PRCTI20230167

City of Puyallup Development & Permitting Services ISSUED PERMIT
Building | Planning | Engineering | Public Works | Fire | Traffic

City of Puyallup Building ACCEPTED
JMontgomery 02/16/2023 10:27:02 AM

CONSULTING ENGINEER
R.T. WHARTON & ASSOCIATES, INC
1268 HIDDEN CREST CT.
MUSQUITE, NV 89034
725-225-1048

MANUFACTURER
PANEL-BUILT, INC.
302 BEASLEY ST.
BLAIRSVILLE, GA 30512
800-636-3873

STRUCTURAL DESIGN FOR

REDDOT BUILDING TWO
MODULAR OFFICE APROXIMATELY 800 SQ. FT.
2504 E. MAIN AVE.
PUYALLUP, WA 98372

THE APPROVED CONSTRUCTION PLANS, DOCUMENTS AND ALL ENGINEERING MUST BE POSTED ON THE JOB AT ALL INSPECTIONS IN A VISIBLE AND READILY ACCESSIBLE LOCATION.

FULL SIZED LEDGIBLE COLOR PLANS ARE REQUIRED TO BE PROVIDED BY THE PERMITEE ON SITE FOR INSPECTION

MATHCAD DEFINED UNITS ARE

$\mathrm{in} \equiv 1L \quad \mathrm{lb} \equiv 1M \quad \mathrm{si} \equiv \mathrm{in}^2 \quad \mathrm{ci} \equiv \mathrm{in}^3 \quad \mathrm{ft} \equiv 12\cdot\mathrm{in} \quad \mathrm{sf} \equiv \mathrm{ft}^2 \quad \mathrm{psf} \equiv \frac{\mathrm{lb}}{\mathrm{ft}^2} \quad \mathrm{plf} \equiv \frac{\mathrm{lb}}{\mathrm{ft}} \quad \mathrm{ksi} := 1000\cdot\mathrm{psi}$

$\mathrm{psi} \equiv \frac{\mathrm{lb}}{\mathrm{in}^2} \quad \mathrm{pcf} := \frac{\mathrm{lb}}{\mathrm{ft}^3} \quad \mathrm{pli} := \frac{\mathrm{lb}}{\mathrm{in}}$

BUILDING CODE: 2018 WSBC

DESIGN LOADS

$LL_r := 0\cdot\mathrm{psf}$ ROOF LIVE LOAD PER MANUFACTURER, NO STORAGE OR WALKING

$DL_r := 9\cdot\mathrm{psf}$ ROOF DEAD LOAD.

see drawings pg N-1

NO WIND LOAD, STRUCTURE IS INTERIOR TO A BUILDING

SEISMIC
SITE CLASS "D" IN LIEU OF A SOILS REPORT.
SEISMIC FORCE-RESISTING SYSTEM A.17 (SHEAR WALLS NOT RATED FOR RESISTANCE)

$I := 1.0$ IMPORTANCE FACTOR PER TABLE 11.5-1 (CATEGORY II)

$R := 2$ RESPONSE MODIFICATION COEFF.

$\Omega_o := 2.5$ SYSTEM OVERSTRENGTH FACTOR

$C_d := 2$ DEFLECTION AMPLIFICATION FACTOR

$S_S := 1.258 \quad S_1 := 0.433$ MAX. GROUND MOTION

$F_a := 1.2 \quad F_v := 1.6$ SITE COEFFICIENTS

$S_{ms} := F_a \cdot S_S \cdot I \quad S_{ms} = 1.51 \quad S_{ds} := \frac{2}{3}\cdot S_{ms} \quad S_{ds} = 1.01$

DESIGN CAT. D $P := 1.0$ REDUNDANCY FACTOR (REGULAR IN PLAN)

THEREFORE $C_s := \dfrac{S_{ds}}{\frac{R}{I}} \quad C_s = 0.5$

$Q_e := C_s \quad Q_e = 0.5$ HORIZONTAL SEISMIC FORCE FACTOR

RONALD T. WHARTON STATE OF WASHINGTON 21705 REGISTERED PROFESSIONAL ENGINEER
EXP. 11/30/22
OCT. 06, 2022

PRCTI20230167

Wharton and Associates
Mesquite, NV

ALLOWABLE STRESS DESIGN

APPLICABLE BASIC LOAD COMBINATIONS

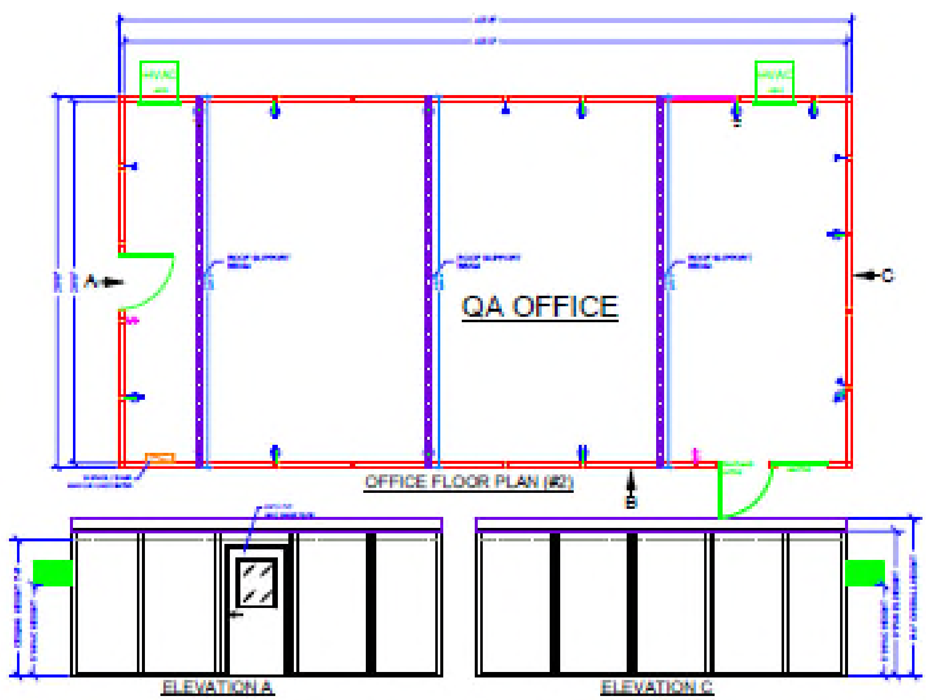
1. D + L
2. (1 + 0.105 Sds)D + 0.75L + 0.525ρQe)
3. (0.6 - 0 .14Sds)D + 0.7ρQe

DEFLECTION & DRIFT LIMITS

VERTICAL PER IBC TABLE 1604.3
HORIZONTAL SEISMIC PER ASCE TAB. 12.12-1

MATERIAL SPECIFICATIONS

STEEL ROOF DECK: 22 GA. "B" 1-1/2" DEPTH, Fy = 38 KSI. PER ESR-2078P
ALUMINUM: ALLOY 6063 - T6, Fb = 25.0 KSI
WALLS: SANDWICH PANELS
CONCRETE: f'c = 2500 PSI
NYLON NAIL ANCHORS: POWERS 1/4" x 1/2"
EXP. ANCHORS: ITW RED HEAD WEDGE PER ESR-2427



ROOF DECK **USE 1-1/2" x 22 GA. B-DECK**

$span := 12.75 \cdot ft$ $spacing := 12 \cdot in$ $Fy := 38 \cdot ksi$ $E := 29500 \cdot ksi$

PROPERTIES PER MANUFACTURER $Sx := 0.1757 \cdot in^3$ $Ix := 0.1485 \cdot in^4$

$M_a := \frac{Sx \cdot Fy}{1.67}$ $M_a = 333.16\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND LOADS $w_g := (DL_r + LL_r) \cdot spacing$ $w_g = 9\,plf$ UNIF. GRAVITY LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := w_g \cdot \frac{span^2}{8}$ $M_r = 182.88\,ft \cdot lb$ $\frac{M_r}{M_a} = 0.55$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot span^4}{384 \cdot E \cdot Ix}$ $\Delta = 1.22\,in$ $\frac{span}{\Delta} = 125.25$ **OK > 120**

Wharton and Associates
Mesquite, NV

MODULAR WALL PANELS IN BEARING

USE 3" 3-PLY G/G PANELS

COMPOSITE PANEL, GYP.BD. FACING BOTH SIDES, POLYSTYRENE CORE.

FIND THE ALLOWABLE BEARING FOR PANEL BASED ON RACKING LOAD TEST PERFORMED BY TWIN CITY TESTING CORP. USE THE AVG. FAILURE LOAD WITH A SAFETY FACTOR OF FOUR. TEST PANEL LENGTH, 8 FT. HEIGHT 10 FT.

$SF := 4$ SAFETY FACTOR

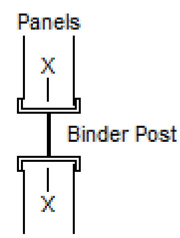
$F_{fail} := 3230 \cdot lb$ AVERAGE LATERAL LOAD PANELS FAILED, DEFLECTION = 2.2" JUST PRIOR TO FAILURE.

$P_{fail} := \frac{F_{fail} \cdot 10 \cdot ft}{8 \cdot ft}$ $P_{fail} = 4037.5\,lb$ RESULTANT AXIAL LOAD AT BINDER POST

$w_a := \frac{P_{fail}}{SF \cdot 4.25 \cdot ft}$ $w_a = 237.5\,plf$ ALLOWABLE UNIFORM LOAD/UNIT WIDTH

$w_g := \left(DL_r + LL_r\right) \cdot \frac{10.5 \cdot ft}{2}$ $w_g = 47.25\,plf$ UNIFORM LOAD $\frac{w_g}{w_a} = 0.2$ **OK < 1.0**

CHECK 5 PSF PARTITION LOAD BENDING MOMENT TO BINDER POST



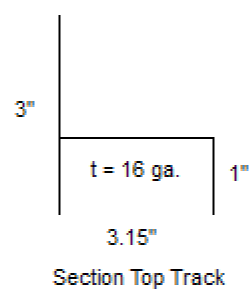
$Sx := 0.529 \cdot in^3$ $F_y := 25.0 \cdot ksi$

$Mr := 5 \cdot psf \cdot 51 \cdot in \cdot \frac{(8 \cdot ft)^2}{8}$ $Mr = 170\,ft \cdot lb$ REQUIRED MOMENT

$Ma := \frac{Sx \cdot F_y}{1.67}$ $Ma = 659.93\,ft \cdot lb$ ALLOWABLE MOMENT

CHECK BINDER POST FOR BENDING $\frac{Mr}{Ma} = 0.26$ **OK < 1.0**

USE 3" x 16 GA ALUMINUM SUPPORT HEADER



$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$

$span_h := 36 \cdot in$ HEADER SPAN OVER DOORS

PROPERTIES PER ANALYSIS $S_x := 0.1679 \cdot in^3$ $I_x := 0.1916 \cdot in^4$

$M_a := \frac{S_x \cdot F_y}{1.67}$ $M_a = 209.46\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \frac{w_g \cdot span_h^2}{8}$ $M_r = 53.16\,ft \cdot lb$ $\frac{M_r}{M_a} = 0.25$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot span_h^4}{384 \cdot E_a \cdot I_x}$ $\Delta = 0.04\,in$ $\frac{span_h}{\Delta} = 800.99$ **OK > 180**

Wharton and Associates
Mesquite, NV

ROOF SUPPORT BEAM, W 8 x 10 $span_b := 20.5 \cdot ft$ BEAM SPAN

PROPERTIES $S_x := 7.81 \cdot in^3$ $I_x := 30.8 \cdot in^4$ $F_y := 50 \cdot ksi$ $E = 29500\, ksi$

$M_a := \frac{S_x \cdot F_y}{1.67}$ $M_a = 19486.03\, ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

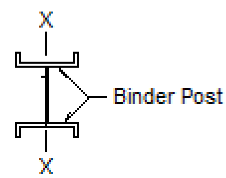
FIND LOADS $w_g := \left[\left(DL_r + LL_r\right) \cdot 12.75 \cdot ft\right] + 10 \cdot plf$ $w_g = 124.75\, plf$ UNIFORM LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \frac{w_g \cdot span_b^{\,2}}{8}$ $M_r = 6553.27\, ft \cdot lb$ $\frac{M_r}{M_a} = 0.34$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot span_b^{\,4}}{384 \cdot E \cdot I_x}$ $\Delta = 0.55\, in$ $\frac{span_b}{\Delta} = 450.89$ **OK > 180**

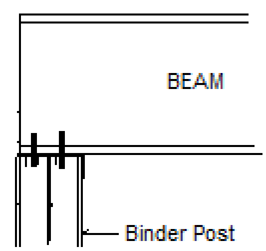
USE BINDER POSTS AS SUPPORT COLUMNS



$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$ $Ht := 8 \cdot ft$ $Trib := 51 \cdot in$

PROPERTIES

$Sx := 0.529 \cdot in^3$ $Ix := 0.792 \cdot in^4$ $A := 0.947 \cdot in^2$ $r_x := 0.914 \cdot in$



$F_e := \frac{\pi^2 \cdot E_a}{\left(\frac{1.0 \cdot Ht}{r_x}\right)^2}$ $F_e = 8946.43\, psi$ LESS THAN

$.44 \cdot F_y = 11000\, psi$

$F_{cr} := .877 \cdot F_e$ $F_{cr} = 7846.02\, psi$ CRITICAL BUCKLING STRESS

$P_n := F_{cr} \cdot A$ $P_n = 7430.18\, lb$ $P_a := \frac{P_n}{1.92}$ $P_a = 3869.88\, lb$

$P_r := \frac{w_g \cdot span_b}{2}$ $P_r = 1278.69\, lb$ POINT LOAD $\frac{P_r}{P_a} = 0.33$ **OK < 1.0**

Wharton and Associates
Mesquite, NV

LATERAL DESIGN FIND SEISMIC FORCE $Area := 800 \cdot ft^2$

$DL_w := \frac{Ht \cdot 4 \cdot psf \cdot (122 \cdot ft)}{2 \cdot Area}$ $DL_w = 2.44\,psf$ WALL DEAD LOAD PER SQ.-FT. FLOOR

$W_s := DL_r + DL_w$ $W_s = 11.44\,psf$ SEISMIC DEAD LOAD AT ROOF LEVEL

$F_s := W_s \cdot Area \cdot 0.7 \cdot P \cdot Q_e$ $F_s = 3223.7\,lb$ SEISMIC FORCE

USE THE MODULAR WALLS AS SHEAR WALL (CHECK LEFT SIDE WALL)

$v := \frac{F_s}{2 \cdot 17.5 \cdot ft}$ $v = 92.11\,plf$ SHEAR

FIND THE ALLOWABLE SHEAR FOR PANEL BASED ON TEST RESULTS.
SEE FOLLOWING TEST RESULTS. A SAFETY FACTOR OF 3 IS USED.

$F_{fail} := 3230 \cdot lb$ AVERAGE FORCE AT FAILURE $SF := 3$ SAFETY FACTOR

$v_a := \frac{F_{fail}}{SF \cdot 8 \cdot ft}$ $v_a = 134.58\,plf$ ALLOWABLE SHEAR CAPACITY $\frac{v}{v_a} = 0.68$ **OK < 1.0**

CHECK WALL ANCHORAGE FOR SHEAR, USE 1/2" x 2-1/2" EMB. WEDGE ANCHORS AT 48"

$V_n := 3206 \cdot lb$ ALLOW.SHEAR $\frac{v \cdot 48 \cdot in}{V_n} = 0.11$ **OK < 1.0** $T_n := 598 \cdot lb$ ALLOW.TENSION

CHECK OVERTURNING

$M_o := v \cdot 8.75 \cdot ft \cdot 8 \cdot ft$ $M_o = 6447.4\,ft \cdot lb$ OVERTURNING MOMENT

$M_{res} := \frac{(8.75 \cdot ft)^2}{2} \cdot \left[\left(DL_r \cdot 2 \cdot ft \right) + 32 \cdot plf + \frac{T_n}{48 \cdot in} \right]$ $M_{res} = 7637.11\,ft \cdot lb$ RESISTING MOMENT

$T_s := \frac{M_o - M_{res} \cdot \left(0.6 - .14 \cdot S_{ds} \right)}{8.75 \cdot ft}$ $T_s = 336.13\,lb$ $\frac{T_s}{T_n} = 0.56$ **OK < 1.0**

FRONT , REAR AND RIGHT SIDE WALLS ARE ADEQUATE BY COMPARISON

END DESIGN

Wharton and Associates
Mesquite, NV

Allowable Load Capacities for Nylon Nailin in Normal-Weight Concrete[1,2,3]

Anchor Diameter	Minimum Embedment Depth	Minimum Concrete Compressive Strength (f'_c)					
		2,000 psi (13.8 MPa)		4,000 psi (27.6 MPa)		6,000 psi (41.4 MPa)	
d in. (mm)	h_v in. (mm)	Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)
3/16 (4.8)	3/4 (19.1)	45 (0.2)	70 (0.3)	50 (0.2)	80 (0.4)	50 (0.2)	80 (0.4)
	1 (25.4)	50 (0.2)	70 (0.3)	55 (0.2)	80 (0.4)	60 (0.3)	80 (0.4)
1/4 (6.4)	5/8 (15.9)	30 (0.1)	80 (0.4)	35 (0.2)	125 (0.6)	45 (0.2)	125 (0.6)
	3/4 (19.1)	55 (0.2)	80 (0.4)	60 (0.3)	125 (0.6)	60 (0.3)	125 (0.6)
	1 (25.4)	60 (0.3)	80 (0.4)	65 (0.3)	125 (0.6)	65 (0.3)	125 (0.6)
	1 1/2 (38.1)	60 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	70 (0.3)	125 (0.6)
	2 (50.8)	65 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	75 (0.3)	125 (0.6)

1. Allowable load capacities listed are calculated using an applied safety factor of 4.0.
2. Linear interpolation may be used to determine allowable loads for intermediate embedments and compressive strengths.
3. Critical and minimum spacing and edge distances as well as reduction factors for intermediate spacing and edge distances are listed in the Design Criteria section.

TAKEN FROM POWERS "SPECIFICATION AND DESIGN MANUAL"

PROJECT NO: 031329 DATE: August 9, 2001
PAGE: 2

RACKING LOAD TEST OF AB/BA PANELS

TEST PROCEDURES: (cont.)

4. Load was applied according to ASTM:E72. At load increment, deflection readings were obtained. Residual deflection (set) readings were obtained after the loads specified in section 14.4.2 of ASTM:E72-95 were released. The samples were loaded until failure.

5. Total deflection was calculated according to section 14.3.5 of ASTM:E72.

TEST RESULTS:

Racking Load Test of AB/BA Panels

Sample Number	Assembly Size	Core Thickness	Load (lbs.) at 1/4"	Deflection Prior to Failure (in)	Failure Load (lbs)
1A	8' x 10'	3"	878	3.146	3400
1B	8' x 10'	3"	1045	1.685	3310
1C	8' x 10'	3"	721	1.787	2980

SAMPLE DESCRIPTION:

Six AB/BA Panels, manufactured by Panel Built, Inc, were submitted to our laboratory on July 25, 2001 by Panel Built, Inc. in Blairsville, Georgia. The sample description is as follows:

# of Panels	Size of Panels	Core Thickness	Skin Material	Adhesive Type	Edge Members
3	4' x 10'	3"	0.024" Stucco-Embossed Aluminum, □" Hardboard	Neoprene	22 Gauge Steel Channel

Panel fabrication was conducted at Panel Built, Inc. in Blairsville, Georgia on July 23, 2001. See enclosed Panel Built drawing for panel configuration. Based on our review, the submitted samples are consistent with the drawings submitted by Panel Built, Inc.

The core used in producing the AB/BA Panels was Resin-Impregnated Structural Kraft Honeycomb produced at Panel Built, Inc.

The skin material consisted of single sheets of four foot by ten foot by 0.024 inch Stucco-Embossed Aluminum and □" compressed hardboard.

The panel edges consisted of 22 gauge steel channels.

The adhesives used to laminate the core to the skins was a Neoprene Adhesive. The panel was then subjected to a combination of heat and pressure.

Each of the test assemblies consisted of two-four foot by ten foot AB/BA Panels connected together using two divider strips and one c-connector along the ten-foot length. A channel was attached across the bottom using #10 ¾" pan head screws spaced every six inches. An eave connector was attached across the top using the same screws spaced every twelve inches. The walls assemblies were fabricated by Twin City Testing Personal on August 7, 2001.

EXCERPT FROM RACKING LOAD TEST

PRCTI20230167

Wharton and Associates
Mesquite, NV

ITW REDHEAD TRUBOLT+ WEDGE ANCHOR PER ESR-2427

1/2" DIA. x 2- 1/2" EMBEDMENT

FIND ALLOWABLE SHEAR AND TENSION

GIVEN:
ONE CARBON STEEL ANCHOR WITH NO CONCRETE THICKNESS OR EDGE DISTANCE CONCERNS.
NORMAL WEIGHT CONCRETE OF 2500 PSI COMPRESSIVE STRENGTH.
CONDITION "B", NO SUPPLEMENTARY REINFORCEMENT.
ASSUME CRACKED CONCRETE. 6" SLAB ON GRADE CONDITION..
ANCHOR IS CONSIDERED A DUCTILE STEEL ELEMENT.

VALUES PER ESR REPORT ARE AS FOLLOWS.

$h_a := 6 \cdot in$ SLAB ON GRADE THICKNESS REQUIRED

$f_c := 3000 \cdot psi$ CONCRETE STRENGTH REQUIRED

$C_{ac} := 6 \cdot in$ CRITICAL EDGE DISTANCE

$h_{ef} := 2.5 \cdot in$ EFFECTIVE EMBEDMENT

$d_a := .50 \cdot in$ ANCHOR DIAMETER

$l_e := 2.5 \cdot in$ BEARING LENGTH FOR ANCHOR

$k_{cr} := 17$ FACTOR, CRACKED CONCRETE

FIND CONCRETE BREAKOUT STRENGTH IN TENSION

$$N_b := k_{cr} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lb \qquad N_b = 3680.61\,lb \qquad 0.65 \cdot N_b = 2392.4\,lb$$

$$A_{nc} := (2 \cdot 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) \qquad A_{nc} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE

$$A_{nco} := 9 \cdot h_{ef}^2 \qquad A_{nco} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE, NO LIMITATIONS

$$N_{cb} := N_b \cdot \frac{A_{nc}}{A_{nco}} \qquad N_{cb} = 3680.61\,lb$$

FIND CONCRETE BREAKOUT STRENGTH IN SHEAR $\Psi_{cv} := 1.0$ $\psi_{ed} := 2$

$$A_{vc} := (2 \cdot 1.5 \cdot C_{ac}) \cdot h_a \qquad A_{vc} = 108\,in^2 \qquad A_{vco} := 4.5 \cdot C_{ac}^2 \qquad A_{vco} = 162\,in^2$$

$$V_b := 7 \cdot \left(\frac{l_e}{d_a}\right)^{.2} \cdot \sqrt{\frac{d_a}{in}} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{C_{ac}}{in}\right)^{1.5} \cdot lb \qquad V_b = 5497.49\,lb$$

$$V_{cb} := \frac{A_{vc}}{A_{vco}} \cdot \Psi_{cv} \cdot \psi_{ed} \cdot V_b \qquad V_{cb} = 7329.99\,lb$$

FIND FACTOR FOR ASD BASED ON 100% DEAD LOAD

$\alpha := 1.2 \cdot 1 + 1.6 \cdot 0$ $\alpha = 1.2$ CONVERSION FACTOR

$$T_{ension} := \frac{0.65 \cdot 0.75 \cdot 0.4 \cdot N_{cb}}{\alpha} \qquad S_{hear} := \frac{0.70 \cdot 0.75 \cdot V_{cb}}{\alpha}$$

$T_{ension} = 598.1\,lb$ ALLOWABLE TENSION

$S_{hear} = 3206.87\,lb$ ALLOWABLE SHEAR

PRCTI20230167

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R.T. WHARTON & ASSOCIATES, INC
1268 HIDDEN CREST CT.
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725-225-1048

MANUFACTURER
PANEL-BUILT, INC.
302 BEASLEY ST.
BLAIRSVILLE, GA 30512
800-636-3873

STRUCTURAL DESIGN FOR

REDDOT BUILDING THREE
MODULAR OFFICE APROXIMATELY 1200 SQ. FT.
2504 E. MAIN AVE.
PUYALLUP, WA 98372

MATHCAD DEFINED UNITS ARE

$in \equiv 1L \quad lb \equiv 1M \quad si \equiv in^2 \quad ci \equiv in^3 \quad ft \equiv 12 \cdot in \quad sf \equiv ft^2 \quad psf \equiv \frac{lb}{ft^2} \quad plf \equiv \frac{lb}{ft} \quad ksi := 1000 \cdot psi$

$psi \equiv \frac{lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad pli := \frac{lb}{in}$

BUILDING CODE: 2018 WSBC

DESIGN LOADS

$LL_r := 0 \cdot psf$ ROOF LIVE LOAD PER MANUFACTURER, NO STORAGE OR WALKING

$DL_r := 9 \cdot psf$ ROOF DEAD LOAD.

NO WIND LOAD, STRUCTURE IS INTERIOR TO A BUILDING

SEISMIC
SITE CLASS "D" IN LIEU OF A SOILS REPORT.
SEISMIC FORCE-RESISTING SYSTEM A.17 (SHEAR WALLS NOT RATED FOR RESISTANCE)

$I := 1.0$ IMPORTANCE FACTOR PER TABLE 11.5-1 (CATEGORY II)

$R := 2$ RESPONSE MODIFICATION COEFF.

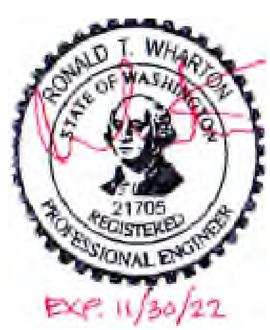
$\Omega_o := 2.5$ SYSTEM OVERSTRENGTH FACTOR

$C_d := 2$ DEFLECTION AMPLIFICATION FACTOR

$S_S := 1.258 \quad S_1 := 0.433$ MAX. GROUND MOTION

$F_a := 1.2 \quad F_v := 1.6$ SITE COEFFICIENTS

$S_{ms} := F_a \cdot S_S \cdot I \quad S_{ms} = 1.51 \quad S_{ds} := \frac{2}{3} \cdot S_{ms} \quad S_{ds} = 1.01$



OCT. 06, 2022

DESIGN CAT. D $P := 1.0$ REDUNDANCY FACTOR (REGULAR IN PLAN)

THEREFORE $C_s := \frac{S_{ds}}{\frac{R}{I}} \quad C_s = 0.5$

$Q_e := C_s \quad Q_e = 0.5$ HORIZONTAL SEISMIC FORCE FACTOR

PRCTI20230167

Wharton and Associates
Mesquite, NV

ALLOWABLE STRESS DESIGN

APPLICABLE BASIC LOAD COMBINATIONS

1. D + L
2. (1 + 0.105 Sds)D + 0.75L + 0.525ρQe)
3. (0.6 - 0 .14Sds)D + 0.7ρQe

DEFLECTION & DRIFT LIMITS

VERTICAL PER IBC TABLE 1604.3
HORIZONTAL SEISMIC PER ASCE TAB. 12.12-1

MATERIAL SPECIFICATIONS

STEEL ROOF DECK: 22 GA. "B" 1-1/2" DEPTH, Fy = 38 KSI. PER ESR-2078P
ALUMINUM: ALLOY 6063 - T6, Fb = 25.0 KSI
WALLS: SANDWICH PANELS
CONCRETE: f'c = 2500 PSI
NYLON NAIL ANCHORS: POWERS 1/4" x 1/2"
EXP. ANCHORS: ITW RED HEAD WEDGE PER ESR-2427

ROOF DECK **USE 1-1/2" x 22 GA. B-DECK**

$span := 12.75 \cdot ft$ $spacing := 12 \cdot in$ $Fy := 38 \cdot ksi$ $E := 29500 \cdot ksi$

PROPERTIES PER MANUFACTURER $Sx := 0.1757 \cdot in^3$ $Ix := 0.1485 \cdot in^4$

$M_a := \frac{Sx \cdot Fy}{1.67}$ $M_a = 333.16 \, ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND LOADS $w_g := (DL_r + LL_r) \cdot spacing$ $w_g = 9 \, plf$ UNIF. GRAVITY LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := w_g \cdot \frac{span^2}{8}$ $M_r = 182.88 \, ft \cdot lb$ $\frac{M_r}{M_a} = 0.55$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot span^4}{384 \cdot E \cdot Ix}$ $\Delta = 1.22 \, in$ $\frac{span}{\Delta} = 125.25$ **OK > 120**

PRCTI20230167

Wharton and Associates
Mesquite, NV

MODULAR WALL PANELS IN BEARING

USE 3" 3-PLY G/G PANELS

COMPOSITE PANEL, GYP.BD. FACING BOTH SIDES, POLYSTYRENE CORE.

FIND THE ALLOWABLE BEARING FOR PANEL BASED ON RACKING LOAD TEST PERFORMED BY TWIN CITY TESTING CORP. USE THE AVG. FAILURE LOAD WITH A SAFETY FACTOR OF FOUR. TEST PANEL LENGTH, 8 FT. HEIGHT 10 FT.

$SF := 4$ SAFETY FACTOR

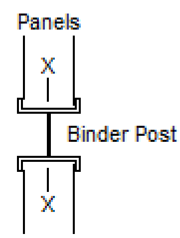
$F_{fail} := 3230 \cdot lb$ AVERAGE LATERAL LOAD PANELS FAILED, DEFLECTION = 2.2" JUST PRIOR TO FAILURE.

$P_{fail} := \dfrac{F_{fail} \cdot 10 \cdot ft}{8 \cdot ft}$ $P_{fail} = 4037.5\,lb$ RESULTANT AXIAL LOAD AT BINDER POST

$w_a := \dfrac{P_{fail}}{SF \cdot 4.25 \cdot ft}$ $w_a = 237.5\,plf$ ALLOWABLE UNIFORM LOAD/UNIT WIDTH

$w_g := \left(DL_r + LL_r\right) \cdot \dfrac{12.75 \cdot ft}{2}$ $w_g = 57.38\,plf$ UNIFORM LOAD $\dfrac{w_g}{w_a} = 0.24$ **OK < 1.0**

CHECK 5 PSF PARTITION LOAD BENDING MOMENT TO BINDER POST



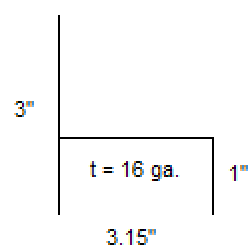
$Sx := 0.529 \cdot in^3$ $F_y := 25.0 \cdot ksi$

$Mr := 5 \cdot psf \cdot 51 \cdot in \cdot \dfrac{(10 \cdot ft)^2}{8}$ $Mr = 265.63\,ft \cdot lb$ REQUIRED MOMENT

$Ma := \dfrac{Sx \cdot F_y}{1.67}$ $Ma = 659.93\,ft \cdot lb$ ALLOWABLE MOMENT

CHECK BINDER POST FOR BENDING $\dfrac{Mr}{Ma} = 0.4$ **OK < 1.0**

USE 3" x 16 GA ALUMINUM SUPPORT HEADER



Section Top Track

$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$

$span_h := 36 \cdot in$ HEADER SPAN OVER DOORS

PROPERTIES PER ANALYSIS $S_x := 0.1679 \cdot in^3$ $I_x := 0.1916 \cdot in^4$

$M_a := \dfrac{S_x \cdot F_y}{1.67}$ $M_a = 209.46\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \dfrac{w_g \cdot span_h^{\,2}}{8}$ $M_r = 64.55\,ft \cdot lb$ $\dfrac{M_r}{M_a} = 0.31$ **OK < 1.0**

$\Delta := \dfrac{5 \cdot w_g \cdot span_h^{\,4}}{384 \cdot E_a \cdot I_x}$ $\Delta = 0.05\,in$ $\dfrac{span_h}{\Delta} = 659.64$ **OK > 180**

PRCTI20230167

ROOF SUPPORT BEAM, W 8 x 10 $span_b := 19.5 \cdot ft$ BEAM SPAN

PROPERTIES $S_x := 7.81 \cdot in^3$ $I_x := 30.8 \cdot in^4$ $F_y := 50 \cdot ksi$ $E = 29500\,ksi$

$M_a := \dfrac{S_x \cdot F_y}{1.67}$ $M_a = 19486.03\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

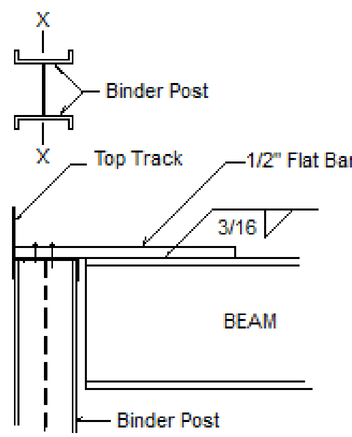
FIND LOADS $w_g := \left[\left(DL_r + LL_r\right) \cdot 12.75 \cdot ft\right] + 10 \cdot plf$ $w_g = 124.75\,plf$ UNIFORM LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \dfrac{w_g \cdot span_b^{\,2}}{8}$ $M_r = 5929.52\,ft \cdot lb$ $\dfrac{M_r}{M_a} = 0.3$ **OK < 1.0**

$\Delta := \dfrac{5 \cdot w_g \cdot span_b^{\,4}}{384 \cdot E \cdot I_x}$ $\Delta = 0.45\,in$ $\dfrac{span_b}{\Delta} = 523.87$ **OK > 180**

USE BINDER POSTS AS SUPPORT COLUMNS



$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$ $Ht := 10 \cdot ft$ $Trib := 51 \cdot in$

PROPERTIES

$Sx := 0.529 \cdot in^3$ $Ix := 0.792 \cdot in^4$ $A := 0.947 \cdot in^2$ $r_x := 0.914 \cdot in$

$F_e := \dfrac{\pi^2 \cdot E_a}{\left(\dfrac{1.0 \cdot Ht}{r_x}\right)^2}$ $F_e = 5725.71\,psi$ LESS THAN

$.44 \cdot F_y = 11000\,psi$

$F_{cr} := .877 \cdot F_e$ $F_{cr} = 5021.45\,psi$ CRITICAL BUCKLING STRESS

$P_n := F_{cr} \cdot A$ $P_n = 4755.31\,lb$ $P_a := \dfrac{P_n}{1.92}$ $P_a = 2476.73\,lb$ ALLOWABLE AXIAL LOAD

$P_r := \dfrac{w_g \cdot span_b}{2}$ $P_r = 1216.31\,lb$ POINT LOAD $\dfrac{P_r}{P_a} = 0.49$ **OK < 1.0**

USE HSS 5" x 5" x 3/16" COLUMNS

$Sx := 5.36 \cdot in^3$ $Ix := 13.4 \cdot in^4$ $A := 3.52 \cdot in^2$ $r := 1.95 \cdot in$ $F_y := 46 \cdot ksi$ $k := 1$ $l_u := 10 \cdot ft$

$M_a := Sx \cdot .66 \cdot F_y$ $M_a = 13560.8\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

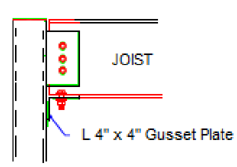
$F_e := \dfrac{\pi^2 \cdot E}{\left(\dfrac{k \cdot l_u}{r}\right)^2}$ $F_e = 76882.68\,psi$ GREATER THAN $.44 \cdot F_y = 20.24\,ksi$ THEREFORE

$F_{cr} := 0.658^{\frac{F_y}{F_e}} \cdot F_y$ $F_{cr} = 35809.63\,psi$ CRITICAL BUCKLING STRESS

$P_a := 0.6 \cdot F_{cr} \cdot A$ $P_a = 75629.93\,lb$ ALLOWABLE AXIAL LOAD

CHECK COLUMN FOR LIVE + DEAD $\dfrac{P_r}{P_a} = 0.02$ **OK < 1.0**

END CONNECT WITH (3) 1/2" DIA. GRADE 5 BOLTS



Note: Number of Bolts Depends on Depth of Beam

$V_{bolt} := .20 \cdot in^2 \cdot 21 \cdot ksi$ $V_{bolt} = 4200\,lb$ ALLOWABLE SHEAR

$\dfrac{w_g \cdot span_b}{2 \cdot 3 \cdot V_{bolt}} = 0.1$ **OK < 1.0**

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LATERAL DESIGN FIND SEISMIC FORCE $Area := 1200 \cdot ft^2$

$DL_w := \frac{Ht \cdot 4 \cdot psf \cdot (160 \cdot ft)}{2 \cdot Area}$ $DL_w = 2.67\,psf$ WALL DEAD LOAD PER SQ.-FT. FLOOR

$W_s := DL_r + DL_w$ $W_s = 11.67\,psf$ SEISMIC DEAD LOAD AT ROOF LEVEL

$F_s := W_s \cdot Area \cdot 0.7 \cdot P \cdot Q_e$ $F_s = 4931.36\,lb$ SEISMIC FORCE

USE THE MODULAR WALLS AS SHEAR WALL (CHECK LEFT SIDE WALL)

$v := \frac{F_s}{2 \cdot 20 \cdot ft}$ $v = 123.28\,plf$ SHEAR

FIND THE ALLOWABLE SHEAR FOR PANEL BASED ON TEST RESULTS.
SEE FOLLOWING TEST RESULTS. A SAFETY FACTOR OF 3 IS USED.

$F_{fail} := 3230 \cdot lb$ AVERAGE FORCE AT FAILURE $SF := 3$ SAFETY FACTOR

$v_a := \frac{F_{fail}}{SF \cdot 8 \cdot ft}$ $v_a = 134.58\,plf$ ALLOWABLE SHEAR CAPACITY $\frac{v}{v_a} = 0.92$ **OK < 1.0**

CHECK WALL ANCHORAGE FOR SHEAR, USE 1/2" x 2-1/2" EMB. WEDGE ANCHORS AT 24"

$V_n := 3206 \cdot lb$ ALLOW.SHEAR $\frac{v \cdot 24 \cdot in}{V_n} = 0.08$ **OK < 1.0** $T_n := 598 \cdot lb$ ALLOW.TENSION

CHECK OVERTURNING

$M_o := v \cdot 20 \cdot ft \cdot 10 \cdot ft$ $M_o = 24656.8\,ft \cdot lb$ OVERTURNING MOMENT

$M_{res} := \frac{(20 \cdot ft)^2}{2} \cdot \left(DL_r \cdot 2 \cdot ft + 40 \cdot plf + \frac{T_n}{24 \cdot in}\right)$ $M_{res} = 71400\,ft \cdot lb$ RESISTING MOMENT

$T_s := \frac{M_o - M_{res} \cdot (0.6 - .14 \cdot S_{ds})}{20 \cdot ft}$ $T_s = -406.16\,lb$ $\frac{T_s}{T_n} = -0.68$ **OK < 1.0**

USE THE MODULAR WALLS AS SHEAR WALL (CHECK FRONT WALL)

$v := \frac{F_s}{2 \cdot 25.5 \cdot ft}$ $v = 96.69\,plf$ SHEAR $\frac{v}{v_a} = 0.72$ **OK < 1.0**

CHECK WALL ANCHORAGE $\frac{v \cdot 24 \cdot in}{V_n} = 0.06$ **OK < 1.0**

CHECK OVERTURNING

$M_o := v \cdot 4.25 \cdot ft \cdot 10 \cdot ft$ $M_o = 4109.47\,ft \cdot lb$ OVERTURNING MOMENT

$M_{res} := \frac{(4.25 \cdot ft)^2}{2} \cdot \left(DL_r \cdot 1 \cdot ft + 40 \cdot plf + \frac{T_n}{24 \cdot in}\right)$ $M_{res} = 3142.88\,ft \cdot lb$ RESISTING MOMENT

$T_s := \frac{M_o - M_{res} \cdot (0.6 - .14 \cdot S_{ds})}{4.25 \cdot ft}$ $T_s = 627.43\,lb$ $\frac{T_s}{T_n} = 1.05$ **OK ~ 1.0**

THE EXISTING WALLS ARE CONSIDERED ADEQUATE TO RESIST SEISMIC SHEAR

END DESIGN

PRCTI20230167

Wharton and Associates
Mesquite, NV

Allowable Load Capacities for Nylon Nailin in Normal-Weight Concrete[1,2,3]

Anchor Diameter d in. (mm)	Minimum Embedment Depth h_v in. (mm)	Minimum Concrete Compressive Strength (f'_c) 2,000 psi (13.8 MPa) Tension lbs. (kN)	2,000 psi (13.8 MPa) Shear lbs. (kN)	4,000 psi (27.6 MPa) Tension lbs. (kN)	4,000 psi (27.6 MPa) Shear lbs. (kN)	6,000 psi (41.4 MPa) Tension lbs. (kN)	6,000 psi (41.4 MPa) Shear lbs. (kN)
3/16 (4.8)	3/4 (19.1)	45 (0.2)	70 (0.3)	50 (0.2)	80 (0.4)	50 (0.2)	80 (0.4)
	1 (25.4)	50 (0.2)	70 (0.3)	55 (0.2)	80 (0.4)	60 (0.3)	80 (0.4)
1/4 (6.4)	5/8 (15.9)	30 (0.1)	80 (0.4)	35 (0.2)	125 (0.6)	45 (0.2)	125 (0.6)
	3/4 (19.1)	55 (0.2)	80 (0.4)	60 (0.3)	125 (0.6)	60 (0.3)	125 (0.6)
	1 (25.4)	60 (0.3)	80 (0.4)	65 (0.3)	125 (0.6)	65 (0.3)	125 (0.6)
	1 1/2 (38.1)	60 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	70 (0.3)	125 (0.6)
	2 (50.8)	65 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	75 (0.3)	125 (0.6)

1. Allowable load capacities listed are calculated using an applied safety factor of 4.0.
2. Linear interpolation may be used to determine allowable loads for intermediate embedments and compressive strengths.
3. Critical and minimum spacing and edge distances as well as reduction factors for intermediate spacing and edge distances are listed in the Design Criteria section.

TAKEN FROM POWERS "SPECIFICATION AND DESIGN MANUAL"

PROJECT NO: 031329 DATE: August 9, 2001
PAGE: 2

RACKING LOAD TEST OF AB/BA PANELS

TEST PROCEDURES: (cont.)

4. Load was applied according to ASTM:E72. At load increment, deflection readings were obtained. Residual deflection (set) readings were obtained after the loads specified in section 14.4.2 of ASTM:E72-95 were released. The samples were loaded until failure.

5. Total deflection was calculated according to section 14.3.5 of ASTM:E72.

TEST RESULTS:

Racking Load Test of AB/BA Panels

Sample Number	Assembly Size	Core Thickness	Load (lbs.) at 1/4"	Deflection Prior to Failure (in)	Failure Load (lbs)
1A	8' x 10'	3"	878	3.146	3400
1B	8' x 10'	3"	1045	1.685	3310
1C	8' x 10'	3"	721	1.787	2980

SAMPLE DESCRIPTION:

Six AB/BA Panels, manufactured by Panel Built, Inc, were submitted to our laboratory on July 25, 2001 by Panel Built, Inc. in Blairsville, Georgia. The sample description is as follows:

# of Panels	Size of Panels	Core Thickness	Skin Material	Adhesive Type	Edge Members
3	4' x 10'	3"	0.024" Stucco-Embossed Aluminum, □" Hardboard	Neoprene	22 Gauge Steel Channel

Panel fabrication was conducted at Panel Built, Inc. in Blairsville, Georgia on July 23, 2001. See enclosed Panel Built drawing for panel configuration. Based on our review, the submitted samples are consistent with the drawings submitted by Panel Built, Inc.

The core used in producing the AB/BA Panels was Resin-Impregnated Structural Kraft Honeycomb produced at Panel Built, Inc.

The skin material consisted of single sheets of four foot by ten foot by 0.024 inch Stucco-Embossed Aluminum and □" compressed hardboard.

The panel edges consisted of 22 gauge steel channels.

The adhesives used to laminate the core to the skins was a Neoprene Adhesive. The panel was then subjected to a combination of heat and pressure.

Each of the test assemblies consisted of two-four foot by ten foot AB/BA Panels connected together using two divider strips and one c-connector along the ten-foot length. A channel was attached across the bottom using #10 ¾" pan head screws spaced every six inches. An eave connector was attached across the top using the same screws spaced every twelve inches. The walls assemblies were fabricated by Twin City Testing Personal on August 7, 2001.

EXCERPT FROM RACKING LOAD TEST

Wharton and Associates
Mesquite, NV

ITW REDHEAD TRUBOLT+ WEDGE ANCHOR PER ESR-2427

1/2" DIA. x 2- 1/2" EMBEDMENT

FIND ALLOWABLE SHEAR AND TENSION

GIVEN:
ONE CARBON STEEL ANCHOR WITH NO CONCRETE THICKNESS OR EDGE DISTANCE CONCERNS.
NORMAL WEIGHT CONCRETE OF 2500 PSI COMPRESSIVE STRENGTH.
CONDITION "B", NO SUPPLEMENTARY REINFORCEMENT.
ASSUME CRACKED CONCRETE. 6" SLAB ON GRADE CONDITION..
ANCHOR IS CONSIDERED A DUCTILE STEEL ELEMENT.

VALUES PER ESR REPORT ARE AS FOLLOWS.

$h_a := 6\cdot in$ SLAB ON GRADE THICKNESS REQUIRED

$f_c := 3000\cdot psi$ CONCRETE STRENGTH REQUIRED

$C_{ac} := 6\cdot in$ CRITICAL EDGE DISTANCE

$h_{ef} := 2.5\cdot in$ EFFECTIVE EMBEDMENT

$d_a := .50\cdot in$ ANCHOR DIAMETER

$l_e := 2.5\cdot in$ BEARING LENGTH FOR ANCHOR

$k_{cr} := 17$ FACTOR, CRACKED CONCRETE

FIND CONCRETE BREAKOUT STRENGTH IN TENSION

$$N_b := k_{cr}\cdot\sqrt{\frac{f_c}{psi}}\cdot\left(\frac{h_{ef}}{in}\right)^{1.5}\cdot lb \qquad N_b = 3680.61\,lb \qquad 0.65\cdot N_b = 2392.4\,lb$$

$$A_{nc} := (2\cdot 1.5\cdot h_{ef})\cdot(2\cdot 1.5\cdot h_{ef}) \qquad A_{nc} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE

$$A_{nco} := 9\cdot h_{ef}^{\,2} \qquad A_{nco} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE, NO LIMITATIONS

$$N_{cb} := N_b\cdot\frac{A_{nc}}{A_{nco}} \qquad N_{cb} = 3680.61\,lb$$

FIND CONCRETE BREAKOUT STRENGTH IN SHEAR $\Psi_{cv} := 1.0$ $\psi_{ed} := 2$

$$A_{vc} := (2\cdot 1.5\cdot C_{ac})\cdot h_a \qquad A_{vc} = 108\,in^2 \qquad A_{vco} := 4.5\cdot C_{ac}^{\,2} \qquad A_{vco} = 162\,in^2$$

$$V_b := 7\cdot\left(\frac{l_e}{d_a}\right)^{.2}\cdot\sqrt{\frac{d_a}{in}}\cdot\sqrt{\frac{f_c}{psi}}\cdot\left(\frac{C_{ac}}{in}\right)^{1.5}\cdot lb \qquad V_b = 5497.49\,lb$$

$$V_{cb} := \frac{A_{vc}}{A_{vco}}\cdot\Psi_{cv}\cdot\psi_{ed}\cdot V_b \qquad V_{cb} = 7329.99\,lb$$

FIND FACTOR FOR ASD BASED ON 100% DEAD LOAD

$\alpha := 1.2\cdot 1 + 1.6\cdot 0$ $\alpha = 1.2$ CONVERSION FACTOR

$$T_{ension} := \frac{0.65\cdot 0.75\cdot 0.4\cdot N_{cb}}{\alpha} \qquad S_{hear} := \frac{0.70\cdot 0.75\cdot V_{cb}}{\alpha}$$

$T_{ension} = 598.1\,lb$ ALLOWABLE TENSION

$S_{hear} = 3206.87\,lb$ ALLOWABLE SHEAR

PRCTI20230167

CONSULTING ENGINEER
R.T. WHARTON & ASSOCIATES, INC
1268 HIDDEN CREST CT.
MUSQUITE, NV 89034
725-225-1048

MANUFACTURER
PANEL-BUILT, INC.
302 BEASLEY ST.
BLAIRSVILLE, GA 30512
800-636-3873

STRUCTURAL DESIGN FOR

REDDOT BUILDING FOUR
MODULAR OFFICE APROXIMATELY 800 SQ. FT.
2504 E. MAIN AVE.
PUYALLUP, WA 98372

MATHCAD DEFINED UNITS ARE

$in \equiv 1L \quad lb \equiv 1M \quad si \equiv in^2 \quad ci \equiv in^3 \quad ft \equiv 12 \cdot in \quad sf \equiv ft^2 \quad psf \equiv \frac{lb}{ft^2} \quad plf \equiv \frac{lb}{ft} \quad ksi := 1000 \cdot psi$

$psi \equiv \frac{lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad pli := \frac{lb}{in}$

BUILDING CODE: 2018 WSBC

DESIGN LOADS

$LL_r := 0 \cdot psf$ ROOF LIVE LOAD PER MANUFACTURER, NO STORAGE OR WALKING

$DL_r := 9 \cdot psf$ ROOF DEAD LOAD.

NO WIND LOAD, STRUCTURE IS INTERIOR TO A BUILDING

SEISMIC
SITE CLASS "D" IN LIEU OF A SOILS REPORT.
SEISMIC FORCE-RESISTING SYSTEM A.17 (SHEAR WALLS NOT RATED FOR RESISTANCE)

$I := 1.0$ IMPORTANCE FACTOR PER TABLE 11.5-1 (CATEGORY II)

$R := 2$ RESPONSE MODIFICATION COEFF.

$\Omega_o := 2.5$ SYSTEM OVERSTRENGTH FACTOR

$C_d := 2$ DEFLECTION AMPLIFICATION FACTOR

$S_S := 1.258 \quad S_1 := 0.433$ MAX. GROUND MOTION

$F_a := 1.2 \quad F_v := 1.6$ SITE COEFFICIENTS

$S_{ms} := F_a \cdot S_S \cdot I \qquad S_{ms} = 1.51 \qquad S_{ds} := \frac{2}{3} \cdot S_{ms} \qquad S_{ds} = 1.01$

DESIGN CAT. D $\quad P := 1.0$ REDUNDANCY FACTOR (REGULAR IN PLAN)

THEREFORE $C_s := \dfrac{S_{ds}}{\frac{R}{I}} \qquad C_s = 0.5$

$Q_e := C_s \qquad Q_e = 0.5$ HORIZONTAL SEISMIC FORCE FACTOR

RONALD T. WHARTON
STATE OF WASHINGTON
21705
REGISTERED
PROFESSIONAL ENGINEER
EXP. 11/30/22
OCT. 06, 2022

PRCTI20230167

Wharton and Associates
Mesquite, NV

ALLOWABLE STRESS DESIGN

APPLICABLE BASIC LOAD COMBINATIONS

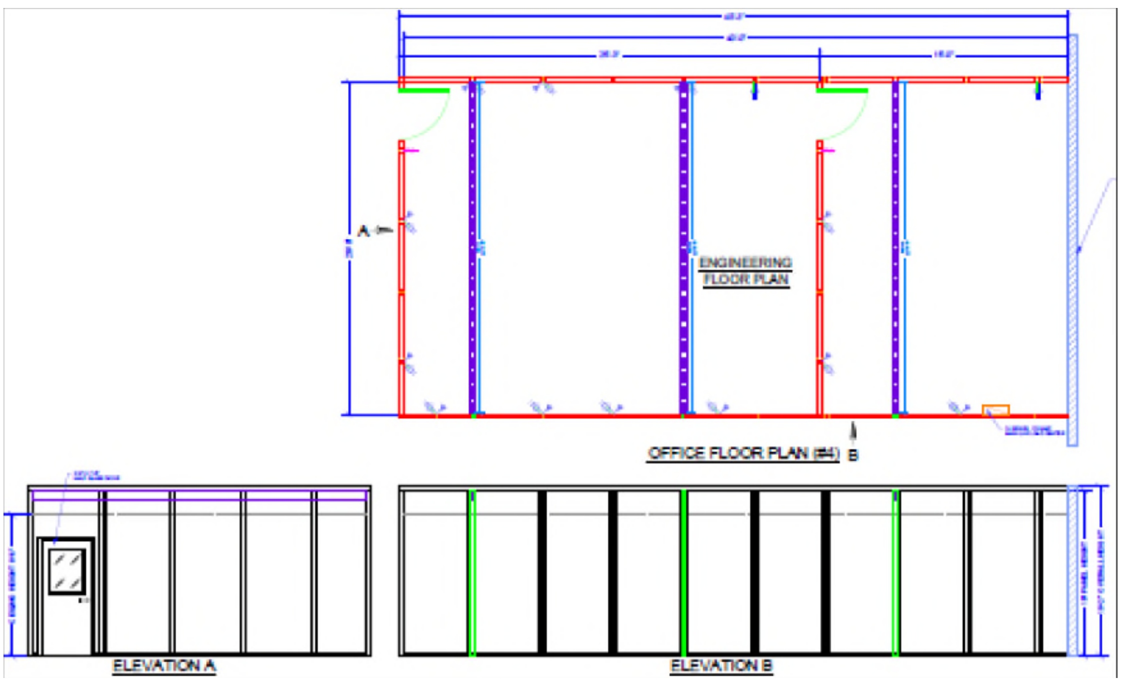
1. D + L
2. (1 + 0.105 Sds)D + 0.75L + 0.525ρQe)
3. (0.6 - 0 .14Sds)D + 0.7ρQe

DEFLECTION & DRIFT LIMITS

VERTICAL PER IBC TABLE 1604.3
HORIZONTAL SEISMIC PER ASCE TAB. 12.12-1

MATERIAL SPECIFICATIONS

STEEL ROOF DECK: 22 GA. "B" 1-1/2" DEPTH, Fy = 38 KSI. PER ESR-2078P
ALUMINUM: ALLOY 6063 - T6, Fb = 25.0 KSI
WALLS: SANDWICH PANELS
CONCRETE: f'c = 2500 PSI
NYLON NAIL ANCHORS: POWERS 1/4" x 1/2"
EXP. ANCHORS: ITW RED HEAD WEDGE PER ESR-2427



ROOF DECK **USE 1-1/2" x 22 GA. B-DECK**

$\text{span} := 12.75 \cdot \text{ft} \qquad \text{spacing} := 12 \cdot \text{in} \qquad Fy := 38 \cdot \text{ksi} \qquad E := 29500 \cdot \text{ksi}$

PROPERTIES PER MANUFACTURER $\quad Sx := 0.1757 \cdot \text{in}^3 \quad Ix := 0.1485 \cdot \text{in}^4$

$M_a := \frac{Sx \cdot Fy}{1.67} \qquad M_a = 333.16\,\text{ft} \cdot \text{lb}$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND LOADS $\quad w_g := (DL_r + LL_r) \cdot \text{spacing} \qquad w_g = 9\,\text{plf}$ UNIF. GRAVITY LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := w_g \cdot \frac{\text{span}^2}{8} \qquad M_r = 182.88\,\text{ft} \cdot \text{lb} \qquad \frac{M_r}{M_a} = 0.55$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot \text{span}^4}{384 \cdot E \cdot Ix} \qquad \Delta = 1.22\,\text{in} \qquad \frac{\text{span}}{\Delta} = 125.25$ **OK > 120**

MODULAR WALL PANELS IN BEARING

USE 3" 3-PLY G/G PANELS

COMPOSITE PANEL, GYP.BD. FACING BOTH SIDES, POLYSTYRENE CORE.

FIND THE ALLOWABLE BEARING FOR PANEL BASED ON RACKING LOAD TEST PERFORMED BY TWIN CITY TESTING CORP. USE THE AVG. FAILURE LOAD WITH A SAFETY FACTOR OF FOUR. TEST PANEL LENGTH, 8 FT. HEIGHT 10 FT.

$SF := 4$ SAFETY FACTOR

$F_{fail} := 3230 \cdot lb$ AVERAGE LATERAL LOAD PANELS FAILED, DEFLECTION = 2.2" JUST PRIOR TO FAILURE.

$P_{fail} := \dfrac{F_{fail} \cdot 10 \cdot ft}{8 \cdot ft}$ $P_{fail} = 4037.5\,lb$ RESULTANT AXIAL LOAD AT BINDER POST

$w_a := \dfrac{P_{fail}}{SF \cdot 4.25 \cdot ft}$ $w_a = 237.5\,plf$ ALLOWABLE UNIFORM LOAD/UNIT WIDTH

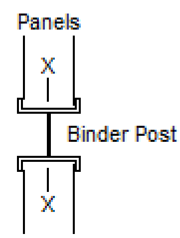
$w_g := (DL_r + LL_r) \cdot \dfrac{12.75 \cdot ft}{2}$ $w_g = 57.38\,plf$ UNIFORM LOAD $\dfrac{w_g}{w_a} = 0.24$ **OK < 1.0**

CHECK 5 PSF PARTITION LOAD BENDING MOMENT TO BINDER POST

$Sx := 0.529 \cdot in^3$ $F_y := 25.0 \cdot ksi$

$Mr := 5 \cdot psf \cdot 51 \cdot in \cdot \dfrac{(10 \cdot ft)^2}{8}$ $Mr = 265.63\,ft \cdot lb$ REQUIRED MOMENT

$Ma := \dfrac{Sx \cdot F_y}{1.67}$ $Ma = 659.93\,ft \cdot lb$ ALLOWABLE MOMENT



CHECK BINDER POST FOR BENDING $\dfrac{Mr}{Ma} = 0.4$ **OK < 1.0**

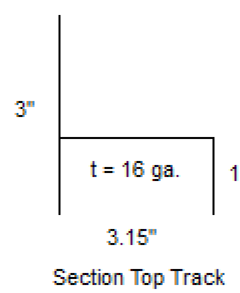
USE 3" x 16 GA ALUMINUM SUPPORT HEADER

$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$

$span_h := 36 \cdot in$ HEADER SPAN OVER DOORS

PROPERTIES PER ANALYSIS $S_x := 0.1679 \cdot in^3$ $I_x := 0.1916 \cdot in^4$

$M_a := \dfrac{S_x \cdot F_y}{1.67}$ $M_a = 209.46\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS



FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \dfrac{w_g \cdot span_h^2}{8}$ $M_r = 64.55\,ft \cdot lb$ $\dfrac{M_r}{M_a} = 0.31$ **OK < 1.0**

$\Delta := \dfrac{5 \cdot w_g \cdot span_h^4}{384 \cdot E_a \cdot I_x}$ $\Delta = 0.05\,in$ $\dfrac{span_h}{\Delta} = 659.64$ **OK > 180**

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Mesquite, NV

ROOF SUPPORT BEAM, W 8 x 10 $span_b := 20 \cdot ft$ BEAM SPAN

PROPERTIES $S_x := 7.81 \cdot in^3$ $I_x := 30.8 \cdot in^4$ $F_y := 50 \cdot ksi$ $E = 29500\,ksi$

$M_a := \frac{S_x \cdot F_y}{1.67}$ $M_a = 19486.03\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

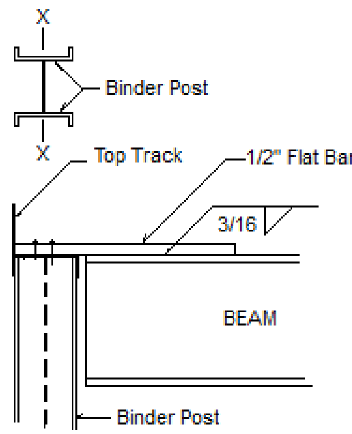
FIND LOADS $w_g := \left[\left(DL_r + LL_r\right) \cdot 12.75 \cdot ft\right] + 10 \cdot plf$ $w_g = 124.75\,plf$ UNIFORM LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \frac{w_g \cdot span_b^{\,2}}{8}$ $M_r = 6237.5\,ft \cdot lb$ $\frac{M_r}{M_a} = 0.32$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot span_b^{\,4}}{384 \cdot E \cdot I_x}$ $\Delta = 0.49\,in$ $\frac{span_b}{\Delta} = 485.56$ **OK > 180**

USE BINDER POSTS AS SUPPORT COLUMNS



$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$ $Ht := 10 \cdot ft$ $Trib := 51 \cdot in$

PROPERTIES

$Sx := 0.529 \cdot in^3$ $Ix := 0.792 \cdot in^4$ $A := 0.947 \cdot in^2$ $r_x := 0.914 \cdot in$

$F_e := \frac{\pi^2 \cdot E_a}{\left(\frac{1.0 \cdot Ht}{r_x}\right)^2}$ $F_e = 5725.71\,psi$ LESS THAN

$.44 \cdot F_y = 11000\,psi$

$F_{cr} := .877 \cdot F_e$ $F_{cr} = 5021.45\,psi$ CRITICAL BUCKLING STRESS

$P_n := F_{cr} \cdot A$ $P_n = 4755.31\,lb$ $P_a := \frac{P_n}{1.92}$ $P_a = 2476.73\,lb$ ALLOWABLE AXIAL LOAD

$P_r := \frac{w_g \cdot span_b}{2}$ $P_r = 1247.5\,lb$ POINT LOAD $\frac{P_r}{P_a} = 0.5$ **OK < 1.0**

Wharton and Associates
Mesquite, NV

LATERAL DESIGN FIND SEISMIC FORCE $Area := 800 \cdot ft^2$

$DL_w := \frac{10 \cdot ft \cdot 4 \cdot psf \cdot (120 \cdot ft)}{2 \cdot Area}$ $DL_w = 3\,psf$ WALL DEAD LOAD PER SQ.-FT. FLOOR

$W_s := DL_r + DL_w$ $W_s = 12\,psf$ SEISMIC DEAD LOAD AT ROOF LEVEL

$F_s := W_s \cdot Area \cdot 0.7 \cdot P \cdot Q_e$ $F_s = 3381.5\,lb$ SEISMIC FORCE

USE THE MODULAR WALLS AS SHEAR WALL (CHECK LEFT SIDE WALL)

$v := \frac{F_s}{2 \cdot 17 \cdot ft}$ $v = 99.46\,plf$ SHEAR

FIND THE ALLOWABLE SHEAR FOR PANEL BASED ON TEST RESULTS.
SEE FOLLOWING TEST RESULTS. A SAFETY FACTOR OF 3 IS USED.

$F_{fail} := 3230 \cdot lb$ AVERAGE FORCE AT FAILURE $SF := 3$ SAFETY FACTOR

$v_a := \frac{F_{fail}}{SF \cdot 8 \cdot ft}$ $v_a = 134.58\,plf$ ALLOWABLE SHEAR CAPACITY $\frac{v}{v_a} = 0.74$ **OK < 1.0**

CHECK WALL ANCHORAGE FOR SHEAR, USE 1/2" x 2-1/2" EMB. WEDGE ANCHORS AT 48"

$V_n := 3206 \cdot lb$ ALLOW.SHEAR $\frac{v \cdot 48 \cdot in}{V_n} = 0.12$ **OK < 1.0** $T_n := 598 \cdot lb$ ALLOW.TENSION

CHECK OVERTURNING

$M_o := v \cdot 17 \cdot ft \cdot 10 \cdot ft$ $M_o = 16907.52\,ft \cdot lb$ OVERTURNING MOMENT

$M_{res} := \frac{(17 \cdot ft)^2}{2} \cdot \left(DL_r \cdot 2 \cdot ft + 40 \cdot plf + \frac{T_n}{48 \cdot in}\right)$ $M_{res} = 29983.75\,ft \cdot lb$ RESISTING MOMENT

$T_s := \frac{M_o - M_{res} \cdot (0.6 - .14 \cdot S_{ds})}{17 \cdot ft}$ $T_s = 184.82\,lb$ $\frac{T_s}{T_n} = 0.31$ **OK < 1.0**

THE FRONT, REAR WALLS ARE CONSIDERED ADEQUATE BY COMPARISON

THE EXISTING SHARED WALL IS CONSIDERED ADEQUATE TO RESIST SEISMIC SHEAR

END DESIGN

Allowable Load Capacities for Nylon Nailin in Normal-Weight Concrete[1,2,3]

Anchor Diameter d in. (mm)	Minimum Embedment Depth h_v in. (mm)	Minimum Concrete Compressive Strength (f'_c)					
		2,000 psi (13.8 MPa)		4,000 psi (27.6 MPa)		6,000 psi (41.4 MPa)	
		Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)
3/16 (4.8)	3/4 (19.1)	45 (0.2)	70 (0.3)	50 (0.2)	80 (0.4)	50 (0.2)	80 (0.4)
	1 (25.4)	50 (0.2)	70 (0.3)	55 (0.2)	80 (0.4)	60 (0.3)	80 (0.4)
1/4 (6.4)	5/8 (15.9)	30 (0.1)	80 (0.4)	35 (0.2)	125 (0.6)	45 (0.2)	125 (0.6)
	3/4 (19.1)	55 (0.2)	80 (0.4)	60 (0.3)	125 (0.6)	60 (0.3)	125 (0.6)
	1 (25.4)	60 (0.3)	80 (0.4)	65 (0.3)	125 (0.6)	65 (0.3)	125 (0.6)
	1 1/2 (38.1)	60 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	70 (0.3)	125 (0.6)
	2 (50.8)	65 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	75 (0.3)	125 (0.6)

1. Allowable load capacities listed are calculated using an applied safety factor of 4.0.
2. Linear interpolation may be used to determine allowable loads for intermediate embedments and compressive strengths.
3. Critical and minimum spacing and edge distances as well as reduction factors for intermediate spacing and edge distances are listed in the Design Criteria section.

TAKEN FROM POWERS "SPECIFICATION AND DESIGN MANUAL"

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RACKING LOAD TEST OF AB/BA PANELS

TEST PROCEDURES: (cont.)

4. Load was applied according to ASTM:E72. At load increment, deflection readings were obtained. Residual deflection (set) readings were obtained after the loads specified in section 14.4.2 of ASTM:E72-95 were released. The samples were loaded until failure.

5. Total deflection was calculated according to section 14.3.5 of ASTM:E72.

TEST RESULTS:

Racking Load Test of AB/BA Panels

Sample Number	Assembly Size	Core Thickness	Load (lbs.) at 1/4"	Deflection Prior to Failure (in)	Failure Load (lbs)
1A	8' x 10'	3"	878	3.146	3400
1B	8' x 10'	3"	1045	1.685	3310
1C	8' x 10'	3"	721	1.787	2980

SAMPLE DESCRIPTION:

Six AB/BA Panels, manufactured by Panel Built, Inc, were submitted to our laboratory on July 25, 2001 by Panel Built, Inc. in Blairsville, Georgia. The sample description is as follows:

# of Panels	Size of Panels	Core Thickness	Skin Material	Adhesive Type	Edge Members
3	4' x 10'	3"	0.024" Stucco-Embossed Aluminum, □" Hardboard	Neoprene	22 Gauge Steel Channel

Panel fabrication was conducted at Panel Built, Inc. in Blairsville, Georgia on July 23, 2001. See enclosed Panel Built drawing for panel configuration. Based on our review, the submitted samples are consistent with the drawings submitted by Panel Built, Inc.

The core used in producing the AB/BA Panels was Resin-Impregnated Structural Kraft Honeycomb produced at Panel Built, Inc.

The skin material consisted of single sheets of four foot by ten foot by 0.024 inch Stucco-Embossed Aluminum and □" compressed hardboard.

The panel edges consisted of 22 gauge steel channels.

The adhesives used to laminate the core to the skins was a Neoprene Adhesive. The panel was then subjected to a combination of heat and pressure.

Each of the test assemblies consisted of two-four foot by ten foot AB/BA Panels connected together using two divider strips and one c-connector along the ten-foot length. A channel was attached across the bottom using #10 ¾" pan head screws spaced every six inches. An eave connector was attached across the top using the same screws spaced every twelve inches. The walls assemblies were fabricated by Twin City Testing Personal on August 7, 2001.

EXCERPT FROM RACKING LOAD TEST

Wharton and Associates
Mesquite, NV

ITW REDHEAD TRUBOLT+ WEDGE ANCHOR PER ESR-2427

1/2" DIA. x 2- 1/2" EMBEDMENT

FIND ALLOWABLE SHEAR AND TENSION

GIVEN:
ONE CARBON STEEL ANCHOR WITH NO CONCRETE THICKNESS OR EDGE DISTANCE CONCERNS.
NORMAL WEIGHT CONCRETE OF 2500 PSI COMPRESSIVE STRENGTH.
CONDITION "B", NO SUPPLEMENTARY REINFORCEMENT.
ASSUME CRACKED CONCRETE. 6" SLAB ON GRADE CONDITION..
ANCHOR IS CONSIDERED A DUCTILE STEEL ELEMENT.

VALUES PER ESR REPORT ARE AS FOLLOWS.

$h_a := 6 \cdot in$ SLAB ON GRADE THICKNESS REQUIRED

$f_c := 3000 \cdot psi$ CONCRETE STRENGTH REQUIRED

$C_{ac} := 6 \cdot in$ CRITICAL EDGE DISTANCE

$h_{ef} := 2.5 \cdot in$ EFFECTIVE EMBEDMENT

$d_a := .50 \cdot in$ ANCHOR DIAMETER

$l_e := 2.5 \cdot in$ BEARING LENGTH FOR ANCHOR

$k_{cr} := 17$ FACTOR, CRACKED CONCRETE

FIND CONCRETE BREAKOUT STRENGTH IN TENSION

$$N_b := k_{cr} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lb \qquad N_b = 3680.61\,lb \qquad 0.65 \cdot N_b = 2392.4\,lb$$

$$A_{nc} := (2 \cdot 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) \qquad A_{nc} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE

$$A_{nco} := 9 \cdot h_{ef}^2 \qquad A_{nco} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE, NO LIMITATIONS

$$N_{cb} := N_b \cdot \frac{A_{nc}}{A_{nco}} \qquad N_{cb} = 3680.61\,lb$$

FIND CONCRETE BREAKOUT STRENGTH IN SHEAR $\Psi_{cv} := 1.0$ $\psi_{ed} := 2$

$$A_{vc} := (2 \cdot 1.5 \cdot C_{ac}) \cdot h_a \qquad A_{vc} = 108\,in^2 \qquad A_{vco} := 4.5 \cdot C_{ac}^2 \qquad A_{vco} = 162\,in^2$$

$$V_b := 7 \cdot \left(\frac{l_e}{d_a}\right)^{.2} \cdot \sqrt{\frac{d_a}{in}} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{C_{ac}}{in}\right)^{1.5} \cdot lb \qquad V_b = 5497.49\,lb$$

$$V_{cb} := \frac{A_{vc}}{A_{vco}} \cdot \Psi_{cv} \cdot \psi_{ed} \cdot V_b \qquad V_{cb} = 7329.99\,lb$$

FIND FACTOR FOR ASD BASED ON 100% DEAD LOAD

$\alpha := 1.2 \cdot 1 + 1.6 \cdot 0$ $\alpha = 1.2$ CONVERSION FACTOR

$$T_{ension} := \frac{0.65 \cdot 0.75 \cdot 0.4 \cdot N_{cb}}{\alpha} \qquad S_{hear} := \frac{0.70 \cdot 0.75 \cdot V_{cb}}{\alpha}$$

$T_{ension} = 598.1\,lb$ ALLOWABLE TENSION

$S_{hear} = 3206.87\,lb$ ALLOWABLE SHEAR

CONSULTING ENGINEER
R.T. WHARTON & ASSOCIATES, INC
1268 HIDDEN CREST CT.
MUSQUITE, NV 89034
725-225-1048

MANUFACTURER
PANEL-BUILT, INC.
302 BEASLEY ST.
BLAIRSVILLE, GA 30512
800-636-3873

STRUCTURAL DESIGN FOR

REDDOT BUILDING FIVE
MODULAR OFFICE APROXIMATELY 480 SQ. FT.
2504 E. MAIN AVE.
PUYALLUP, WA 98372

MATHCAD DEFINED UNITS ARE

$in \equiv 1L \quad lb \equiv 1M \quad si \equiv in^2 \quad ci \equiv in^3 \quad ft \equiv 12 \cdot in \quad sf \equiv ft^2 \quad psf \equiv \frac{lb}{ft^2} \quad plf \equiv \frac{lb}{ft} \quad ksi := 1000 \cdot psi$

$psi \equiv \frac{lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad pli := \frac{lb}{in}$

BUILDING CODE: 2018 WSBC

DESIGN LOADS

$LL_r := 0 \cdot psf$ ROOF LIVE LOAD PER MANUFACTURER, NO STORAGE OR WALKING

$DL_r := 9 \cdot psf$ ROOF DEAD LOAD.

NO WIND LOAD, STRUCTURE IS INTERIOR TO A BUILDING

SEISMIC
SITE CLASS "D" IN LIEU OF A SOILS REPORT.
SEISMIC FORCE-RESISTING SYSTEM A.17 (SHEAR WALLS NOT RATED FOR RESISTANCE)

$I := 1.0$ IMPORTANCE FACTOR PER TABLE 11.5-1 (CATEGORY II)

$R := 2$ RESPONSE MODIFICATION COEFF.

$\Omega_o := 2.5$ SYSTEM OVERSTRENGTH FACTOR

$C_d := 2$ DEFLECTION AMPLIFICATION FACTOR

$S_S := 1.258 \quad S_1 := 0.433$ MAX. GROUND MOTION

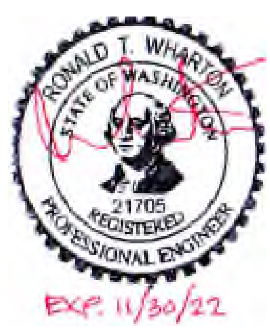
$F_a := 1.2 \quad F_v := 1.6$ SITE COEFFICIENTS

$S_{ms} := F_a \cdot S_S \cdot I \qquad S_{ms} = 1.51 \qquad S_{ds} := \frac{2}{3} \cdot S_{ms} \qquad S_{ds} = 1.01$

DESIGN CAT. D $P := 1.0$ REDUNDANCY FACTOR
(REGULAR IN PLAN)

THEREFORE $C_s := \frac{S_{ds}}{\frac{R}{I}} \qquad C_s = 0.5$

$Q_e := C_s \qquad Q_e = 0.5$ HORIZONTAL SEISMIC FORCE FACTOR



OCT. 06, 2022

PRCTI20230167

Wharton and Associates
Mesquite, NV

ALLOWABLE STRESS DESIGN

APPLICABLE BASIC LOAD COMBINATIONS

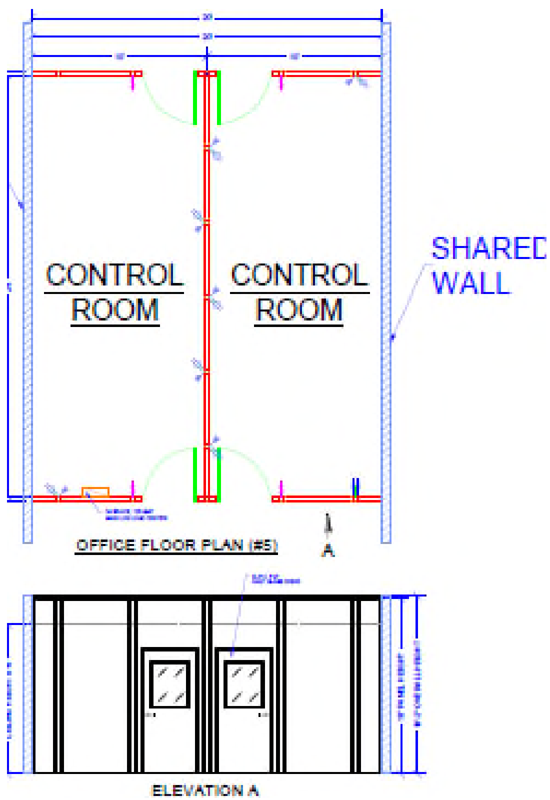
1. D + L
2. (1 + 0.105 Sds)D + 0.75L + 0.525ρQe)
3. (0.6 - 0 .14Sds)D + 0.7ρQe

DEFLECTION & DRIFT LIMITS

VERTICAL PER IBC TABLE 1604.3
HORIZONTAL SEISMIC PER ASCE TAB. 12.12-1

MATERIAL SPECIFICATIONS

STEEL ROOF DECK: 22 GA. "B" 1-1/2" DEPTH, Fy = 38 KSI. PER ESR-2078P
ALUMINUM: ALLOY 6063 - T6, Fb = 25.0 KSI
WALLS: SANDWICH PANELS
CONCRETE: f'c = 2500 PSI
NYLON NAIL ANCHORS: POWERS 1/4" x 1/2"
EXP. ANCHORS: ITW RED HEAD WEDGE PER ESR-2427



ROOF DECK **USE 1-1/2" x 22 GA. B-DECK**

$span := 10 \cdot ft$ $spacing := 12 \cdot in$ $Fy := 38 \cdot ksi$ $E := 29500 \cdot ksi$

PROPERTIES PER MANUFACTURER $Sx := 0.1757 \cdot in^3$ $Ix := 0.1485 \cdot in^4$

$M_a := \frac{Sx \cdot Fy}{1.67}$ $M_a = 333.16\, ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND LOADS $w_g := (DL_r + LL_r) \cdot spacing$ $w_g = 9\, plf$ UNIF. GRAVITY LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := w_g \cdot \frac{span^2}{8}$ $M_r = 112.5\, ft \cdot lb$ $\frac{M_r}{M_a} = 0.34$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot span^4}{384 \cdot E \cdot Ix}$ $\Delta = 0.46\, in$ $\frac{span}{\Delta} = 259.6$ **OK > 120**

MODULAR WALL PANELS IN BEARING

USE 3" 3-PLY G/G PANELS

COMPOSITE PANEL, GYP.BD. FACING BOTH SIDES, POLYSTYRENE CORE.

FIND THE ALLOWABLE BEARING FOR PANEL BASED ON RACKING LOAD TEST PERFORMED BY TWIN CITY TESTING CORP. USE THE AVG. FAILURE LOAD WITH A SAFETY FACTOR OF FOUR. TEST PANEL LENGTH, 8 FT. HEIGHT 10 FT.

$SF := 4$ SAFETY FACTOR

$F_{fail} := 3230 \cdot lb$ AVERAGE LATERAL LOAD PANELS FAILED, DEFLECTION = 2.2" JUST PRIOR TO FAILURE.

$P_{fail} := \frac{F_{fail} \cdot 10 \cdot ft}{8 \cdot ft}$ $P_{fail} = 4037.5\,lb$ RESULTANT AXIAL LOAD AT BINDER POST

$w_a := \frac{P_{fail}}{SF \cdot 4.25 \cdot ft}$ $w_a = 237.5\,plf$ ALLOWABLE UNIFORM LOAD/UNIT WIDTH

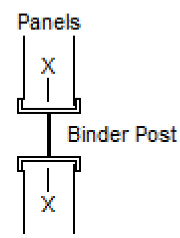
$w_g := \left(DL_r + LL_r\right) \cdot \frac{3 \cdot ft}{2}$ $w_g = 47.25\,plf$ UNIFORM LOAD $\frac{w_g}{w_a} = 0.06$ **OK < 1.0**

CHECK 5 PSF PARTITION LOAD BENDING MOMENT TO BINDER POST

$Sx := 0.529 \cdot in^3$ $F_y := 25.0 \cdot ksi$

$Mr := 5 \cdot psf \cdot 51 \cdot in \cdot \frac{(10 \cdot ft)^2}{8}$ $Mr = 265.63\,ft \cdot lb$ REQUIRED MOMENT

$Ma := \frac{Sx \cdot F_y}{1.67}$ $Ma = 659.93\,ft \cdot lb$ ALLOWABLE MOMENT



CHECK BINDER POST FOR BENDING $\frac{Mr}{Ma} = 0.4$ **OK < 1.0**

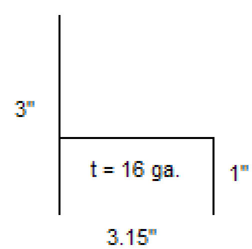
USE 3" x 16 GA ALUMINUM SUPPORT HEADER

$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$

$span_h := 36 \cdot in$ HEADER SPAN OVER DOORS

PROPERTIES PER ANALYSIS $S_x := 0.1679 \cdot in^3$ $I_x := 0.1916 \cdot in^4$

$M_a := \frac{S_x \cdot F_y}{1.67}$ $M_a = 209.46\,ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS



Section Top Track

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \frac{w_g \cdot span_h^2}{8}$ $M_r = 15.19\,ft \cdot lb$ $\frac{M_r}{M_a} = 0.07$ **OK < 1.0**

$\Delta := \frac{5 \cdot w_g \cdot span_h^4}{384 \cdot E_a \cdot I_x}$ $\Delta = 0.05\,in$ $\frac{span_h}{\Delta} = 659.64$ **OK > 180**

PRCTI20230167

Wharton and Associates
Mesquite, NV

LATERAL DESIGN FIND SEISMIC FORCE $Area := 480 \cdot ft^2$

$DL_w := \frac{10 \cdot ft \cdot 4 \cdot psf \cdot (88 \cdot ft)}{2 \cdot Area}$ $DL_w = 3.67\,psf$ WALL DEAD LOAD PER SQ.-FT. FLOOR

$W_s := DL_r + DL_w$ $W_s = 12.67\,psf$ SEISMIC DEAD LOAD AT ROOF LEVEL

$F_s := W_s \cdot Area \cdot 0.7 \cdot P \cdot Q_e$ $F_s = 2141.62\,lb$ SEISMIC FORCE

USE THE MODULAR WALLS AS SHEAR WALL (CHECK CENTER WALL)

$v := \frac{F_s}{2 \cdot 24 \cdot ft}$ $v = 44.62\,plf$ SHEAR

FIND THE ALLOWABLE SHEAR FOR PANEL BASED ON TEST RESULTS.
SEE FOLLOWING TEST RESULTS. A SAFETY FACTOR OF 3 IS USED.

$F_{fail} := 3230 \cdot lb$ AVERAGE FORCE AT FAILURE $SF := 3$ SAFETY FACTOR

$v_a := \frac{F_{fail}}{SF \cdot 8 \cdot ft}$ $v_a = 134.58\,plf$ ALLOWABLE SHEAR CAPACITY $\frac{v}{v_a} = 0.33$ **OK < 1.0**

CHECK WALL ANCHORAGE FOR SHEAR, USE 1/2" x 2-1/2" EMB. WEDGE ANCHORS AT 48"

$V_n := 3206 \cdot lb$ ALLOW.SHEAR $\frac{v \cdot 48 \cdot in}{V_n} = 0.06$ **OK < 1.0** $T_n := 598 \cdot lb$ ALLOW.TENSION

CHECK OVERTURNING

$M_o := v \cdot 20 \cdot ft \cdot 10 \cdot ft$ $M_o = 8923.41\,ft \cdot lb$ OVERTURNING MOMENT

$M_{res} := \frac{(20 \cdot ft)^2}{2} \cdot \left(DL_r \cdot 10 \cdot ft + 40 \cdot plf + \frac{T_n}{48 \cdot in}\right)$ $M_{res} = 55900\,ft \cdot lb$ RESISTING MOMENT

$T_s := \frac{M_o - M_{res} \cdot (0.6 - .14 \cdot S_{ds})}{20 \cdot ft}$ $T_s = -837.03\,lb$ NEGATIVE, NONE

THE FRONT, REAR WALLS ARE CONSIDERED ADEQUATE BY COMPARISON

THE EXISTING SHARED WALLS ARE CONSIDERED ADEQUATE TO RESIST SEISMIC SHEAR

END DESIGN

Allowable Load Capacities for Nylon Nailin in Normal-Weight Concrete[1,2,3]

Anchor Diameter d in. (mm)	Minimum Embedment Depth h_v in. (mm)	Minimum Concrete Compressive Strength (f'_c)					
		2,000 psi (13.8 MPa)		**4,000 psi** (27.6 MPa)		**6,000 psi** (41.4 MPa)	
		Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)
3/16 (4.8)	3/4 (19.1)	45 (0.2)	70 (0.3)	50 (0.2)	80 (0.4)	50 (0.2)	80 (0.4)
	1 (25.4)	50 (0.2)	70 (0.3)	55 (0.2)	80 (0.4)	60 (0.3)	80 (0.4)
1/4 (6.4)	5/8 (15.9)	30 (0.1)	80 (0.4)	35 (0.2)	125 (0.6)	45 (0.2)	125 (0.6)
	3/4 (19.1)	55 (0.2)	80 (0.4)	60 (0.3)	125 (0.6)	60 (0.3)	125 (0.6)
	1 (25.4)	60 (0.3)	80 (0.4)	65 (0.3)	125 (0.6)	65 (0.3)	125 (0.6)
	1 1/2 (38.1)	60 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	70 (0.3)	125 (0.6)
	2 (50.8)	65 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	75 (0.3)	125 (0.6)

1. Allowable load capacities listed are calculated using an applied safety factor of 4.0.
2. Linear interpolation may be used to determine allowable loads for intermediate embedments and compressive strengths.
3. Critical and minimum spacing and edge distances as well as reduction factors for intermediate spacing and edge distances are listed in the Design Criteria section.

TAKEN FROM POWERS "SPECIFICATION AND DESIGN MANUAL"

PROJECT NO: 031329 DATE: August 9, 2001
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RACKING LOAD TEST OF AB/BA PANELS

TEST PROCEDURES: (cont.)

4. Load was applied according to ASTM:E72. At load increment, deflection readings were obtained. Residual deflection (set) readings were obtained after the loads specified in section 14.4.2 of ASTM:E72-95 were released. The samples were loaded until failure.

5. Total deflection was calculated according to section 14.3.5 of ASTM:E72.

TEST RESULTS:

Racking Load Test of AB/BA Panels

Sample Number	Assembly Size	Core Thickness	Load (lbs.) at 1/4"	Deflection Prior to Failure (in)	Failure Load (lbs)
1A	8' x 10'	3"	878	3.146	3400
1B	8' x 10'	3"	1045	1.685	3310
1C	8' x 10'	3"	721	1.787	2980

SAMPLE DESCRIPTION:

Six AB/BA Panels, manufactured by Panel Built, Inc, were submitted to our laboratory on July 25, 2001 by Panel Built, Inc. in Blairsville, Georgia. The sample description is as follows:

# of Panels	Size of Panels	Core Thickness	Skin Material	Adhesive Type	Edge Members
3	4' x 10'	3"	0.024" Stucco-Embossed Aluminum, □" Hardboard	Neoprene	22 Gauge Steel Channel

Panel fabrication was conducted at Panel Built, Inc. in Blairsville, Georgia on July 23, 2001. See enclosed Panel Built drawing for panel configuration. Based on our review, the submitted samples are consistent with the drawings submitted by Panel Built, Inc.

The core used in producing the AB/BA Panels was Resin-Impregnated Structural Kraft Honeycomb produced at Panel Built, Inc.

The skin material consisted of single sheets of four foot by ten foot by 0.024 inch Stucco-Embossed Aluminum and □" compressed hardboard.

The panel edges consisted of 22 gauge steel channels.

The adhesives used to laminate the core to the skins was a Neoprene Adhesive. The panel was then subjected to a combination of heat and pressure.

Each of the test assemblies consisted of two-four foot by ten foot AB/BA Panels connected together using two divider strips and one c-connector along the ten-foot length. A channel was attached across the bottom using #10 ¾" pan head screws spaced every six inches. An eave connector was attached across the top using the same screws spaced every twelve inches. The walls assemblies were fabricated by Twin City Testing Personal on August 7, 2001.

EXCERPT FROM RACKING LOAD TEST

PRCTI20230167

Wharton and Associates
Mesquite, NV

ITW REDHEAD TRUBOLT+ WEDGE ANCHOR PER ESR-2427

1/2" DIA. x 2- 1/2" EMBEDMENT

FIND ALLOWABLE SHEAR AND TENSION

GIVEN:
ONE CARBON STEEL ANCHOR WITH NO CONCRETE THICKNESS OR EDGE DISTANCE CONCERNS.
NORMAL WEIGHT CONCRETE OF 2500 PSI COMPRESSIVE STRENGTH.
CONDITION "B", NO SUPPLEMENTARY REINFORCEMENT.
ASSUME CRACKED CONCRETE. 6" SLAB ON GRADE CONDITION..
ANCHOR IS CONSIDERED A DUCTILE STEEL ELEMENT.

VALUES PER ESR REPORT ARE AS FOLLOWS.

$h_a := 6 \cdot in$ SLAB ON GRADE THICKNESS REQUIRED

$C_{ac} := 6 \cdot in$ CRITICAL EDGE DISTANCE

$d_a := .50 \cdot in$ ANCHOR DIAMETER

$k_{cr} := 17$ FACTOR, CRACKED CONCRETE

$f_c := 3000 \cdot psi$ CONCRETE STRENGTH REQUIRED

$h_{ef} := 2.5 \cdot in$ EFFECTIVE EMBEDMENT

$l_e := 2.5 \cdot in$ BEARING LENGTH FOR ANCHOR

FIND CONCRETE BREAKOUT STRENGTH IN TENSION

$$N_b := k_{cr} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lb \qquad N_b = 3680.61\,lb \qquad 0.65 \cdot N_b = 2392.4\,lb$$

$$A_{nc} := (2 \cdot 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) \qquad A_{nc} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE

$$A_{nco} := 9 \cdot h_{ef}^{\,2} \qquad A_{nco} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE, NO LIMITATIONS

$$N_{cb} := N_b \cdot \frac{A_{nc}}{A_{nco}} \qquad N_{cb} = 3680.61\,lb$$

FIND CONCRETE BREAKOUT STRENGTH IN SHEAR $\Psi_{cv} := 1.0$ $\psi_{ed} := 2$

$$A_{vc} := (2 \cdot 1.5 \cdot C_{ac}) \cdot h_a \qquad A_{vc} = 108\,in^2 \qquad A_{vco} := 4.5 \cdot C_{ac}^{\,2} \qquad A_{vco} = 162\,in^2$$

$$V_b := 7 \cdot \left(\frac{l_e}{d_a}\right)^{.2} \cdot \sqrt{\frac{d_a}{in}} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{C_{ac}}{in}\right)^{1.5} \cdot lb \qquad V_b = 5497.49\,lb$$

$$V_{cb} := \frac{A_{vc}}{A_{vco}} \cdot \Psi_{cv} \cdot \psi_{ed} \cdot V_b \qquad V_{cb} = 7329.99\,lb$$

FIND FACTOR FOR ASD BASED ON 100% DEAD LOAD

$\alpha := 1.2 \cdot 1 + 1.6 \cdot 0$ $\alpha = 1.2$ CONVERSION FACTOR

$$T_{ension} := \frac{0.65 \cdot 0.75 \cdot 0.4 \cdot N_{cb}}{\alpha} \qquad S_{hear} := \frac{0.70 \cdot 0.75 \cdot V_{cb}}{\alpha}$$

$T_{ension} = 598.1\,lb$ ALLOWABLE TENSION

$S_{hear} = 3206.87\,lb$ ALLOWABLE SHEAR

CONSULTING ENGINEER
R.T. WHARTON & ASSOCIATES, INC
1268 HIDDEN CREST CT.
MUSQUITE, NV 89034
725-225-1048

MANUFACTURER
PANEL-BUILT, INC.
302 BEASLEY ST.
BLAIRSVILLE, GA 30512
800-636-3873

STRUCTURAL DESIGN FOR

REDDOT BUILDING SIX
MODULAR OFFICE APROXIMATELY 1437 SQ. FT.
2504 E. MAIN AVE.
PUYALLUP, WA 98372

MATHCAD DEFINED UNITS ARE

$in \equiv 1L \quad lb \equiv 1M \quad si \equiv in^2 \quad ci \equiv in^3 \quad ft \equiv 12 \cdot in \quad sf \equiv ft^2 \quad psf \equiv \frac{lb}{ft^2} \quad plf \equiv \frac{lb}{ft} \quad ksi := 1000 \cdot psi$

$psi \equiv \frac{lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad pli := \frac{lb}{in}$

BUILDING CODE: 2018 WSBC

DESIGN LOADS

$LL_r := 0 \cdot psf$ ROOF LIVE LOAD PER MANUFACTURER, NO STORAGE OR WALKING

$DL_r := 9 \cdot psf$ ROOF DEAD LOAD.

NO WIND LOAD, STRUCTURE IS INTERIOR TO A BUILDING

SEISMIC
SITE CLASS "D" IN LIEU OF A SOILS REPORT.
SEISMIC FORCE-RESISTING SYSTEM A.17 (SHEAR WALLS NOT RATED FOR RESISTANCE)

$I := 1.0$ IMPORTANCE FACTOR PER TABLE 11.5-1 (CATEGORY II)

$R := 2$ RESPONSE MODIFICATION COEFF.

$\Omega_o := 2.5$ SYSTEM OVERSTRENGTH FACTOR

$C_d := 2$ DEFLECTION AMPLIFICATION FACTOR

$S_S := 1.258 \quad S_1 := 0.433$ MAX. GROUND MOTION

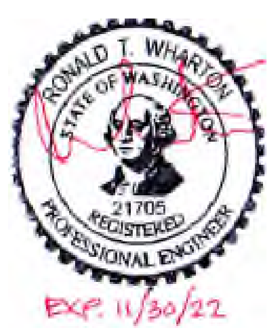
$F_a := 1.2 \quad F_v := 1.6$ SITE COEFFICIENTS

$S_{ms} := F_a \cdot S_S \cdot I \quad S_{ms} = 1.51 \quad S_{ds} := \frac{2}{3} \cdot S_{ms} \quad S_{ds} = 1.01$

DESIGN CAT. D $P := 1.0$ REDUNDANCY FACTOR
(REGULAR IN PLAN)

THEREFORE $C_s := \frac{S_{ds}}{\frac{R}{I}} \quad C_s = 0.5$

$Q_e := C_s \quad Q_e = 0.5$ HORIZONTAL SEISMIC FORCE FACTOR



OCT 06, 2022

Wharton and Associates
Mesquite, NV

ALLOWABLE STRESS DESIGN

APPLICABLE BASIC LOAD COMBINATIONS

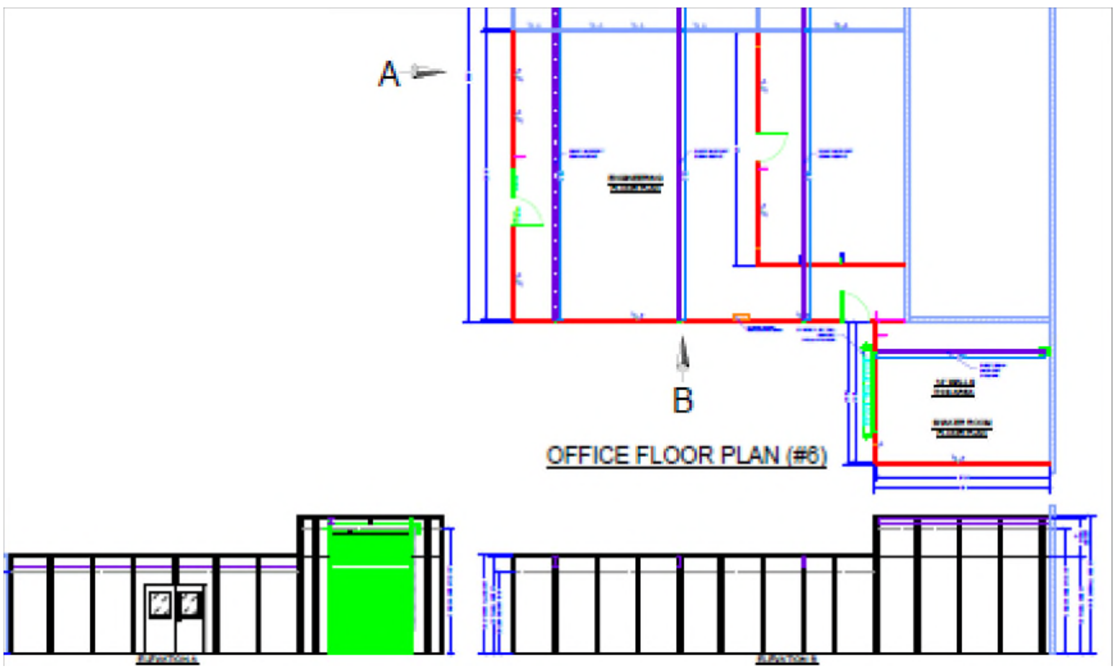
1. D + L
2. (1 + 0.105 Sds)D + 0.75L + 0.525ρQe)
3. (0.6 - 0 .14Sds)D + 0.7ρQe

DEFLECTION & DRIFT LIMITS

VERTICAL PER IBC TABLE 1604.3
HORIZONTAL SEISMIC PER ASCE TAB. 12.12-1

MATERIAL SPECIFICATIONS

STEEL ROOF DECK: 22 GA. "B" 1-1/2" DEPTH, Fy = 38 KSI. PER ESR-2078P
ALUMINUM: ALLOY 6063 - T6, Fb = 25.0 KSI
WALLS: SANDWICH PANELS
CONCRETE: f'c = 2500 PSI
NYLON NAIL ANCHORS: POWERS 1/4" x 1/2"
EXP. ANCHORS: ITW RED HEAD WEDGE PER ESR-2427



ROOF DECK **USE 1-1/2" x 22 GA. B-DECK**

$span := 12.75\cdot ft$ $spacing := 12\cdot in$ $Fy := 38\cdot ksi$ $E := 29500\cdot ksi$

PROPERTIES PER MANUFACTURER $Sx := 0.1757\cdot in^3$ $Ix := 0.1485\cdot in^4$

$M_a := \frac{Sx\cdot Fy}{1.67}$ $M_a = 333.16\,ft\cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND LOADS $w_g := (DL_r + LL_r)\cdot spacing$ $w_g = 9\,plf$ UNIF. GRAVITY LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := w_g\cdot\frac{span^2}{8}$ $M_r = 182.88\,ft\cdot lb$ $\frac{M_r}{M_a} = 0.55$ **OK < 1.0**

$\Delta := \frac{5\cdot w_g\cdot span^4}{384\cdot E\cdot Ix}$ $\Delta = 1.22\,in$ $\frac{span}{\Delta} = 125.25$ **OK > 120**

MODULAR WALL PANELS IN BEARING

USE 3" 3-PLY G/G PANELS

COMPOSITE PANEL, GYP.BD. FACING BOTH SIDES, POLYSTYRENE CORE.

FIND THE ALLOWABLE BEARING FOR PANEL BASED ON RACKING LOAD TEST PERFORMED BY TWIN CITY TESTING CORP. USE THE AVG. FAILURE LOAD WITH A SAFETY FACTOR OF FOUR. TEST PANEL LENGTH, 8 FT. HEIGHT 10 FT.

$SF := 4$ SAFETY FACTOR

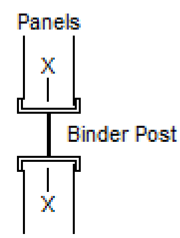
$F_{fail} := 3230 \cdot lb$ AVERAGE LATERAL LOAD PANELS FAILED, DEFLECTION = 2.2" JUST PRIOR TO FAILURE.

$P_{fail} := \dfrac{F_{fail} \cdot 10 \cdot ft}{8 \cdot ft}$ $P_{fail} = 4037.5\, lb$ RESULTANT AXIAL LOAD AT BINDER POST

$w_a := \dfrac{P_{fail}}{SF \cdot 4.25 \cdot ft}$ $w_a = 237.5\, plf$ ALLOWABLE UNIFORM LOAD/UNIT WIDTH

$w_g := \left(DL_r + LL_r\right) \cdot \dfrac{12.75 \cdot ft}{2}$ $w_g = 57.38\, plf$ UNIFORM LOAD $\dfrac{w_g}{w_a} = 0.24$ **OK < 1.0**

CHECK 5 PSF PARTITION LOAD BENDING MOMENT TO BINDER POST



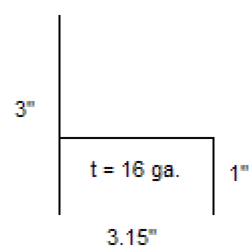
$Sx := 0.529 \cdot in^3$ $F_y := 25.0 \cdot ksi$

$Mr := 5 \cdot psf \cdot 51 \cdot in \cdot \dfrac{(14 \cdot ft)^2}{8}$ $Mr = 520.63\, ft \cdot lb$ REQUIRED MOMENT

$Ma := \dfrac{Sx \cdot F_y}{1.67}$ $Ma = 659.93\, ft \cdot lb$ ALLOWABLE MOMENT

CHECK BINDER POST FOR BENDING $\dfrac{Mr}{Ma} = 0.79$ **OK < 1.0**

USE 3" x 16 GA ALUMINUM SUPPORT HEADER



Section Top Track

$F_y := 25.0 \cdot ksi$ $E_a := 10000 \cdot ksi$

$span_h := 36 \cdot in$ HEADER SPAN OVER DOORS

PROPERTIES PER ANALYSIS $S_x := 0.1679 \cdot in^3$ $I_x := 0.1916 \cdot in^4$

$M_a := \dfrac{S_x \cdot F_y}{1.67}$ $M_a = 209.46\, ft \cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \dfrac{w_g \cdot span_h^{\,2}}{8}$ $M_r = 64.55\, ft \cdot lb$ $\dfrac{M_r}{M_a} = 0.31$ **OK < 1.0**

$\Delta := \dfrac{5 \cdot w_g \cdot span_h^{\,4}}{384 \cdot E_a \cdot I_x}$ $\Delta = 0.05\, in$ $\dfrac{span_h}{\Delta} = 659.64$ **OK > 180**

PRCTI20230167

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Mesquite, NV

ROOF SUPPORT BEAM, W 12 x 14 $span_b := 29.5\cdot ft$ BEAM SPAN

PROPERTIES $S_x := 14.9\cdot in^3$ $I_x := 88.6\cdot in^4$ $F_y := 50\cdot ksi$ $E = 29500\,ksi$

$M_a := \dfrac{S_x\cdot F_y}{1.67}$ $M_a = 37175.65\,ft\cdot lb$ ALLOWABLE MOMENT FOR BENDING STRESS

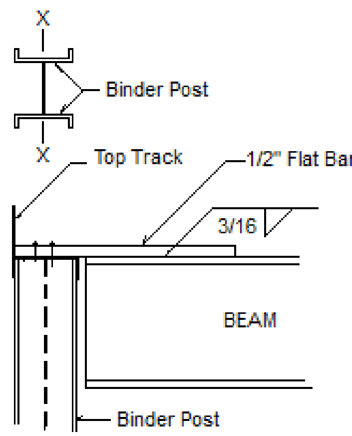
FIND LOADS $w_g := \left[\left(DL_r + LL_r\right)\cdot 12.75\cdot ft\right] + 14\cdot plf$ $w_g = 128.75\,plf$ UNIFORM LOAD

FIND REQUIRED MOMENT, STRESS RATIO AND DEFLECTION

$M_r := \dfrac{w_g\cdot span_b^{\,2}}{8}$ $M_r = 14005.59\,ft\cdot lb$ $\dfrac{M_r}{M_a} = 0.38$ **OK < 1.0**

$\Delta := \dfrac{5\cdot w_g\cdot span_b^{\,4}}{384\cdot E\cdot I_x}$ $\Delta = 0.84\,in$ $\dfrac{span_b}{\Delta} = 421.74$ **OK > 180**

USE BINDER POSTS AS SUPPORT COLUMNS (10 FT. HEIGHT)



$F_y := 25.0\cdot ksi$ $E_a := 10000\cdot ksi$ $Ht := 10\cdot ft$ $Trib := 51\cdot in$

PROPERTIES

$Sx := 0.529\cdot in^3$ $Ix := 0.792\cdot in^4$ $A := 0.947\cdot in^2$ $r_x := 0.914\cdot in$

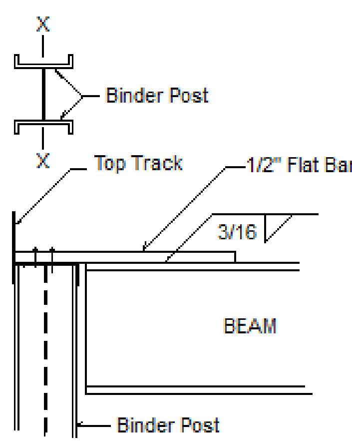
$F_e := \dfrac{\pi^2\cdot E_a}{\left(\dfrac{1.0\cdot Ht}{r_x}\right)^2}$ $F_e = 5725.71\,psi$ LESS THAN $.44\cdot F_y = 11000\,psi$

$F_{cr} := .877\cdot F_e$ $F_{cr} = 5021.45\,psi$ CRITICAL BUCKLING STRESS

$P_n := F_{cr}\cdot A$ $P_n = 4755.31\,lb$ $P_a := \dfrac{P_n}{1.92}$ $P_a = 2476.73\,lb$ ALLOWABLE AXIAL LOAD

$P_r := \dfrac{w_g\cdot span_b}{2}$ $P_r = 1899.06\,lb$ POINT LOAD $\dfrac{P_r}{P_a} = 0.77$ **OK < 1.0**

USE BINDER POSTS AS SUPPORT COLUMNS (14 FT. HEIGHT)



$F_y := 25.0\cdot ksi$ $E_a := 10000\cdot ksi$ $Ht := 14\cdot ft$ $Trib := 51\cdot in$

PROPERTIES

$Sx := 0.529\cdot in^3$ $Ix := 0.792\cdot in^4$ $A := 0.947\cdot in^2$ $r_x := 0.914\cdot in$

$F_e := \dfrac{\pi^2\cdot E_a}{\left(\dfrac{1.0\cdot Ht}{r_x}\right)^2}$ $F_e = 2921.28\,psi$ LESS THAN $.44\cdot F_y = 11000\,psi$

$F_{cr} := .877\cdot F_e$ $F_{cr} = 2561.96\,psi$ CRITICAL BUCKLING STRESS

$P_n := F_{cr}\cdot A$ $P_n = 2426.18\,lb$ $P_a := \dfrac{P_n}{1.92}$ $P_a = 1263.64\,lb$ ALLOWABLE AXIAL LOAD

$P_r := \dfrac{66\cdot plf\cdot 18\cdot ft}{2}$ $P_r = 594\,lb$ POINT LOAD $\dfrac{P_r}{P_a} = 0.47$ **OK < 1.0**

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LATERAL DESIGN (10 TALL AREA) FIND SEISMIC FORCE $Area := 1188 \cdot ft^2$

$DL_w := \frac{10 \cdot ft \cdot 4 \cdot psf \cdot (140 \cdot ft)}{2 \cdot Area}$ $DL_w = 2.36\,psf$ WALL DEAD LOAD PER SQ.-FT. FLOOR

$W_s := DL_r + DL_w$ $W_s = 11.36\,psf$ SEISMIC DEAD LOAD AT ROOF LEVEL

$F_s := W_s \cdot Area \cdot 0.7 \cdot P \cdot Q_e$ $F_s = 4752.42\,lb$ SEISMIC FORCE

USE THE MODULAR WALLS AS SHEAR WALL (CHECK LEFT SIDE WALL)

$v := \frac{F_s}{2 \cdot 23.5 \cdot ft}$ $v = 101.12\,plf$ SHEAR

FIND THE ALLOWABLE SHEAR FOR PANEL BASED ON TEST RESULTS.
SEE FOLLOWING TEST RESULTS. A SAFETY FACTOR OF 3 IS USED.

$F_{fail} := 3230 \cdot lb$ AVERAGE FORCE AT FAILURE $SF := 3$ SAFETY FACTOR

$v_a := \frac{F_{fail}}{SF \cdot 8 \cdot ft}$ $v_a = 134.58\,plf$ ALLOWABLE SHEAR CAPACITY $\frac{v}{v_a} = 0.75$ **OK < 1.0**

CHECK WALL ANCHORAGE FOR SHEAR, USE 1/2" x 2-1/2" EMB. WEDGE ANCHORS AT 48"

$V_n := 3206 \cdot lb$ ALLOW.SHEAR $\frac{v \cdot 48 \cdot in}{V_n} = 0.13$ **OK < 1.0** $T_n := 598 \cdot lb$ ALLOW.TENSION

CHECK OVERTURNING

$M_o := v \cdot 9.5 \cdot ft \cdot 10 \cdot ft$ $M_o = 9605.96\,ft \cdot lb$ OVERTURNING MOMENT

$M_{res} := \frac{(9.5 \cdot ft)^2}{2} \cdot \left(DL_r \cdot 2 \cdot ft + 40 \cdot plf + \frac{T_n}{48 \cdot in}\right)$ $M_{res} = 9363.44\,ft \cdot lb$ RESISTING MOMENT

$T_s := \frac{M_o - M_{res} \cdot (0.6 - .14 \cdot S_{ds})}{9.5 \cdot ft}$ $T_s = 558.65\,lb$ $\frac{T_s}{T_n} = 0.93$ **OK < 1.0**

THE FRONT, REAR WALLS ARE CONSIDERED ADEQUATE BY COMPARISON

THE EXISTING SHARED WALL IS CONSIDERED ADEQUATE TO RESIST SEISMIC SHEAR

LATERAL DESIGN (14 TALL AREA) FIND SEISMIC FORCE $Area := 260 \cdot ft^2$

$DL_w := \frac{14 \cdot ft \cdot 4 \cdot psf \cdot (64 \cdot ft)}{2 \cdot Area}$ $DL_w = 6.89\,psf$ WALL DEAD LOAD PER SQ.-FT. FLOOR

$W_s := DL_r + DL_w$ $W_s = 15.89\,psf$ SEISMIC DEAD LOAD AT ROOF LEVEL

$F_s := W_s \cdot Area \cdot 0.7 \cdot P \cdot Q_e$ $F_s = 1455.46\,lb$ SEISMIC FORCE

USE THE MODULAR WALLS AS SHEAR WALL (CHECK LEFT SIDE WALL)

$v := \frac{F_s}{2 \cdot 14.5 \cdot ft}$ $v = 50.19\,plf$ SHEAR $\frac{v}{v_a} = 0.37$ **OK < 1.0**

CHECK OVERTURNING

$M_o := v \cdot 14.66 \cdot ft \cdot 14 \cdot ft$ $M_o = 10300.61\,ft \cdot lb$ OVERTURNING MOMENT

$M_{res} := \frac{(14.66 \cdot ft)^2}{2} \cdot \left(DL_r \cdot 1 \cdot ft + 40 \cdot plf + \frac{T_n}{48 \cdot in}\right)$ $M_{res} = 21330.37\,ft \cdot lb$ RESISTING MOMENT

$T_s := \frac{M_o - M_{res} \cdot (0.6 - .14 \cdot S_{ds})}{14.66 \cdot ft}$ $T_s = 34.64\,lb$ $\frac{T_s}{T_n} = 0.06$ **OK < 1.0**

END DESIGN

PRCTI20230167

Allowable Load Capacities for Nylon Nailin in Normal-Weight Concrete[1,2,3]

Anchor Diameter d in. (mm)	Minimum Embedment Depth h_v in. (mm)	Minimum Concrete Compressive Strength (f'_c)					
		2,000 psi (13.8 MPa)		4,000 psi (27.6 MPa)		6,000 psi (41.4 MPa)	
		Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)	Tension lbs. (kN)	Shear lbs. (kN)
3/16 (4.8)	3/4 (19.1)	45 (0.2)	70 (0.3)	50 (0.2)	80 (0.4)	50 (0.2)	80 (0.4)
	1 (25.4)	50 (0.2)	70 (0.3)	55 (0.2)	80 (0.4)	60 (0.3)	80 (0.4)
1/4 (6.4)	5/8 (15.9)	30 (0.1)	80 (0.4)	35 (0.2)	125 (0.6)	45 (0.2)	125 (0.6)
	3/4 (19.1)	55 (0.2)	80 (0.4)	60 (0.3)	125 (0.6)	60 (0.3)	125 (0.6)
	1 (25.4)	60 (0.3)	80 (0.4)	65 (0.3)	125 (0.6)	65 (0.3)	125 (0.6)
	1 1/2 (38.1)	60 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	70 (0.3)	125 (0.6)
	2 (50.8)	65 (0.3)	80 (0.4)	70 (0.3)	125 (0.6)	75 (0.3)	125 (0.6)

1. Allowable load capacities listed are calculated using an applied safety factor of 4.0.
2. Linear interpolation may be used to determine allowable loads for intermediate embedments and compressive strengths.
3. Critical and minimum spacing and edge distances as well as reduction factors for intermediate spacing and edge distances are listed in the Design Criteria section.

TAKEN FROM POWERS "SPECIFICATION AND DESIGN MANUAL"

PROJECT NO: 031329 **DATE:** August 9, 2001
PAGE: 2

RACKING LOAD TEST OF AB/BA PANELS

TEST PROCEDURES: (cont.)

4. Load was applied according to ASTM:E72. At load increment, deflection readings were obtained. Residual deflection (set) readings were obtained after the loads specified in section 14.4.2 of ASTM:E72-95 were released. The samples were loaded until failure.

5. Total deflection was calculated according to section 14.3.5 of ASTM:E72.

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Racking Load Test of AB/BA Panels

Sample Number	Assembly Size	Core Thickness	Load (lbs.) at 1/4"	Deflection Prior to Failure (in)	Failure Load (lbs)
1A	8' x 10'	3"	878	3.146	3400
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Panel fabrication was conducted at Panel Built, Inc. in Blairsville, Georgia on July 23, 2001. See enclosed Panel Built drawing for panel configuration. Based on our review, the submitted samples are consistent with the drawings submitted by Panel Built, Inc.

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The adhesives used to laminate the core to the skins was a Neoprene Adhesive. The panel was then subjected to a combination of heat and pressure.

Each of the test assemblies consisted of two-four foot by ten foot AB/BA Panels connected together using two divider strips and one c-connector along the ten-foot length. A channel was attached across the bottom using #10 ¾" pan head screws spaced every six inches. An eave connector was attached across the top using the same screws spaced every twelve inches. The walls assemblies were fabricated by Twin City Testing Personal on August 7, 2001.

EXCERPT FROM RACKING LOAD TEST

Wharton and Associates
Mesquite, NV

ITW REDHEAD TRUBOLT+ WEDGE ANCHOR PER ESR-2427

1/2" DIA. x 2- 1/2" EMBEDMENT

FIND ALLOWABLE SHEAR AND TENSION

GIVEN:
ONE CARBON STEEL ANCHOR WITH NO CONCRETE THICKNESS OR EDGE DISTANCE CONCERNS.
NORMAL WEIGHT CONCRETE OF 2500 PSI COMPRESSIVE STRENGTH.
CONDITION "B", NO SUPPLEMENTARY REINFORCEMENT.
ASSUME CRACKED CONCRETE. 6" SLAB ON GRADE CONDITION..
ANCHOR IS CONSIDERED A DUCTILE STEEL ELEMENT.

VALUES PER ESR REPORT ARE AS FOLLOWS.

$h_a := 6 \cdot in$	SLAB ON GRADE THICKNESS REQUIRED	$f_c := 3000 \cdot psi$	CONCRETE STRENGTH REQUIRED
$C_{ac} := 6 \cdot in$	CRITICAL EDGE DISTANCE	$h_{ef} := 2.5 \cdot in$	EFFECTIVE EMBEDMENT
$d_a := .50 \cdot in$	ANCHOR DIAMETER	$l_e := 2.5 \cdot in$	BEARING LENGTH FOR ANCHOR
$k_{cr} := 17$	FACTOR, CRACKED CONCRETE		

FIND CONCRETE BREAKOUT STRENGTH IN TENSION

$$N_b := k_{cr} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lb \qquad N_b = 3680.61\,lb \qquad 0.65 \cdot N_b = 2392.4\,lb$$

$$A_{nc} := (2 \cdot 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) \qquad A_{nc} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE

$$A_{nco} := 9 \cdot h_{ef}^2 \qquad A_{nco} = 56.25\,in^2$$ PROJECTED AREA OF CONCRETE FAILURE, NO LIMITATIONS

$$N_{cb} := N_b \cdot \frac{A_{nc}}{A_{nco}} \qquad N_{cb} = 3680.61\,lb$$

FIND CONCRETE BREAKOUT STRENGTH IN SHEAR $\quad \Psi_{cv} := 1.0 \quad \psi_{ed} := 2$

$$A_{vc} := (2 \cdot 1.5 \cdot C_{ac}) \cdot h_a \qquad A_{vc} = 108\,in^2 \qquad A_{vco} := 4.5 \cdot C_{ac}^2 \qquad A_{vco} = 162\,in^2$$

$$V_b := 7 \cdot \left(\frac{l_e}{d_a}\right)^{.2} \cdot \sqrt{\frac{d_a}{in}} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{C_{ac}}{in}\right)^{1.5} \cdot lb \qquad V_b = 5497.49\,lb$$

$$V_{cb} := \frac{A_{vc}}{A_{vco}} \cdot \Psi_{cv} \cdot \psi_{ed} \cdot V_b \qquad V_{cb} = 7329.99\,lb$$

FIND FACTOR FOR ASD BASED ON 100% DEAD LOAD

$$\alpha := 1.2 \cdot 1 + 1.6 \cdot 0 \qquad \alpha = 1.2$$ CONVERSION FACTOR

$$T_{ension} := \frac{0.65 \cdot 0.75 \cdot 0.4 \cdot N_{cb}}{\alpha} \qquad S_{hear} := \frac{0.70 \cdot 0.75 \cdot V_{cb}}{\alpha}$$

$T_{ension} = 598.1\,lb$ ALLOWABLE TENSION $\qquad S_{hear} = 3206.87\,lb$ ALLOWABLE SHEAR