C&R
DESIGN • REMODEL

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LITTLE GEM GROCERY
GILBERT HOUSE CHILDREN'S MUSEUM

116 Marion Street NE, Salem, Oregon

PORCH ROOF ADDITION

STRUCTURAL CALCULATIONS

24 September 2024

REGISTERED ARCHITECT
Steven N. Schwanke
Steven N Schwanke
Salem, Oregon
1958
STATE OF OREGON

Snow Loads

Latitude : 44.9447° 44°56'41" N

Longitude: −123.0412° 123°02'28" W

Ground snow load:

Oregon Snow Load Map (snowload.seao.org):

10 psf 800 meter grid cell

Latitude-Longitude Lookup (snowload.seao.org):

ground snow load: 9.0 psf

modeled elevation: 180 feet

Elevation at building site: approx 151 feet 46 m

Local Elevation Adjustment not required: 151' < 180'

Design ground snow load, $p_g = 10$ psf

Flat roof snow load, $p_f = 0.7 C_e C_t I_s p_g$

$C_e = 1.0$

$C_t = 1.2$

$I_s = 1.00$ Risk Category II

$p_f = (0.7)(1.0)(1.2)(1.00)(10\text{ psf}) = 8.40$ psf

Sloped roof snow load, $p_s = C_s p_f$

roof slope, $\theta = 22.62°$ 5 in 12

$C_s = 1.0$ roof slope < 12 in 12

$p_s = (1.0)(8.40\text{ psf}) = 8.40$ psf

Snow density, $\gamma = 0.13 p_g + 14 \leq 30$ pcf

$\gamma = (0.13)(10\text{ psf}) + 14 = 15.3$ pcf

Drift Load

$h_b = 8.40 \text{ psf}/15.3 \text{ pcf} = 0.55'$

$\ell_u \simeq 22'$

$$\frac{h_d}{\sqrt{I_s}} = \left(0.43\sqrt[3]{\ell_u}\sqrt[4]{p_g+10}\right) - 1.5$$

$$h_d = \left(0.43\sqrt[3]{\ell_u}\sqrt[4]{p_g+10}\right) - 1.5 \text{ when } I_s = 1.00$$

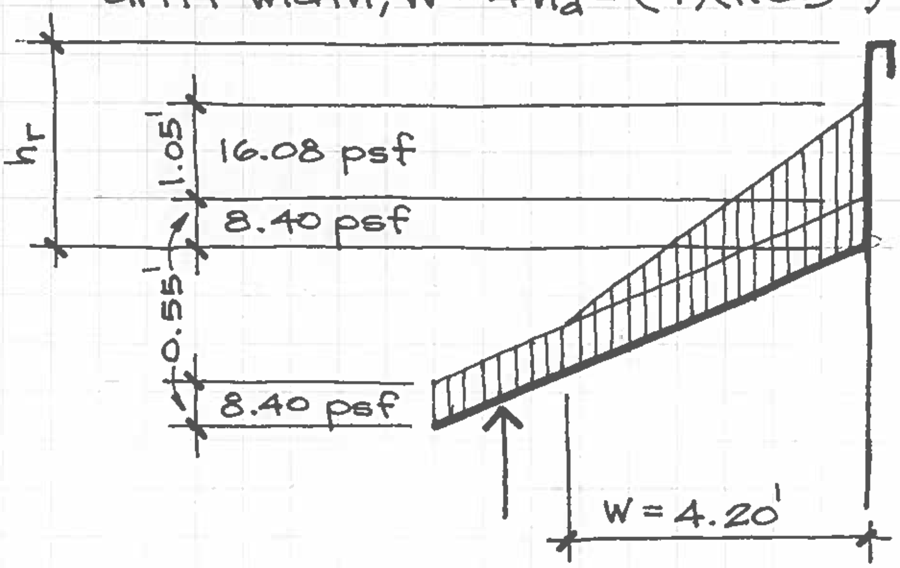
drift height, $h_d = \left(0.43\sqrt[3]{22}\sqrt[4]{10+10}\right) - 1.5 = 1.05'$

$h_b + h_d = 0.55 + 1.05 = 1.60'$

vertical dimension btwn roof & parapet top, $h_r = 4.50'$

maximum snow load $= (15.3 \text{ pcf})(1.60') = 24.48$ psf

drift width, $W = 4h_d = (4)(1.05') = 4.20'$



Minimum allowable design snow load = 25 psf > 24.48 psf

∴ Design roof snow load = 25 psf

Wind Loads

Risk Category II

$I_W = 1.00$

Basic wind speed, $V = 98$ mph

Wind directionality factor, $K_d = 0.85$

Surface Roughness Categories:

Sector 1: Categories C, D (Willamette River), and B

Sectors 2, 3, and 4: Category B

Sector 5: Categories B, C, and D

Sectors 6, 7, and 8: Categories C and D

Exposure: C

Topographic factor, K_{zt}:

$H = 30'$ escarpment (river bank)

$L_h = 60'/2 = 30'$

$H/L_h = 30'/30' = 1$

$K_1 = 0.43$

$x = 0'$ (building site at crest)

$x/2H = 0'/60' = 0$

$K_2 = 1.00$

$z = 30'$

$z/2H = 30'/60' = 0.5$

$K_3 = 0.29$

$K_{zt} = (1 + K_1 K_2 K_3)^2$

$K_{zt} = [1 + (0.43)(1.00)(0.29)]^2 = 1.26$

Ground elevation factor, K_e:

$K_e = e^{-0.0000362 z_g}$

$K_e = e^{-(0.0000362)(151)} = 0.995$

W1

Gust-effect factor, $G = 0.85$

Enclosure Classification: Partially Open Building
(roof structure)

Internal Pressure Coefficient, $(GC_{pi}) = +0.18$ and -0.18

Mean roof height, $h = 11.63'$

$K_h = 0.85$

$K_z = 0.85$

$q_z = q_h = 0.00256 K_z K_{zt} K_d K_e V^2$

$q_z = q_h = (0.00256)(0.85)(1.26)(0.85)(0.995)(98)^2$

$q_z = q_h = 22.27$ psf

Roof plane angle from horizontal, $\theta = 22.62°$ 5 in 12

East-west wind:

$B = 14.00'$

$L = 26.00'$

$h/L = 11.63/26.00 = 0.45$

North-south wind:

$B = 26.00'$

$L = 14.00'$

$h/L = 11.63/14.00 = 0.83$

East-west wind:

$C_{p1} = -0.328$ windward roof

$C_{p2} = +0.134$ windward roof

$C_{p3} = -0.600$ leeward roof

W2

North-south wind:

$C_{p4} = -1.16$ uplift, 0 to h/2 distance from windward edge
$C_{p5} = -0.18$ uplift, 0 to h/2
$C_{p6} = -0.76$ uplift, h/2 to h
$C_{p7} = -0.18$ uplift, h/2 to h
$C_{p8} = -0.63$ uplift, h to 2h
$C_{p9} = -0.18$ uplift, h to 2h

$h/2 = 5.82'$
$h = 11.63'$
$2h = 23.26'$

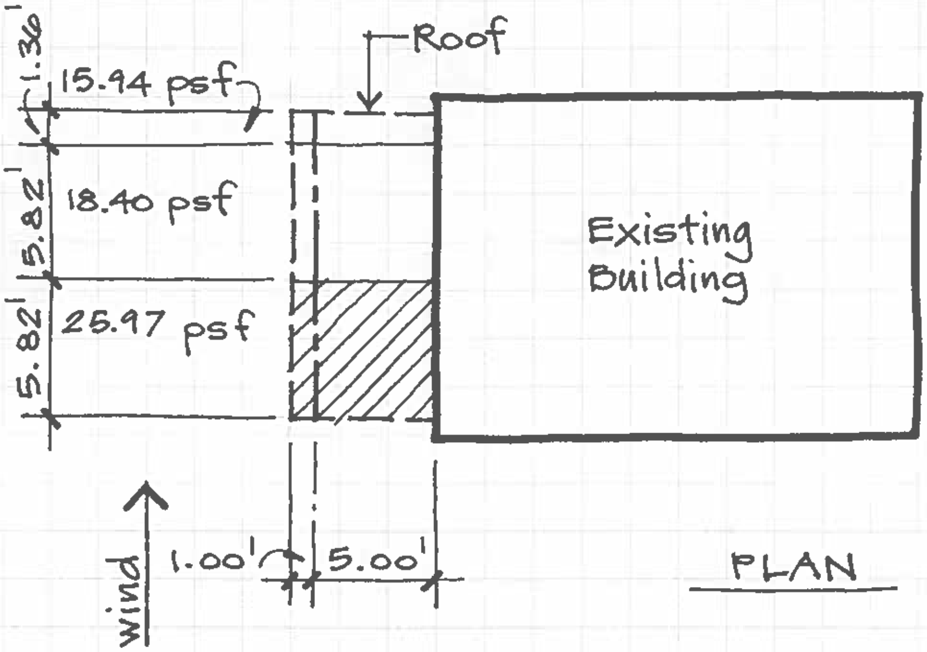
$p = qGC_p - q_i(GC_{pi})$
$q_i = q_h = 22.27$ psf

$p_4 = (22.27)(0.85)(-1.16) - (22.27)(\pm 0.18)$
$p_4 = -21.96$ psf ± 4.01 psf
$p_4 = -21.96 + (-4.01) = -25.97$ psf maximum uplift

$p_6 = (22.27)(0.85)(-0.76) - (22.27)(\pm 0.18)$
$p_6 = -14.39$ psf ± 4.01 psf
$p_6 = -14.39 + (-4.01) = -18.40$ psf

$p_8 = (22.27)(0.85)(-0.63) - (22.27)(\pm 0.18)$
$p_8 = -11.93$ psf ± 4.01 psf
$p_8 = -11.93 + (-4.01) = -15.94$ psf

$p_5 = (22.27)(0.85)(-0.18) - (22.27)(\pm 0.18)$
$p_5 = -3.41$ psf ± 4.01 psf
$p_5 = -3.41 + (-4.01) = -7.42$ psf


Roof
1.36'
15.94 psf
5.82'
18.40 psf
5.82'
25.97 psf
Existing Building
wind
1.00'
5.00'

PLAN

W4

Roof Joists

2x4 @ 27" oc (2.25' oc)

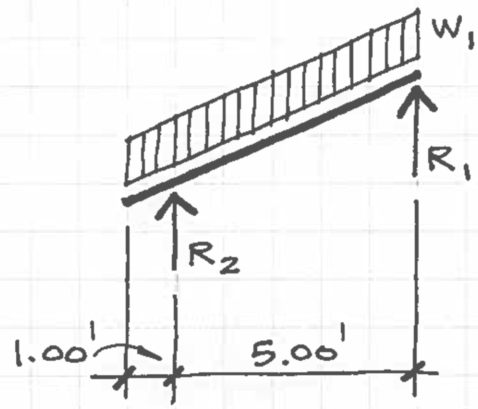
snow load = 25 psf

dead load = 9 psf

2.50 psf shingles + 2.50 reroof + 1.13 plywood + 2.00 1x T&G + 0.50 2x4 framing = 8.63 psf

total load = 25 + 9 = 34 psf

$w_1 = (34 \text{ psf})(2.25') = 77 \text{ plf}$



$R_1 = \frac{77}{10.00}(25.00 - 1.00) = 185\#$

$R_2 = \frac{77}{10.00}(5.00 + 1.00)^2 = 277\#$

$M_1 = \frac{77}{200}(5.00 + 1.00)^2(5.00 - 1.00)^2 = 222\#'$ btwn R_1 & R_2

$M_2 = \frac{(77)(1.00)^2}{2} = 39\#'$ @ R_2

$F_b' = F_b C_D C_L C_F C_r$

$F_b' = (900)(1.15)(1.0)(1.5)(1.0) = 1553$ psi DF-L No 2 2x4

$S_x = \frac{(222)(12)}{1553} = 1.715 \text{ in}^3$ required

RJ1

$S_x = 3.063\ in^3$ actual 2x4 > $1.715\ in^3$ OK
2x4 DF-L No 2 acceptable in bending

Check deflection:

$w_1 = 77\ plf = 6.42\ \#/inch$ DL + SL
$w_2 = (25\ psf)(2.25') = 56\ plf = 4.67\ \#/inch$ SL
$\ell = 60''$
$a = 12''$
$x = \frac{60}{2}\left(1 - \frac{144}{3600}\right) = 28.8''$
$E' = E = 1{,}600{,}000\ psi$ DF-L No 2
$I_x = 5.359\ in^4$ 2x4

$$\Delta_x = \frac{wx}{24EI\ell}(\ell^4 - 2\ell^2x^2 + \ell x^3 - 2a^2\ell^2 + 2a^2x^2)$$

$$\ell^4 - 2\ell^2x^2 + \ell x^3 - 2a^2\ell^2 + 2a^2x^2 = 7{,}623{,}383$$

$$\Delta_x = \frac{w(28.8)(7{,}623{,}383)}{24E(5.359)(60)} = \frac{28{,}451w}{E}$$

$$\Delta_{x_1} = \frac{(28{,}451)(6.42)}{1{,}600{,}000} = 0.114''\ @\ x\ (\text{btwn } R_1 \,\&\, R_2)$$

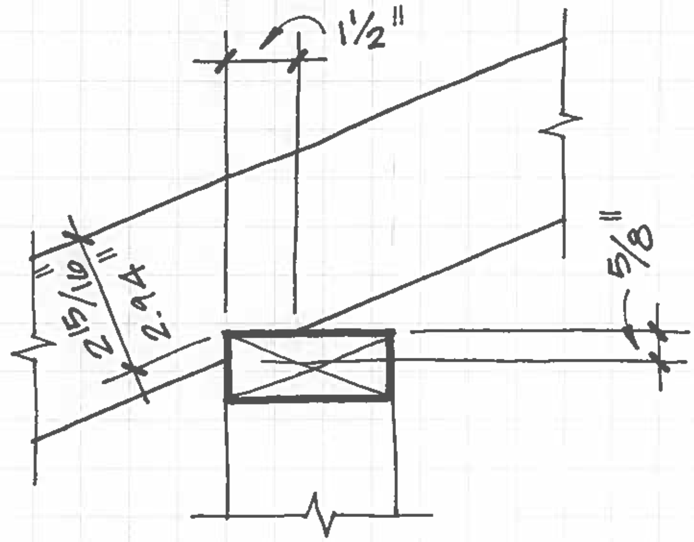
$0.114''$ TL$\Delta = \ell/526$ OK

$$\Delta_{x2} = \frac{(28{,}451)(4.67)}{1{,}600{,}000} = 0.083''\ @\ x$$

$0.083''$ SL$\Delta = \ell/723$ OK

2x4 DF-L No 2 acceptable in deflection between supports

RJ2



1.50" x 2.94" section

$$S_x = \frac{(1.50)(2.94)^2}{6} = 2.161 \text{ in}^3$$

$$I_x = \frac{(1.50)(2.94)^3}{12} = 3.177 \text{ in}^4$$

$$S_x = \frac{(39)(12)}{1553} = 0.301 \text{ in}^3 \text{ required} < 2.161 \text{ in}^3 \qquad \text{OK}$$

Deflection at roof joist overhang end:

$$\Delta = \frac{wa}{24EI}(3a^3 + 4a^2\ell - \ell^3)$$

$$\Delta = \frac{(6.42)(12)(5184 + 34{,}560 - 216{,}000)}{(24)(1{,}600{,}000)(3.177)} = -0.111"$$

effective cantilever length for deflection calculations = 2a = (2)(12") = 24"

$$0.111" \text{ TL}\Delta = \frac{24"}{0.111"} = \ell/216 < \ell/120 \qquad \text{OK}$$

RJ3

$$\Delta = \frac{(4.67)(12)(5184 + 34{,}560 - 216{,}000)}{(24)(1{,}600{,}000)(3.177)} = -0.081''$$

$0.081''$ SLΔ = $\ell/296 < \ell/180$ OK

Consider bending and deflection due to wind uplift:

$w = (25.97 \text{ psf max})(2.25') = 58$ plf

58 plf < 77 plf used in calculations on previous pages

Roof joist bearing:

area $= (1.50'')(1.50'') = 2.25$ sq in

$F_{c\perp}' = F_{c\perp} = 625$ psi

allowable load = (2.25 sq in)(625 psi) = 1406#

$1406\# > 277\# = R_2$ OK

RJ4

Roof Joist Connection to Beam

maximum wind uplift = 25.97 psf

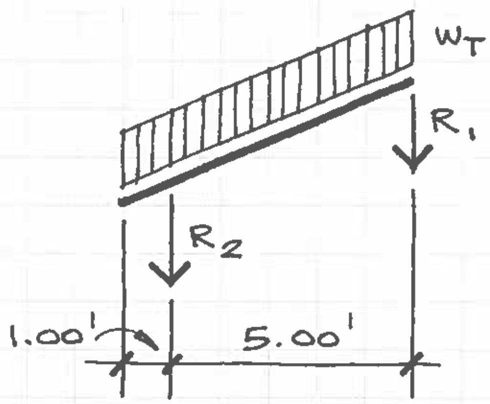
$w = (25.97\ \text{psf})(2.25') = 58\ \text{plf maximum}$

dead load = 6.50 psf (excluding reroof weight)

effective dead load = $(6.50)(0.67) = 4.40$ psf

$w_{DL} = (4.40)(2.25') = 9.90$ plf DL resisting uplift

$w_T = 58\ \text{plf} - 10\ \text{plf} = 48\ \text{plf}$



$R_1 = \dfrac{48}{10.00}(25.00 - 1.00) = 115^{\#}$ @ wall

$R_2 = \dfrac{48}{10.00}(5.00 + 1.00)^2 = 173^{\#}$ @ beam

6" FastenMaster TimberLOK screw:

thread length 2"

$W = 220\,^{\#}/\text{inch}$ of thread penetration in main member

$P = 160\,^{\#}/\text{screw}$ (head pull-through)

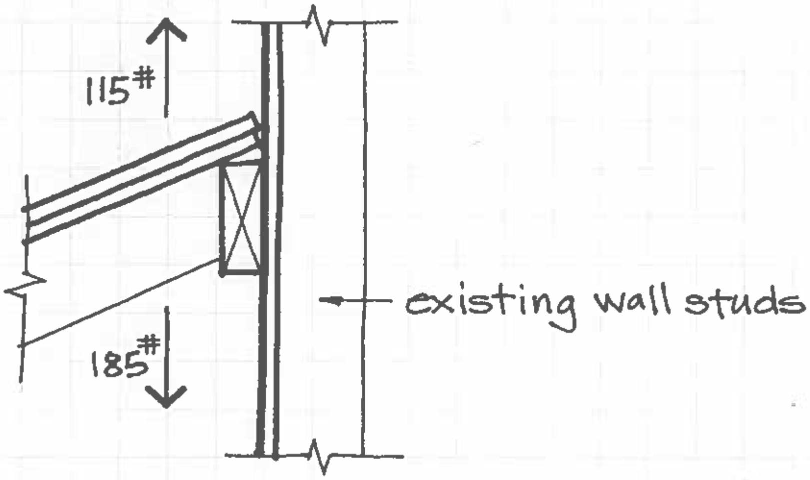
$W' = W C_D C_M = (220\,^{\#}/\text{in})(2")(1.60)(0.7) = 493\,^{\#}/\text{screw}$

$493^{\#} > 173^{\#}$ OK

$P' = P C_D C_M = (160^{\#})(1.60)(0.7) = 179\,^{\#}/\text{screw} > 173^{\#}$

(1) - 6" TimberLOK screw acceptable

Roof Joist Connection to Wall Ledger



10d box nail $3'' \times 0.128''$

$Z = 93\#/\text{nail}$ $\quad\quad W = 31\#/\text{inch}$

$Z' = Z C_D C_M C_{tn}$

$Z' = (93\#/\text{nail})(1.15)(1.0)(0.83) = 89\#/\text{nail}$

$(89\#/\text{nail})(4\ \text{nails}) = 356\# > 185\#$

$W' = W C_D C_{tn}$

$W' = (31\#/\text{in})(1.60)(0.67) = 33\#/\text{inch}$

$(33\#/\text{in})$(2" penetration into ledger & exst board shthg) $= 66\#/\text{nail}$

$(66\#/\text{nail})(4\ \text{nails}) = 264\#$

(4) - 10d box toenails acceptable

RJG

Ledger Connection to Wall

R_1 = 185# @ 2.25' centers

w = 185#/2.25' = 82 plf along ledger

existing stud spacing unknown

studs 16"oc: (82 plf)(1.33') = 109# @ each stud

studs 24"oc: (82 plf)(2.00') = 164# @ each stud

4½" FastenMaster HeadLok screw:

thread length 2"

W = 230#/inch

P = 520#/screw

$Z_\perp$ = 260#/screw

2¼" screw length in stud > 2" thread length

$Z_\perp' = Z_\perp C_D = (260)(1.15) = 299$#/screw

299# > 164#

(1) - 4½" HeadLok screw into each stud acceptable

L1

Beam

$R_2 = 277\#$ @ $27''$ centers (page 8)

$w = 277\#/2.25' = 123$ plf

estimated beam weight = 5 plf

$w_T = 123 + 5 = 128$ plf

beam span = $11'-1'' = 11.08' = 133''$ simple span

$R = (128)(11.08)(0.5) = 709\#$

$$M_{max} = \frac{(128)(11.08)^2}{8} = 1964\#'$$

$F_b' = F_b C_D C_L C_F$

$F_b' = (1000)(1.15)(1.0)(1.3) = 1495$ psi DF-L No 1 4x6, 4x8

$$S_x = \frac{(1964)(12)}{1495} = 15.76 \text{ in}^3 \text{ required}$$

4x6 $S_x = 17.65 \text{ in}^3 > 15.76 \text{ in}^3$ OK

Check deflection:

$I_x = 48.53 \text{ in}^4$

$E' = E = 1{,}700{,}000$ psi

$\ell = 133''$

$w_1 = 128$ plf $= 10.67\#/$inch SL + DL

$w_2 = 90$ plf $= 7.50\#/$inch SL

$w_3 = 38$ plf $= 3.17\#/$inch DL

$$\Delta_{max} = \frac{5w\ell^4}{384EI}$$

$$\Delta_{max} = \frac{(5)(10.67)(133)^4}{(384)(1{,}700{,}000)(48.53)} = 0.527'' \quad SL + DL$$

$0.527''$ TLΔ = $\ell/252 < \ell/120$ OK

$$\Delta_{max} = \frac{(5)(7.50)(133)^4}{(384)(1{,}700{,}000)(48.53)} = 0.370'' \quad SL$$

$0.370''$ SLΔ = $\ell/359 < \ell/180$ OK

$0.370'' = \Delta_{max\ ST}$

$$\Delta_{max\ LT} = \frac{(5)(3.17)(133)^4}{(384)(1{,}700{,}000)(48.53)} = 0.157'' \quad DL$$

Long-term loading:

$\Delta_T = K_{cr}\Delta_{LT} + \Delta_{ST}$

$\Delta_T = (1.5)(0.157'') + 0.370'' = 0.236 + 0.370 = 0.606''$

$0.606'' = \ell/219 < \ell/120$ OK

4x6 DF-L No 1 beam acceptable

Beam to Column Connection

uplift @ each roof joist, $R_2 = 173^{\#}$ (page 12)

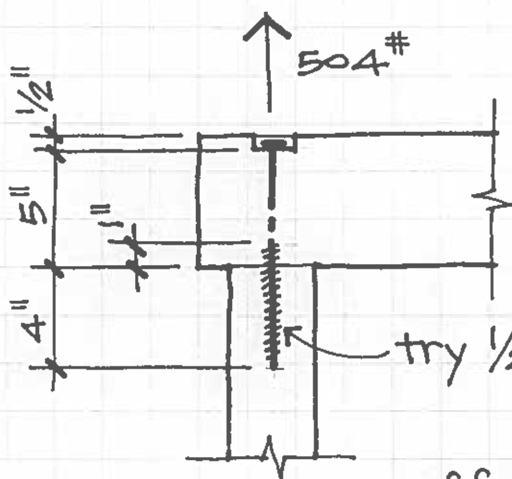
$173^{\#}/2.25' = 77$ plf on beam @ windward end

$R_{max} = (77\text{ plf})(13.08'\text{ roof length})(0.5) = 504^{\#}$

$1/2"\phi$ lag screw $W = 378^{\#}$/inch thread penetration

$W' = W C_D C_M C_{eg}$

$W' = (378^{\#}/\text{in})(1.60)(1.0)(0.75) = 454^{\#}/\text{inch}$



try $1/2"\phi \times 9"$ lag screw $T-E = 4\frac{11}{16}"$

$T = 5"$

effective thread length in column $= 4\frac{11}{16}" - 1" = 3\frac{11}{16}"$

$3\frac{11}{16}" = 3.6875"$

$(454^{\#}/\text{inch})(3.6875") = 1674^{\#} > 504^{\#}$ OK

If $C_M = 0.7$ is applied:

$(378^{\#}/\text{inch})(1.60)(0.7)(0.75)(3.6875") = 1171^{\#} > 504^{\#}$

$1/2"\phi \times 9"$ lag screw acceptable

Column Connection at Concrete

Simpson CPT44Z with cast-in-place anchorage

load acting down, P = 709# (page 15)
uplift = 504# (page 17)

allowable load, down = 11,455# > 709#

allowable uplift, wood to steel = 3035# > 504#

allowable uplift, anchors in concrete = 695# min > 504#