JOHNSON
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DESIGN CALCULATIONS FOR GRUBER LIVESTOCK SOUTH GDU BARN

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<u>FOOTING ANALYSIS</u>

In accordance with ACI318: See attachment for additional ACI-350 Reinforcement Requirements if they differ from those in ACI-318

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.957

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	2.9			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.435	1.5	0.957	Pass

Strip footing details - considering a one foot strip

Length of footing; L_x = **1** ft

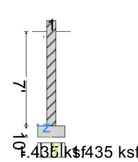
Width of footing; L_y = **2** ft

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

Depth of footing; h = **10** in

Depth of soil over footing; h_{soil} = **84** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall; l_{y1} = **8** in

position in y-axis; y_1 = **12** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **1.5** ksf;

Density of soil; γ_{soil} = **115.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

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

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **125** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **805** psf

Wall no.1 loads per linear foot

Dead load in z; F_{Dz1} = **0.8** kips

Live load in z; F_{Lz1} = **0.3** kips

Live roof load in z; F_{Lrz1} = **0.5** kips

Snow load in z; F_{Sz1} = **0.5** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.708)
1.0D + 1.0L (0.791)
1.0D + 1.0Lr (0.874)
1.0D + 1.0S (0.862)
1.0D + 1.0R (0.708)
1.0D + 0.75L + 0.75Lr (0.895)
1.0D + 0.75L + 0.75S (0.886)
1.0D + 0.75L + 0.75R (0.770)
1.0D + 0.6W (0.708)
$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.807)
1.0D + 0.75L + 0.75Lr + 0.45W (0.895)
1.0D + 0.75L + 0.75S + 0.45W (0.886)
1.0D + 0.75L + 0.75R + 0.45W (0.770)
$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.957)
0.6D + 0.6W (0.425)
$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.326)

Combination 14 results: $(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E

Forces on footing per linear foot

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **2.9** kips

Moments on footing per linear foot

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **2.9** kip_ft

Uplift verification

Vertical force; F_{dz} = **2.87** kips

PASS - Footing is not subject to uplift

Stability against sliding

Resistance due to base friction; $F_{RFriction} = \max(F_{dz}, 0\ kN) \times \tan(\delta_{bb}) =$ **1.657** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0.000** in

Strip base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1\ ft) =$ **1.435** ksf

$q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1\ ft) =$ **1.435** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2) =$ **1.435** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2) =$ **1.435** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} =$ **1.5** ksf

$q_{max} / q_{allow} =$ **0.957**

PASS - Allowable bearing capacity exceeds design base pressure

RC WALL DESIGN

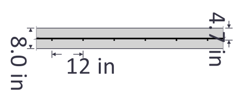
In accordance with ACI 318-14: See attachment for additional ACI-350 Reinforcement Requirements if they differ from those in ACI-318

Tedds calculation version 1.2.08

Design summary

Overall design status; PASS

Description	Unit	Required	Provided	Utilization	Result
Axial	kips/ft	1.9	3.9	0.480	PASS
Euler load	kips/ft	1.9	104.0	0.018	PASS
Moment	kip_ft/ft	3.9	7.3	0.534	PASS
Shear	kips/ft	2.6	5.2	0.496	PASS



Geometry of wall

Depth of wall; h = **8.00** in

Clear cover to reinforcement (both sides); c_c = **3.00** in

Unsupported height of wall; l_u = **96.0** in

Effective height factor; k = **2.00**

Reinforcement of wall

Numbers of reinforcement layers; N_l = **1**

Vertical steel bar diameter number; D_{ver_num} = **5**

Spacing of vertical steel; s_v = **12.00** in

Diameter of vertical steel bar; D_{ver} = **0.625** in

Horizontal steel bar diameter number; D_{hor_num} = **5**

Spacing of horizontal steel; s_h = **7.00** in

Diameter of horizontal bar; D_{hor} = **0.625** in

Specified yield strength of reinforcement; f_y = **60000** psi

Specified compressive strength of concrete; f'_c = **4000** psi

Modulus of elasticity of bar reinforcement; E_s = **29×10^6** psi

Modulus of elasticity of concrete; $E_c = 57000 \times f'_c{}^{1/2} \times (1\text{psi})^{1/2}$ = **3604997** psi

Ultimate design strain; ε_c = **0.003** in/in

Compression-controlled strain limit ε_{ty} = **0.002**

Check for minimum area of vertical steel of single layer reinforcement wall to cl. 11.6.1

Gross area of wall per running foot length; $A_g = h \times 12\text{in}$ = **96.000** in^2

Numbers of vertical bars per running foot length; $N_v = 12\text{in}/s_v$ = **1.000**

Area of vertical steel per running foot length; $A_{st\_v} = N_v \times (\pi \times D_{ver}^2) / 4$ = **0.307** in^2

Minimum area of vertical steel required; $A_{st\_v\_min}$= **0.115** in^2

PASS- Minimum vertical steel check

Check for minimum area of horizontal steel of single layer reinforcement wall to cl. 11.6.1

Gross area of wall per running foot height; $A_g = h \times 12in =$ **96.000** in^2

Numbers of horizontal bar per running foot height; $N_h = 12in / s_h =$ **1.714**

Area of horizontal steel per running foot height; $A_{st\_h} = N_h \times (\pi \times D_{hor}^2) / 4 =$ **0.526** in^2

Minimum area of horizontal steel required; $A_{st\_h\_min}$= **0.192** in^2

PASS- Minimum horizontal steel check

Braced wall slenderness check to 6.2.5

Permissible slenderness ratio; $s_{r\_perm} = \min(34 + 12 \times (M_{1\_act} / M_{2\_act}),40) =$ **40.0**

Radius of gyration; $r_{min} = 0.3 \times h =$ **2.40** in

Actual slenderness ratio; $s_{r\_act} = k \times l_u / r_{min} =$ **80.00**

Wall is braced slender wall

Design loads and moments for wall subjected to shear, axial load and bending

Ultimate axial force per running foot; $P_{u\_act} =$ **1.88** kips/ft

Ultimate large end moment per running foot; $M_{2\_act} =$ **3.90** kips_ft/ft

Ultimate small end moment per running foot; $M_{1\_act} =$ **3.90** kips_ft/ft

Ultimate shear force per running foot; $V_{u\_act} =$ **2.60** kips/ft

Ratio of DL moment to total moment; $\beta_d =$ **0.900**

Magnified moment for braced slender wall to 6.6.4

Moment of inertia of section; $I_g = (12in \times h^3)/12 =$ **512.000** in^4

Euler's buckling load; $P_c = (\pi^2 \times 0.4 \times E_c \times I_g)/((1+\beta_d) \times (k \times l_u)^2 \times 1ft) =$ **104.035** kips/ft

PASS - Euler's buckling load exceeds ultimate axial force

Correction factor for actual to equiv. mmt. diagram; $C_m = 0.6 - (0.4 \times M_{1\_act} / M_{2\_act}) =$ **0.200**

Moment magnifier; $\delta_{ns} = \max(1.0, C_m / (1 - (P_{u\_act} / (0.75 \times P_c)))) =$ **1.000**

Minimum uniaxial moment for slender section; $M_{2\_min} = P_{u\_act} \times (0.6 \text{ in} + 0.03 \times h) =$ **0.132** kip_ft/ft

Magnified uniaxial moment; $M_c = \delta_{ns} \times \max(M_{2\_min}, M_{2\_act}) =$ **3.900** kip_ft/ft**Axial load capacity of single layer reinforcement wall subjected to bending**

c/d_t ratio; r = **0.140**

Effective cover to reinforcement; $d' = c_c + (D_{ver}/2) =$ **3.312** in

Depth of tension steel; $d_t = h - d' =$ **4.688** in

Depth of NA from extreme compression face; $c = r \times d_t =$ **0.656** in

Factor of depth of compressive stress block; β_1= **0.850**

Depth of equivalent rectangular stress block; $a = \min((\beta_1 \times c), h)=$ **0.558** in

Strain in 'tension' reinforcement; $\varepsilon_s = \varepsilon_c \times (1 - d_t / c) =$ **-0.018429**

f_s Stress in 'tension' reinforcement; $f_s = \max(E_s \times \varepsilon_s, -f_y) =$;**-60000.0**; psi

Compression force in concrete; $C_c = 0.85 \times f'_c \times a \times 12in/1ft=$ **22.759** kips/ft

Area of vertical tension steel per running foot; $A_s = A_{st\_v} =$ **0.307** in^2

Force in 'tension' steel; $T_s = A_s \times f_s/1ft =$ **-18.408** kips/ft

Nominal axial load strength; $P_n = C_c + T_s =$ **4.351** kips/ft

Strength reduction factor; $\phi = 0.9 =$ **0.9**

Ultimate axial load carrying capacity of wall; $P_u = \phi \times P_n =$ **3.916** kips/ft

Check for axial load capacity of wall

PASS- Wall is safe in axial loading

Bending capacity of single layer reinforcement wall

Centroid of wall;	$y = h \times 0.5 =$ **4.000** in
Nominal moment strength;	$M_n = C_c \times (y - 0.5 \times a) - T_s \times (d_t - y) =$ **8.112** kip_ft/ft
Ultimate moment strength capacity of wall;	$M_u = \phi \times M_n =$ **7.301**kip_ft/ft

Check for uniaxial bending capacity of wall

Wall is safe for bending

Check for shear capacity of wall subjected to shear, axial load and bending cl. 22.5

Strength reduction factor;	$\phi_v =$ **0.75**
Effective cover to reinforcement;	$d' = c_c + (D_{ver}/2) =$ **3.312** in
Depth of tension steel;	$d_t = h - d' =$ **4.688** in
Factored moment for axial compression;	$M_m = M_{2\_act} - (P_{u\_act} \times ((4 \times h) - d_t) / (8 \times 12in) \times 1ft) =$ **3.365** kips_ft/ft
Shear force capacity of wall;	$V_{c1} = ((1.9 \times \lambda \times \sqrt{(f'_c \times 1psi)} \times d_t \times 12in) / 1ft) + (2500\ psi \times A_s / 1\ ft \times \min(1,(V_{u\_act} \times d_t / M_m)))$ = **6.991** kips/ft
Maximum shear force resisting capacity of wall;	$V_{max} = (3.5 \times \lambda \times \sqrt{(f'_c \times 1psi)} \times h \times 12in \times \sqrt{(1kips / ft^3 + P_{u\_act} / (500 \times A_g))} \times \sqrt{(1ft / 1kips)}$ = **21.310** kips/ft
Shear force resisting capacity of wall;	$\phi V_c = \phi_v \times \min(V_{c1}, V_{max})$ = 5.243 kips/ft;

PASS- Wall is safe in shear force

FOOTING ANALYSIS

In accordance with ACI318: See attachment for additional ACI-350 Reinforcement Requirements if they differ from those in ACI-318

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.696

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	3.1			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.044	1.5	0.696	Pass

Strip footing details - considering a one foot strip

Length of footing; L_x = **1** ft

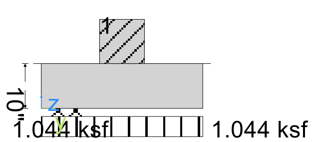
Width of footing; L_y = **3** ft

Footing area; $A = L_x \times L_y$ = **3** ft^2

Depth of footing; h = **10** in

Depth of soil over footing; h_{soil} = **0** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall; l_{y1} = **10** in

position in y-axis; y_1 = **18** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **1.5** ksf;

Density of soil; γ_{soil} = **65.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

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

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **125** psf

Wall no.1 loads per linear foot

Dead load in z; F_{Dz1} = **1.5** kips

Live load in z; F_{Lz1} = **0.5** kips

Live roof load in z; F_{Lrz1} = **1.0** kips

Snow load in z; F_{Sz1} = **0.9** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.417)

1.0D + 1.0L (0.528)

1.0D + 1.0Lr (0.639)

1.0D + 1.0S (0.622)

1.0D + 1.0R (0.417)

1.0D + 0.75L + 0.75Lr (0.667)

1.0D + 0.75L + 0.75S (0.654)

1.0D + 0.75L + 0.75R (0.500)

1.0D + 0.6W (0.417)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.475)

1.0D + 0.75L + 0.75Lr + 0.45W (0.667)

1.0D + 0.75L + 0.75S + 0.45W (0.654)

1.0D + 0.75L + 0.75R + 0.45W (0.500)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.696)

0.6D + 0.6W (0.250)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.192)

Combination 14 results: $(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E

Forces on footing per linear foot

Force in z-axis; $F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **3.1** kips

Moments on footing per linear foot

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **4.7** kip_ft

Uplift verification

Vertical force; F_{dz} = **3.131** kips

PASS - Footing is not subject to uplift

Stability against sliding

Resistance due to base friction; $F_{RFriction} = \max(F_{dz}, 0\ kN) \times \tan(\delta_{bb})$ = **1.808** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **1.044** ksf

$q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **1.044** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2)$ = **1.044** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2)$ = **1.044** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

q_{max} / q_{allow} = **0.696**

PASS - Allowable bearing capacity exceeds design base pressure

RC WALL DESIGN

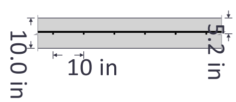
In accordance with ACI 318-14: See attachment for additional ACI-350 Reinforcement Requirements if they differ from those in ACI-318

Tedds calculation version 1.2.08

Design summary

Overall design status; PASS

Description	Unit	Required	Provided	Utilization	Result
Axial	kips/ft	3.7	11.0	0.332	PASS
Euler load	kips/ft	3.7	203.2	0.018	PASS
Moment	kip_ft/ft	3.9	12.1	0.323	PASS
Shear	kips/ft	2.6	5.9	0.440	PASS



Geometry of wall

Depth of wall; h = **10.00** in

Clear cover to reinforcement (both sides); c_c = **4.50** in

Unsupported height of wall; l_u = **96.0** in

Effective height factor; k = **2.00**

Reinforcement of wall

Numbers of reinforcement layers; N_l = **1**

Vertical steel bar diameter number; D_{ver_num} = **5**

Spacing of vertical steel; s_v = **10.00** in

Diameter of vertical steel bar; D_{ver} = **0.625** in

Horizontal steel bar diameter number; D_{hor_num} = **5**

Spacing of horizontal steel; s_h = **6.00** in

Diameter of horizontal bar; D_{hor} = **0.625** in

Specified yield strength of reinforcement; f_y = **60000** psi

Specified compressive strength of concrete; f'_c = **4000** psi

Modulus of elasticity of bar reinforcement; $E_s = \mathbf{29 \times 10^6}$ psi

Modulus of elasticity of concrete; $E_c = 57000 \times f'_c{}^{1/2} \times (1\text{psi})^{1/2}$ = **3604997** psi

Ultimate design strain; ε_c = **0.003** in/in

Compression-controlled strain limit ε_{ty} = **0.002**

Check for minimum area of vertical steel of single layer reinforcement wall to cl. 11.6.1

Gross area of wall per running foot length; $A_g = h \times 12\text{in}$ = **120.000** in^2

Numbers of vertical bars per running foot length; N_v = 12in/s_v = **1.200**

Area of vertical steel per running foot length; $A_{st\_v} = N_v \times (\pi \times D_{ver}^2) / 4$ = **0.368** in^2

Minimum area of vertical steel required; $A_{st\_v\_min}$= **0.144** in^2

PASS- Minimum vertical steel check

Check for minimum area of horizontal steel of single layer reinforcement wall to cl. 11.6.1

Gross area of wall per running foot height; $A_g = h \times 12in$ = **120.000** in^2

Numbers of horizontal bar per running foot height; N_h = 12in / s_h = **2.000**

Area of horizontal steel per running foot height; $A_{st\_h} = N_h \times (\pi \times D_{hor}^2) / 4$ = **0.614** in^2

Minimum area of horizontal steel required; $A_{st\_h\_min}$= **0.240** in^2

PASS- Minimum horizontal steel check

Braced wall slenderness check to 6.2.5

Permissible slenderness ratio; $s_{r\_perm} = \min(34 + 12 \times (M_{1\_act} / M_{2\_act}),40)$ = **40.0**

Radius of gyration; $r_{min} = 0.3 \times h$ = **3.00** in

Actual slenderness ratio; $s_{r\_act} = k \times l_u / r_{min}$ = **64.00**

Wall is braced slender wall

Design loads and moments for wall subjected to shear, axial load and bending

Ultimate axial force per running foot; P_{u_act} = **3.65** kips/ft

Ultimate large end moment per running foot; M_{2_act} = **3.90** kips_ft/ft

Ultimate small end moment per running foot; M_{1_act} = **3.90** kips_ft/ft

Ultimate shear force per running foot; V_{u_act} = **2.60** kips/ft

Ratio of DL moment to total moment; β_d = **0.900**

Magnified moment for braced slender wall to 6.6.4

Moment of inertia of section; $I_g = (12in \times h^3)/12$ = **1000.000** in^4

Euler's buckling load; $P_c = (\pi^2 \times 0.4 \times E_c \times I_g)/((1+\beta_d)\times(k \times l_u)^2 \times 1ft)$ = **203.193** kips/ft

PASS - Euler's buckling load exceeds ultimate axial force

Correction factor for actual to equiv. mmt. diagram; $C_m = 0.6 - (0.4 \times M_{1\_act} / M_{2\_act})$ = **0.200**

Moment magnifier; $\delta_{ns} = \max(1.0, C_m / (1 - (P_{u\_act} / (0.75 \times P_c))))$ = **1.000**

Minimum uniaxial moment for slender section; $M_{2\_min} = P_{u\_act} \times (0.6\text{ in} + 0.03 \times h)$ = **0.274** kip_ft/ft

Magnified uniaxial moment; $M_c = \delta_{ns} \times \max(M_{2\_min}, M_{2\_act})$ = **3.900** kip_ft/ft**Axial load capacity of single layer reinforcement wall subjected to bending**

c/d_t ratio; r = **0.191**

Effective cover to reinforcement; $d' = c_c + (D_{ver}/2)$ = **4.813** in

Depth of tension steel; $d_t = h - d'$ = **5.188** in

Depth of NA from extreme compression face; $c = r \times d_t$ = **0.989** in

Factor of depth of compressive stress block; β_1= **0.850**

Depth of equivalent rectangular stress block; $a = \min((\beta_1 \times c), h)$= **0.840** in

Strain in 'tension' reinforcement; $\varepsilon_s = \varepsilon_c \times (1 - d_t / c)$ = **-0.012740**

f_s Stress in 'tension' reinforcement; $f_s = \max(E_s \times \varepsilon_s, -f_y)$ = ;**-60000.0**; psi

Compression force in concrete; $C_c = 0.85 \times f'_c \times a \times 12in/1ft$= **34.289** kips/ft

Area of vertical tension steel per running foot; $A_s = A_{st\_v}$ = **0.368** in^2

Force in 'tension' steel; $T_s = A_s \times f_s/1ft$ = **-22.089** kips/ft

Nominal axial load strength; $P_n = C_c + T_s$ = **12.200** kips/ft

Strength reduction factor; $\phi = 0.9 = \mathbf{0.9}$

Ultimate axial load carrying capacity of wall; $P_u = \phi \times P_n =$ **10.980** kips/ft

Check for axial load capacity of wall

PASS- Wall is safe in axial loading

Bending capacity of single layer reinforcement wall

Centroid of wall; $y = h \times 0.5 =$ **5.000** in

Nominal moment strength; $M_n = C_c \times (y - 0.5 \times a) - T_s \times (d_t - y) =$ **13.432** kip_ft/ft

Ultimate moment strength capacity of wall; $M_u = \phi \times M_n =$ **12.088**kip_ft/ft

Check for uniaxial bending capacity of wall

Wall is safe for bending

Check for shear capacity of wall subjected to shear, axial load and bending cl. 22.5

Strength reduction factor; $\phi_v =$ **0.75**

Effective cover to reinforcement; $d' = c_c + (D_{ver}/2) =$ **4.813** in

Depth of tension steel; $d_t = h - d' =$ **5.188** in

Factored moment for axial compression; $M_m = M_{2\_act} - (P_{u\_act} \times ((4 \times h) - d_t) / (8 \times 12in) \times 1ft) =$ **2.576** kips_ft/ft

Shear force capacity of wall; $V_{c1} = ((1.9 \times \lambda \times \sqrt{(f'_c \times 1psi)} \times d_t \times 12in) / 1ft) + (2500\ psi \times A_s / 1\ ft \times \min(1,(V_{u\_act} \times d_t / M_m)))$ = **7.882** kips/ft

Maximum shear force resisting capacity of wall; $V_{max} = (3.5 \times \lambda \times \sqrt{(f'_c \times 1psi)} \times h \times 12in \times \sqrt{(1kips / ft^3 + P_{u\_act} / (500 \times A_g))}) \times \sqrt{(1ft / 1kips)}$ = **26.679** kips/ft

Shear force resisting capacity of wall; $\phi V_c = \phi_v \times \min(V_{c1}, V_{max}) = 5.911$ kips/ft;

PASS- Wall is safe in shear force

FOOTING ANALYSIS

In accordance with ACI318-14: See attachment for additional ACI-350 Reinforcement Requirements if they differ from those in ACI-318

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.987

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	13.3			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.48	1.5	0.987	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	2.4	23.6	0.101	Pass
Moment, positive, y-direction	kip_ft	2.4	23.6	0.101	Pass
Shear, one-way, x-direction	kips	2.2	23.1	0.097	Pass
Shear, one-way, y-direction	kips	2.2	21.3	0.105	Pass
Shear, two-way, Col 1	psi	21.596	189.737	0.114	Pass
Min.area of reinf, bot., x-direction	in^2	0.648	0.800		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.5		Pass
Min.area of reinf, bot., y-direction	in^2	0.648	0.800		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.5		Pass

Pad footing details

Length of footing; L_x = **3** ft

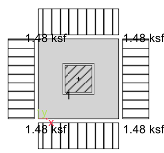
Width of footing; L_y = **3** ft

Footing area; $A = L_x \times L_y$ = **9** ft^2

Depth of footing; h = **10** in

Depth of soil over footing; h_{soil} = **0** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column; l_{x1} = **12.00** in

Width of column; l_{y1} = **12.00** in

position in x-axis; x_1 = **18.00** in

position in y-axis; y_1 = **18.00** in

Height of pedestal; h_{ped1} = **82.00** in

Length of pedestal; $l_{x,ped1}$ = **14.00** in

Width of pedestal; $l_{y,ped1}$ = **14.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **1.5** ksf;

Density of soil; γ_{soil} = **63.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

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

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **125** psf

Column no.1 loads

Pedestal self weight; F_{SWz1} = **1.4** kips

Dead load in z; F_{Dz1} = **4.8** kips

Live load in z; F_{Lz1} = **6.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.542)

1.0D + 1.0L (0.987)

1.0D + 1.0Lr (0.542)

1.0D + 1.0S (0.542)

1.0D + 1.0R (0.542)

1.0D + 0.75L + 0.75Lr (0.876)

1.0D + 0.75L + 0.75S (0.876)

1.0D + 0.75L + 0.75R (0.876)

$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.618)

1.0D + 0.75L + 0.75Lr + 0.45W (0.876)

1.0D + 0.75L + 0.75S + 0.45W (0.876)

1.0D + 0.75L + 0.75R + 0.45W (0.876)

$(1.0 + 0.10 \times S_{DS})D + 0.75L + 0.75S + 0.525E$ (0.930)

0.6D + 0.6W (0.325)

$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.249)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **13.3** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **20.0** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **20.0** kip_ft

Uplift verification

Vertical force; F_{dz} = **13.32** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

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

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

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.48** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.48** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.48** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.48** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.48** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.48** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

q_{max} / q_{allow} = **0.987**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; f'_c = **4000** psi

Yield strength of reinforcement; f_y = **60000** psi

Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**

Cover to top of footing; c_{nom_t} = **2** in

Cover to side of footing; c_{nom_s} = **2** in

Cover to bottom of footing; c_{nom_b} = **3** in

Concrete type; Normal weight

Concrete modification factor; λ = **1.00**

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.058)

1.2D + 1.6L + 0.5Lr (0.114)

1.2D + 1.6L + 0.5S (0.114)

1.2D + 1.6L + 0.5R (0.114)

1.2D + 1.0L + 1.6Lr (0.114)

1.2D + 1.0L + 1.6S (0.114)

1.2D + 1.0L + 1.6R (0.114)

1.2D + 1.6Lr + 0.5W (0.114)

1.2D + 1.6S + 0.5W (0.114)

1.2D + 1.6R + 0.5W (0.114)

1.2D + 1.0L + 0.5Lr + 1.0W (0.114)

1.2D + 1.0L + 0.5S + 1.0W (0.114)

1.2D + 1.0L + 0.5R + 1.0W (0.114)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.2S + 1.0E (0.114)

0.9D + 1.0W (0.114)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.114)

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

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$

= **18.4** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (((F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **27.6** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **27.6** 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

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.043** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.043** ksf

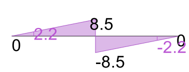
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.043** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.043** ksf

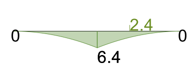
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.043** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.043** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max}$ = **2.386** kip_ft

Tension reinforcement provided; 4 No.4 bottom bars (10.5 in c/c)

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

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h$ = **0.648** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); $s_{max} = \min(2 \times h, 18\text{ in})$ = **18** 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; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.392** in

Neutral axis factor; β_1 = **0.85**

Depth to neutral axis; $c = a / \beta_1$ = **0.461** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04089**

Minimum tensile strain(8.3.3.1); ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **26.216** 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; $\phi M_n = \phi_f \times M_n$ = **23.594** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.101**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **2.248** kips

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

Shear strength reduction factor; ϕ_v = **0.75**

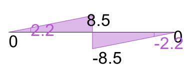
Nominal shear capacity (Eq. 22.5.5.1); $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v$ = **30.737** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **23.053** kips

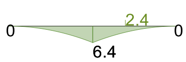
$V_{u.x} / \phi V_n$ = **0.097**

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; $M_{u.y.max}$ = **2.386** kip_ft

Tension reinforcement provided; 4 No.4 bottom bars (10.5 in c/c)

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

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **0.648** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); $s_{max} = \min(2 \times h, 18 \text{ in})$ = **18** 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; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.392** in

Neutral axis factor; β_1 = **0.85**

Depth to neutral axis; $c = a / \beta_1$ = **0.461** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03764**

Minimum tensile strain(8.3.3.1); ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **24.216** 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; $\phi M_n = \phi_f \times M_n =$ **21.794** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.109**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} =$ **2.248** 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); $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v =$ **28.46** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **21.345** kips

$V_{u.y} / \phi V_n =$ **0.105**

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); $l_{xp} =$ **20.500** in

Shear perimeter width (22.6.4); $l_{yp} =$ **20.500** in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x,ped1} + d_{v2}) + 2 \times (l_{y,ped1} + d_{v2}) =$ **82.000** in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} =$ **420.250** in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x,ped1} \times l_{y,ped1} =$ **224.250** in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} =$ **2.043** ksf

Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} - q_{up.avg} \times A_p =$ **11.511** kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) =$ **21.596** psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y,ped1} / l_{x,ped1} =$ **1.00**

Column location factor (22.6.5.3); $\alpha_s =$**40**

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **327.026** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi

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

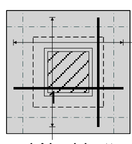
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **252.982** psi

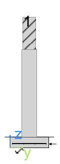
Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **189.737** psi

$v_{ug} / \phi v_n =$ **0.114**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Johnson Engineering Group PO Box 384 Windom, MN 56101 507-832-8450	Project Gruber Livestock South GDU				Job no. 0169-01B	
	Calcs for Pit Column Footing				Start page no./Revision 8	
	Calcs by MPJ	Calcs date 7/22/2025	Checked by DJJ	Checked date 7/22/2025	Approved by DJJ	Approved date 7/22/2025





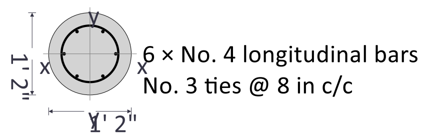
RC COLUMN DESIGN

In accordance with ACI318-14: See attachment for additional ACI-350 Reinforcement Requirements if they differ from those in ACI-318

Tedds calculation version 2.2.07

Design summary

Description	Unit	Capacity	Applied	Utilization	Result
Axial	kips	243.0	20.6	0.085	-



Applied loads

Ultimate axial force acting on column; P_{u_act} = **20.64** kips

Ratio of DL moment to total moment; β_d = **0.600**

Geometry of column

Column diameter; h = **14.0** in

Clear cover to all reinforcement; c_c = **2.00** in

Unsupported height of column about x axis; l_{ux} = **6.8** ft

Effective height factor about x axis; k_x = **1.00**

Column state about the x axis; Braced

Unsupported height of column about y axis; l_{uy} = **6.8** ft

Effective height factor about y axis; k_y = **1.00**

Column state about the y axis; Braced

Reinforcement of column

Numbers of bars of longitudinal steel; N = **6**

Longitudinal steel bar diameter number; D_{bar_num} = **4**

Diameter of longitudinal bar; D_{long} = **0.500** in

Stirrup bar diameter number; D_{stir_num} = **3**

Diameter of stirrup bar; D_{stir} = **0.375** in

Specified yield strength of reinforcement; f_y = **60000** psi

Specified compressive strength of concrete; f'_c = **4000** psi

Modulus of elasticity of bar reinforcement; E_s = **29 × 10⁶** psi

Modulus of elasticity of concrete; $E_c = 57000 \times f'_c{}^{1/2} \times (1psi)^{1/2}$ = **3604997** psi

Yield strain; $\varepsilon_y = f_y / E_s$ = **0.00207**

Ultimate design strain; ε_c = **0.003** in/in

Check for minimum area of steel - 10.6.1.1

Gross area of column; $A_g = \pi \times h^2 / 4$ = **153.938** in²

Area of steel; $A_{st} = N \times (\pi \times D_{long}^2) / 4$ = **1.178** in²

Minimum area of steel required; $A_{st\_min} = 0.01 \times A_g$ = **1.539** in²

Area of reinforcement provided (A_{st}) is less than the minimum required (A_{st_min}) therefore apply ACI 318 clause 10.3.1.2

Reduced column area to satisfy min reinft; $A_{g\_red} = A_{st} / 0.01$ = **117.810** in²

PASS- Reduced effective area is not less than half gross area

Check for maximum area of steel - 10.6.1.1

Permissible maximum area of steel; $A_{st\_max} = 0.08 \times A_g$ = **12.315** in²

$A_{st} < A_{st\_max}$, PASS - Maximum steel check

Design of column ties - 25.7.2

Spacing of lateral ties; s_{v_ties} = **8.000** in

16 times longitudinal bar diameter; $s_{v1} = 16 \times D_{long}$ = **8.000** in

48 times tie bar diameter; $s_{v2} = 48 \times D_{stir}$ = **18.000** in

Least column dimension; s_{v3} = min (h,b) = **14.000** in

Required tie spacing ; $s = \min(s_{v1}, s_{v2}, s_{v3})$ = **8.000** in

$s_{v\_ties} < s$ PASS

Slenderness check about x axis

Radius of gyration; $r_x = 0.25 \times h$ = **3.5** in

Actual slenderness ratio; $s_{rx\_act} = k_x \times l_{ux} / r_x$ = **23.42**

Permissible slenderness ratio; $s_{rx\_perm} = \min(34 - 12 \times (M_{1x\_act} / M_{2x\_act}), 40)$ = **34**

Slenderness effects may be neglected about the X axis

Slenderness check about y axis

Radius of gyration; $r_y = 0.25 \times h$ = **3.5** in

Actual slenderness ratio; $s_{ry\_act} = k_y \times l_{uy} / r_y$ = **23.42**

Permissible slenderness ratio; $s_{ry\_perm} = \min(34 - 12 \times (M_{1y\_act} / M_{2y\_act}), 40)$ = **34**

Slenderness effects may be neglected about the Y axis

Axial load capacity of axially loaded column

Strength reduction factor; ϕ = **0.650**

Area of steel on compression face; $A'_s = A_{st} / 2$ = **0.589** in²

Area of steel on tension face; $A_s = A_{st} / 2$ = **0.589** in²

Net axial load capacity of column; $P_n = 0.8 \times (0.85 \times f'_c \times (A_{g\_red} - A_{st}) + f_y \times A_{st})$ = **373.787** kips

Ultimate axial load capacity of column; $P_u = \phi \times P_n$ = **242.961** kips

PASS : Column is safe in axial loading

;

8" Exterior Pit Wall	
Minimum Flexural Reinforcement	
ACI-350 Required	**Provided**
1. $0.003A_g$ — $0.288\ in^2$ (Total Vertical) 2. $200\ b_w d/f_y$ — $0.14\ in^2$ (Flexural Only) 3. $(3\sqrt{f'_c}/f_y)b_w d$ — $0.13\ in^2$ (Flexural Only)	#5's @ 12" O.C. A_s=0.31 in^2
Minimum Shrinkage & Temperature Steel	
ACI-350 Required	**Provided**
1. $0.005A_g$ — $0.48\ in^2$ (Total Horizontal)	#5's @ 7" O.C. A_s=0.53 in^2
10" Pit Divide Wall	
Minimum Flexural Reinforcement	
ACI-350 Required	**Provided**
1. $0.003A_g$ — $0.36\ in^2$ (Total Vertical) 2. $200\ b_w d/f_y$ — $0.23\ in^2$ (Flexural Only) 3. $(3\sqrt{f'_c}/f_y)b_w d$ — $0.22\ in^2$ (Flexural Only)	#5's @ 10" O.C. A_s=0.37 in^2
Minimum Shrinkage & Temperature Steel	
ACI-350 Required	**Provided**
1. $0.005A_g$ — $0.6\ in^2$ (Total Horizontal)	#5's @ 6" O.C. A_s=0.62 in^2
5" Pit Floor	
ACI-350 Required	**Provided**
1. $0.005A_g$ — $0.3\ in^2$	#4's @ 8" O.C. A_s=0.3 in^2

Gruber Livestock - Pumpout Lintel

- Loading - only floor Load supported
  - Tributary width = 5 ft
  - Slat DL = 40 psf * 5 ft = 200 plf
  - slat LL = 50 psf * 5 ft = 250 plf

  - Span of Lintel: 6 ft

- LRFD Load Combinations
  - $1.4D$: $1.4 * 200\,plf = 280\,plf = 0.28\,klf$
  - $1.2D + 1.6L$: $(1.2 * 200\,plf) + (1.6 * 250\,plf) = 640\,plf = 0.64\,klf$ ⇒ Controls

- Design Moment
  - $M_u = wu l^2/8 = \frac{(0.64)(6)^2}{8} = 2.88\,ft\text{-}K$

- Angle Design (AISC F10)
  - Angle braced by slat ⇒ no buckling checks
  - Angle is compact ⇒ no leg buckling
  - Angle Size: 4 x 4 x 1/4
    - $F_y = 36\,ksi$
    - $A = 1.93\,in^2$
    - $S = 1.03\,in^3$

  - Yielding Moment capacity: $\phi M_n = \phi\, 1.5\, F_y\, S_x$

$$= 0.9(1.5)(36)(1.03)$$

$$\boxed{\phi M_n = 50.05\,in\text{-}K = 4.17\,ft\text{-}K > M_u}$$