# STRUCTURAL CALCULATION REPORT

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

| DOCUMENT NO: | REVISION: 00 |
|---|---|

### Revision and Issue Records

| Document No. | Rev. | Date | Remarks |
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Document Review and Approval

| Contractor | Prepared by | Checked by | Approved by |
|---|---|---|---|
| **Name** | | | |
| **Job Title** | | | |
| **Signature** | | | |
| **Date** | | | |

## Contents

## 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

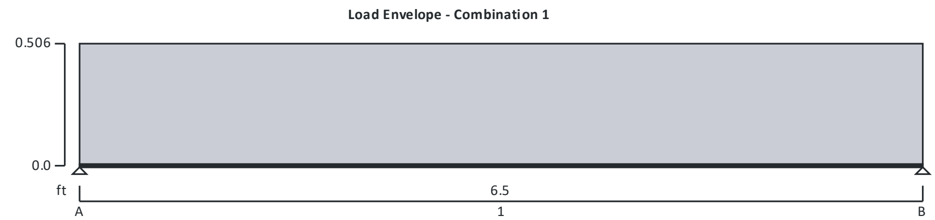
Douglas-Fir #2 U.N.O. for Studs

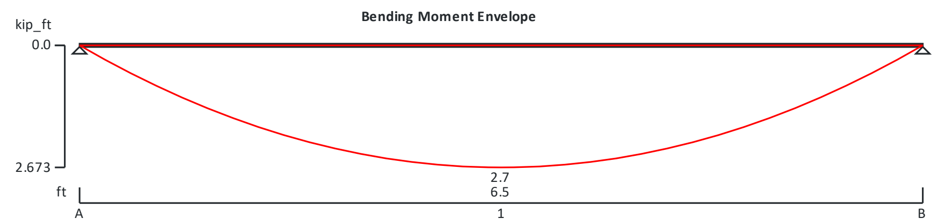
Machine Bolt = ASTM A307 Anchor Bolt = ASTM F1554

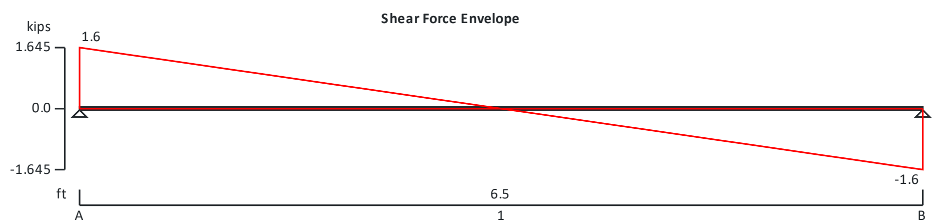
## STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

**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 × 1

Dead full UDL 300 lb/ft

Live full UDL 200 lb/ft

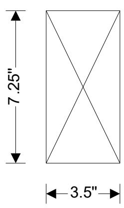
### Load combinations

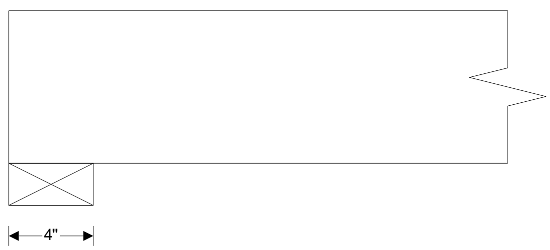
| | | |
|---|---|---|
| Load combination 1 | Support A | Dead × 1.00 |
| | | Live × 1.00 |
| | Span 1 | Dead × 1.00 |
| | | Live × 1.00 |
| | Support B | Dead × 1.00 |
| | | Live × 1.00 |

### Analysis results

| | | |
|---|---|---|
| Maximum moment | $M_{max}$ = **2673** lb_ft | $M_{min}$ = **0** lb_ft |

| | |
|---|---|
| Design moment | $M = max(abs(M_{max}),abs(M_{min}))$ = **2673** lb_ft |
| Maximum shear | $F_{max}$ = **1645** lb    $F_{min}$ = **-1645** lb |
| Design shear | $F = max(abs(F_{max}),abs(F_{min}))$ = **1645** lb |
| Total load on member | $W_{tot}$ = **3290** lb |
| Reaction at support A | $R_{A\_max}$ = **1645** lb    $R_{A\_min}$ = **1645** lb |
| Unfactored dead load reaction at support A | $R_{A\_Dead}$ = **995** lb |
| Unfactored live load reaction at support A | $R_{A\_Live}$ = **650** lb |
| Reaction at support B | $R_{B\_max}$ = **1645** lb    $R_{B\_min}$ = **1645** lb |
| Unfactored dead load reaction at support B | $R_{B\_Dead}$ = **995** lb |
| Unfactored live load reaction at support B | $R_{B\_Live}$ = **650** lb |




**Sawn lumber section details**

| | |
|---|---|
| Nominal breadth of sections | $b_{nom}$ = **4** in |
| Dressed breadth of sections | $b$ = **3.5** in |
| Nominal depth of sections | $d_{nom}$ = **8** in |
| Dressed depth of sections | $d$ = **7.25** in |
| Number of sections in member | $N$ = **1** |
| Overall breadth of member | $b_b = N \times b$ = **3.5** in |
| Species, grade and size classification | Douglas Fir-Larch, No.1 grade, 2" & wider |
| Bending parallel to grain | $F_b$ = **1000** lb/in$^2$ |
| Tension parallel to grain | $F_t$ = **675** lb/in$^2$ |
| Compression parallel to grain | $F_c$ = **1500** lb/in$^2$ |
| Compression perpendicular to grain | $F_{c\_perp}$ = **625** lb/in$^2$ |
| Shear parallel to grain | $F_v$ = **180** lb/in$^2$ |
| Modulus of elasticity | $E$ = **1700000** lb/in$^2$ |
| Modulus of elasticity, stability calculations | $E_{min}$ = **620000** lb/in$^2$ |
| Mean shear modulus | $G_{def} = E / 16$ = **106250** lb/in$^2$ |

**Member details**

| | |
|---|---|
| Service condition | **Dry** |
| Length of span | $L_{s1}$ = **6.5** ft |
| Length of bearing | $L_b$ = **4** in |
| Load duration | **Ten years** |

**Section properties**

| | |
|---|---|
| Cross sectional area of member | $A = N \times b \times d$ = **25.37** in$^2$ |
| Section modulus | $S_x = N \times b \times d^2 / 6$ = **30.66** in$^3$ |
| | $S_y = d \times (N \times b)^2 / 6$ = **14.80** in$^3$ |
| Second moment of area | $I_x = N \times b \times d^3 / 12$ = **111.15** in$^4$ |

$I_y = d \times (N \times b)^3 / 12 =$ **25.90** 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** lb/in$^2$

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) =$ **118** 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** lb/in$^2$

Actual bending stress $f_b = M / S_x =$ **1046** 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** lb/in$^2$

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) =$ **97** 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** lb/in$^2$

Design deflection $\delta_{adm} = 0.003 \times L_{s1} =$ **0.234** in

Total deflection $\delta_{b\_s1} =$ **0.108** in

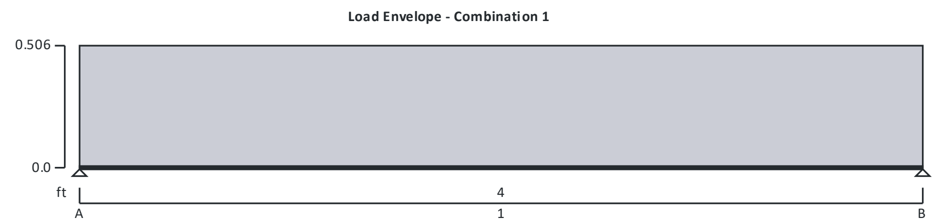
$\delta_{b\_s1} / \delta_{adm} =$ **0.460**

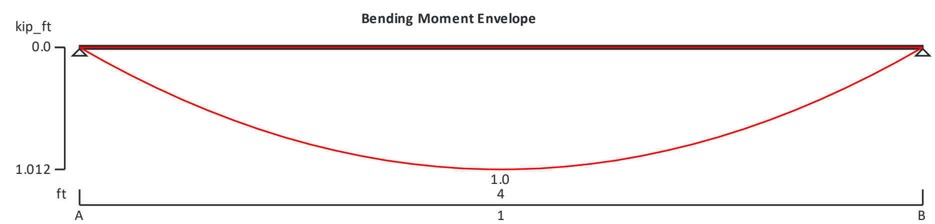
***PASS - Total deflection is less than design deflection***

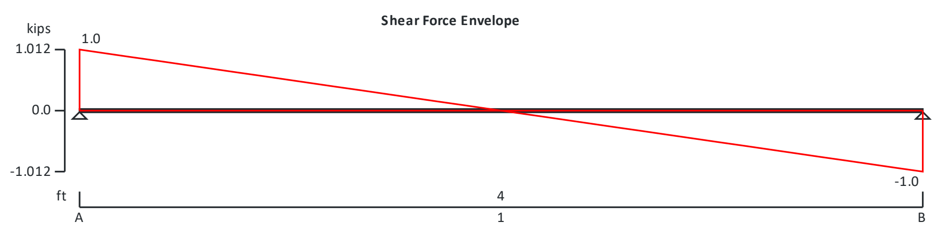
## STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

**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 × 1

Dead full UDL 300 lb/ft

Live full UDL 200 lb/ft

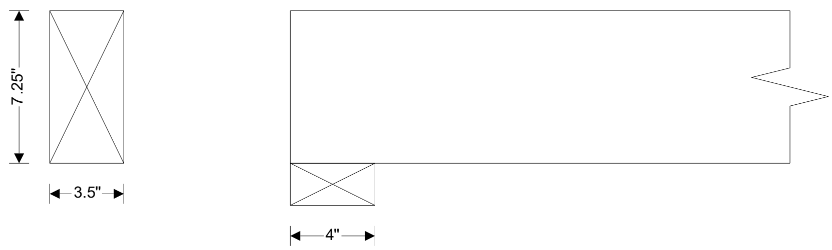
**Load combinations**

| | | |
|---|---|---|
| Load combination 1 | Support A | Dead × 1.00 |
| | | Live × 1.00 |
| | Span 1 | Dead × 1.00 |
| | | Live × 1.00 |
| | Support B | Dead × 1.00 |
| | | Live × 1.00 |

**Analysis results**

Maximum moment $M_{max}$ = **1012** lb_ft $M_{min}$ = **0** lb_ft

| | | |
|---|---|---|
| Design moment | $M = \max(\mathrm{abs}(M_{max}),\mathrm{abs}(M_{min}))$ = **1012** lb_ft | |
| Maximum shear | $F_{max}$ = **1012** lb | $F_{min}$ = **-1012** lb |
| Design shear | $F = \max(\mathrm{abs}(F_{max}),\mathrm{abs}(F_{min}))$ = **1012** lb | |
| Total load on member | $W_{tot}$ = **2025** lb | |
| Reaction at support A | $R_{A\_max}$ = **1012** lb | $R_{A\_min}$ = **1012** lb |
| Unfactored dead load reaction at support A | $R_{A\_Dead}$ = **612** lb | |
| Unfactored live load reaction at support A | $R_{A\_Live}$ = **400** lb | |
| Reaction at support B | $R_{B\_max}$ = **1012** lb | $R_{B\_min}$ = **1012** lb |
| Unfactored dead load reaction at support B | $R_{B\_Dead}$ = **612** lb | |
| Unfactored live load reaction at support B | $R_{B\_Live}$ = **400** lb | |




**Sawn lumber section details**

| | |
|---|---|
| Nominal breadth of sections | $b_{nom}$ = **4** in |
| Dressed breadth of sections | $b$ = **3.5** in |
| Nominal depth of sections | $d_{nom}$ = **8** in |
| Dressed depth of sections | $d$ = **7.25** in |
| Number of sections in member | $N$ = **1** |
| Overall breadth of member | $b_b = N \times b$ = **3.5** in |
| Species, grade and size classification | Douglas Fir-Larch, No.1 grade, 2" & wider |
| Bending parallel to grain | $F_b$ = **1000** lb/in$^2$ |
| Tension parallel to grain | $F_t$ = **675** lb/in$^2$ |
| Compression parallel to grain | $F_c$ = **1500** lb/in$^2$ |
| Compression perpendicular to grain | $F_{c\_perp}$ = **625** lb/in$^2$ |
| Shear parallel to grain | $F_v$ = **180** lb/in$^2$ |
| Modulus of elasticity | $E$ = **1700000** lb/in$^2$ |
| Modulus of elasticity, stability calculations | $E_{min}$ = **620000** lb/in$^2$ |
| Mean shear modulus | $G_{def} = E / 16$ = **106250** lb/in$^2$ |

**Member details**

| | |
|---|---|
| Service condition | **Dry** |
| Length of span | $L_{s1}$ = **4** ft |
| Length of bearing | $L_b$ = **4** in |
| Load duration | **Ten years** |

**Section properties**

| | |
|---|---|
| Cross sectional area of member | $A = N \times b \times d$ = **25.37** in$^2$ |
| Section modulus | $S_x = N \times b \times d^2 / 6$ = **30.66** in$^3$ |
| | $S_y = d \times (N \times b)^2 / 6$ = **14.80** in$^3$ |
| Second moment of area | $I_x = N \times b \times d^3 / 12$ = **111.15** in$^4$ |

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

**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**

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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** $lb/in^2$

Applied compression stress perpendicular to grain — $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b) =$ **72** $lb/in^2$

$f_{c\_perp} / F_{c\_perp}' =$ **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** $lb/in^2$

Actual bending stress — $f_b = M / S_x =$ **396** $lb/in^2$

$f_b / F_b' =$ **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** $lb/in^2$

Actual shear stress - eq.3.4-2 — $f_v = 3 \times F / (2 \times A) =$ **60** $lb/in^2$

$f_v / F_v' =$ **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** $lb/in^2$

Design deflection — $\delta_{adm} = 0.003 \times L_{s1} =$ **0.144** in

Total deflection — $\delta_{b\_s1} =$ **0.015** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.107**

***PASS - Total deflection is less than design deflection***

## 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

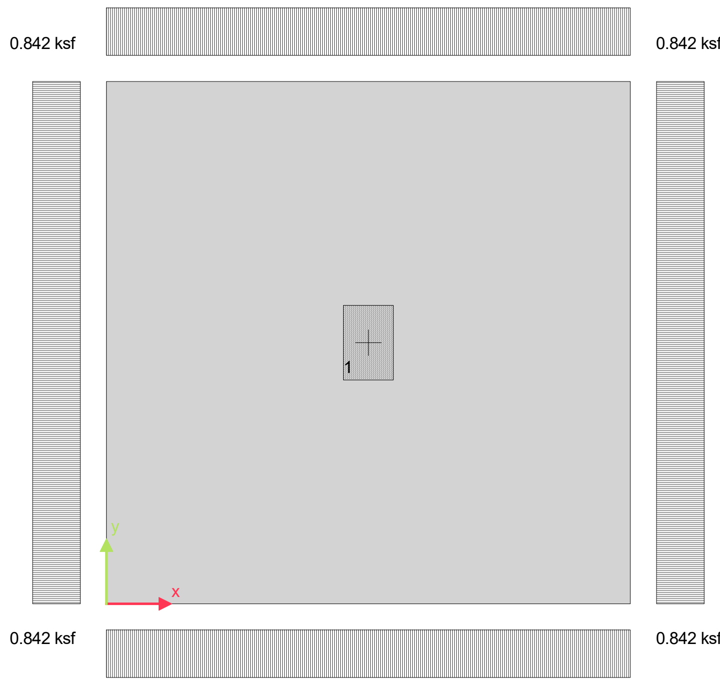
Width of footing $L_y$ = **3.5** ft

Footing area $A = L_x \times L_y$ = **12.25** $ft^2$

Depth of footing h = **8** in

Depth of soil over footing $h_{soil}$ = **24** in

Density of concrete $\gamma_{conc}$ = **150.0** $lb/ft^3$




**Column no.1 details**

Length of column $l_{x1}$ = **4.00** in

Width of column $l_{y1}$ = **6.00** in

position in x-axis $x_1$ = **21.00** in

position in y-axis $y_1$ = **21.00** in

**Soil properties**

Gross allowable bearing pressure $q_{allow\_Gross}$ = **3** ksf

Density of soil $\gamma_{soil}$ = **120.0** lb/ft$^3$

Angle of internal friction $\phi_b$ = **30.0** deg

Design base friction angle $\delta_{bb}$ = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

**Footing loads**

Self weight $F_{swt} = h \times \gamma_{conc}$ = **100** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **240** psf

**Column no.1 loads**

Dead load in z $F_{Dz1}$ = **1.7** kips

Live load in z $F_{Lz1}$ = **4.5** kips

Snow load in z $F_{Sz1}$ = **3.4** kips

**Footing analysis for soil and stability**

**Load combinations per ASCE 7-16**

1.0D (0.158)

1.0D + 1.0L (0.281)

**Combination 2 results: 1.0D + 1.0L**

**Forces on footing**

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{10.3}$ kips

**Moments on footing**

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{18.0}$ kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{18.0}$ kip_ft

**Uplift verification**

Vertical force — $F_{dz} = \mathbf{10.312}$ kips

***PASS - Footing is not subject to uplift***

**Bearing resistance**

**Eccentricity of base reaction**

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0}$ in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

**Pad base pressures**

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

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

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

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

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{0.842}$ ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{0.842}$ ksf

**Allowable bearing capacity**

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} = \mathbf{3}$ ksf

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

***PASS - Allowable bearing capacity exceeds design base pressure***

**<u>84265 EDWARDS RD, OR, US.</u>**

**Footing design in accordance with ACI318-14**

Tedds calculation version 3.3.08

**Material details**

| | |
|---|---|
| Compressive strength of concrete | $f'_c = \mathbf{2500}$ psi |
| Yield strength of reinforcement | $f_y = \mathbf{60000}$ psi |
| Compression-controlled strain limit (21.2.2) | $\varepsilon_{ty} = \mathbf{0.00200}$ |
| Cover to top of footing | $c_{nom\_t} = \mathbf{1}$ in |
| Cover to side of footing | $c_{nom\_s} = \mathbf{1}$ in |
| Cover to bottom of footing | $c_{nom\_b} = \mathbf{1}$ in |
| Concrete type | Normal weight |
| Concrete modification factor | $\lambda = \mathbf{1.00}$ |
| Column type | Concrete |

**Analysis and design of concrete footing**

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

**Moments on footing**

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **24.8** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **24.8** kip_ft

**Eccentricity of base reaction**

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

**Pad base pressures**

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

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

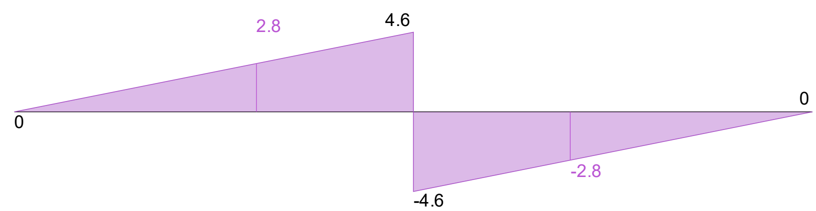
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.157** ksf

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

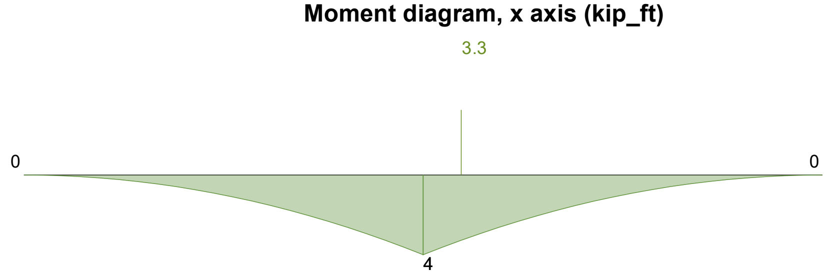
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.157** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.157** ksf

**Shear diagram, x axis (kips)**




**Moment diagram, x axis (kip_ft)**




**Moment design, x direction, positive moment**

Ultimate bending moment $M_{u.x.max}$ = **3.286** kip_ft

Tension reinforcement provided 4 No.4 bottom bars (13.1 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **0.8** $in^2$

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h =$ **0.605** 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** in

***PASS - Maximum permissible reinforcement spacing exceeds actual spacing***

Depth to tension reinforcement $d = h - c_{nom\_b} - \phi_{x.bot} / 2 =$ **6.750** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.538** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **0.633** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.02900**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004 =$ **0.00400**

***PASS - Tensile strain exceeds minimum required***

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **25.924** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **23.332** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.141**

***PASS - Design moment capacity exceeds ultimate moment load***

**One-way shear design, x direction**

Ultimate shear force $V_{u.x} =$ **2.786** kips

Depth to reinforcement $d_v = h - c_{nom\_b} - \phi_{x.bot} / 2 =$ **6.75** in

Shear strength reduction factor $\phi_v =$ **0.75**

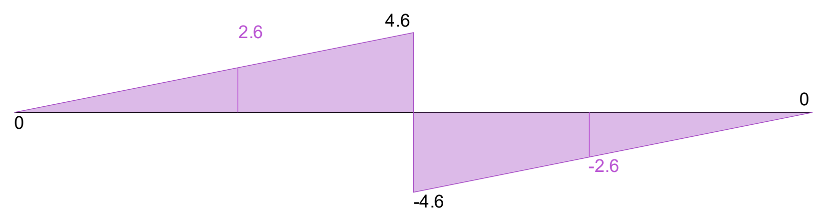
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v =$ **28.35** kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **21.263** kips

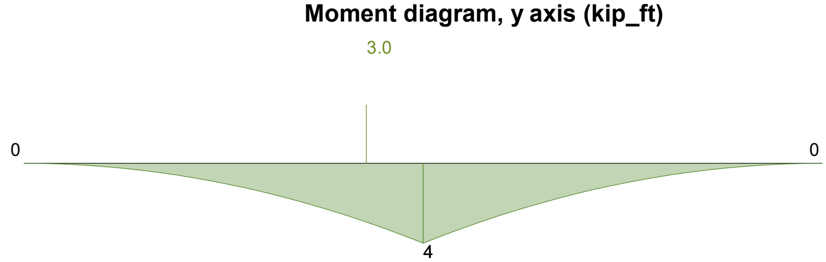
$V_{u.x} / \phi V_n =$ **0.131**

***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 $M_{u.y.max} =$ **2.952** kip_ft

Tension reinforcement provided 4 No.4 bottom bars (13.1 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **0.8** in$^2$

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **0.605** in$^2$

***PASS - Area of reinforcement provided exceeds minimum***

Maximum spacing of reinforcement (8.7.2.2) $s_{max}$ = min(2 × h, 18 in) = **16** 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** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.538** in

Neutral axis factor $\beta_1$ = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.633** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02663**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004$ = **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** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **21.532** 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** kips

Depth to reinforcement $d_v = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **6.25** 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** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **19.688** 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** in

Shear perimeter length (22.6.4) $l_{xp}$ = **10.500** in

Shear perimeter width (22.6.4) $l_{yp}$ = **12.500** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **46.000** in

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

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **107.250** in$^2$

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **1.157** 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** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi})$ = **28.246** 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** psi

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

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

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

Shear strength reduction factor $\phi_v$ = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **200.000** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v \times v_n$ = **150.000** psi

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

***PASS - Design shear stress capacity exceeds ultimate shear stress load***