

STRUCTURAL CALCULATION REPORT

DESIGN CALCULATIONS FOR HEADERS AND EXPANDED FOOTINGS 84265 Edwards Rd, OR, US

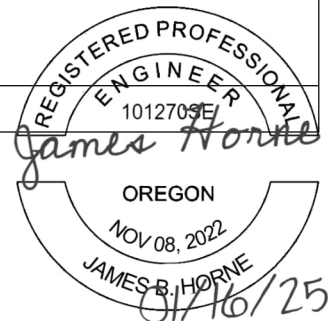
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Revision and Issue Records

Document No.	Rev.	Date	Remarks

Document Review and Approval

Contractor	Prepared by	Checked by	Approved by
Name			
Job Title			
Signature			
Date			



Revision	Title
00	DESIGN CALCULATIONS FOR HEADERS and Expanded Footings - 84265 Edwards Rd, OR, US.

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1. Executive Summary

This design report presents the sizing of headers, and expanded footings for the proposed second floor at 84265 Edwards Rd, OR, US.

2. Design Codes, Standards & Reference Documents

Design of structural engineering systems for the project is carried out in accordance with the regulations of below mentioned codes. The listed issue or edition of the design code or standard documents is applicable unless otherwise noted. The following codes and standards have been identified as applicable, in whole or in part, to structural engineering design and construction of this project:

- a) IBC 2015: International Building Code
- b) ASCE 7-16: Minimum Design Loads for buildings and other structures
- c) Oregon Residential Specialty Code (ORSC)

3. Materials Data

Structural Lumber is Douglas-Fir Larch

Douglas-Fir #1 U.N.O. for Posts

Douglas-Fir #1 U.N.O. for Beams

Douglas-Fir #2 U.N.O. for Studs

Machine Bolt = ASTM A307 Anchor Bolt = ASTM F1554

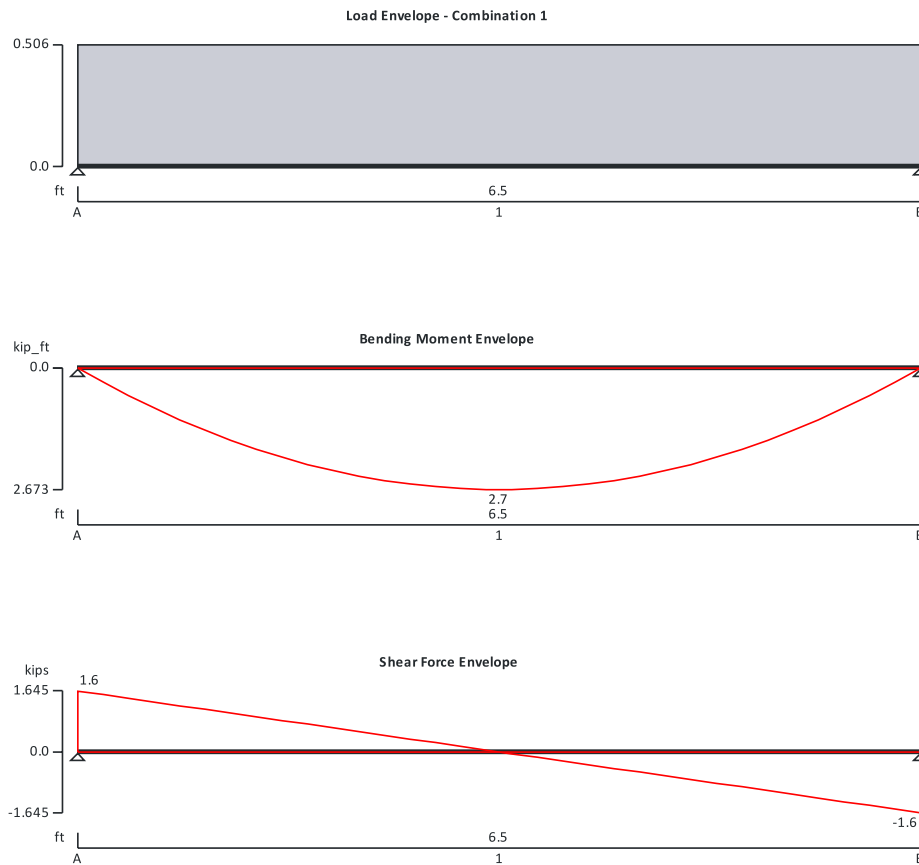
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00	DESIGN CALCULATIONS FOR HEADERS and Expanded Footings - 84265 Edwards Rd, OR, US.

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STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

Tedds calculation version 1.7.10



Applied loading

Beam loads

Dead self weight of beam $\times 1$
Dead full UDL 300 lb/ft
Live full UDL 200 lb/ft

Load combinations

Load combination 1

Support A	Dead $\times 1.00$ Live $\times 1.00$
Span 1	Dead $\times 1.00$ Live $\times 1.00$
Support B	Dead $\times 1.00$ Live $\times 1.00$

Analysis results

Maximum moment

$M_{\max} = 2673 \text{ lb}_\text{ft}$

$M_{\min} = 0 \text{ lb}_\text{ft}$

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Design moment

$$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2673 \text{ lb_ft}$$

Maximum shear

$$F_{\max} = 1645 \text{ lb} \quad F_{\min} = -1645 \text{ lb}$$

Design shear

$$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 1645 \text{ lb}$$

Total load on member

$$W_{\text{tot}} = 3290 \text{ lb}$$

Reaction at support A

$$R_{A_{\max}} = 1645 \text{ lb} \quad R_{A_{\min}} = 1645 \text{ lb}$$

Unfactored dead load reaction at support A

$$R_{A_{\text{Dead}}} = 995 \text{ lb}$$

Unfactored live load reaction at support A

$$R_{A_{\text{Live}}} = 650 \text{ lb}$$

Reaction at support B

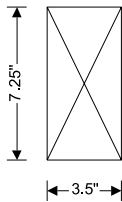
$$R_{B_{\max}} = 1645 \text{ lb} \quad R_{B_{\min}} = 1645 \text{ lb}$$

Unfactored dead load reaction at support B

$$R_{B_{\text{Dead}}} = 995 \text{ lb}$$

Unfactored live load reaction at support B

$$R_{B_{\text{Live}}} = 650 \text{ lb}$$



Sawn lumber section details

Nominal breadth of sections

$$b_{\text{nom}} = 4 \text{ in}$$

Dressed breadth of sections

$$b = 3.5 \text{ in}$$

Nominal depth of sections

$$d_{\text{nom}} = 8 \text{ in}$$

Dressed depth of sections

$$d = 7.25 \text{ in}$$

Number of sections in member

$$N = 1$$

Overall breadth of member

$$b_b = N \times b = 3.5 \text{ in}$$

Species, grade and size classification

Douglas Fir-Larch, No.1 grade, 2" & wider

Bending parallel to grain

$$F_b = 1000 \text{ lb/in}^2$$

Tension parallel to grain

$$F_t = 675 \text{ lb/in}^2$$

Compression parallel to grain

$$F_c = 1500 \text{ lb/in}^2$$

Compression perpendicular to grain

$$F_{c_{\text{perp}}} = 625 \text{ lb/in}^2$$

Shear parallel to grain

$$F_v = 180 \text{ lb/in}^2$$

Modulus of elasticity

$$E = 1700000 \text{ lb/in}^2$$

Modulus of elasticity, stability calculations

$$E_{\min} = 620000 \text{ lb/in}^2$$

Mean shear modulus

$$G_{\text{def}} = E / 16 = 106250 \text{ lb/in}^2$$

Member details

Service condition

Dry

Length of span

$$L_{s1} = 6.5 \text{ ft}$$

Length of bearing

$$L_b = 4 \text{ in}$$

Load duration

Ten years

Section properties

Cross sectional area of member

$$A = N \times b \times d = 25.37 \text{ in}^2$$

Section modulus

$$S_x = N \times b \times d^2 / 6 = 30.66 \text{ in}^3$$

$$S_y = d \times (N \times b)^2 / 6 = 14.80 \text{ in}^3$$

Second moment of area

$$I_x = N \times b \times d^3 / 12 = 111.15 \text{ in}^4$$

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$$I_y = d \times (N \times b)^3 / 12 = 25.90 \text{ in}^4$$

Adjustment factors

Load duration factor - Table 2.3.2	$C_D = 1.00$
Temperature factor - Table 2.3.3	$C_t = 1.00$
Size factor for bending - Table 4A	$C_{Fb} = 1.30$
Size factor for tension - Table 4A	$C_{Ft} = 1.20$
Size factor for compression - Table 4A	$C_{Fc} = 1.05$
Flat use factor - Table 4A	$C_{fu} = 1.05$
Incising factor for modulus of elasticity - Table 4.3.8	

$$C_{IE} = 1.00$$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$$C_i = 1.00$$

Incising factor for perpendicular compression - Table 4.3.8

$$C_{ic_perp} = 1.00$$

Repetitive member factor - cl.4.3.9

$$C_r = 1.00$$

Bearing area factor - cl.3.10.4

$$C_b = 1.00$$

Depth-to-breadth ratio

$$d_{nom} / (N \times b_{nom}) = 2.00$$

- Beam is fully restrained

Beam stability factor - cl.3.3.3

$$C_L = 1.00$$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c_perp}' = F_{c_perp} \times C_t \times C_{ic_perp} \times C_b = 625 \text{ lb/in}^2$

Applied compression stress perpendicular to grain $f_{c_perp} = R_{A_max} / (N \times b \times L_b) = 118 \text{ lb/in}^2$

$$f_{c_perp} / F_{c_perp}' = 0.188$$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress

$$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = 1300 \text{ lb/in}^2$$

Actual bending stress

$$f_b = M / S_x = 1046 \text{ lb/in}^2$$

$$f_b / F_b' = 0.805$$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress

$$F_v' = F_v \times C_D \times C_t \times C_i = 180 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2

$$f_v = 3 \times F / (2 \times A) = 97 \text{ lb/in}^2$$

$$f_v / F_v' = 0.540$$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection

$$E' = E \times C_{ME} \times C_t \times C_{IE} = 1700000 \text{ lb/in}^2$$

Design deflection

$$\delta_{adm} = 0.003 \times L_{s1} = 0.234 \text{ in}$$

Total deflection

$$\delta_{b_s1} = 0.108 \text{ in}$$

$$\delta_{b_s1} / \delta_{adm} = 0.460$$

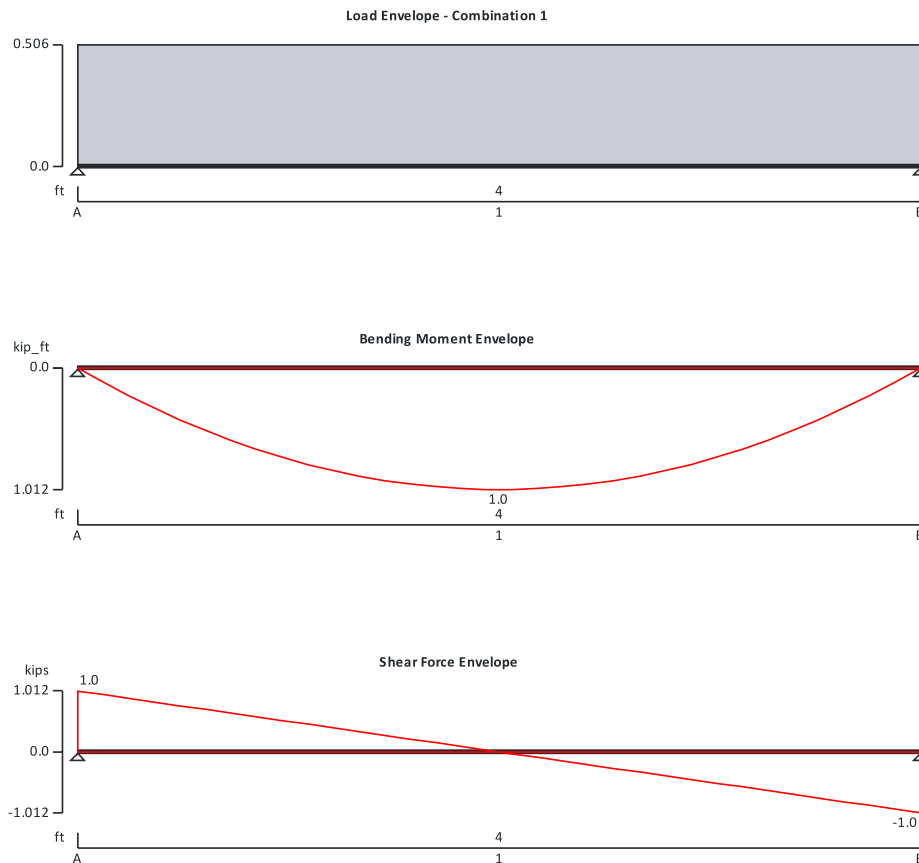
PASS - Total deflection is less than design deflection

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STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

Tedds calculation version 1.7.10



Applied loading

Beam loads

Dead self weight of beam $\times 1$
Dead full UDL 300 lb/ft
Live full UDL 200 lb/ft

Load combinations

Load combination 1

Support A	Dead $\times 1.00$ Live $\times 1.00$
Span 1	Dead $\times 1.00$ Live $\times 1.00$
Support B	Dead $\times 1.00$ Live $\times 1.00$

Analysis results

Maximum moment

$M_{\max} = 1012 \text{ lb_ft}$

$M_{\min} = 0 \text{ lb_ft}$

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Design moment

$$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 1012 \text{ lb_ft}$$

Maximum shear

$$F_{\max} = 1012 \text{ lb} \quad F_{\min} = -1012 \text{ lb}$$

Design shear

$$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 1012 \text{ lb}$$

Total load on member

$$W_{\text{tot}} = 2025 \text{ lb}$$

Reaction at support A

$$R_{A_{\max}} = 1012 \text{ lb} \quad R_{A_{\min}} = 1012 \text{ lb}$$

Unfactored dead load reaction at support A

$$R_{A_{\text{Dead}}} = 612 \text{ lb}$$

Unfactored live load reaction at support A

$$R_{A_{\text{Live}}} = 400 \text{ lb}$$

Reaction at support B

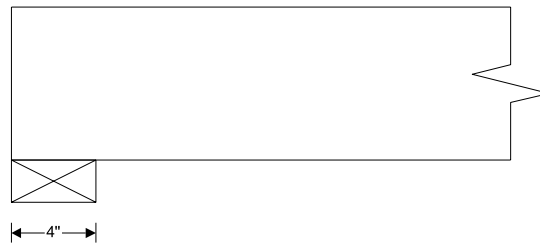
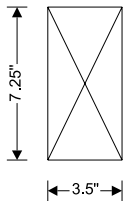
$$R_{B_{\max}} = 1012 \text{ lb} \quad R_{B_{\min}} = 1012 \text{ lb}$$

Unfactored dead load reaction at support B

$$R_{B_{\text{Dead}}} = 612 \text{ lb}$$

Unfactored live load reaction at support B

$$R_{B_{\text{Live}}} = 400 \text{ lb}$$



Sawn lumber section details

Nominal breadth of sections

$$b_{\text{nom}} = 4 \text{ in}$$

Dressed breadth of sections

$$b = 3.5 \text{ in}$$

Nominal depth of sections

$$d_{\text{nom}} = 8 \text{ in}$$

Dressed depth of sections

$$d = 7.25 \text{ in}$$

Number of sections in member

$$N = 1$$

Overall breadth of member

$$b_b = N \times b = 3.5 \text{ in}$$

Species, grade and size classification

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$$F_t = 675 \text{ lb/in}^2$$

Compression parallel to grain

$$F_c = 1500 \text{ lb/in}^2$$

Compression perpendicular to grain

$$F_{c_{\text{perp}}} = 625 \text{ lb/in}^2$$

Shear parallel to grain

$$F_v = 180 \text{ lb/in}^2$$

Modulus of elasticity

$$E = 1700000 \text{ lb/in}^2$$

Modulus of elasticity, stability calculations

$$E_{\min} = 620000 \text{ lb/in}^2$$

Mean shear modulus

$$G_{\text{def}} = E / 16 = 106250 \text{ lb/in}^2$$

Member details

Service condition

Dry

Length of span

$$L_{s1} = 4 \text{ ft}$$

Length of bearing

$$L_b = 4 \text{ in}$$

Load duration

Ten years

Section properties

Cross sectional area of member

$$A = N \times b \times d = 25.37 \text{ in}^2$$

Section modulus

$$S_x = N \times b \times d^2 / 6 = 30.66 \text{ in}^3$$

$$S_y = d \times (N \times b)^2 / 6 = 14.80 \text{ in}^3$$

Second moment of area

$$I_x = N \times b \times d^3 / 12 = 111.15 \text{ in}^4$$

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$$I_y = d \times (N \times b)^3 / 12 = 25.90 \text{ in}^4$$

Adjustment factors

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$$C_{IE} = 1.00$$

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$$C_i = 1.00$$

Incising factor for perpendicular compression - Table 4.3.8

$$C_{ic_perp} = 1.00$$

Repetitive member factor - cl.4.3.9

$$C_r = 1.00$$

Bearing area factor - cl.3.10.4

$$C_b = 1.00$$

Depth-to-breadth ratio

$$d_{nom} / (N \times b_{nom}) = 2.00$$

- Beam is fully restrained

Beam stability factor - cl.3.3.3

$$C_L = 1.00$$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c_perp}' = F_{c_perp} \times C_t \times C_{ic_perp} \times C_b = 625 \text{ lb/in}^2$

Applied compression stress perpendicular to grain $f_{c_perp} = R_{B_max} / (N \times b \times L_b) = 72 \text{ lb/in}^2$

$$f_{c_perp} / F_{c_perp}' = 0.116$$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress

$$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = 1300 \text{ lb/in}^2$$

Actual bending stress

$$f_b = M / S_x = 396 \text{ lb/in}^2$$

$$f_b / F_b' = 0.305$$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress

$$F_v' = F_v \times C_D \times C_t \times C_i = 180 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2

$$f_v = 3 \times F / (2 \times A) = 60 \text{ lb/in}^2$$

$$f_v / F_v' = 0.332$$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection

$$E' = E \times C_{ME} \times C_t \times C_{IE} = 1700000 \text{ lb/in}^2$$

Design deflection

$$\delta_{adm} = 0.003 \times L_{s1} = 0.144 \text{ in}$$

Total deflection

$$\delta_{b_s1} = 0.015 \text{ in}$$

$$\delta_{b_s1} / \delta_{adm} = 0.107$$

PASS - Total deflection is less than design deflection

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84265 EDWARDS RD, OR, US.

Footing analysis in accordance with ACI318-14

Tedds calculation version 3.3.08

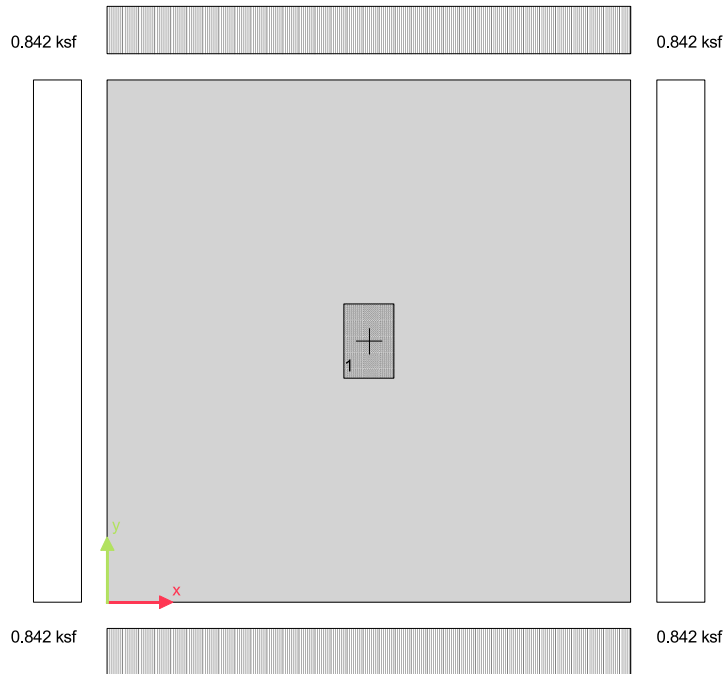
Summary results

Overall design status **PASS**
Overall design utilisation **0.281**

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	10.3			Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	0.842	3	0.281	Pass
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	3.3	23.3	0.141	Pass
Moment, positive, y-direction	kip_ft	3.0	21.5	0.137	Pass
Shear, one-way, x-direction	kips	2.8	21.3	0.131	Pass
Shear, one-way, y-direction	kips	2.6	19.7	0.130	Pass
Shear, two-way, Col 1	psi	28.246	150.000	0.188	Pass
Min.area of reinf, bot., x-direction	in ²	0.605	0.800		Pass
Max.reinf.spacing, bot, x-direction	in	16.0	13.1		Pass
Min.area of reinf, bot., y-direction	in ²	0.605	0.800		Pass
Max.reinf.spacing, bot, y-direction	in	16.0	13.1		Pass

Pad footing details

Length of footing **$L_x = 3.5$ ft**
Width of footing **$L_y = 3.5$ ft**
Footing area **$A = L_x \times L_y = 12.25$ ft²**
Depth of footing **$h = 8$ in**
Depth of soil over footing **$h_{soil} = 24$ in**
Density of concrete **$\gamma_{conc} = 150.0$ lb/ft³**



Column no.1 details

Length of column

$$l_{x1} = 4.00 \text{ in}$$

Width of column

$$l_{y1} = 6.00 \text{ in}$$

position in x-axis

$$x_1 = 21.00 \text{ in}$$

position in y-axis

$$y_1 = 21.00 \text{ in}$$

Soil properties

Gross allowable bearing pressure

$$Q_{\text{allow_Gross}} = 3 \text{ ksf}$$

Density of soil

$$\gamma_{\text{soil}} = 120.0 \text{ lb/ft}^3$$

Angle of internal friction

$$\phi_b = 30.0 \text{ deg}$$

Design base friction angle

$$\delta_{bb} = 30.0 \text{ deg}$$

Coefficient of base friction

$$\tan(\delta_{bb}) = 0.577$$

Footing loads

Self weight

$$F_{\text{swt}} = h \times \gamma_{\text{conc}} = 100 \text{ psf}$$

Soil weight

$$F_{\text{soil}} = h_{\text{soil}} \times \gamma_{\text{soil}} = 240 \text{ psf}$$

Column no.1 loads

Dead load in z

$$F_{Dz1} = 1.7 \text{ kips}$$

Live load in z

$$F_{Lz1} = 4.5 \text{ kips}$$

Snow load in z

$$F_{Sz1} = 3.4 \text{ kips}$$

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.158)

1.0D + 1.0L (0.281)

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Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis

$$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{10.3 \text{ kips}}$$

Moments on footing

Moment in x-axis, about x is 0

$$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{18.0 \text{ kip_ft}}$$

Moment in y-axis, about y is 0

$$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{18.0 \text{ kip_ft}}$$

Uplift verification

Vertical force

$$F_{dz} = \mathbf{10.312 \text{ kips}}$$

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.842 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.842 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.842 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.842 \text{ ksf}}$$

Minimum base pressure

$$q_{min} = \min(q_1, q_2, q_3, q_4) = \mathbf{0.842 \text{ ksf}}$$

Maximum base pressure

$$q_{max} = \max(q_1, q_2, q_3, q_4) = \mathbf{0.842 \text{ ksf}}$$

Allowable bearing capacity

Allowable bearing capacity

$$Q_{allow} = Q_{allow_Gross} = \mathbf{3 \text{ ksf}}$$

$$q_{max} / Q_{allow} = \mathbf{0.281}$$

PASS - Allowable bearing capacity exceeds design base pressure

84265 EDWARDS RD, OR, US.

Footing design in accordance with ACI318-14

Tedds calculation version 3.3.08

Material details

Compressive strength of concrete

$$f'_c = \mathbf{2500 \text{ psi}}$$

Yield strength of reinforcement

$$f_y = \mathbf{60000 \text{ psi}}$$

Compression-controlled strain limit (21.2.2)

$$\epsilon_{ty} = \mathbf{0.00200}$$

Cover to top of footing

$$c_{nom_t} = \mathbf{1 \text{ in}}$$

Cover to side of footing

$$c_{nom_s} = \mathbf{1 \text{ in}}$$

Cover to bottom of footing

$$c_{nom_b} = \mathbf{1 \text{ in}}$$

Concrete type

$$\text{Normal weight}$$

Concrete modification factor

$$\lambda = \mathbf{1.00}$$

Column type

$$\text{Concrete}$$

Analysis and design of concrete footing

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Load combinations per ASCE 7-16

1.4D (0.046)

1.2D + 1.6L + 0.5Lr (0.188)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis

$$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{14.2 \text{ kips}}$$

Moments on footing

Ultimate moment in x-axis, about x is 0

$$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{24.8 \text{ kip_ft}}$$

Ultimate moment in y-axis, about y is 0

$$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{24.8 \text{ kip_ft}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.157 \text{ ksf}}$$

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.157 \text{ ksf}}$$

$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.157 \text{ ksf}}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.157 \text{ ksf}}$$

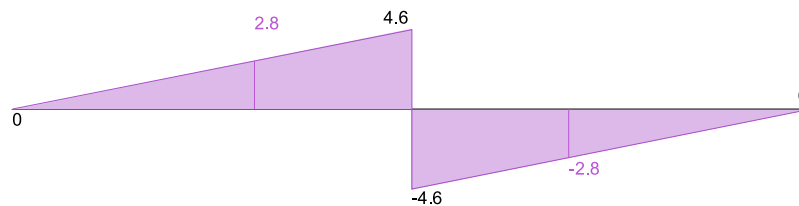
Minimum ultimate base pressure

$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.157 \text{ ksf}}$$

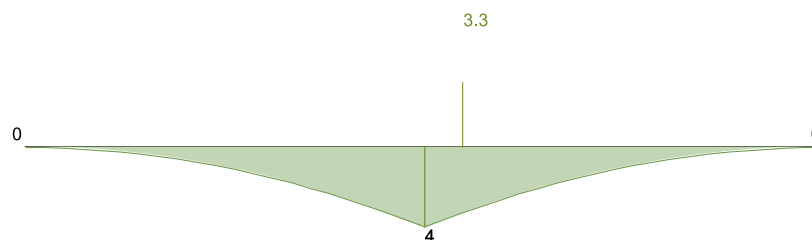
Maximum ultimate base pressure

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{1.157 \text{ ksf}}$$

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment

$$M_{u,x,max} = \mathbf{3.286 \text{ kip_ft}}$$

Tension reinforcement provided

$$\mathbf{4 \text{ No.4 bottom bars (13.1 in c/c)}}$$

Area of tension reinforcement provided

$$A_{sx,bot,prov} = \mathbf{0.8 \text{ in}^2}$$

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Minimum area of reinforcement (8.6.1.1)

$$A_{s,min} = 0.0018 \times L_y \times h = \mathbf{0.605 \text{ in}^2}$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)

$$s_{max} = \min(2 \times h, 18 \text{ in}) = \mathbf{16 \text{ in}}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement

$$d = h - c_{nom,b} - \phi_{x,bot} / 2 = \mathbf{6.750 \text{ in}}$$

Depth of compression block

$$a = A_{sx,bot,prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.538 \text{ in}}$$

Neutral axis factor

$$\beta_1 = \mathbf{0.85}$$

Depth to neutral axis

$$c = a / \beta_1 = \mathbf{0.633 \text{ in}}$$

Strain in tensile reinforcement

$$\epsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.02900}$$

Minimum tensile strain(8.3.3.1)

$$\epsilon_{min} = 0.004 = \mathbf{0.00400}$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity

$$M_n = A_{sx,bot,prov} \times f_y \times (d - a / 2) = \mathbf{25.924 \text{ kip_ft}}$$

Flexural strength reduction factor

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$$

Design moment capacity

$$\phi M_n = \phi_f \times M_n = \mathbf{23.332 \text{ kip_ft}}$$

$$M_{u,x,max} / \phi M_n = \mathbf{0.141}$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force

$$V_{u,x} = \mathbf{2.786 \text{ kips}}$$

Depth to reinforcement

$$d_v = h - c_{nom,b} - \phi_{x,bot} / 2 = \mathbf{6.75 \text{ in}}$$

Shear strength reduction factor

$$\phi_v = \mathbf{0.75}$$

Nominal shear capacity (Eq. 22.5.5.1)

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = \mathbf{28.35 \text{ kips}}$$

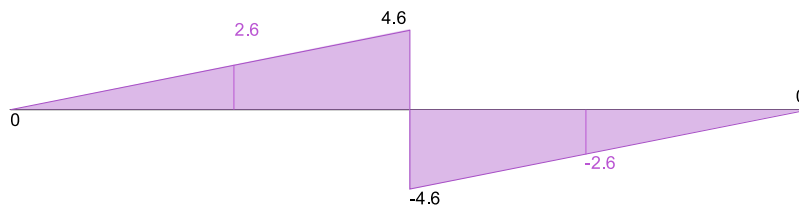
Design shear capacity

$$\phi V_n = \phi_v \times V_n = \mathbf{21.263 \text{ kips}}$$

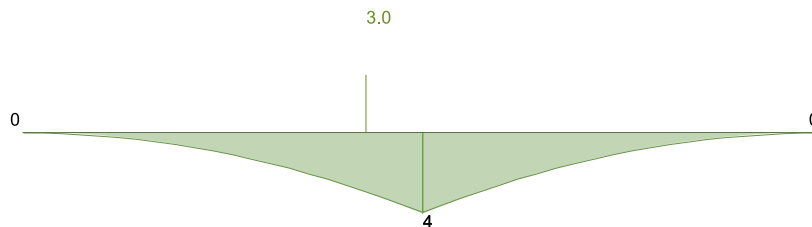
$$V_{u,x} / \phi V_n = \mathbf{0.131}$$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment

$$M_{u,y,max} = \mathbf{2.952 \text{ kip_ft}}$$

Tension reinforcement provided

$$\mathbf{4 \text{ No.4 bottom bars (13.1 in c/c)}}$$

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Area of tension reinforcement provided
Minimum area of reinforcement (8.6.1.1)

$$A_{sy,bot,prov} = 0.8 \text{ in}^2$$

$$A_{s,min} = 0.0018 \times L_x \times h = 0.605 \text{ in}^2$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 16 \text{ in}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement

$$d = h - c_{nom,b} - \phi_{x,bot} - \phi_{y,bot} / 2 = 6.250 \text{ in}$$

Depth of compression block

$$a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.538 \text{ in}$$

Neutral axis factor

$$\beta_1 = 0.85$$

Depth to neutral axis

$$c = a / \beta_1 = 0.633 \text{ in}$$

Strain in tensile reinforcement

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.02663$$

Minimum tensile strain(8.3.3.1)

$$\epsilon_{min} = 0.004 = 0.00400$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity

$$M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = 23.924 \text{ kip_ft}$$

Flexural strength reduction factor

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$$

Design moment capacity

$$\phi M_n = \phi_f \times M_n = 21.532 \text{ kip_ft}$$

$$M_{u,y,max} / \phi M_n = 0.137$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force

$$V_{u,y} = 2.569 \text{ kips}$$

Depth to reinforcement

$$d_v = h - c_{nom,b} - \phi_{x,bot} - \phi_{y,bot} / 2 = 6.25 \text{ in}$$

Shear strength reduction factor

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1)

$$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v = 26.25 \text{ kips}$$

Design shear capacity

$$\phi V_n = \phi_v \times V_n = 19.688 \text{ kips}$$

$$V_{u,y} / \phi V_n = 0.130$$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement

$$d_{v2} = 6.5 \text{ in}$$

Shear perimeter length (22.6.4)

$$l_{xp} = 10.500 \text{ in}$$

Shear perimeter width (22.6.4)

$$l_{yp} = 12.500 \text{ in}$$

Shear perimeter (22.6.4)

$$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = 46.000 \text{ in}$$

Shear area

$$A_p = l_{x,perim} \times l_{y,perim} = 131.250 \text{ in}^2$$

Surcharge loaded area

$$A_{sur} = A_p - l_{x1} \times l_{y1} = 107.250 \text{ in}^2$$

Ultimate bearing pressure at center of shear area

$$q_{up,avg} = 1.157 \text{ ksf}$$

Ultimate shear load

$$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up,avg} \times A_p = 8.446 \text{ kips}$$

Ultimate shear stress from vertical load

$$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = 28.246 \text{ psi}$$

Column geometry factor (Table 22.6.5.2)

$$\beta = l_{y1} / l_{x1} = 1.50$$

Column location factor (22.6.5.3)

$$\alpha_s = 40$$

Concrete shear strength (22.6.5.2)

$$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 233.333 \text{ psi}$$

$$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 382.609 \text{ psi}$$

$$v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 200.000 \text{ psi}$$

$$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = 200.000 \text{ psi}$$

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Shear strength reduction factor

$$\phi_v = 0.75$$

Nominal shear stress capacity (Eq. 22.6.1.2)

$$v_n = v_{cp} = 200.000 \text{ psi}$$

Design shear stress capacity (8.5.1.1(d))

$$\phi v_n = \phi_v \times v_n = 150.000 \text{ psi}$$

$$v_{ug} / \phi v_n = 0.188$$

PASS - Design shear stress capacity exceeds ultimate shear stress load

