETWEEN LAMINATIONS - 2X8 LAMINATIONS

Nail Spacing (in)	s	18	(2 rows of nails, 18" spacing between nails in a row)		
Interlayer Shear Capacity (lbf/in)	ISC	15.0	(EP 559.1, Table 4)		
Total Shear Load per Interface, lbf	V	270	V = (ISC)s		
Shear Load Taken by Glue, lbf		0	0% (Percentage of Load Taken by Glue)		
Net Shear Load per Interface, lbf	V	270	Includes Effect of Glue		
NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD					
Nail Size		8d		*Intermediate Calculations*	
Number of nails per interface per splice length	N	2		K_D	2.2
Nail Diameter (in)	D	0.131		F_{yb} (psi)	100000
Nail Length (in)	L	n/a		$2+R_e$ =	3.0
Width of Side Member (in)	l_s	1.5		$1+2R_e$ =	3.0
Width of Main Member (in)	l_m	1.5		k_1 =	1.07
Nail Penetration into main member (in)	p	1.5		k_2 =	1.07
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	0.8		F_{em} =	5526
Specific Gravity of Side Member	G_s	0.55		F_{es} =	5526
Specific Gravity of Main Member	G_m	0.55		R_e =	1.0
Lateral Design Value (lbs)	Z	106	*(controlling yield value)*	p =	1.50
Duration Factor	C_D	1.6		$1+R_e$ =	2.0
Wet Service Factor	C_M	1		I_s	494
Temperature Factor	C_t	1		III_m	176
Toe Nail Factor	C_{tn}	1		III_s	176
End Grain Factor	C_{eg}	1		IV	106
Total Allowable Lateral Capacity (lbs)	Z'	339	← $Z' = N \times Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Actual/Allowable = *270 ÷ 339*	=	*0.80*	< *1.0* *PASS*		

APPENDIX 3
(Supporting Calculations Modeling)

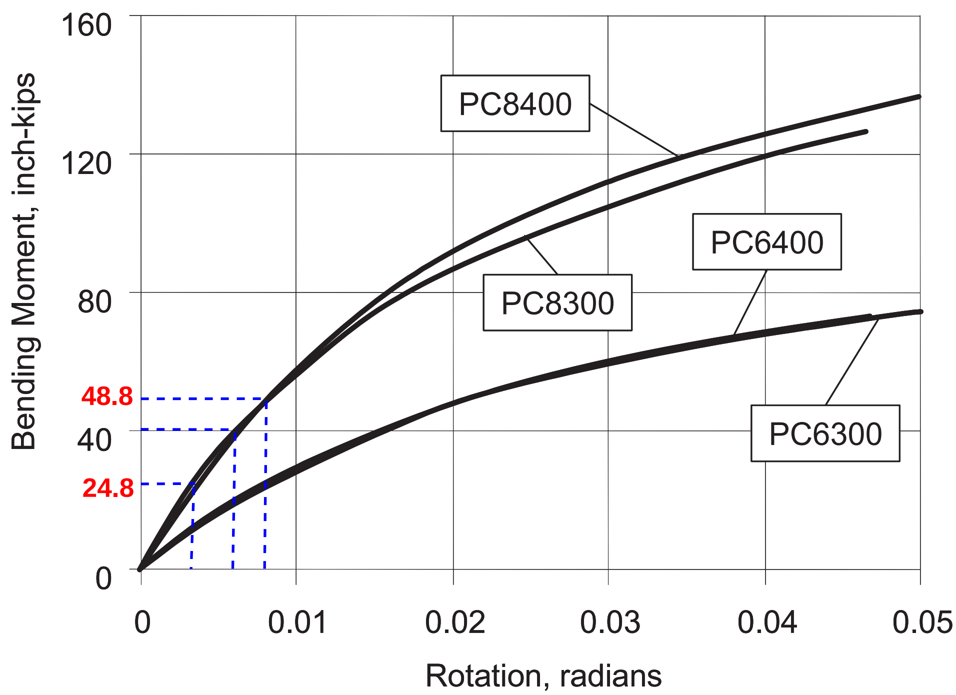


Figure B.4. Joint rotation versus bending moment for steel bracket-to-concrete connection. Plotted from data compiled in Table B.2.

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

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

CONCRETE TO STEEL BRACKET CONNECTION MODELING

			PC6300	PC6400	PC6600	PC8300	PC8400	PC8500
Rotation	θ_r	(rad)	0.0090	0.0103	0.0103	0.0104	0.0118	0.0118
Moment at 0.01 radians	$M_{0.01}$	(in-k)	25.7	29.9	29.9	57.7	64.7	64.7
Modulus of Elasticity	E	(ksi)	29000	29000	29000	29000	29000	29000
Length of Modeling Element	L_e	(in)	2.5	2.5	2.5	2.5	2.5	2.5
Moment of Inertia*	I	(in^4)	0.2462	0.2515	0.2515	0.4769	0.4711	0.4711
Rotational Stiffness	$M_{0.01}/\Theta$	(in-kip/rad)	2570	2990	2990	5770	6470	6470
Square Profile Side Width	b	(in)	1.3110	1.3180	1.3180	1.5467	1.5420	1.5420

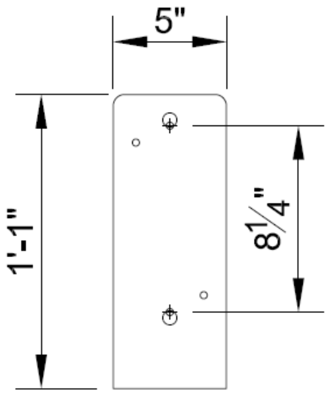
*$I = ML_e / (E\theta_r)$

STEEL BRACKET TO WOOD CONNECTION MODELING

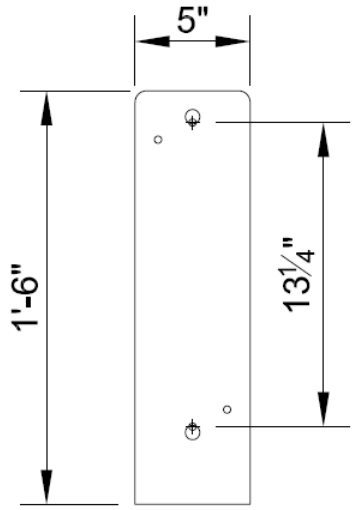
			PC6300	PC6400	PC6600	PC8300	PC8400	PC8500
Modulus of Elasticity	E	(ksi)	29000	29000	29000	29000	29000	29000
Spacing Between Groups	s	(in)	8	13	8	11	11	11
Length of Modeling Element	L_e	(in)	8	13	8	11	11	11
Fastener Group Stiffness	k	(lb/in)	142900	142900	142900	200300	200300	200300
Moment of Inertia*	I	(in^4)	1.2615	5.4130	1.2615	4.5965	4.5965	4.5965
Rotational Stiffness**	M/Θ	(in-kip/rad)	4573	12075	4573	12118	12118	12118
Square Profile Side Width	b	(in)	1.9725	2.8389	1.9725	2.7252	2.7252	2.7252

*$I = ks^2L_e / (2E)$

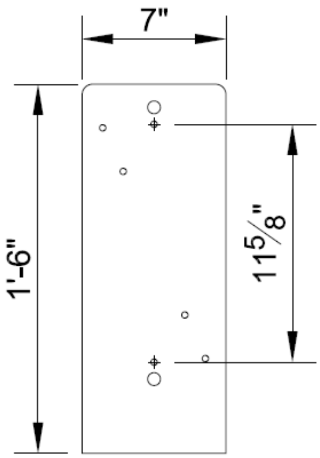
**$M/\Theta = ks^2/2$ per equation 6.2.4 of the Perma-Column Test Report



PC6300 & PC6600
+ Centroid of Fastener Group



PC6400
+ Centroid of Fastener Group



PC8300, PC8400, & PC8500
+ Centroid of Fastener Group

MODELING OF PERMA-COLUMN ELEMENT IN VISUAL ANALYSIS

The size of the Perma-Column in Visual Analysis is selected such that the moment of inertia of the gross section of the modeling element is approximately equal to the effective moment of inertia listed in this table using the maximum bending moment from the Visual Analysis output as the "Actual Bending Moment" in this table.

TABLE BELOW IS COPIED FROM ENGINEERING DESIGN MANUAL BY DAVID R. BOHNHOFF, Ph.D., P.E.

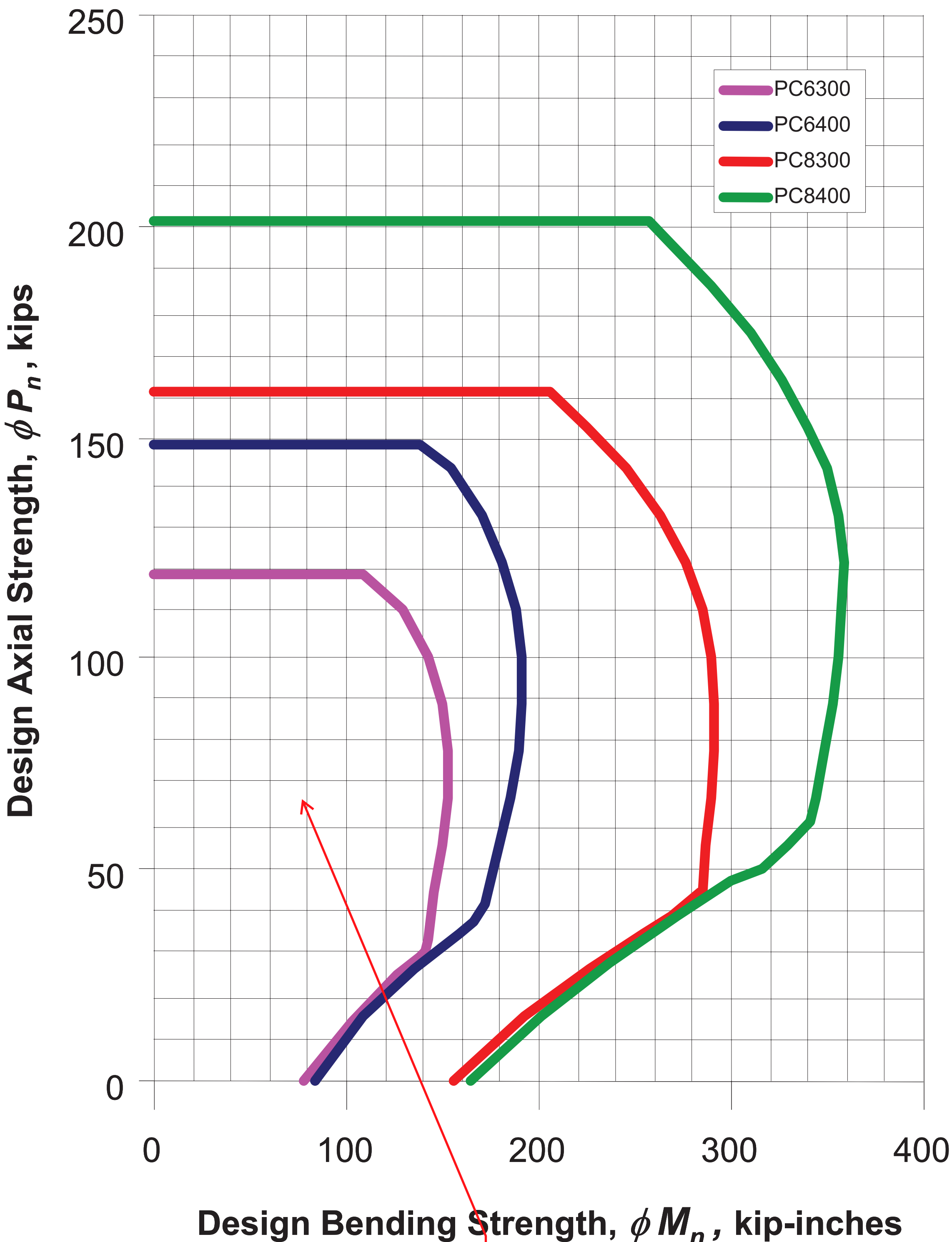
Table 5.2.1: Perma Column Moment of Inertia Values

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

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

AXIAL AND BENDING STRENGTH CHECK OF PERMA-COLUMN

CHART BELOW IS COPIED FROM ENGINEERING DESIGN MANUAL BY DAVID R. BOHNHOFF, Ph.D., P.E.



Factored maximum axial load and bending moments for all Perma-Column assemblies in this report fall inside their respective curvature satisfying the axial and bending strength requirements. See Visual Analysis report for actual axial and bending forces for each of the column assemblies.

SHEAR STRENGTH CHECK OF PERMA-COLUMN

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3.4 Shear Strength

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

For members subjected to shear and flexure only, V_c can be taken as the greater of:

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

or

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

but not greater than:

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

or

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

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

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

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

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

See Visual Analysis report for actual shear forces for each of the column assemblies.

APPENDIX 4
(Calculations for Design Example)

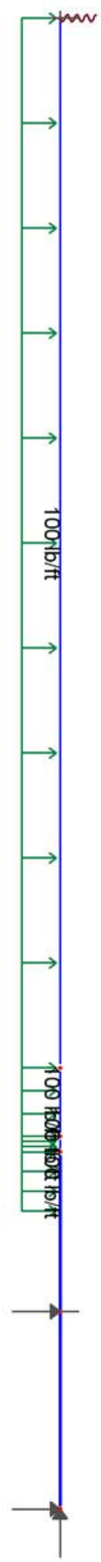
Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Kevin Salyer
Dec 22, 2014; 03:31 PM
IES VisualAnalysis 11.00.0013

Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Kevin Salyer
Dec 22, 2014; 03:42 PM
IES VisualAnalysis 11.00.0013

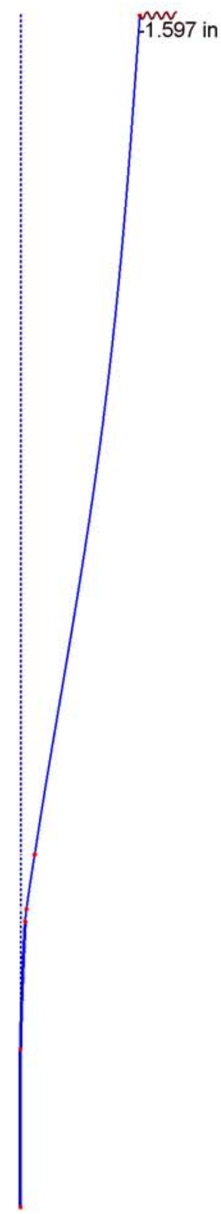
Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Kevin Salyer
Dec 22, 2014; 03:43 PM
Load Case: Dead loads
IES VisualAnalysis 11.00.0013

Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Kevin Salyer
Dec 22, 2014; 03:43 PM
Load Case: Snow load
IES VisualAnalysis 11.00.0013

Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Kevin Salyer
Dec 22, 2014; 03:44 PM
Load Case: Wind -X loads
IES VisualAnalysis 11.00.0013



Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Brent Leatherman
Dec 22, 2014; 10:13 AM
Result Case: D+H+F+W »-X Second Order
IES VisualAnalysis 11.00.0013



Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Brent Leatherman
Dec 22, 2014; 10:40 AM
Result Case: 1 .2D+1.6W+.5L+Lpa+.5S »-X Second Order
Member Mz, moment
IES VisualAnalysis 11.00.0013

Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Brent Leatherman
Dec 22, 2014; 11:32 AM
Result Case: LRFD Envelope Low Extreme
Member Mz, moment
IES VisualAnalysis 11.00.0013

Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Brent Leatherman
Dec 22, 2014; 11:20 AM
Result Case: ASD Envelope Low Extreme
Member Mz, moment
IES VisualAnalysis 11.00.0013

Report Example PC8300 - NO 1 SYP Eave III
TIMBER TECH ENGINEERING, INC., Brent Leatherman
Dec 22, 2014; 10:18 AM
Result Case: D+H+F+W »-X Second Order
Member Mz, moment
IES VisualAnalysis 11.00.0013

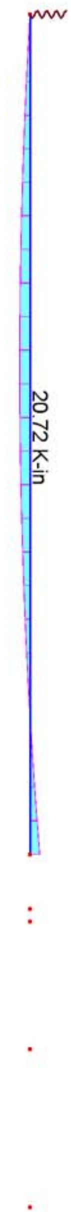


Table of Contents

Section Properties

Section	Beta	Theta	Ax	J	Iy	Iz	Sz(+y)	Sz(-y)	Sy(+z)	Sy(-z)
	deg	deg	ft^2	ft^4	ft^4	ft^4	ft^3	ft^3	ft^3	ft^3
3-ply 2x8	0.000	0.000	0.225	0.006	0.003	0.007	0.022	0.022	0.014	0.014
Square 1.55	0.000	0.000	0.017	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Square 2.73	0.000	0.000	0.052	0.000	0.000	0.000	0.002	0.002	0.002	0.002
Square 5.67	0.000	0.000	0.223	0.007	0.004	0.004	0.018	0.018	0.018	0.018

Nodes

Node	X	Y	Z	Fix DX	Fix DY	Fix DZ	Fix RX	Fix RY	Fix RZ	Scissor?
	ft	ft	ft							
N005	40.000	-4.000	0.000	Yes	Yes	Yes	No	No	No	No
N012	40.000	-1.350	0.000	Yes	No	Yes	No	Yes	No	No
N019	40.000	0.792	0.000	No	No	No	No	No	No	No
N026	40.000	1.000	0.000	No	No	No	No	No	No	No
N033	40.000	1.917	0.000	No	No	No	No	No	No	No
N040	40.000	16.000	0.000	No	No	Yes	No	Yes	No	No

Member Elements

Member	Section	Material	(1)Node	(2)Node	Length	Rz1	Rz2	One Way	Framing
					ft				
PC	Square 5.67	Concrete (F'c = 10 ksi)	N005	N019	4.792	Rigid	Rigid	Normal (2-way)	Column
SC	Square 1.55	ASTM A992 Grade 50	N019	N026	0.208	Rigid	Rigid	Normal (2-way)	Column
SW	Square 2.73	ASTM A992 Grade 50	N026	N033	0.917	Rigid	Rigid	Normal (2-way)	Column
W	3-ply 2x8	Southern Pine #1 (8" wide)	N033	N040	14.083	Rigid	Rigid	Normal (2-way)	Column

Member Uniform Loads

Load Case	Member	Direction	Offset	End Offset	Force	Moment
			ft	ft	lb/ft	ft-lb/ft
Wind -X loads	PC	Force X	4.000	4.792	100.000	-NA-
Wind -X loads	SC	Force X	0.000	0.208	100.000	-NA-
Wind -X loads	SW	Force X	0.000	0.917	100.000	-NA-
Wind -X loads	W	Force X	0.000	14.083	100.000	-NA-

Load Cases

Load Case	Design Checks	Seismic Type	Results	Analyze?	Envelope?
(1)Dead loads	-NA-	-NA-	Yes (2 sets)	Yes	Yes
(3)Snow load	-NA-	-NA-	Yes (2 sets)	Yes	Yes
(4)Wind -X loads	-NA-	-NA-	Yes (2 sets)	Yes	Yes
(5).6D+.7Di	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No

(6).6D+W »-X	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No
(7).9D+1.6W+1.6H »-X	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(8).9D+Di	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(9)1 .2D+1.6W+.5L+Lpa+.5S »-X	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(10)1.2(D+F+T)+1.6(L+H)+.5Lr	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(11)1.2(D+F+T)+1.6(L+H)+.5S	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(12)1.2D+1.6Lr+W »-X	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(13)1.2D+1.6S+.5L+Lpa	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(14)1.2D+1.6S+W »-X	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(15)1.2D+1.6W+.5L+Lpa+.5Lr »-X	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(16)1.4(D+F)	Strength (LRFD)	-NA-	Yes (2 sets)	Yes	No
(17)D+F	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No
(18)D+H+F+.75(L+S)	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No
(19)D+H+F+.75(W+L+Lr) »-X	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No
(20)D+H+F+.75(W+L+S) »-X	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No
(21)D+H+F+S	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No
(22)D+H+F+W »-X	Allowable (ASD)	-NA-	Yes (2 sets)	Yes	No

Load Combination Summary

Equation Combination: .6D+.7Di
Design Category: Allowable (ASD)
Combination: 0.60D
Contributing Cases & Source
Dead loads (D)

Equation Combination: .6D+W »-X
Design Category: Allowable (ASD)
Combination: 0.60D + W-X
Contributing Cases & Source
Dead loads (D)
Wind -X loads (W-X)

Equation Combination: .9D+1.6W+1.6H »-X
Design Category: Strength (LRFD)
Combination: 0.90D + 1.60W-X
Contributing Cases & Source
Dead loads (D)
Wind -X loads (W-X)

Equation Combination: .9D+Di
Design Category: Strength (LRFD)
Combination: 0.90D
Contributing Cases & Source
Dead loads (D)

Equation Combination: 1 .2D+1.6W+.5L+Lpa+.5S »-X
Design Category: Strength (LRFD)
Combination: 1.20D + 0.50S + 1.60W-X
Contributing Cases & Source
Dead loads (D)
Snow load (S)
Wind -X loads (W-X)

Equation Combination: 1.2(D+F+T)+1.6(L+H)+.5Lr
Design Category: Strength (LRFD)
Combination: 1.20D
Contributing Cases & Source
Dead loads (D)

Equation Combination: 1.2(D+F+T)+1.6(L+H)+.5S
Design Category: Strength (LRFD)

Combination: 1.20D + 0.50S
Contributing Cases & Source
Dead loads (D)
Snow load (S)

Equation Combination: 1.2D+1.6Lr+W »-X
Design Category: Strength (LRFD)
Combination: 1.20D + 0.80W-X
Contributing Cases & Source
Dead loads (D)
Wind -X loads (W-X)

Equation Combination: 1.2D+1.6S+.5L+Lpa
Design Category: Strength (LRFD)
Combination: 1.20D + 1.60S
Contributing Cases & Source
Dead loads (D)
Snow load (S)

Equation Combination: 1.2D+1.6S+W »-X
Design Category: Strength (LRFD)
Combination: 1.20D + 1.60S + 0.80W-X
Contributing Cases & Source
Dead loads (D)
Snow load (S)
Wind -X loads (W-X)

Equation Combination: 1.2D+1.6W+.5L+Lpa+.5Lr »-X
Design Category: Strength (LRFD)
Combination: 1.20D + 1.60W-X
Contributing Cases & Source
Dead loads (D)
Wind -X loads (W-X)

Equation Combination: 1.4(D+F)
Design Category: Strength (LRFD)
Combination: 1.40D
Contributing Cases & Source
Dead loads (D)

Equation Combination: D+F
Design Category: Allowable (ASD)
Combination: D
Contributing Cases & Source
Dead loads (D)

Equation Combination: D+H+F+.75(L+S)
Design Category: Allowable (ASD)
Combination: D + 0.75S
Contributing Cases & Source
Dead loads (D)
Snow load (S)

Equation Combination: D+H+F+.75(W+L+Lr) »-X
Design Category: Allowable (ASD)
Combination: D + 0.75W-X
Contributing Cases & Source
Dead loads (D)
Wind -X loads (W-X)

Equation Combination: D+H+F+.75(W+L+S) »-X
Design Category: Allowable (ASD)

Combination: D + 0.75S + 0.75W-X

Contributing Cases & Source

Dead loads (D)

Snow load (S)

Wind -X loads (W-X)

Equation Combination: D+H+F+S

Design Category: Allowable (ASD)

Combination: D + S

Contributing Cases & Source

Dead loads (D)

Snow load (S)

Equation Combination: D+H+F+W »-X

Design Category: Allowable (ASD)

Combination: D + W-X

Contributing Cases & Source

Dead loads (D)

Wind -X loads (W-X)

Nodal Loads

Load Case	Node	Direction	Force	Moment
			lb	lb-ft
Dead loads	N040	DY	-4500.000	0.000
Snow load	N040	DY	-13500.000	0.000

Design Groups

Group/Mesh	Elements	LL Factor	Unity	Design Shape	Overstrength
Wood Column 16' to Eave	1	1.000	0.83	3-ply 2x8	No

Design Group Results

Design Group: Wood Column 16' to Eave per NDS 2005 ASD

Designed As: 3-ply 2x8, Material: \NDS Lumber Supplement 13 (Visual)\No. 1\Southern Pine #1 (8" wide)

Combined Check

Member Name	Result Case	Offset ft	Code Ref.	Unity Check	Details
W	D+H+F+.75(W+L+S) »-X Second Order	0.000	3.9-3	0.82 OK	Cr = 1.350 , CD = 1.600 , KLz = 15.000 ft, KLy = 0.000 ft, Lb = 0.000 ft

Axial Check

Member Name	Result Case	Offset ft	Demand fa psf	Capacity Fa psf	Code Ref.	Unity Check	Details
W	D+H+F+S	0.000	80597.987	97083.116	3.6.3	0.83 OK	CD = 1.150 , Fx = 18109.360 lb, CP = 0.391 , KLz = 15.000 ft, KLy = 0.000 ft

Strong Flexure Check

Member Name	Result Case	Offset ft	Demand fbz psf	Capacity Fbz psf	Code Ref.	Unity Check	Details

W	.6D+W »-X	8.168	78558.667	388724.322	3.3-1	0.20 OK	Cr = 1.350 , Mz = 1762.664 lb-ft, Lb = 0.000 ft, CD = 1.600

Strong Shear Check

Member Name	Result Case	Offset ft	Demand fvz psf	Capacity Fvz psf	Code Ref.	Unity Check	Details
W	D+H+F+W »-X	0.000	5438.116	40319.999	3.4-2	0.13 OK	V = 814.584 lb, CD = 1.600

STEEL BRACKET CONNECTION DESIGN FOR PC8300 (STEEL BRACKET TO SYP WOOD COLUMN)

Maximum Moment in Bracket "Element", in-lb	M_{max}	29300	(ASD)
Spacing Between Fastener Groups, in	s	11.6	← *Actual centroidal spacing based on fastener stiffnesses*
Load on Fastener Group, lbf	R	2526	
Slip Modulus for 1/2" Bolt, lbf/in	k_b	85500	
Slip Modulus for 1/4" SDS Screw, lbf/in	k_{SDS}	28700	
Number of SDS Screws per Fastener Group	N_{SDS}	4	(two from each side, near bolt)
Load on Bolt, lbf	P_b	1078	$P_b = Rk_b / (k_b + N_{SDS}k_{SDS})$
Load on SDS screws, lbf	P_{SDS}	1448	$P_{SDS} = RN_{SDS}k_{SDS} / (k_b + N_{SDS}k_{SDS})$

BOLTS - WOOD TO STEEL PLATES - DOUBLE SHEAR - NDS 2005, ASD

Number of bolts	N	1		$F_{em, par}$	6160
Bolt Diameter (in)	D	0.5		$F_{em, perp}$	3626
Main Member Thickness (in)	t_m	4.5		F_{em}	3626
Side Member Thickness (in)	t_s	0.25		R_e	0.042
Dowel Bearing Strength (psi)	F_{es}	87000		K_θ	1.250
Bolt Yield Strength (psi)	F_{yb}	85000	*Proof Stress for SAE Gr5 bolt*	$1+R_e$	1.042
Max Angle Load to Grain (deg)	θ	90		$2+R_e$	2.042
Specific Gravity	G	0.55		k_3	12.328
Reference Lateral Design Value (Z)	Z	1368	← *NDS 2005, Table 11LA-11*	I_m	1631
Duration Factor	C_D	1.6		III_s	1368
Geometry Factor	C_Δ	1		IV	1755
Wet Service Factor	C_M	1			
Adjusted Lateral Design Value (lbs)	Z'	2189	$Z'=Z \times C_D \times C_{eg} \times C_{tn} \times C_M \times C_t$		
Total Allowable Connection Capacity (lbs)	V	2189	← V=NZ'		
Actual/Allowable = *1078 ÷ 2189*	=	*0.49*	< *1.0* *PASS*		

SDS SIMPSON SCREWS - SHEAR - STEEL TO WOOD (ESR-2236)

Number of screws	N	4	
Screw Diameter (in)	D	0.242	
Screw Length (in)	L	3	
Thickness of Steel Plate (in)	t_s	0.25	
Thickness of Wood Member (in)	t_w	4.31	
Screw Penetration into main member (in)	p	2.75	
Specific Gravity of Wood Member		0.55	
Lateral Design Value (lbs)	Z	420	*(ESR-2236)*
Duration Factor	C_D	1.6	
End Grain Factor	C_{eg}	1	
Toe Nail Factor	C_{tn}	1	
Geometry Factor	C_Δ	1	*(ESR-2236)*
Wet Service Factor	C_M	1	
Adjusted Lateral Design Value (lbs)	Z'	672	$Z'=Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$
Total Allowable Connection Capacity (lbs)	V	2688	V=NZ'
Actual/Allowable = *1448 ÷ 2688*	=	*0.54*	< *1.0* *PASS*

STEEL BRACKET CONNECTION DESIGN FOR PC8300 (STEEL BRACKET TO CONCRETE)

Factored Moment in Bracket "Element", in-lb	M_U	54000	(LRFD)
Factored Shear in Bracket "Element", lbf	V_U	1332	(LRFD)

BENDING STRENGTH

Ultimate Bending Strength, lbf-in, at 0.0437 Radians		M_{ult}	123900	(Engineering Design Manual, Table B.2)	
Bending Moment at 0.03 Radians Rotation, lbf-in		$M_{0.03}$	103700		
Reduction Factor		φ	0.9		
Design Bending Moment, k-in			93330		
Actual/Allowable =	*54000 ÷ 93330*	=	*0.58*	< *1.0*	*PASS*

SHEAR STRENGTH

Tests show that shear strength of the bracket is at least as strong as the shear strength of concrete Perma-Column base, see Section 4.2 of the Perma-Column Engineering Design Manual by David R. Bohnhoff, Ph.D., P.E. The shear strength of the concrete Perma-Column base is shown below.

Reduction Factor		φ	0.75		
Design Shear Strength, lbf		φV_n	4531	(W/O Increases from Axial Compressive Forces)	
Actual/Allowable =	*1332 ÷ 4531*	=	*0.29*	< *1.0*	*PASS*

APPENDIX 5
(Sample Uplift Resistance Calculations)

PC6300 Foundation Uplift Calculations With Uplift Anchors

Presumed Soil Properties (ASAE EP486.1 2003)

Class of Material Class Number	Class Description	Lateral Pressure Per Unit Depth, S lb/sf per ft	Lateral Sliding Coefficient -	Vertical Pressure, S lb/sf	Friction Angle degrees	Density lb/cf
1	Massive crystalline bedrock	1200	0.79	4000	-	-
2	Sedimentary & foliated rock	400	0.35	2000	-	-
3	Firm sandy gravel/gravel	300	-	2000	38	120
4	Loose sandy gravel/gravel	200	0.35	2000	32	90
5	Firm Sand . . .	200	-	1500	30	105
6	Loose Sand . . .	150	0.25	1500	26	85
7	Medium Clay	130	-	1000	15	120
8	Soft Clay . . .	100	-	1000	10	90

FOUNDATION UPLIFT CALCULATIONS - PC6300 WITH 2X2X8.5X0.134 STEEL ANGLES

NON-CONSTRAINED FOOTING WITH BOTTOM STEEL COLLAR (ASAE EP486.1 2003)			
Soil Class		6	Loose Sand . . .
Depth to top of footing, (in)	d	48	= *4.000* ft
Collar thickness (footing to bottom flange distance), (in)	t	2	= *0.167* ft
Collar width, (in)	w	9.38	= *0.782* ft
Anchor Angle length, (in)	l	8.5	= *0.708* ft
Angle horz. leg length, (in)	l_h	2	
Angle Gage (3/6.5/8/10/12/14)		10	
Steel yield strength, (psi)	F_y	50000	
Number of Anchor Angles		2	
Column Width, (in)	b	5.38	
Column Depth, (in)	h	5.44	
Gravity Acceleration, (lbf/lb)	G	1	(EP486.1 allows 25% of concrete weight to be included)
Perma Column Weight, (lb)	C	152	= 150/1728 x (b) x (h) x (60)
1 - UPLIFT RESISTANCE OF SOIL CONE			
Soil Friction Angle, (deg)	θ	26	
Soil Density, (pcf)	α	85	
Post Cross Section Area, (ft^2)	A_p	0.203	
Potential Uplift Resistance[1, 2], (lbs)	**U**	**2203**	
1. $U=aG[(wl - A_p)(d - t) + (w + l)(d - t)^2 \tan\theta + 0.33\pi(d - t)^3 \tan^2\theta]$			
2. *This calculation is for allowable uplift capacity based on uplift soil resistance and weight of foundation concrete*			
2 - BOLT SHEAR CAPACITY (1/2" GRADE 2 DOUBLE SHEAR)			
Nominal Shear Strength, (ksi)	F_{nv}	27	AISC 14th Ed. Table J3.2
Bolt Area, (in^2)	A_{bolt}	0.196	
ASD	Ω	*2.0*	
Bolt Capacity, (lbs)	**R_n/Ω**	**5292**	$[(F_{nv}/\Omega)(A_{bolt})(2)(1000)]$
3 - SHEAR RUPTURE ON EFFECTIVE AREA			
Steel Ultimate Strength, (psi)	F_u	65000	
Bolt Hole Diameter, (in)	d_{bolt}	0.563	
Distance from edge of hole to edge of plate, (in)	a	0.719	*(Measured parallel to the direction of the force)*
Plate thickness, (in)	t	0.134	
Effective Shear Area, (in^2)	A_{sf}	0.268	$A_{sf} = 2t(a+d/2)$, *(AISC Equation D5-2)*
ASD	Ω	*2.0*	
Shear on Effective Area, (lbs)	**P_n/Ω**	**5226**	$P_n/\Omega = (0.6F_uA_{sf})/2$
4 - ANCHOR ANGLE BENDING CAPACITY			
Angle thickness, (in)	t_a	0.134	
Angle Effective Length, (in)	L_{eff}	5.1	*(Assume 60% of angle length is effective)*
ASD	Ω	1.67	
Bending Capacity of (1) angle, (lbs)	P_a	1028	$P_a = (3F_yL_{eff}t_a^2)/(4l_h\Omega)$
Total Bending Capacity of (2) Angles, (lbs)		**2056**	

Net Uplift Resistance = Min Value from the Four Failure Modes Above + 25% of PC Weight = 2094 lbs

SOIL - UPLIFT; PC6300, 18" WIDE X 12" THICK CONCRETE COLLAR

NON-CONSTRAINED FOOTING WITH BOTTOM CONCRETE COLLAR (ASAE EP486.1 2003)

Actual post embedment depth, d	(ft)	4
Collar width, w	(ft)	1.5
Collar thickness, t_c	(ft)	1
Concrete Density, C	(pcf)	150
Soil Density, α	(pcf)	85
Gravity Acceleration, G	(lbf/lb)	1.0
Soil Friction Angle, q	(deg)	26
Post Cross Section Area, A_p	(in^2)	29.3
Soil and Foundation Uplift Resistance[1, 2], U	(lbs)	2134

1. $U = \alpha G[0.33\pi\{[(d-t)+0.5w/\tan\theta]^3(\tan\theta)^2 - 0.125w^3/\tan\theta\} - A_p(d-t)] + 0.25C\pi w^2 tG + 0.25CA_p 60$
2. *This calculation is for allowable uplift capacity based on uplift soil resistance and weight of foundation concrete*

SOIL - UPLIFT; PC6300, 24" WIDE X 12" THICK CONCRETE COLLAR

NON-CONSTRAINED FOOTING WITH BOTTOM CONCRETE COLLAR (ASAE EP486.1 2003)

Actual post embedment depth, d	(ft)	4
Collar width, w	(ft)	2
Collar thickness, t_c	(ft)	1
Concrete Density, C	(pcf)	150
Soil Density, α	(pcf)	85
Gravity Acceleration, G	(lbf/lb)	1.0
Soil Friction Angle, q	(deg)	26
Post Cross Section Area, A_p	(in^2)	29.3
Soil and Foundation Uplift Resistance[1, 2], U	(lbs)	2977

1. $U = \alpha G[0.33\pi\{[(d-t)+0.5w/\tan\theta]^3(\tan\theta)^2 - 0.125w^3/\tan\theta\} - A_p(d-t)] + 0.25C\pi w^2 tG + 0.25CA_p 60$
2. *This calculation is for allowable uplift capacity based on uplift soil resistance and weight of foundation concrete*

SOIL - UPLIFT; PC6300, 18" WIDE X 12" THICK CONCRETE COLLAR WITH 12" EXTENDER

NON-CONSTRAINED FOOTING WITH BOTTOM CONCRETE COLLAR (ASAE EP486.1 2003)

Actual post embedment depth, d	(ft)	4
Collar width, w	(ft)	1.5
Collar thickness, t_c	(ft)	1
Footing thickness, t_f	(ft)	1
Concrete Density, C	(pcf)	150
Soil Density, α	(pcf)	85
Gravity Acceleration, G	(lbf/lb)	1.0
Soil Friction Angle, q	(deg)	26
Post Cross Section Area, A_p	(in^2)	29.3
Soil and Foundation Uplift Resistance[1, 2], U	(lbs)	2399

1. $U = \alpha G\,[0.33\pi\{[(d\text{-}t)+0.5w/\tan\theta]^3(\tan\theta)^2-0.125w^3/\tan\theta\}-A_p(d\text{-}t)]+0.25C\pi w^{2(}t_c+t_f)G+0.25CA_p60$

2. This calculation is for allowable uplift capacity based on uplift soil resistance and weight of foundation concrete

SOIL - UPLIFT; PC6400, 18" WIDE X 12" THICK CONCRETE COLLAR WITH 12" EXTENDER

NON-CONSTRAINED FOOTING WITH BOTTOM CONCRETE COLLAR (ASAE EP486.1 2003)

Actual post embedment depth, d	(ft)	4
Collar width, w	(ft)	1.5
Collar thickness, t_c	(ft)	1
Footing thickness, t_f	(ft)	1
Concrete Density, C	(pcf)	150
Soil Density, α	(pcf)	85
Gravity Acceleration, G	(lbf/lb)	1.0
Soil Friction Angle, q	(deg)	26
Post Cross Section Area, A_p	(in^2)	37.4
Soil and Foundation Uplift Resistance[1, 2], U	(lbs)	2395

1. $U = \alpha G\,[0.33\pi\{[(d\text{-}t)+0.5w/\tan\theta]^3(\tan\theta)^2-0.125w^3/\tan\theta\}-A_p(d\text{-}t)]+0.25C\pi w^{2(}t_c+t_f)G+0.25CA_p60$

2. This calculation is for allowable uplift capacity based on uplift soil resistance and weight of foundation concrete