

Calc. By RJS Checked By JH Date 1/11/2021

Structural Calculations For:

# **Balderston Auto**

# Lee's Summit, MO

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Project Balderston Au	ito	Project No. 20-467
		,
Calc. By RS	Checked By_JH	Date 02/01/2021

# <u>Summary</u>

Loads for the project referenced above were determined based on the governing International Building Code (IBC) and the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7).

All vertical/gravity loads were determined as follow: All dead loads were determined based on the building composition and all live loads were determined based on the expected occupancy for each of the spaces within the building. Snow loads were determined based on the building dimensions, the roof profile and the project location.

All lateral loads were determined as follows: All wind loading was based on the building dimensions and project location. All seismic loads were determined based on the building composition, the type of lateral stability system and the project location.

The following section of calculations covers the process used to determine the gravity and lateral loads for the project referenced above. Refer to all other sections for the application of these loads.



Project Balderston Auto	Project No. 20-467
	FI0JECT NO. 20-407

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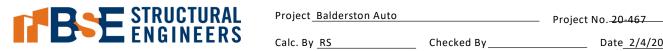
# **DEAD LOADING**

DEAD LOAD CONSTRUCTION	
Roof Dead Load	15 psf

# LIVE LOADING

LIVE LOAD CONSTRUCTION

Roof Live Load IBC 2012, Table 1607.1 20 psf



Calc. By <u>RS</u> Checked By Date <u>2/4/2021</u>

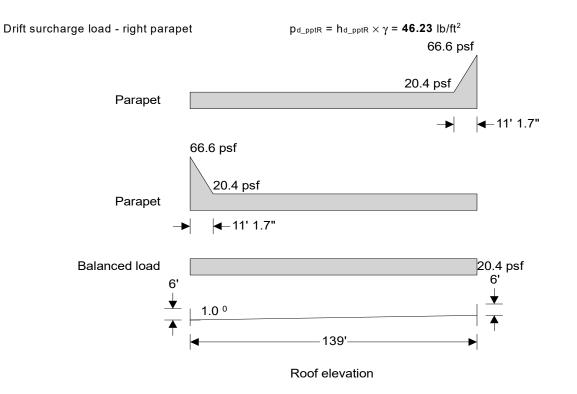
SNOW LOADING	
In accordance with ASCE7-10	
	Tedds calculation version 1.0.09
Building details	
Roof type	Monopitch
Width of roof	b = <b>139.00</b> ft
Slope of roof 1	α = <b>1.00</b> deg
Ground snow load	
Ground snow load (Figure 7-1)	p <sub>g</sub> = <b>20.00</b> lb/ft <sup>2</sup>
Density of snow	$\gamma = min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3) = 16.60 lb/ft^3$
Terrain typeSect. 26.7	C
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	Ce = 1.00
Thermal condition (Table 7-3)	Structures kept just above freezing
Thermal factor (Table 7-3)	Ct = <b>1.10</b>
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	ls = 1.00
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f_{min}} = I_s \times p_g = 20.00 \text{ lb/ft}^2$
Rain on snow surcharge (Sect 7.10)	p <sub>f_sur</sub> = <b>5.00</b> lb/ft <sup>2</sup>
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g + p_{f_sur} = 20.40 \text{ lb}/ft^2$
Unbalanced flat roof snow load (Sect 7.3)	$p_{f\_unbal}$ = max( $p_{f\_min}$ , 0.7 × C <sub>e</sub> × C <sub>t</sub> × I <sub>s</sub> × $p_g$ ) = <b>20.00</b> lb/ft <sup>2</sup>
Cold roof slope factor (Ct > 1.0)	
Roof surface type	Slippery
Ventilation	Ventilated
Thermal resistance (R-value)	R = <b>30.00</b> °F h ft <sup>2</sup> / Btu
Roof slope factor Fig 7-2b (solid line)	$C_{s} = 1.00$
Monoslope	
Sloped roof snow load (CI.7.4)	$p_{s} = max(C_{s} \times p_{f}, p_{f_{min}}) = 20.40 \text{ lb/ft}^{2}$
Left parapet	
Balanced snow load height	$h_b = p_f \times C_s / \gamma = 1.23 ft$
Height of left parapet	$h_{pptL} = 6.00 \text{ ft}$
Height from balance load to top of left parapet	$h_{c_{pptL}} = h_{pptL} - h_{b} = 4.77 \text{ ft}$
Length of roof - left parapet	$l_{u \text{ pptL}} = b_1 = 139.00 \text{ ft}$
Drift height windward drift - left parpet	$h_{d\_pptL} = 0.75 \times (0.43 \times (max(20 \text{ ft, } l_{u\_pptL}) \times 1 \text{ ft}^2)^{1/3} \times (p_g / 1 \text{ lb/ft}^2 + 10)^{1/4})^{1/4}$
Dint neight windward dint - ien parpet	-1.5ft) = 2.78 ft
Drift height - left parapet	hd_pptL = min(hd_l_pptL, hpptL - hb) = 2.78 ft
Drift width	$W_{d_pptL} = min(4 \times h_{d_l_pptL}, 8 \times (h_{pptL} - h_b), b) = 11.14 \text{ ft}$
Drift surcharge load - left parapet	$p_{d_pptL} = h_{d_pptL} \times \gamma = 46.23 \text{ Ib/ft}^2$
Right parapet	
Height of right parapet	h <sub>pptR</sub> = <b>6.00</b> ft
Height from balance load to top of right parapet	$h_{c_pptR} = h_{pptR} - h_b = 4.77$ ft
Length of roof - right parapet	Iu_pptR = b1 = <b>139.00</b> ft
Drift height windward drift - right parpet	$h_{d\_pptR} = 0.75 \times (0.43 \times (max(20 \text{ ft, } l_{u\_pptR}) \times 1 \text{ ft}^2)^{1/3} \times (p_g / 1 \text{ lb/ft}^2 + 1 \text{ lb/ft}^2)^{1/3}$
	$10)^{1/4}$ - 1.5ft) = <b>2.78</b> ft
Drift height - right parapet	$h_{d_{pptR}} = min(h_{d_{l_{pptR}}}, h_{pptR} - h_b) = 2.78$ ft
Drift width	$W_{d_pptR} = min(4 \times h_{d_pptR}, 8 \times (h_{pptR} - h_b), b) = 11.14 \text{ ft}$
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Project Balderston Auto \_\_\_\_\_ Project No.-<u>20-467</u>\_\_\_

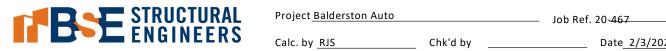
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Date 2/4/2021



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Sht. No. 2 A4<sup>of</sup> 11 2



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## **SEISMIC FORCES (ASCE 7-10)**

Tedds calculation version 3.1.00

	led
Site parameters	
Site class	D
Mapped acceleration parameters (Section 11.4.1)	
at short period	Ss = 0.099
at 1 sec period	S <sub>1</sub> = <b>0.068</b>
Site coefficientat short period (Table 11.4-1)	$F_{a} = 1.600$
at 1 sec period (Table 11.4-2)	$F_{v} = 2.400$
Spectral response acceleration parameters	
at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S = 0.158$
at 1 sec period (Eq. 11.4-2)	S <sub>M1</sub> = F <sub>v</sub> × S <sub>1</sub> = <b>0.163</b>
Design spectral acceleration parameters (Sect 1	1.4.4)
at short period (Eq. 11.4-3)	SDS = 2/3 × SMS = 0.106
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2/3 \times S_{M1} = 0.109$
	301 - 273 × 3m1 - 0.103
Seismic design category	
Risk category (Table 1.5-1)	II
Seismic design category based on short period res	
Seismic design category based on 1 sec period res	
Solomia design estagony	B
Seismic design category	В
Approximate fundamental period	
Height above base to highest level of building	hn = <b>17.2</b> ft
From Table 12.8-2:	
	All other overtome
Structure type	All other systems Ct = <b>0.02</b>
Building period parameter Ct	x = 0.75
Building period parameter x	x = 0.75
Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1 \sec t / (1ft)^x = 0.169 \sec t$
Building fundamental period (Sect 12.8.2)	$T = T_a = 0.169 \text{ sec}$
Long-period transition period	$T_{L} = 12 \text{ sec}$
Seismic response coefficient	D DUU DING EDAME OVOTEMO
Seismic force-resisting system (Table 12.2-1)	B_BUILDING_FRAME_SYSTEMS
	18. Ordinary reinforced masonry shear walls
Response modification factor (Table 12.2-1)	R = 2
Seismic importance factor (Table 1.5-2)	le = 1.000
Seismic response coefficient (Sect 12.8.1.1)	0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
Calculated (Eq 12.8-3)	$C_{s_{calc}} = S_{DS} / (R / I_e) = 0.0528$
Maximum (Eq 12.8-3)	$C_{s_{max}} = S_{D1} / ((T / 1 \text{ sec}) \times (R / I_e)) = 0.3220$
Minimum (Eq 12.8-5)	$C_{\text{s}\_\text{min}} = max(0.044 \times S_{\text{DS}} \times I_{\text{e}}, 0.01) = \textbf{0.0100}$
Seismic response coefficient	C <sub>s</sub> = <b>0.0528</b>
Seismic base shear (Sect 12.8.1)	
Effective seismic weight of the structure	W = <b>375.0</b> kips



\_\_\_\_\_ Job Ref. 20-467\_\_\_

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Seismic response coefficient	Cs = 0.0528
Seismic base shear (Eq 12.8-1)	$V$ = $C_s \times W$ = <b>19.8</b> kips

## Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12)

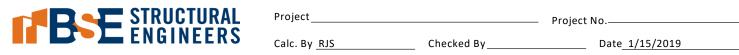
 $C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$ 

Lateral force induced at level i (Eq 12.8-11)  $F_x = C_{vx} \times V$ 

 $\nabla x = W \times W + 2(W \times W)$ 

Vertical force distribution table

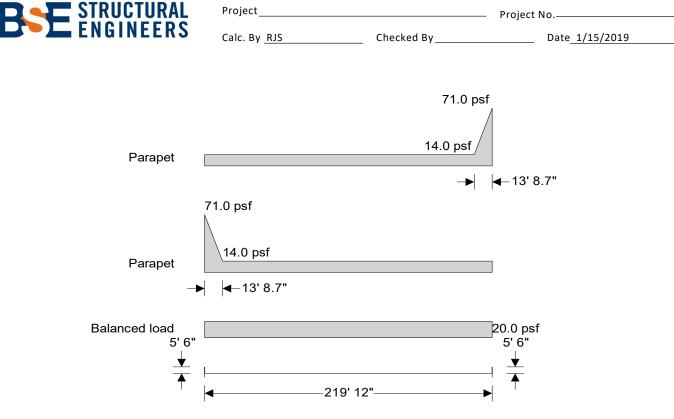
Level	Height from base to Level i (ft), h <sub>x</sub>	Portion of effective seismic weight assigned to Level i (kips), w <sub>x</sub>	Distribution exponent related to building period, k	Vertical distribution factor, C <sub>vx</sub>	Lateral force induced at Level i (kips), F <sub>x</sub>
1	17.2;	375.0;	1.00;	1.000;	19.8



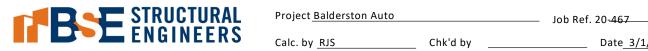
# SNOW LOADING (ASCE7-10)

Tedds calculation version 1.0.06

Building details	
Roof type	Flat
Width of roof	b = <b>220.00</b> ft
Ground snow load	
Ground snow load	pg = <b>20.00</b> lb/ft <sup>2</sup>
Density of snow	$\gamma = min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3) = 16.60 lb/ft^3$
Terrain type	С
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	C <sub>e</sub> = 1.00
Thermal condition (Table 7-3)	All
Thermal factor (Table 7-3)	Ct = 1.00
Importance category (Table 1-1)	II
Importance factor (Table 7-4)	ls = 1.00
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f_{min}} = I_s \times p_g = 20.00 \text{ lb/ft}^2$
Flat roof snow load (Sect 7.3)	$p_{f} = 0.7 \times C_{e} \times C_{t} \times I_{s} \times p_{g} = 14.00 \text{ lb/ft}^{2}$
Left parapet	
Balanced snow load height	$h_{\rm b} = p_{\rm f} / \gamma = 0.84  {\rm ft}$
Height of left parapet	h <sub>pptL</sub> = <b>5.50</b> ft
Height from balance load to top of left parapet	hc_pptL = hpptL - hb = <b>4.66</b> ft
Length of roof - left parapet	$l_{u_pptL} = b = 220.00 \text{ ft}$
Drift height windward drift - left parpet	$h_{d_{\perp}pptL} = 0.75 \times (0.43 \times (max(20 \text{ ft, } l_{u_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4}$
Drift height left nerenet	-1.5ft = 3.43 ft
Drift height - left parapet	$h_{d_pptL} = min(h_{d_pptL}, h_{pptL} - h_b) = 3.43 \text{ ft}$
Drift width	$W_{d_pptL} = min(4 \times h_{d_pptL}, 8 \times (h_{pptL} - h_b)) = 13.73 \text{ ft}$
Drift surcharge load - left parapet	$p_{d_pptL} = h_{d_pptL} \times \gamma = 56.96 \text{ lb/ft}^2$
Right parapet	
Height of right parapet	$h_{pptR} = 5.50 \text{ ft}$
Height from balance load to top of right parapet	$h_{c_{pptR}} = h_{pptR} - h_b = 4.66 \text{ ft}$
Length of roof - right parapet	$I_{u_{pptR}} = b = 220.00 \text{ ft}$
Drift height windward drift - right parpet	$h_{d\_l\_pptR}$ = 0.75 × (0.43 × (max(20 ft, Iu_pptR) × 1ft <sup>2</sup> ) <sup>1/3</sup> × (pg / 1Ib/ft <sup>2</sup> +
	10) <sup>1/4</sup> - 1.5ft) = <b>3.43</b> ft
Drift height - right parapet	$h_{d_pptR} = min(h_{d_pptR}, h_{pptR} - h_b) = 3.43 \text{ ft}$
Drift width	$W_{d_pptR} = min(4 \times h_{d_pptR}, 8 \times (h_{pptR} - h_b)) = 13.73 \text{ ft}$
Drift surcharge load - right parapet	$p_{d_pptR} = h_{d_pptR} \times \gamma = 56.96 \text{ lb/ft}^2$



Roof elevation



Calc. by <u>RJS</u> Chk'd by \_\_\_\_\_ Date <u>3/1/2021</u>

### WIND LOADING

#### In accordance with ASCE7-10

#### Using the components and cladding design method

	Tedds calculation version 2.1.05
€ © © V V V V V V V V V V V V V V V V V	$ \begin{array}{c}                                     $
<b>Building data</b> Type of roof Length of building Width of building Height to eaves Height of parapet Mean height	Flat b = 220.00 ft d = 66.00 ft H = 22.00 ft h <sub>P</sub> = 5.00 ft h = 22.00 ft
General wind load requirements Basic wind speed Risk category Velocity pressure exponent coef (Table 26.6-1) Exposure category (cl 26.7.3) Enclosure classification (cl.26.10) Internal pressure coef +ve (Table 26.11-1) Internal pressure coef -ve (Table 26.11-1) Parapet internal pressure coef +ve (Table 26.11-1) Parapet internal pressure coef -ve (Table 26.11-1) Gust effect factor	
<b>Topography</b> Topography factor not significant <b>Velocity pressure</b> Velocity pressure coefficient (T.30.3-1) Velocity pressure	$K_{zt} = 1.0$ $K_{z} = 0.92$ $q_{h} = 0.00256 \times K_{z} \times K_{zt} \times K_{d} \times V^{2} \times 1psf/mph^{2} = 26.4 psf$
<b>Velocity pressure at parapet</b> Velocity pressure coefficient (T.30.3-1) Velocity pressure	$K_z = 0.96$ $q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1psf/mph^2 = 27.5 psf$
Peak velocity pressure for internal pressure Peak velocity pressure – internal (as roof press.) Equations used in tables Net pressure Parapet net pressure	$q_i = 26.36 \text{ psf}$ $p = q_h \times [GC_p - GC_{pi}]$ $p = q_p \times [GC_p - GC_{pi\_p}]$

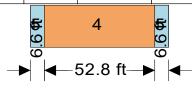


\_\_\_\_\_ Job Ref. 20-467\_\_\_

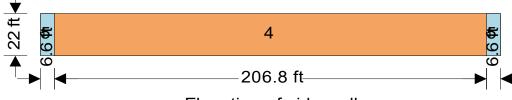
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#### Components and cladding pressures - Wall (Table 30.4-1 and Figure 30.4-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft <sup>2</sup> )	+GCp	-GCp	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	4	-	-	10.0	0.90	-0.99	28.5	-30.8
50sf	4	-	-	50.0	0.79	-0.88	25.5	-27.9
200sf	4	-	-	200.0	0.69	-0.78	23.0	-25.4
>500sf	4	-	-	500.0	0.63	-0.72	21.4	-23.7
<10sf	5	-	-	10.0	0.90	-1.26	28.5	-38.0
50sf	5	-	-	50.0	0.79	-1.04	25.5	-32.1
200sf	5	-	-	200.0	0.69	-0.85	23.0	-27.1
>500sf	5	-	-	500.0	0.63	-0.72	21.4	-23.7
20 sff	5	-	-	20.0	0.85	-1.16	27.2	-35.4
20 sff	4	-	-	20.0	0.85	-0.94	27.2	-29.6



Elevation of gable wall



Elevation of side wall

## Components and cladding pressures - Roof (Figure 30.4-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft <sup>2</sup> )	+GC <sub>p</sub>	-GCp	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	1	-	-	10.0	0.30	-1.00	12.7 #	-31.1
25sf	1	-	-	25.0	0.26	-0.96	11.6 #	-30.1
50sf	1	-	-	50.0	0.23	-0.93	10.8 #	-29.3
>100sf	1	-	-	100.0	0.20	-0.90	10.0 #	-28.5
<10sf	2	-	-	10.0	0.90	-1.80	28.5	-52.2
25sf	2	-	-	25.0	0.84	-1.52	26.8	-44.9
50sf	2	-	-	50.0	0.79	-1.31	25.5	-39.3
>100sf	2	-	-	100.0	0.74	-1.10	24.3	-33.7
<10sf	3	-	-	10.0	0.90	-1.80	28.5	-52.2
25sf	3	-	-	25.0	0.84	-1.52	26.8	-44.9
50sf	3	-	-	50.0	0.79	-1.31	25.5	-39.3

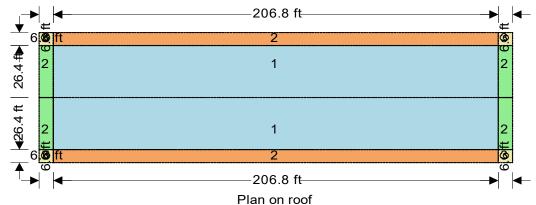


 Project Balderston Auto
 Job Ref. 20-467

 Calc. by RJS
 Chk'd by
 Date 3/1/2021

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft <sup>2</sup> )	+GC <sub>p</sub>	-GCp	Pres (+ve) (psf)	Pres (-ve) (psf)
>100sf	3	-	-	100.0	0.74	-1.10	24.3	-33.7

# The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction





Project Balderston A	uto P	Project No. 20-467
Calc. By_RS	Checked By_JB	Date_01/2/2020

# **Summary**

The gravity structure system of the project referenced above consists primarily of steel joists and girders, load-bearing CMU walls and steel columns. Loading criteria and depth criteria are shown for all roof framing members. These members are to be designed by the Project joist manufacturer and reviewed by BSE. Interior steel framing is supported by steel columns. The locations of all roof framing and columns are indicated on the structural framing plans.

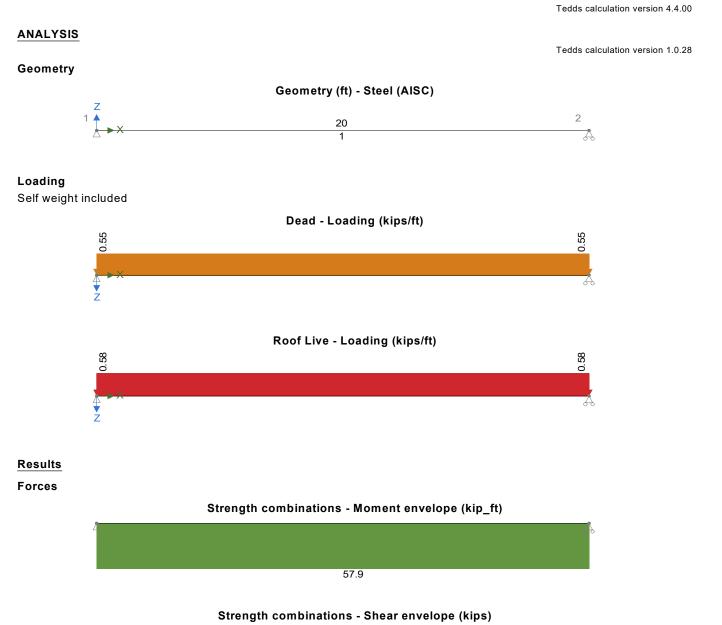
The following section of calculations covers the complete design of the roof framing system for project referenced above, including the design of the gravity columns and canopy framing. Refer to the "Loads" section of these calculations for the determination of all dead, live, roof live, and snow loads.



Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>2/10/2021</u>

#### **STEEL MEMBER ANALYSIS & DESIGN (AISC 360)**

In accordance with AISC360 14th Edition published 2010 using the ASD method







Project No. <u>20-467</u>

Calc. By <u>RJS</u>

Checked By\_JH

Flange thickness,  $t_{\rm f}$ , 0.345 in Web thickness,  $t_{\rm w}$ , 0.25 in Area of section, A, 7.7 in<sup>2</sup>

Radius of gyration about x-axis, r  $_{\rm x}$ , 6.26 in Radius of gyration about y-axis, r  $_{\rm y}$ , 1.12 in

Elastic section modulus about x-axis,  $S_x$ , 38.4 in<sup>3</sup> Elastic section modulus about y-axis,  $S_y$ , 3.49 in<sup>3</sup> Plastic section modulus about x-axis,  $Z_x$ , 44.2 in<sup>3</sup> Plastic section modulus about y-axis,  $Z_y$ , 5.48 in<sup>3</sup> Second moment of area about x-axis,  $I_x$ , 301 in<sup>4</sup>

Second moment of area about y-axis, I  $_{\rm y}^{*}$ , 9.59 in<sup>4</sup>

Safety factors				
Shear				Ω <sub>v</sub> = <b>1.67</b>
Flexure				$\Omega_{\rm b}$ = 1.67
Tensile yielding				Ω <sub>t,y</sub> = <b>1.67</b>
Tensile rupture				Ω <sub>t,r</sub> = <b>2.00</b>
Compression				Ωc = <b>1.67</b>
Beam design				
Section details				
Section type				W 16x26 (AISC 15th Edn (v15.0))
ASTM steel designation				A992
Steel yield stress				F <sub>y</sub> = <b>50</b> ksi
Steel tensile stress				Fu = <b>65</b> ksi
Modulus of elasticity				E = <b>29000</b> ksi
		H0.35"		
	Ť	<u>↓</u> ↑		W 16x26 (AISC 15th Edn (v15.0)) Section depth, d, 15.7 in Section breadth, $b_p$ 5.5 in Weight of section, Weight, 26 lbf/ft

#### Lateral restraint

Top flange has lateral restraint at supports plus 6ft, 12ft & 18ft Bottom flange has lateral restraint at supports only

▶ 10.35

4

15.7

#### Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)				
Width to thickness ratio	bf / (2 × tf) = <b>7.97</b>			
Limiting ratio for compact section	$\lambda_{\text{pff}}$ = 0.38 × $\sqrt{[E / F_y]}$ = 9.15			
Limiting ratio for non-compact section	$\lambda_{rff}$ = 1.0 × $\sqrt{[E / F_y]}$ = 24.08	Compact		
Classification of web in flexure - Table B4.1b (case 15)				
Width to thickness ratio	(d - 2 × k) / t <sub>w</sub> = <b>56.82</b>			
Limiting ratio for compact section	$\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} = 90.55$			

-0.25"

-5.5"------

->

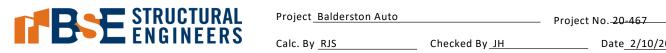
Limiting ratio for compact section	$\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} = 90.55$	
Limiting ratio for non-compact section	$\lambda_{rwf}$ = 5.70 × $\sqrt{[E / F_y]}$ = <b>137.27</b>	Compact

Section is compact in flexure

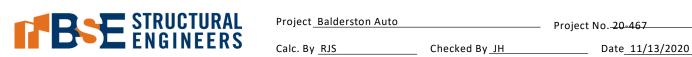


Check design at start of span	
Design of members for shear - Chapter G	
Required shear strength	V <sub>r,x</sub> = <b>11.6</b> kips
Web area	$A_w = d \times t_w = 3.925 in^2$
Web plate buckling coefficient	k <sub>v</sub> = 5
	(d - 2 × k) / t <sub>w</sub> > 2.24 × $\sqrt{(E / F_y)}$
Web shear coefficient - eq G2-3	C <sub>v</sub> = <b>1.000</b>
Nominal shear strength - eq G2-1	$V_{n,x}$ = 0.6 × F <sub>y</sub> × A <sub>w</sub> × C <sub>v</sub> = <b>117.8</b> kips
Safety factor	$\Omega_{\rm V} = 1.67$
Allowable shear strength	V <sub>c,x</sub> = V <sub>n,x</sub> / Ω <sub>v</sub> = <b>70.5</b> kips
	V <sub>r,x</sub> / V <sub>c,x</sub> = <b>0.164</b>
	PASS - Allowable shear strength exceeds required shear strength
Check design 10ft along span	
Design of members for flexure - Chapter F	
Required flexural strength	M <sub>r,x</sub> = <b>57.9</b> kips_ft
Yielding - Section F2.1	
Nominal flexural strength for yielding - eq F2-1	$M_{n,yld,x} = M_{p,x} = F_y \times Z_x = 184.2 \text{ kips_ft}$
Lateral-torsional buckling - Section F2.2 Unbraced length	L <sub>b</sub> = <b>6</b> ft
-	$L_p = 1.76 \times r_y \times \sqrt{(E / F_y)} = 3.956 \text{ ft}$
Limiting unbraced length for yielding - eq F2-5	$L_p = 1.76 \times 19 \times 1(E 7 Fy) = 3.936 ft$ h <sub>o</sub> = <b>15.4</b> in
Distance between flange centroids	c = 1
	r <sub>ts</sub> = <b>1.38</b> in
Limiting unbraced length for inelastic LTB - eg E2-f	$5 \text{ Lr} = 1.95 \times \text{rt}_{\text{s}} \times \text{E} / (0.7 \times \text{F}_{\text{y}}) \times \sqrt{((J \times \text{c} / (S_{\text{x}} \times \text{h}_{\text{o}})) + \sqrt{((J \times \text{c} / (S_{\text{x}} \times \text{h})) + \sqrt{((J \times \text{c} / (S_{\text{x}} \times \text{h}$
	$h_{0}))^{2} + 6.76 \times (0.7 \times F_{y} / E)^{2})) = 11.167 \text{ ft}$
Moment at quarter point of segment	$M_A = 54.2$ kips ft
Moment at center-line of segment	$M_B = 57.3 \text{ kips_ft}$
Moment at three quarter point of segment	Mc = <b>57.7</b> kips_ft
Maximum moment in segment	$M_{max} = 57.9$ kips ft
LTB modification factor - eq F1-1	$C_b = 12.5 \times M_{max} / (2.5 \times M_{max} + 3 \times M_A + 4 \times M_B + 3 \times M_c) = 1.019$
Nominal flexural strength for lateral-torsional buckli	
-	$M_{n,ltb,x} = min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x})$
	= 166.8 kips_ft
Allowable flexural strength - F1	
Nominal flexural strength	M <sub>n,x</sub> = min(M <sub>n,yld,x</sub> , M <sub>n,ltb,x</sub> ) = <b>166.8</b> kips_ft
Allowable flexural strength	M <sub>c,x</sub> = M <sub>n,x</sub> / Ω <sub>b</sub> = <b>99.9</b> kips_ft
	M <sub>r,x</sub> / M <sub>c,x</sub> = <b>0.580</b>
P	ASS - Allowable flexural strength exceeds required flexural strength
Check design 10ft along span	
Design of members for x-x axis deflection	
Maximum deflection	δ <sub>x</sub> = <b>0.493</b> in
Allowable deflection	$\delta_{x,Allowable} = L_{m1_{s1}} / 360 = 0.667$ in
	$\delta_x / \delta_{x,Allowable} = 0.74$

PASS - Allowable deflection exceeds design deflection



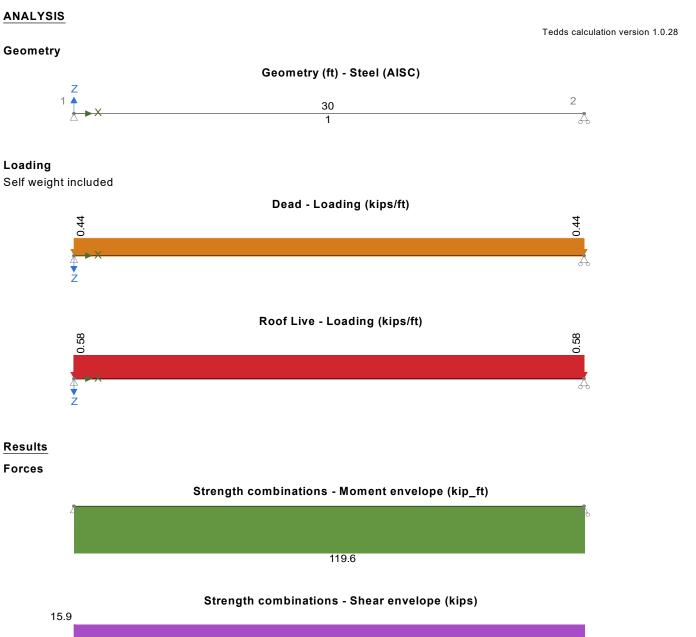
Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>2/10/2021</u>



Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>11/13/2020</u>

#### **STEEL MEMBER ANALYSIS & DESIGN (AISC 360)**

In accordance with AISC360 14th Edition published 2010 using the ASD method





Tedds calculation version 4.4.00



Project No.-20-467

Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>11/13/2020</u>

Safatu faatara				
Safety factors Shear				Ω <sub>v</sub> = <b>1.67</b>
Flexure				$\Omega_{\rm b} = 1.67$
Tensile yielding				$\Omega_{t,y} = 1.67$
Tensile rupture				$\Omega_{t,r} = 2.00$
Compression				$\Omega_{c} = 1.67$
Beam design				
Section details				
Section type				W 21x48 (AISC 15th Edn (v15.0))
ASTM steel designation				A992
Steel yield stress				F <sub>y</sub> = <b>50</b> ksi
Steel tensile stress				F <sub>u</sub> = <b>65</b> ksi
Modulus of elasticity				E = <b>29000</b> ksi
		"€+-0.43"		
	▲	¥		W 21x48 (AISC 15th Edn (v15.0))
		т	1	Section depth, d, 20.6 in Section breadth, b, 8.14 in
				Weight of section, Weight, 48 lbf/ft
				Flange thickness, $t_{p}$ , 0.43 in Web thickness, $t_{w}$ , 0.35 in
				Area of section, A, 14.1 in <sup>2</sup> Radius of gyration about x-axis, r <sub>y</sub> , 8.24 in
				Radius of gyration about y-axis, $r_{y}$ , 1.66 in
	-20.6			Elastic section modulus about x-axis, S $_x$ , 93 in <sup>3</sup> Elastic section modulus about y-axis, S $_y$ , 9.52 in <sup>3</sup>
				Plastic section modulus about x-axis, $Z_{x'}$ , 107 in <sup>3</sup>
			→ -0.35"	Plastic section modulus about y-axis, $Z_y$ , 14.9 in <sup>3</sup> Second moment of area about x-axis, $I_x$ , 959 in <sup>4</sup>
				Second moment of area about y-axis, I $_{\rm y},38.7$ in $^{\rm 4}$
		0.43"		
	<b>↓</b>	⊥ ⊥		
		T		

-8.14"-----

#### Lateral restraint

Top flange has lateral restraint at supports plus 6ft, 12ft & 18ft Bottom flange has lateral restraint at supports only

#### Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4	l.1b (case 10)			
Width to thickness ratio	bf / (2 × tf) = <b>9.47</b>			
Limiting ratio for compact section	$\lambda_{\text{pff}}$ = 0.38 $\times$ $\sqrt{[\text{E} / F_y]}$ = 9.15			
Limiting ratio for non-compact section	$\lambda_{rff}$ = 1.0 × $\sqrt{[E / F_y]}$ = 24.08	Noncompact		
Classification of web in flexure - Table B4.1b (case 15)				
Width to thickness ratio	(d - 2 × k) / t <sub>w</sub> = <b>53.54</b>			
Limiting ratio for compact section	$\lambda_{pwf}$ = 3.76 × $\sqrt{[E / F_y]}$ = 90.55			

Section is noncompact in flexure



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Design of members for shear - Chapter G	
Required shear strength	V <sub>r,x</sub> = <b>15.9</b> kips
Web area	$A_{w} = d \times t_{w} = 7.21 in^{2}$
Web plate buckling coefficient	k <sub>v</sub> = <b>5</b>
	$(d - 2 \times k) / t_w \le 2.24 \times \sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C <sub>v</sub> = <b>1.000</b>
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 \times F_y \times A_w \times C_v = 216.3$ kips
Safety factor	$\Omega_{\rm v}$ = 1.50
Allowable shear strength	V <sub>c,x</sub> = V <sub>n,x</sub> / Ω <sub>v</sub> = <b>144.2</b> kips
	V <sub>r,x</sub> / V <sub>c,x</sub> = <b>0.111</b>
	PASS - Allowable shear strength exceeds required shear strength

#### Check design 15ft along span

M <sub>r,x</sub> = <b>119.6</b> kips_ft
$M_{p,x} = F_y \times Z_x = 445.8 \text{ kips_ft}$
$L_{b} = 6 ft$
$L_p = 1.76 \times r_y \times \sqrt{(E / F_y)} = 5.863 \text{ ft}$
h <sub>o</sub> = <b>20.2</b> in
c = 1
rts = <b>2.05</b> in
$L_r = 1.95 \times r_{ts} \times E \ / \ (0.7 \times F_y) \times ((J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(J \times c \ / \ (S_x \times h_o)) + \sqrt{(S_x \times h_o) + \sqrt{(S_x \times h_o)) + \sqrt{(S_x \times h_o) + \sqrt$
h₀)) <sup>2</sup> + 6.76 × (0.7 × F <sub>y</sub> / E) <sup>2</sup> )) = <b>16.548</b> ft
MA = 118.4 kips_ft
MB = <b>119.6</b> kips_ft
Mc = <b>118.4</b> kips_ft
M <sub>max</sub> = <b>119.6</b> kips_ft
$C_{\text{b}} = 12.5 \times M_{\text{max}} / (2.5 \times M_{\text{max}} + 3 \times M_{\text{A}} + 4 \times M_{\text{B}} + 3 \times M_{\text{C}}) = 1.005$
ng - eq F2-2

Compression flange local buckling - Section F3.2

# $\lambda = b_f / (2 \times t_f) = 9.465$

= 445.7 kips\_ft

 $M_{n,ltb,x} = min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x})$ 

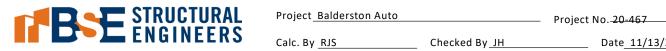
Nominal flexural strength for compression flange local buckling - eq F3-1

	0		0	5 1
				$M_{n,fib,x} = M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (\lambda - \lambda_{pff}) / (\lambda_{rff} - \lambda_{pff}) = 442.2 \text{ kips_ft}$
Allowable flexur	al strength	- F1		
Nominal flexural s	strength			M <sub>n,x</sub> = min(M <sub>n,Itb,x</sub> , M <sub>n,Itb,x</sub> ) = <b>442.2</b> kips_ft
Allowable flexural	strength			M <sub>c,x</sub> = M <sub>n,x</sub> / Ω <sub>b</sub> = <b>264.8</b> kips_ft
				Mr,x / Mc,x = <b>0.452</b>
			F	ASS - Allowable flexural strength exceeds required flexural strength

#### Check design 15ft along span

Design of members for x-x axis deflection Maximum deflection Allowable deflection

 $\delta_x = 0.714$  in  $\delta_{x,\text{Allowable}}$  =  $L_{m1\_s1}$  / 360 = 1 in



Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>11/13/2020</u>

 $\delta_x / \delta_{x,Allowable} = 0.714$ 

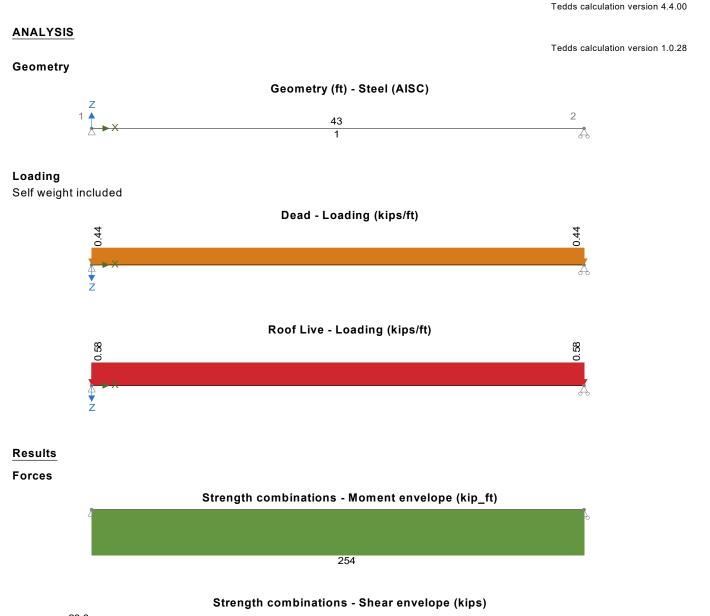
PASS - Allowable deflection exceeds design deflection



Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>11/13/2020</u>

### **STEEL MEMBER ANALYSIS & DESIGN (AISC 360)**

In accordance with AISC360 14th Edition published 2010 using the ASD method







Project Balderston Auto Project No.-20-467

Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>11/13/2020</u>

Safety factorsShear $\Omega_v = 1.67$ Flexure $\Omega_b = 1.67$ Tensile yielding $\Omega_{1y} = 1.67$ Tensile rupture $\Omega_{1r} = 2.00$ Compression $\Omega_c = 1.67$ Beam designSection detailsSection detailsSection typeSteel designationA992Steel yield stressFy = 50 ksiSteel tensile stressFu = 65 ksiModulus of elasticityE = 29000 ksiV 24x84 (AISC 15th Edn (v15.0))Section depth. d. 24.1 in Section breath, b. 902 in Veithickness. t., 0.47 in Area of section, Weight, 84 Ibfth Flange thickness. t., 0.47 in Area of section, Weight, 84 Jbfth Flange thickness. t., 0.47 in Area of section, Weight, 84 Jbfth Flange thickness. t., 0.47 in Area of section, Weight, 84 Jbfth Flange thickness. t., 0.47 in Area of section, Weight, 84 Jbfth Flange thickness. t., 0.47 in Area of section, Weight, 84 Jbfth Flange thickness. t., 0.27 in Radius of gration about yaxis. S., 198 in Blastic section modulus about yaxis. S., 208 in Second moment of area about yaxis. J., 244 in* Second moment of area about yaxis. J., 244 in*				
Flexure $\Omega_b = 1.67$ Tensile yielding $\Omega_{ty} = 1.67$ Tensile rupture $\Omega_{tr} = 2.00$ Compression $\Omega_c = 1.67$ Beam designSection detailsSection detailsSection typeSection typeW 24x84 (AISC 15th Edn (v15.0))ASTM steel designationA992Steel yield stressFy = 50 ksiSteel tensile stressFu = 65 ksiModulus of elasticityE = 29000 ksiV 24x84 (AISC 15th Edn (v15.0))Section depth, d, 24.1 in Section depth, d, 24.1 in Section A24.7 in Area of section, A24.7 in Area of section about xaxis, r., 9.79 in Radius of gyration about xaxis, r., 9.79 in Radius about xaxis, r., 9.79 in Radius about xaxis, r., 9.70 in Plastic section modulus about xaxis, r., 2.224 in <sup>3</sup> Plastic section modulus about xaxis, r., 2.230 in <sup>3</sup>	Safety factors			
Tensile yielding Tensile rupture Compression $\Omega_{c} = 1.67$ Beam design Section details Section details Section type ASTM steel designation Steel yield stress Steel tensile stress Modulus of elasticity V 24x84 (AISC 15th Edn (v15.0)) ASTM steel designation Steel yield stress Fu = 65 ksi E = 29000 ksi V 24x84 (AISC 15th Edn (v15.0)) Section depth, d, 24.1 in Section breadth, b <sub>x</sub> , 902 in Weight of section, Weight, 24 lbf/ft Flange thickness, t <sub>w</sub> , 0.77 in We bitickness, t <sub>w</sub> , 0.77 in We bitickness, t <sub>w</sub> , 0.77 in Neb thickness, t <sub>w</sub> , 0.79 in Radius of gyration about yeaks, s <sub>x</sub> , 196 in <sup>3</sup> Elastic section modulus about yeaks, s <sub>x</sub> , 2.224 in <sup>3</sup> Plastic section modulus about yeaks, s <sub>x</sub> , 2.224 in <sup>3</sup> Plastic section modulus about yeaks, s <sub>x</sub> , 2.24 in <sup>3</sup> Second moment of area about xeaks, t <sub>x</sub> , 2.270 in <sup>4</sup>	Shear			Ω <sub>v</sub> = <b>1.67</b>
Tensile rupture $\Omega_{t,r} = 2.00$ Compression $\Omega_c = 1.67$ Beam designSection detailsSection detailsW 24x84 (AISC 15th Edn (v15.0))ASTM steel designationA992Steel yield stress $F_y = 50$ ksiSteel tensile stress $F_u = 65$ ksiModulus of elasticityE = 29000 ksiVV24x84 (AISC 15th Edn (v15.0))Section depth, d, 24.1 in Section breadth, b, 9.02 in Weight of section, Weight, 84 lbf/ft Flange tickness, $t_v$ , 0.77 in Web tickness, $t_v$ , 0.77 in Web tickness, $t_v$ , 0.47 in Area of section, A24.7 in2 Radius of gyration about x-axis, $r_v$ , 9.99 in Radius of gyration about x-axis, $r_v$ , 1.95 in Elastic section modulus about x-axis, $r_v$ , 2.24 in3 Plastic section modulus about x-axis, $r_v$ , 2.24 in3 Pla	Flexure			Ω <sub>b</sub> = <b>1.67</b>
Compression $\Omega_c = 1.67$ Beam designSection detailsSection detailsSection typeW 24x84 (AISC 15th Edn (v15.0))ASTM steel designationA992Steel yield stressFy = 50 ksiSteel tensile stressFu = 65 ksiModulus of elasticityW 24x84 (AISC 15th Edn (v15.0))Section depti, d, 24.1 in Section depti, d, 24.1 in Section type of section Weight of section Weight, 84 lbf/th Flange thickness, t_u, 0.47 in Area of section, A, 24.7 in <sup>2</sup> Radius of gyration about x-axis, r_u, 195 in Elastic section modulus about x-axis, s_u, 196 in <sup>3</sup> Elastic section modulus about x-axis, z_u, 224 in <sup>3</sup> Plastic section modulus about x-axis, z_u, 32.6 in <sup>3</sup> Second moment of area about x-axis, z_u, 32.6 in <sup>3</sup> Second moment of area about x-axis, z_u, 23.0 in <sup>4</sup>	Tensile yielding			Ω <sub>t,y</sub> = <b>1.67</b>
Beam designSection detailsSection typeW 24x84 (AISC 15th Edn (v15.0))ASTM steel designationA992Steel yield stress $F_y = 50 \text{ ksi}$ Steel tensile stress $F_u = 65 \text{ ksi}$ Modulus of elasticity $E = 29000 \text{ ksi}$ W 24x84 (AISC 15th Edn (v15.0))M 24x84 (AISC 15th Edn (v15.0))Section depth, d, 24.1 in Section breadth, b, 9.02 in Weight of section, Weight, 84 lbi/ft Flange thickness, t_u, 0.77 in Web thickness, t_u, 0.47 in Area of section modulus about x-axis, s_v, 1.95 in Elastic section modulus about x-axis, s_v, 2.92 in a Plastic section modulus about x-axis, s_v, 2.92 in a Plastic section modulus about x-axis, s_v, 2.22 in a Plastic section modulus about x-axis, s_v, 2.23 in a Second moment of area about x-axis, 1, 2.370 in4	Tensile rupture			Ω <sub>t,r</sub> = <b>2.00</b>
Section detailsSection typeW 24x84 (AISC 15th Edn (v15.0))ASTM steel designationA992Steel yield stressFy = 50 ksiSteel tensile stressFu = 65 ksiModulus of elasticityE = 29000 ksiW 24x84 (AISC 15th Edn (v15.0))Section depth, d, 24.1 in Section depth, d, 24.1 in Section breadth, br, 9.02 in Weight of section, Weight, 84 lb/ft Flange thickness, tr, 0.77 in Web thickness, tr, 0.77 in Area of section, X, 24.7 in² Radius of gyration about x-axis, r, 9.79 in Radius of gyration about x-axis, r, 1.95 in Elastic section modulus about x-axis, s, 196 in³ Elastic section modulus about x-axis, s, 2.24 in³ Plastic section modulus about x-axis, t, 230 in³	Compression			Ωc = <b>1.67</b>
Section type Section type ASTM steel designation Steel yield stress Steel tensile stress Modulus of elasticity $F_y = 50$ ksi $F_u = 65$ ksi E = 29000 ksi E = 29000 ksi $F_u = 65$ ksi $F_u =$	Beam design			
ASTM steel designation Steel yield stress Steel tensile stress Modulus of elasticity $F_y = 50 \text{ ksi}$ $F_u = 65 \text{ ksi}$ E = 29000  ksi W 24x84 (AISC 15th Edn (v15.0)) Section depth, d, 24.1 in Section breath, b <sub>y</sub> .02 in Weight of section, Weight, 84 lbf/ft Flange thickness, t <sub>w</sub> . 0.47 in Area of section, A, 24.7 in <sup>2</sup> Radius of gyration about x-axis, r <sub>w</sub> . 9.79 in Radius of gyration about x-axis, s <sub>w</sub> . 196 in <sup>3</sup> Elastic section modulus about y-axis, s <sub>w</sub> . 20 in <sup>3</sup> Plastic section modulus about y-axis, s <sub>w</sub> . 20 in <sup>3</sup> Plastic section modulus about y-axis, s <sub>w</sub> . 2370 in <sup>4</sup>	Section details			
Steel yield stress $F_y = 50 \text{ ksi}$ Steel tensile stress $F_u = 65 \text{ ksi}$ Modulus of elasticity $E = 29000 \text{ ksi}$ Image: tensile stress $F_u = 65 \text{ ksi}$ Modulus of elasticity $E = 29000 \text{ ksi}$ Image: tensile stress $F_u = 65 \text{ ksi}$ <td>Section type</td> <td></td> <td></td> <td>W 24x84 (AISC 15th Edn (v15.0))</td>	Section type			W 24x84 (AISC 15th Edn (v15.0))
Steel tensile stress $F_u = 65 \text{ ksi}$ Modulus of elasticity $E = 29000 \text{ ksi}$ $\underbrace{F_u} = 65 \text{ ksi}$ $\underbrace{F_u} = 29000 \text{ ksi}$ $\underbrace{F_u} = 65 \text{ ksi}$ $\underbrace{F_u} = 29000 \text{ ksi}$	ASTM steel designation			A992
Modulus of elasticity E = 29000  ksi $W 24x84  (AISC 15th Edn (v15.0))$ Section depth, d, 24.1 in Section breadth, b <sub>0</sub> .9.02 in Weight of section, Weight, 84 lbf/ft Flange thickness, t <sub>n</sub> . 0.77 in Web thickness, t <sub>n</sub> . 0.47 in Area of section, A, 24.7 in <sup>2</sup> Radius of gyration about x-axis, r <sub>x</sub> . 9.79 in Radius of gyration about x-axis, r <sub>x</sub> . 9.79 in Elastic section modulus about x-axis, s <sub>x</sub> . 196 in <sup>3</sup> Elastic section modulus about x-axis, s <sub>x</sub> . 209 in <sup>3</sup> Plastic section modulus about y-axis, s <sub>x</sub> . 224 in <sup>3</sup> Second moment of area about x-axis, 1 <sub>x</sub> . 2370 in <sup>4</sup>	Steel yield stress			F <sub>y</sub> = <b>50</b> ksi
W 24x84 (AISC 15th Edn (v15.0)) Section depth, d, 24.1 in Section breadth, b, 9.02 in Weight of section, Weight, 84 lbf/ft Flange thickness, t <sub>w</sub> . 0.77 in Web thickness, t <sub>w</sub> . 0.77 in Web thickness, t <sub>w</sub> . 0.47 in Area of section, A, 24.7 in <sup>2</sup> Radius of gyration about x-axis, r <sub>x</sub> . 9.79 in Radius of gyration about x-axis, s <sub>x</sub> . 196 in <sup>3</sup> Elastic section modulus about x-axis, S <sub>x</sub> , 209 in <sup>3</sup> Plastic section modulus about y-axis, S <sub>x</sub> , 224 in <sup>3</sup> Plastic section modulus about y-axis, S <sub>x</sub> , 230 in <sup>4</sup>	Steel tensile stress			Fu = <b>65</b> ksi
<ul> <li>★ United (allocation)</li> <li>Section depth, d, 24.1 in Section breadth, b<sub>1</sub>, 9.02 in Weight of section, Weight, 84 lbf/ft Flange thickness, t<sub>w</sub>, 0.77 in Web thickness, t<sub>w</sub>, 0.47 in Area of section, A, 24.7 in<sup>2</sup> Radius of gyration about x-axis, r<sub>x</sub>, 9.79 in Radius of gyration about x-axis, r<sub>x</sub>, 9.79 in Elastic section modulus about x-axis, S<sub>x</sub>, 196 in<sup>3</sup> Elastic section modulus about y-axis, S<sub>x</sub>, 20.9 in<sup>3</sup> Plastic section modulus about y-axis, Z<sub>y</sub>, 32.6 in<sup>3</sup> Second moment of area about x-axis, I<sub>x</sub>, 2370 in<sup>4</sup></li> </ul>	Modulus of elasticity			E = <b>29000</b> ksi
<ul> <li>★ United (allocation)</li> <li>Section depth, d, 24.1 in Section breadth, b<sub>1</sub>, 9.02 in Weight of section, Weight, 84 lbf/ft Flange thickness, t<sub>w</sub>, 0.77 in Web thickness, t<sub>w</sub>, 0.47 in Area of section, A, 24.7 in<sup>2</sup> Radius of gyration about x-axis, r<sub>x</sub>, 9.79 in Radius of gyration about x-axis, r<sub>x</sub>, 9.79 in Elastic section modulus about x-axis, S<sub>x</sub>, 196 in<sup>3</sup> Elastic section modulus about y-axis, S<sub>x</sub>, 20.9 in<sup>3</sup> Plastic section modulus about y-axis, Z<sub>y</sub>, 32.6 in<sup>3</sup> Second moment of area about x-axis, I<sub>x</sub>, 2370 in<sup>4</sup></li> </ul>		77.0		
Section breadth, b <sub>r</sub> , 9.02 in Weight of section, Weight, 84 lbf/ft Flange thickness, t <sub>w</sub> , 0.77 in Web thickness, t <sub>w</sub> , 0.47 in Area of section, A, 24.7 in <sup>2</sup> Radius of gyration about x-axis, r <sub>x</sub> , 9.79 in Radius of gyration about x-axis, s <sub>x</sub> , 196 in <sup>3</sup> Elastic section modulus about y-axis, S <sub>x</sub> , 196 in <sup>3</sup> Elastic section modulus about y-axis, S <sub>y</sub> , 20.9 in <sup>3</sup> Plastic section modulus about y-axis, Z <sub>x</sub> , 224 in <sup>3</sup> Plastic section modulus about y-axis, I <sub>x</sub> , 2370 in <sup>4</sup>		$\overline{+}$		W 24x84 (AISC 15th Edn (v15.0))
Weight of section, Weight, 84 lbf/ft         Flange thickness, t <sub>x</sub> , 0.77 in         Web thickness, t <sub>x</sub> , 0.47 in         Area of section, A, 24.7 in <sup>2</sup> Radius of gyration about x-axis, r <sub>x</sub> , 9.79 in         Radius of gyration about x-axis, r <sub>x</sub> , 1.95 in         Elastic section modulus about y-axis, S <sub>x</sub> , 196 in <sup>3</sup> Elastic section modulus about y-axis, S <sub>x</sub> , 20.9 in <sup>3</sup> Plastic section modulus about y-axis, S <sub>x</sub> , 2.24 in <sup>3</sup> Plastic section modulus about y-axis, S <sub>x</sub> , 2.32.6 in <sup>3</sup> Second moment of area about x-axis, I <sub>x</sub> , 2.370 in <sup>4</sup>		1	)(	
Web thickness, t <sub>w</sub> , 0.47 in Area of section, A, 24.7 in <sup>2</sup> Radius of gyration about x-axis, r <sub>x</sub> , 9.79 in Radius of gyration about y-axis, r <sub>y</sub> , 1.95 in Elastic section modulus about y-axis, S <sub>x</sub> , 196 in <sup>3</sup> Elastic section modulus about x-axis, S <sub>y</sub> , 20.9 in <sup>3</sup> Plastic section modulus about y-axis, S <sub>y</sub> , 22.0 in <sup>3</sup> Plastic section modulus about y-axis, Z <sub>y</sub> , 32.6 in <sup>3</sup> Second moment of area about x-axis, I <sub>y</sub> , 2370 in <sup>4</sup>				Weight of section, Weight, 84 lbf/ft
Radius of gyration about x-axis, r <sub>x</sub> , 9.79 in Radius of gyration about y-axis, r <sub>y</sub> , 1.95 in Elastic section modulus about y-axis, S <sub>y</sub> , 196 in <sup>3</sup> Elastic section modulus about y-axis, S <sub>y</sub> , 20.9 in <sup>3</sup> Plastic section modulus about x-axis, Z <sub>y</sub> , 224 in <sup>3</sup> Plastic section modulus about y-axis, Z <sub>y</sub> , 32.6 in <sup>3</sup> Second moment of area about x-axis, I <sub>x</sub> , 2370 in <sup>4</sup>				
Radius of gyration about y-axis, r <sub>y</sub> , 1.95 in Elastic section modulus about x-axis, S <sub>y</sub> , 196 in <sup>3</sup> Elastic section modulus about x-axis, S <sub>y</sub> , 20.9 in <sup>3</sup> Plastic section modulus about x-axis, Z <sub>y</sub> , 224 in <sup>3</sup> Plastic section modulus about y-axis, Z <sub>y</sub> , 32.6 in <sup>3</sup> Second moment of area about x-axis, I <sub>x</sub> , 2370 in <sup>4</sup>				
Elastic section modulus about x-axis, S <sub>x</sub> , 196 in <sup>3</sup> Elastic section modulus about y-axis, S <sub>y</sub> , 20.9 in <sup>3</sup> Plastic section modulus about y-axis, S <sub>y</sub> , 224 in <sup>3</sup> Plastic section modulus about y-axis, Z <sub>y</sub> , 32.6 in <sup>3</sup> Second moment of area about x-axis, I <sub>x</sub> , 2370 in <sup>4</sup>				
Plastic section modulus about x-axis, $Z_x$ , 224 in <sup>3</sup> $\rightarrow$ Plastic section modulus about y-axis, $Z_y$ , 32.6 in <sup>3</sup> Second moment of area about x-axis, $I_x$ , 2370 in <sup>4</sup>		24.1		Elastic section modulus about x-axis, S x, 196 in3
Second moment of area about x-axis, I <sub>x</sub> , 2370 in <sup>4</sup>				
			→ -0.47"	,
		↓ ¥		7
		<u>→</u> <b>→</b>		-

#### Lateral restraint

Top flange has full lateral restraint Bottom flange has lateral restraint at supports only

#### Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)						
Width to thickness ratio	bf / (2 × tf) = <b>5.86</b>					
Limiting ratio for compact section	$\lambda_{\text{pff}}$ = 0.38 $\times$ $\sqrt{[\text{E} / F_y]}$ = <b>9.15</b>					
Limiting ratio for non-compact section	$\lambda_{rff}$ = 1.0 × $\sqrt{[E / F_y]}$ = 24.08	Compact				
		Classification of web in flexure - Table B4.1b (case 15)				
Classification of web in flexure - Table B4.1b (c	ase 15)					
Classification of web in flexure - Table B4.1b (c Width to thickness ratio	ase 15) (d - 2 × k) / t <sub>w</sub> = 45.87					
· ·	,					

Section is compact in flexure



Calc. By RJS

Checked By JH Date 11/13/2020

#### Check design at start of span

Design of members for shear - Chapter G	
Required shear strength	V <sub>r,x</sub> = <b>23.6</b> kips
Web area	$A_w = d \times t_w = 11.327 \text{ in}^2$
Web plate buckling coefficient	k <sub>v</sub> = <b>5</b>
	(d - 2 $\times$ k) / tw <= 2.24 $\times$ $\sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C <sub>v</sub> = 1.000
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 \times F_y \times A_w \times C_v = 339.8$ kips
Safety factor	$\Omega_v = 1.50$
Allowable shear strength	V <sub>c,x</sub> = V <sub>n,x</sub> / Ω <sub>v</sub> = <b>226.5</b> kips
	V <sub>r,x</sub> / V <sub>c,x</sub> = <b>0.104</b>
	PASS - Allowable shear strength exceeds required shear strength

#### Check design 21ft 6in along span

**Design of members for flexure - Chapter F** Required flexural strength

**Yielding - Section F2.1** Nominal flexural strength for yielding - eq F2-1

Allowable flexural strength - F1 Nominal flexural strength

Allowable flexural strength

M<sub>r,x</sub> = **254** kips\_ft

 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x = 933.3 \text{ kips_ft}$ 

$$\begin{split} M_{n,x} &= M_{n,y|d,x} = \textbf{933.3 kips_ft} \\ M_{c,x} &= M_{n,x} \; / \; \Omega_{b} = \textbf{558.9 kips_ft} \\ M_{r,x} \; / \; M_{c,x} = \textbf{0.454} \end{split}$$

#### PASS - Allowable flexural strength exceeds required flexural strength

#### Check design 21ft 6in along span

**Design of members for x-x axis deflection** Maximum deflection Allowable deflection

 $\delta_x = 1.254$  in  $\delta_{x,\text{Allowable}} = L_{m1_{s1}} / 360 = 1.433$  in  $\delta_x / \delta_{x,\text{Allowable}} = 0.875$ 

PASS - Allowable deflection exceeds design deflection



Project	Balderston Auto			t No	20-467
Calc. By	RJS	Checked By	JH	Date	03/01/21

### FLEXURAL ANALYSIS OF ANGLES

# AISC "F10"

b d t L 5 x 5 x 3/8	Fy = <u>36</u> ksi
Cb =	1.00 (conservatively)
L =	48.00 in
Iz =	3.55 in <sup>4</sup>
rz =	0.99 in
t =	0.38 in
βw =	0.00 (see table C-F10.1 in commentary)
S =	2.41 in <sup>3</sup> (in direction of bending)

Mi	n/Ω =	5.57	K-ft	=	66.84	K-in	]
							_
IVI	n/12 = 1	33./11/1	K-IN	=	11.1426	κ-π	Eq. F10-8
							•
M	n/O = 6	7 412415	K-in	=	5.6177	K-ft	Eg. F10-7
4.) LEG LOC	AL BUCK	LING			NEED THI	S CALC	
M	n/Ω =	66.84	K-in	=	5.56982	K-ft	Eq. F10-2 & Eq. F10-3
	Me = 9	77.05078	K-in	=	81.4209	K-ft	Eq. F10-6
3.) LATERAL	TORSIO	NAL BUCK	LING			(using maj	or principal axis)
M	n/Ω =	73.59	K-in	=	6.13228	K-ft	Eq. F10-2 & Eq. F10-3
					354.53	-	Eq. F10-4a & Eq. F10-4b
	0	•			ension (C c	,	Т
	2.) LATERAL TORSIONAL BUCKLING				_		metric axis)
M	n/Ω =	77.93	K-in	=	6.49401	K-ft	Eq. F10-1
1.) YEILDING	G						



Calc. by <u>RS</u>

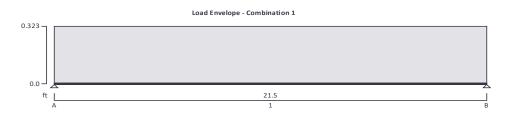
\_\_\_\_\_ Job Ref. 20-467\_\_\_\_

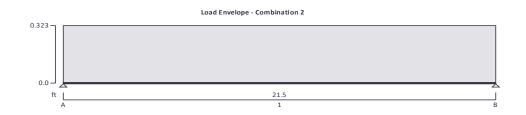
\_\_\_\_\_ Chk'd by \_\_\_\_\_ Date 2/8/2021

# STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

#### In accordance with AISC360-16 using the ASD method

Tedds calculation version 3.0.14





Load Envelope - Combination 3 0.653 -0.0 -21.5 ft 

Load Envelope - Combination 4 0.783 -

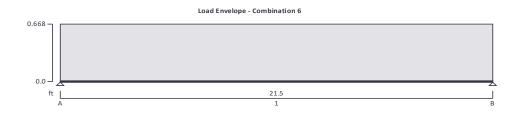


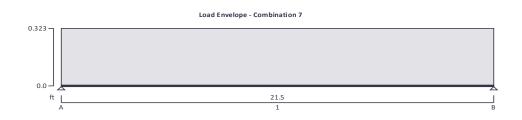
Load Envelope - Combination 5 0.571 -0.0 21.5 ft I 1



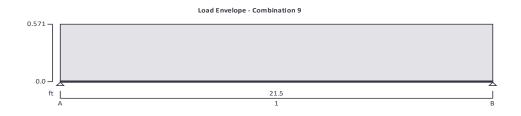
Job Ref. 20-467\_\_\_\_\_

Calc. by <u>RS</u> Chk'd by \_\_\_\_\_ Date\_<u>2/8/2021</u>





Load Envelope - Combination 8 0.323 <mark>-</mark> 0.0 21.5 ft 1



Load Envelope - Combination 10 0.668 0.0 21.5 ft L A 1 В

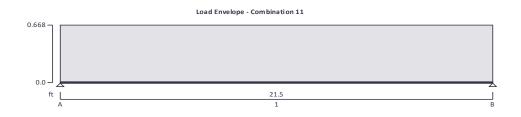
Sht. No. 2 B15 of 33 8

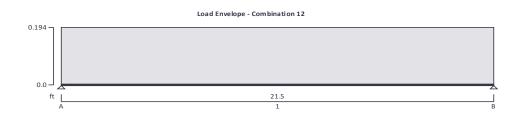


\_\_\_\_\_ Job Ref. 20-467\_

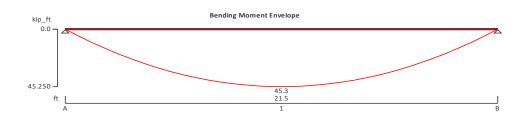
Calc. by <u>RS</u> Chk'd by \_\_\_\_\_

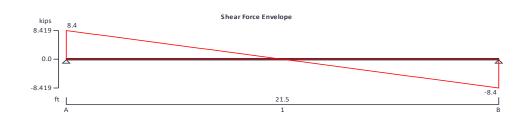
Date 2/8/2021





Load Envelope - Combination 13



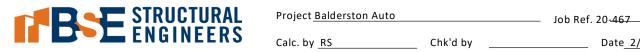


# Support conditions

Support A

Support B

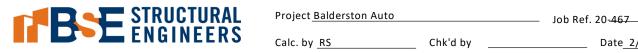
Vertically restrained Rotationally free Vertically restrained Rotationally free



Calc. by <u>RS</u> Chk'd by \_\_\_\_\_ Date\_<u>2/8/2021</u>

Applied loading		
Beam loads	Dead full UDL 0.297 kips/i	
	Snow full UDL 0.46 kips/ft	
	Roof live full UDL 0.33 kip	
	Dead self weight of beam	× 1
Load combinations		
Load combination 1 - D	Support A	$Dead \times 1.00$
		Dead  imes 1.00
	Support B	$Dead \times 1.00$
Load combination 2 - D + L	Support A	Dead  imes 1.00
		$Live \times 1.00$
		$Dead \times 1.00$
		$Live \times 1.00$
	Support B	Dead  imes 1.00
		$Live \times 1.00$
Load combination 3 - D + Lr	Support A	$Dead \times 1.00$
		Roof live $\times$ 1.00
		Dead  imes 1.00
		Roof live $\times$ 1.00
	Support B	Dead  imes 1.00
		Roof live $\times$ 1.00
Load combination 4 - D + S	Support A	$Dead \times 1.00$
		Snow × 1.00
		Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00
Load combination 5 - D+0.75L+0.75Lr	Support A	Dead × 1.00
		Live × 0.75
		Roof live $\times 0.75$
		Dead × 1.00
		Live × 0.75
		Roof live $\times 0.75$
	Support B	Dead $\times$ 1.00
		Live $\times$ 0.75
		Roof live $\times 0.75$
Load combination 6 - D+0.75L+0.75S	Support A	Dead $\times$ 1.00
	Support A	Live $\times$ 0.75
		Snow $\times$ 0.75
		Dead × 1.00
		Live $\times$ 0.75
	Support P	Snow × 0.75
	Support B	Dead × 1.00
		Live $\times$ 0.75
		Snow $\times$ 0.75
		Sht No. 4 -f C
		Sht. No. <u>4</u> B17 of <u>33</u> 8

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	Calc. by <u>RS</u>	Chk'd hy		
	Calc. by <u>KS</u>	Chk'd by	Date <u>2/8/2021</u>	
Load combination 7 - D+ 0.6W	Support A		$Dead \times 1.00$	
	oupportit		Wind $\times$ 0.60	
			Dead $\times$ 1.00	
			Wind $\times$ 0.60	
	Support B		Dead × 1.00	
			Wind $\times$ 0.60	
Load combination 8 - D+0.7E	Support A		$Dead \times 1.00$	
			Seismic × 0.70	
			Dead × 1.00	
			Seismic × 0.70	
	Support B		Dead $\times$ 1.00	
			Seismic × 0.70	
Load combination 9 - D+0.75L+0.75	5(0.6W)+0.75Lr Support A		Dead $\times$ 1.00	
	(0.01.) 00 <u>-</u> . 0		$Live \times 0.75$	
			Roof live $\times 0.75$	
			Wind $\times$ 0.45	
			Dead $\times$ 1.00	
			Live $\times$ 0.75	
			Roof live $\times 0.75$	
			Wind $\times$ 0.45	
	Support B		Dead $\times$ 1.00	
	oupport D		Live $\times$ 0.75	
			Roof live $\times 0.75$	
			Wind $\times 0.45$	
Load combination 10 - D+0.75L+0.7	75(0.6W)+0.75S Support A		Dead $\times$ 1.00	
	0(0.0W) 0.700 0upport /		Live $\times 0.75$	
			Snow × 0.75	
			Wind $\times 0.45$	
			Dead $\times$ 1.00	
			Live $\times$ 0.75	
			Snow $\times$ 0.75	
			Wind $\times$ 0.45	
	Support B		Dead $\times$ 1.00	
	Ouppoir D		Live $\times 0.75$	
			Snow $\times$ 0.75	
			Wind $\times$ 0.45	
Load combination 11 - D+0.75L+0.7	75(0.7E)+0.758 Support A		Dead $\times$ 1.00	
			Live $\times$ 0.75	
			Snow $\times$ 0.75	
			Seismic $\times$ 0.53	
			Dead $\times$ 1.00	
			Live $\times$ 0.75	
			Snow $\times$ 0.75	
			Seismic $\times$ 0.75	
	Support B		Dead $\times$ 1.00	
	Support B			
			Sht. No. 5 profess 8	
			Sht. No. <u>5</u> B18 of <u>33</u> 8	



Calc. by <u>RS</u> Chk'd by \_\_\_\_\_ Date\_<u>2/8/2021</u>

		Live $\times$ 0.75
		Snow $\times 0.75$
		Seismic × 0.53
Load combination 12 - 0.6D + 0.6W	Support A	Dead $\times$ 0.60
		Wind $\times$ 0.60
		$Dead \times 0.60$
		Wind $\times$ 0.60
	Support B	Dead  imes 0.60
		Wind $\times$ 0.60
Load combination 13 - 0.6D + 0.7E	Support A	$\text{Dead}\times 0.60$
		Seismic $ imes$ 0.70
		Dead  imes 0.60
		Seismic × 0.70
	Support B	Dead  imes 0.60
		Seismic $ imes$ 0.70
Analysis results		
Maximum moment	M <sub>max</sub> = <b>45.3</b> kips_ft	M <sub>min</sub> = <b>0</b> kips_ft
/laximum moment span 1 segment 1	M <sub>s1_seg1_max</sub> = <b>40.2</b> kips_ft	M <sub>s1_seg1_min</sub> = <b>0</b> kips_f
Maximum moment span 1 segment 2	Ms1_seg2_max = <b>45.3</b> kips_ft	Ms1_seg2_min = 0 kips_f
Maximum moment span 1 segment 3	Ms1_seg3_max = <b>40.2</b> kips_ft	Ms1_seg3_min = 0 kips_f
/laximum shear	V <sub>max</sub> = <b>8.4</b> kips	V <sub>min</sub> = <b>-8.4</b> kips
Maximum shear span 1 segment 1	V <sub>s1_seg1_max</sub> = <b>8.4</b> kips	Vs1_seg1_min = <b>0</b> kips
Maximum shear span 1 segment 2	V <sub>s1_seg2_max</sub> = <b>2.8</b> kips	Vs1_seg2_min = <b>-2.8</b> kips
Maximum shear span 1 segment 3	Vs1_seg3_max = 0 kips	Vs1_seg3_min = <b>-8.4</b> kips
Deflection segment 4	$\delta_{max} = 0.6$ in	$\delta_{min} = 0$ in
laximum reaction at support A	R <sub>A_max</sub> = <b>8.4</b> kips	RA_min = <b>2.1</b> kips
Jnfactored dead load reaction at support A	RA_Dead = 3.5 kips	
Infactored roof live load reaction at support A	RA_Roof live = 3.5 kips	
Infactored snow load reaction at support A	RA_Snow = <b>4.9</b> kips	
Maximum reaction at support B	R <sub>в_max</sub> <b>= 8.4</b> kips	R <sub>B_min</sub> = 2.1 kips
Unfactored dead load reaction at support B	R <sub>B_Dead</sub> = 3.5 kips	
Unfactored roof live load reaction at support B	R <sub>B_Roof</sub> live = <b>3.5</b> kips	
Jnfactored snow load reaction at support B	R <sub>B_Snow</sub> = 4.9 kips	
Section details		
Section type	W 16x26 (AISC 15th Edn (v1	5.0))
ASTM steel designation	A992	
Steel yield stress	F <sub>y</sub> = <b>50</b> ksi	
Steel tensile stress	Fu = <b>65</b> ksi	
Modulus of elasticity	E = <b>29000</b> ksi	

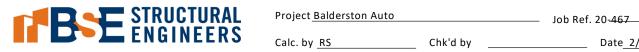


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ť	<b>◄</b> ——5.5"—— <b>→</b>			
	· ·			
Safety factors				
Safety factor for tensile yielding	$\Omega_{ty} = 1.67$			
Safety factor for tensile rupture	$\Omega_{\rm tr}$ = 2.00			
Safety factor for compression	Ω <sub>c</sub> = <b>1.67</b>			
Safety factor for flexure	Ω <sub>b</sub> = <b>1.67</b>			
Lateral bracing				
	Span 1 has lateral bracing at sup	ports plus third points		
Classification of sections for local buckling - Se	ection B4.1			
Classification of flanges in flexure - Table B4.1t	o (case 10)			
Width to thickness ratio	bf / (2 × tf) = <b>7.97</b>			
Limiting ratio for compact section	$\lambda_{\text{pff}}$ = 0.38 × $\sqrt{[E / F_y]}$ = 9.15			
Limiting ratio for non-compact section	$\lambda_{rff}$ = 1.0 × $\sqrt{[E / F_y]}$ = 24.08	Compact		
Classification of web in flexure - Table B4.1b (c	ase 15)			
Width to thickness ratio	(d - 2 × k) / t <sub>w</sub> = <b>56.82</b>			
Limiting ratio for compact section	$\lambda_{pwf}$ = 3.76 × $\sqrt{[E / F_y]}$ = 90.55			
Limiting ratio for non-compact section	$\lambda_{rwf}$ = 5.70 × $\sqrt{[E / F_y]}$ = <b>137.27</b>	Compact		
		Section is compact in flexure		
Design of members for shear - Chapter G				
Required shear strength	$V_r = max(abs(V_{max}), abs(V_{min})) = 8$	3 <b>.419</b> kips		
Web area	$A_w = d \times t_w = 3.925 \text{ in}^2$			
Web plate buckling coefficient	k <sub>v</sub> = <b>5.34</b>			
Web shear coefficient - eq G2-3	C <sub>v1</sub> = <b>1</b>			
Nominal shear strength – eq G6-1	$V_n = 0.6 \times F_y \times A_w \times C_{v1} = 117.750$	) kips		
Safety factor for shear	Ω <sub>v</sub> = <b>1.67</b>			
Allowable shear strength	$V_{c}$ = $V_{n}$ / $\Omega_{v}$ = <b>70.509</b> kips			
	PASS - Allowable shear streng	th exceeds required shear strength		
Design of members for flexure in the major axis at span 1 segment 2 - Chapter F				
Required flexural strength	Mr = max(abs(Ms1_seg2_max), abs(M	s1_seg2_min)) = <b>45.25</b> kips_ft		
Yielding - Section F2.1				
Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y \times Z_x = 184.167 \text{ kip}$	s_ft		



Calc. by <u>RS</u> Chk'd by \_\_\_\_\_ Date\_<u>2/8/2021</u>

Lateral-torsional buckling - Section F2.2	
Unbraced length	$L_{b} = L_{s1\_seg2} = 86$ in
Limiting unbraced length for yielding - eq F2-5	$L_p = 1.76 \times r_y \times \sqrt{[E / F_y]} = 47.473$ in
Distance between flange centroids	h₀ = d - t <sub>f</sub> = <b>15.355</b> in
-	c = 1
	$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]} = 1.385$ in
Limiting unbraced length for inelastic LTB - eq F2-	6
$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J = 1.05) \times 10^{-5}]}$	× c / (S <sub>x</sub> × h <sub>o</sub> )) + $\sqrt{((J × c / (S_x × h_o))^2 + 6.76 × (0.7 × F_y / E)^2)]} = 134.473$ in
Cross-section mono-symmetry parameter	R <sub>m</sub> = 1.000
Moment at quarter point of segment	M <sub>A</sub> = <b>43.993</b> kips_ft
Moment at center-line of segment	M <sub>B</sub> = <b>45.250</b> kips_ft
Moment at three quarter point of segment	Mc = <b>43.993</b> kips_ft
Maximum moment in segment	M <sub>abs</sub> = <b>45.250</b> kips_ft
Lateral torsional buckling modification factor - eq F	= 1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] =$
	1.014
Nominal flexural strength for lateral torsional buck	ling - eq F2-2 $M_{\text{nitb}} = C_b \times [M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)]$
	= <b>154.265</b> kips_ft
Nominal flexural strength	Mn = min(Mnyld, Mnltb) = <b>154.265</b> kips_ft
Allowable flexural strength	Mc = Mn / Ωb = <b>92.374</b> kips_ft
P	ASS - Allowable flexural strength exceeds required flexural strength
Design of members for vertical deflection	
Consider deflection due to dead, live, roof live, sno	ow, wind and seismic loads
Limiting deflection	δ <sub>lim</sub> = L <sub>s1</sub> / 360 = <b>0.717</b> in
Maximum deflection span 1	$\delta = max(abs(\delta_{max}), abs(\delta_{min})) = 0.613$ in
	PASS - Maximum deflection does not exceed deflection limit

#### Joists With Drift Along Length of Joist

Joists with Drift Along	<u>t Length of</u>	JOIST	
Dead Load=	15	psf Area	-
Balance Load=	20.4	psf	
Total Load=	35.4	psf (DL+Balance)	
Max. Drift Load	47	psf	
Length of Drift=	11	ft	
			L
Joist Space #1=	4.6	ft	
Joist Space #2=	4.6	ft	
Joist Space #3=	4.6	ft	
Joist Space #4=	4.6	ft	
Joist Space #5=	5.1	ft	
Joist Space #6=	5.1	ft	
Joist Space #7=	5.1	ft	
Pressure Load #1=	62.95909	psf	
Pressure Load #2=	43.51818	psf	
Pressure Load #3=	35.4	psf	
Pressure Load #4=	35.4	psf	
Pressure Load #5=	35.4	psf	
Pressure Load #6=	35.4	psf	
Uniform Load #1=	286.4639	lb/ft (Joist #1)	
Uniform Load #2=	198.0077	lb/ft (Joist #2)	
Uniform Load #3=	161.07	lb/ft (Joist #3)	
Uniform Load #4=	170.805	lb/ft (Joist #4)	
Uniform Load #5=	180 54	lb/ft (loist #5)	

K	Parape	et Wall						
	Max Drift I	Load= 47	PSF					
							Balanc	e Load + DL= 35.4 PSF
	4.55 <u>1</u>	4.55 <u>2</u>	4.55 <u>3</u>	5 4.55 <u>4</u>	5.1 <u>5</u>	5.1 <u>6</u>	5.1 <u>Z</u>	Joist Space (ft) Joist Space #
	t+ t+1	T# JSIDC	Joist #2	Joist #3	Joist #4	Joist #5	Joist #6	

UIIIUIIII LUdu #1-	200.4059	ID/IL (JOISL #1)
Uniform Load #2=	198.0077	lb/ft (Joist #2)
Uniform Load #3=	161.07	lb/ft (Joist #3)
Uniform Load #4=	170.805	lb/ft (Joist #4)
Uniform Load #5=	180.54	lb/ft (Joist #5)
Uniform Load #6=	180.54	lb/ft (Joist #6)

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Max Loading =	286.4639 lb/ft
Joist Span =	29 ft
Joist Selection =	24K4
Joist Capacity =	290 lb/ft
% Capacity =	98.781%

OK use 24K4

Interior Joists - Non-Drift Loaded Joists

#### Joists With Drift Along Length of Joist

Joists with Drift Along Length of Joist					
Dead Load=	15	psf Area -	-		
Balance Load=	20.4	psf			
Total Load=	35.4	psf (DL+Balance)			
Max. Drift Load	8	psf			
Length of Drift=	11.5	ft			
Joist Space #1=	6.0	ft			
Joist Space #2=	6.0	ft			
Joist Space #3=	6.0	ft			
Joist Space #4=	6.0	ft			
Joist Space #5=	6.0	ft			
Joist Space #6=	6.0	ft			
Joist Space #7=	6.0	ft			
Pressure Load #1=	39.22609	psf			
Pressure Load #2=	35.4	psf			
Pressure Load #3=	35.4	psf			
Pressure Load #4=	35.4	psf			
Pressure Load #5=	35.4	psf			
Pressure Load #6=	35.4	psf			
Uniform Load #1=	235.3565	lb/ft (Joist #1)			
Uniform Load #2=	212.4	lb/ft (Joist #2)			
Uniform Load #3=	212.4	lb/ft (Joist #3)			
Uniform Load #4=	212.4	lb/ft (Joist #4)			
Uniform Load #5=	212.4	lb/ft (Joist #5)			

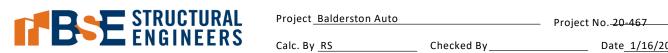
K	Parap	et Wall							
	Max Drift	Load= 8 PSI					Bal	ance Lo	pad + DL= 35.4 PSF
	6 <u>1</u>	6 2	6 <u>3</u>	6 <u>4</u>	6 5		6 6	6 <u>7</u>	Joist Space (ft) Joist Space #
		T# 1SIOL	7# 1310L	Joist #3	Joist #4	Joist #5	Joist #6		

Uniform Load #1=	235.3565	lb/ft (Joist #1)
Uniform Load #2=	212.4	lb/ft (Joist #2)
Uniform Load #3=	212.4	lb/ft (Joist #3)
Uniform Load #4=	212.4	lb/ft (Joist #4)
Uniform Load #5=	212.4	lb/ft (Joist #5)
Uniform Load #6=	212.4	lb/ft (Joist #6)

Max Loading =	235.3565 lb/ft
Joist Span =	29 ft
Joist Selection =	20K6
Joist Capacity =	242 lb/ft
% Capacity =	97.255%

OK use 20K6

Interior Joists - Non-Drift Loaded Joists



Calc. By <u>RS</u> Checked By \_\_\_\_\_ Date <u>1/16/2019</u>

### STEEL COLUMN DESIGN

In accordance with AISC360-10 and the ASD method

ڻ **4**−0.23" V 4 6"-

Tedds calculation version 1.0.09

Column	and	loading	details
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Column details	
Column section	HSS 6x6x1/4
Design loading	
Required axial strength	Pr = <b>45</b> kips (Compression)
Maximum moment about x axis	M <sub>x</sub> = <b>0.0</b> kips_ft
Maximum moment about y axis	M <sub>y</sub> = <b>0.0</b> kips_ft
Maximum shear force parallel to y axis	V <sub>ry</sub> = <b>0.0</b> kips
Maximum shear force parallel to x axis	V <sub>rx</sub> = <b>0.0</b> kips
Material details	
Steel grade	A500 Gr. B
Yield strength	F <sub>y</sub> = <b>46</b> ksi
Ultimate strength	Fu = <b>58</b> ksi
Modulus of elasticity	E = <b>29000</b> ksi
Shear modulus of elasticity	G = <b>11200</b> ksi
Unbraced lengths	
For buckling about x axis	L <sub>x</sub> = <b>198</b> in
For buckling about y axis	L <sub>y</sub> = <b>198</b> in
For torsional buckling	L <sub>z</sub> = <b>198</b> in
Effective length factors	
For buckling about x axis	K <sub>x</sub> = <b>1.00</b>
For buckling about y axis	K <sub>y</sub> = <b>1.00</b>
For torsional buckling	Kz = 1.00
Section classification	
Section classification for local buckling (cl. B4)	
Critical flange width	b = b <sub>f</sub> - 3 × t = <b>5.301</b> in

<b>E</b> STRUCTURAL ENGINEERS	Toject <u>balde</u>		— Project No. <del>20-467</del>
	Calc. By <u>RS</u>	Checked By	Date_ <u>1/16/2019</u>
Width to thickness ratio of flange (	compression)	λ <sub>f_c</sub> = b / t = <b>22.751</b>	
Width to thickness ratio of web (co	mpression)	λ <sub>w_c</sub> = h / t = <b>22.751</b>	
Width to thickness ratio of flange (	major flexure)	$\lambda_{f_{fx}} = b / t = 22.751$	
Width to thickness ratio of web (ma	ajor flexure)	$\lambda_{w_{fx}} = h / t = 22.751$	
Width to thickness ratio of flange (	minor flexure)	$\lambda_{f_{fy}} = h / t = 22.751$	
Width to thickness ratio of web (mi	nor flexure)	$\lambda_{w_{fy}} = b / t = 22.751$	
Compression			
Limit for nonslender section		$\lambda_{r_c}$ = 1.40 × $\sqrt{(E / F_y)}$ = 35.152	
		The	section is nonslender in comp
Slenderness			
Member slenderness			
Slenderness ratio about x axis		SR <sub>x</sub> = K <sub>x</sub> × L <sub>x</sub> / r <sub>x</sub> = <b>84.6</b>	
Slenderness ratio about y axis		$SR_y = K_y \times L_y / r_y = 84.6$	
Reduction factor for slender elements Reduction factor for slender elements The section does not contain any s	ments (E7)		
-	slender element		
Slender element reduction factor	slender element	s therefore:- Q = <b>1.0</b>	
-	slender element		
Slender element reduction factor Compressive strength Flexural buckling about x axis (c		Q = <b>1.0</b>	
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress		Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 = 40.0$ ks	si
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor	sl. E3)	Q = 1.0 $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ks}$ $Q_x = Q = 1.000$	
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis	sl. E3)	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2$ = 40.0 ks Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>QxxFy/Fex</sup> ) × F <sub>y</sub> =	
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength	: <b>I. E3)</b> :is	Q = 1.0 $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ks}$ $Q_x = Q = 1.000$	
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis Nominal flexural buckling strength Flexural buckling about y axis (or	: <b>I. E3)</b> :is	Q = 1.0 $F_{ex} = (\pi^2 \times E) / (SR_x)^2 =$ 40.0 ks $Q_x = Q =$ 1.000 $F_{crx} = Q_x \times (0.658^{Q_{XX}Fy/Fex}) \times F_y =$ $P_{nx} = F_{crx} \times A_g =$ 148.9 kips	= <b>28.4</b> ksi
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress	: <b>I. E3)</b> :is	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ks Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>Qx</sup> ×Fy/Fex) × Fy = P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ks	= <b>28.4</b> ksi
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress Reduction factor	: <b>I. E3)</b> :is : <b>I. E3</b> )	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ks Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>Qx×Fy/Fex</sup> ) × Fy = P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ks Q <sub>y</sub> = Q = 1.000	= <b>28.4</b> ksi si
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about y axis	: <b>I. E3)</b> :is : <b>I. E3</b> )	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ks Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>Qx×Fy/Fex</sup> ) × F <sub>y</sub> = P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ks Q <sub>y</sub> = Q = 1.000 F <sub>cry</sub> = Q <sub>y</sub> × (0.658 <sup>Qy×Fy/Fey</sup> ) × F <sub>y</sub> =	= <b>28.4</b> ksi si
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about y axis Nominal flexural buckling stress about y axis Nominal flexural buckling strength	: <b>I. E3)</b> :is : <b>I. E3</b> ) :is	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ks Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>Qx×Fy/Fex</sup> ) × Fy = P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ks Q <sub>y</sub> = Q = 1.000	= <b>28.4</b> ksi si
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (or Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis Nominal flexural buckling strength Flexural buckling about y axis (or Elastic critical buckling stress Reduction factor Flexural buckling stress about y axis Nominal flexural buckling stress about y axis Nominal flexural buckling strength Allowable compressive strength	: <b>I. E3)</b> :is : <b>I. E3</b> ) :is	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ks Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × $(0.658^{Qx \times Fy/Fex}) \times F_y =$ P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ks Q <sub>y</sub> = Q = 1.000 F <sub>cry</sub> = Q <sub>y</sub> × $(0.658^{Qy \times Fy/Fey}) \times F_y =$ P <sub>ny</sub> = F <sub>cry</sub> × A <sub>g</sub> = 148.9 kips	= <b>28.4</b> ksi si
Slender element reduction factor <u>Compressive strength</u> Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about y axis Nominal flexural buckling stress Reduction factor Flexural buckling stress about y axis Nominal flexural buckling strength Allowable compressive strength	: <b>I. E3)</b> :is : <b>I. E3</b> ) :is	Q = 1.0 Fex = $(\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ks}$ Qx = Q = 1.000 Forx = Qx × $(0.658^{Qx \times Fy/Fex}) \times Fy =$ Pnx = Forx × Ag = 148.9 kips Fey = $(\pi^2 \times E) / (SR_y)^2 = 40.0 \text{ ks}$ Qy = Q = 1.000 Fory = Qy × $(0.658^{Qy \times Fy/Fey}) \times Fy =$ Pny = Fory × Ag = 148.9 kips $\Omega_c = 1.67$	= 28.4 ksi si = 28.4 ksi
Slender element reduction factor Compressive strength Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x axis Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about y axis Nominal flexural buckling strength Allowable compressive strength	:I. E3) :is :I. E3) :is (CI. E1)	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ks Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × $(0.658^{Qx \times Fy/Fex}) \times F_y =$ P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ks Q <sub>y</sub> = Q = 1.000 F <sub>cry</sub> = Q <sub>y</sub> × $(0.658^{Qy \times Fy/Fey}) \times F_y =$ P <sub>ny</sub> = F <sub>cry</sub> × A <sub>g</sub> = 148.9 kips	= <b>28.4</b> ksi si = <b>28.4</b> ksi kips

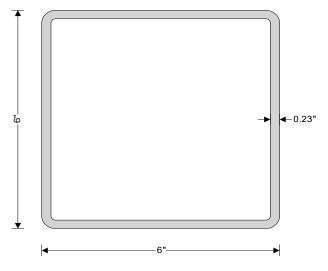


Project\_\_\_\_\_ Project No.\_\_\_\_\_ Calc. By <u>RS</u> Checked By\_\_\_\_\_ Date\_<u>1/16/2019</u>

### STEEL COLUMN DESIGN

In accordance with AISC360-10 and the ASD method

Tedds calculation version 1.0.09



### Column and loading details

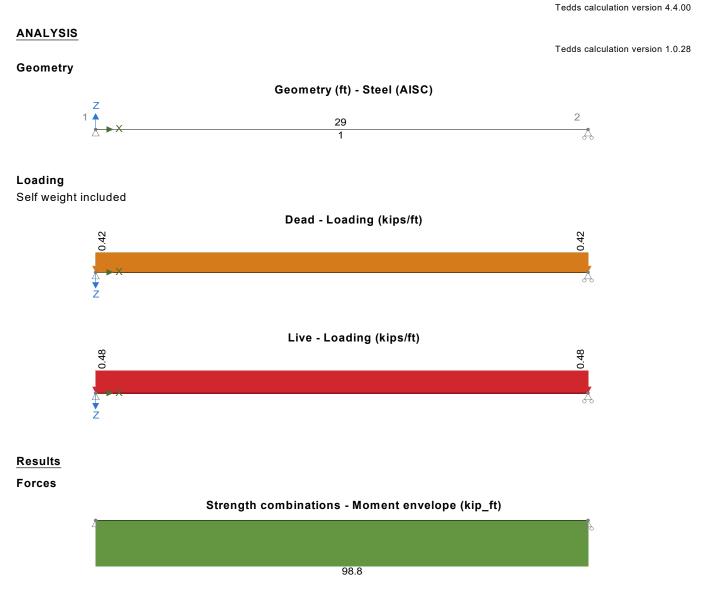
Column details	
Column section	HSS 6x6x1/4
Design loading	
Required axial strength	Pr = 9 kips (Compression)
Maximum moment about x axis	M <sub>x</sub> = <b>0.0</b> kips_ft
Maximum moment about y axis	M <sub>y</sub> = <b>0.0</b> kips_ft
Maximum shear force parallel to y axis	V <sub>ry</sub> = <b>0.0</b> kips
Maximum shear force parallel to x axis	V <sub>rx</sub> = <b>0.0</b> kips
Material details	
Steel grade	A500 Gr. B
Yield strength	F <sub>y</sub> = <b>46</b> ksi
Ultimate strength	Fu = <b>58</b> ksi
Modulus of elasticity	E = <b>29000</b> ksi
Shear modulus of elasticity	G = <b>11200</b> ksi
Unbraced lengths	
For buckling about x axis	L <sub>x</sub> = <b>198</b> in
For buckling about y axis	L <sub>y</sub> = <b>198</b> in
For torsional buckling	Lz = <b>198</b> in
Effective length factors	
For buckling about x axis	K <sub>x</sub> = 1.00
For buckling about y axis	K <sub>y</sub> = <b>1.00</b>
For torsional buckling	Kz = 1.00
Section classification	
Section classification for local buckling (cl. B4)	
Critical flange width	b = b <sub>f</sub> - 3 $\times$ t = <b>5.301</b> in
Critical web width	h = d - 3 × t = <b>5.301</b> in

	Project		Project No
	Calc. By <u>RS</u>	Checked By	Date_1/16/2019
Width to thickness ratio of flange (	compression)	λ <sub>f_c</sub> = b / t = <b>22.751</b>	
Width to thickness ratio of web (co	mpression)	λ <sub>w_c</sub> = h / t = <b>22.751</b>	
Width to thickness ratio of flange (	major flexure)	$\lambda_{f_{fx}} = b / t = 22.751$	
Width to thickness ratio of web (ma	ajor flexure)	$\lambda_{w_{fx}} = h / t = 22.751$	
Width to thickness ratio of flange (	minor flexure)	$\lambda_{f_{fy}} = h / t = 22.751$	
Width to thickness ratio of web (mi	nor flexure)	$\lambda_{w_{fy}} = b / t = 22.751$	
Compression			
Limit for nonslender section		$\lambda_{r_c}$ = 1.40 × $\sqrt{(E / F_y)}$ = 35.152	
		The sectior	n is nonslender in compres
Slenderness			
Member slenderness			
Slenderness ratio about x axis		SR <sub>x</sub> = K <sub>x</sub> × L <sub>x</sub> / r <sub>x</sub> = <b>84.6</b>	
Slenderness ratio about y axis		$SR_y = K_y \times L_y / r_y = 84.6$	
Reduction factor for slender elements Reduction factor for slender elements and the statement of the stateme			
<b>Reduction factor for slender ele</b> The section does not contain any s	ments (E7)		
Reduction factor for slender elements	ments (E7)	s therefore:- Q = <b>1.0</b>	
Reduction factor for slender elements The section does not contain any s Slender element reduction factor Compressive strength	ments (E7) slender elements		
<b>Reduction factor for slender elen</b> The section does not contain any s Slender element reduction factor	ments (E7) slender elements		
Reduction factor for slender elements The section does not contain any s Slender element reduction factor Compressive strength Flexural buckling about x axis (c	ments (E7) slender elements	Q = 1.0	
Reduction factor for slender elements The section does not contain any section factor Slender element reduction factor Compressive strength Flexural buckling about x axis (of Elastic critical buckling stress	ments (E7) slender elements sl. E3)	Q = 1.0 F <sub>ex</sub> = (π <sup>2</sup> × E) / (SR <sub>x</sub> ) <sup>2</sup> = 40.0 ksi	3.4 ksi
Reduction factor for slender elements The section does not contain any section factor Slender element reduction factor Compressive strength Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor	ments (E7) slender elements sl. E3)	Q = 1.0 $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ksi}$ Qx = Q = 1.000	3.4 ksi
Reduction factor for slender element The section does not contain any s Slender element reduction factor Compressive strength Flexural buckling about x axis (o Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax	ments (E7) slender elements sl. E3)	Q = 1.0 $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ksi}$ $Q_x = Q = 1.000$ $F_{crx} = Q_x \times (0.658^{Qx \times Fy/Fex}) \times F_y = 28$	3.4 ksi
Reduction factor for slender element The section does not contain any s Slender element reduction factor Compressive strength Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength	ments (E7) slender elements sl. E3)	Q = 1.0 $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ksi}$ $Q_x = Q = 1.000$ $F_{crx} = Q_x \times (0.658^{Qx \times Fy/Fex}) \times F_y = 28$	3.4 ksi
Reduction factor for slender element The section does not contain any s Slender element reduction factor Compressive strength Flexural buckling about x axis (o Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength Flexural buckling about y axis (o	ments (E7) slender elements sl. E3)	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ksi Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>Qx×Fy/Fex</sup> ) × F <sub>y</sub> = 28 P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ksi Q <sub>y</sub> = Q = 1.000	
Reduction factor for slender element The section does not contain any s Slender element reduction factor Compressive strength Flexural buckling about x axis (o Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength Flexural buckling about y axis (o Elastic critical buckling stress Reduction factor Flexural buckling stress Reduction factor Flexural buckling stress	ments (E7) slender elements sl. E3) is	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2$ = 40.0 ksi Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>Qx×Fy/Fex</sup> ) × F <sub>y</sub> = 28 P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2$ = 40.0 ksi	
Reduction factor for slender element The section does not contain any s Slender element reduction factor Compressive strength Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress Reduction factor	ments (E7) slender elements sl. E3) is	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 =$ 40.0 ksi Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × (0.658 <sup>Qx×Fy/Fex</sup> ) × F <sub>y</sub> = 28 P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 =$ 40.0 ksi Q <sub>y</sub> = Q = 1.000	
Reduction factor for slender element The section does not contain any s Slender element reduction factor Compressive strength Flexural buckling about x axis (o Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength Flexural buckling about y axis (o Elastic critical buckling stress Reduction factor Flexural buckling stress Reduction factor Flexural buckling stress	ments (E7) slender elements sl. E3) is is	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ksi}$ Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × $(0.658^{Qx \times Fy/Fex}) \times F_y = 28$ P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 = 40.0 \text{ ksi}$ Q <sub>y</sub> = Q = 1.000 F <sub>cry</sub> = Q <sub>y</sub> × $(0.658^{Qy \times Fy/Fey}) \times F_y = 28$	
Reduction factor for slender element The section does not contain any section factor Compressive strength Flexural buckling about x axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress about x ax Nominal flexural buckling strength Flexural buckling about y axis (of Elastic critical buckling stress Reduction factor Flexural buckling stress Reduction factor Flexural buckling stress about y ax Nominal flexural buckling stress	ments (E7) slender elements sl. E3) is is	Q = 1.0 F <sub>ex</sub> = $(\pi^2 \times E) / (SR_x)^2 = 40.0 \text{ ksi}$ Q <sub>x</sub> = Q = 1.000 F <sub>crx</sub> = Q <sub>x</sub> × $(0.658^{Qx \times Fy/Fex}) \times F_y = 28$ P <sub>nx</sub> = F <sub>crx</sub> × A <sub>g</sub> = 148.9 kips F <sub>ey</sub> = $(\pi^2 \times E) / (SR_y)^2 = 40.0 \text{ ksi}$ Q <sub>y</sub> = Q = 1.000 F <sub>cry</sub> = Q <sub>y</sub> × $(0.658^{Qy \times Fy/Fey}) \times F_y = 28$	



### **STEEL MEMBER ANALYSIS & DESIGN (AISC 360)**

In accordance with AISC360 14th Edition published 2010 using the ASD method



#### Strength combinations - Shear envelope (kips)





Calc. By <u>RJS</u>

Checked By\_JH

\_\_\_\_\_ Project No.-<u>20-467</u>\_\_\_

Date 2/23/2021

Safety factors		
Shear		Ω <sub>v</sub> = <b>1.67</b>
Flexure		Ω <sub>b</sub> = <b>1.67</b>
Tensile yielding		Ω <sub>t,y</sub> = <b>1.67</b>
Tensile rupture		Ω <sub>t,r</sub> = <b>2.00</b>
Compression		Ωc = <b>1.67</b>
Beam design		
Section details		
Section type		W 18x40 (AISC 15th Edn (v15.0))
ASTM steel designation		A992
Ũ		
Steel yield stress		F <sub>y</sub> = <b>50</b> ksi
Steel tensile stress		Fu = <b>65</b> ksi
Modulus of elasticity		E = <b>29000</b> ksi
	-0.53	
	o ↓	
	$\uparrow$ $\mp$	W 18x40 (AISC 15th Edn (v15.0)) Section depth, d, 17.9 in
		Section depth, d, $17.9$ in Section breadth, b, 6.02 in
		Weight of section, Weight, 40 lbf/ft
		Flange thickness, t <sub>f</sub> , 0.525 in
		Web thickness, $t_w$ , 0.315 in
		Area of section, A, 11.8 in <sup>2</sup> Radius of gyration about x-axis, r <sub>v</sub> , 7.21 in
		Dedius of gyration about v avia, r <sub>x</sub> , r.2 r in

#### Lateral restraint

Top flange has full lateral restraint Bottom flange has lateral restraint at supports only

### Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

17.9"-

▶ ►0.53"

◄

Classification of flanges in flexure - Table B4.1	b (case 10)	
Width to thickness ratio	bf / (2 × tf) = <b>5.73</b>	
Limiting ratio for compact section	$\lambda_{\text{pff}}$ = 0.38 $\times$ $\sqrt{[\text{E / F}_y]}$ = <b>9.15</b>	
Limiting ratio for non-compact section	$\lambda_{\text{rff}}$ = 1.0 × $\sqrt{[E / F_y]}$ = 24.08	Compact
Classification of web in flexure - Table B4.1b (c	ase 15)	
Classification of web in flexure - Table B4.1b (c Width to thickness ratio	ase 15) (d - 2 × k) / t <sub>w</sub> = 50.94	
· ·	,	

-0.32"

Radius of gyration about y-axis,  $r_y$ , 1.27 in

Elastic section modulus about x-axis, S<sub>x</sub>, 68.4 in<sup>3</sup> Elastic section modulus about y-axis, S<sub>y</sub>, 6.35 in<sup>3</sup> Plastic section modulus about x-axis, Z<sub>x</sub>, 78.4 in<sup>3</sup> Plastic section modulus about y-axis, Z<sub>y</sub>, 10 in<sup>3</sup>

Second moment of area about x-axis,  $I_{x^{\prime}}^{2}$  612 in<sup>4</sup> Second moment of area about y-axis,  $I_{y^{\prime}}$  19.1 in<sup>4</sup>

Compact

Section is compact in flexure



Calc. By RJS

Checked By JH Date 2/23/2021

Design of members for shear - Chapter G	
Required shear strength	V <sub>r,x</sub> = <b>13.6</b> kips
Web area	$A_w = d \times t_w = 5.638 in^2$
Web plate buckling coefficient	k <sub>v</sub> = <b>5</b>
	(d - 2 $\times$ k) / t <sub>w</sub> <= 2.24 $\times$ $\sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C <sub>v</sub> = <b>1.000</b>
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 \times F_y \times A_w \times C_v = 169.2$ kips
Safety factor	$\Omega_{\rm v}=1.50$
Allowable shear strength	$V_{c,x} = V_{n,x} / \Omega_v = 112.8$ kips
	V <sub>r,x</sub> / V <sub>c,x</sub> = <b>0.121</b>
	PASS - Allowable shear strength exceeds required shear strength

#### Check design 14ft 6in along span

Design of members for flexure - Chapter F Required flexural strength

Yielding - Section F2.1 Nominal flexural strength for yielding - eq F2-1

Allowable flexural strength - F1 Nominal flexural strength

Allowable flexural strength

M<sub>r,x</sub> = **98.8** kips\_ft  $M_{n,yld,x} = M_{p,x} = F_y \times Z_x = 326.7 \text{ kips_ft}$ 

 $M_{n,x} = M_{n,yld,x} = 326.7$  kips ft  $M_{c,x} = M_{n,x} / \Omega_b = 195.6 \text{ kips_ft}$ Mr,x / Mc,x = 0.505

PASS - Allowable flexural strength exceeds required flexural strength

#### Check design 14ft 6in along span

Design of members for x-x axis deflection Maximum deflection Allowable deflection

 $\delta_x = 0.862$  in  $\delta_{x,Allowable} = L_{m1_s1} / 360 = 0.967$  in  $\delta_x / \delta_{x.Allowable} = 0.891$ 

PASS - Allowable deflection exceeds design deflection



#### **STEEL MEMBER ANALYSIS & DESIGN (AISC 360)**

In accordance with AISC360 14th Edition published 2010 using the ASD method



Forces



308.4

#### Strength combinations - Shear envelope (kips)



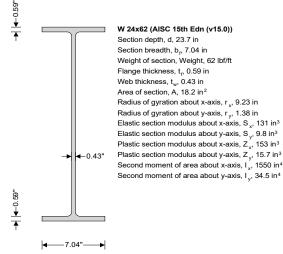


Project No. <u>20-467</u>

Calc. By <u>RJS</u>

Checked By JH

Safety factors	
Shear	Ω <sub>v</sub> = <b>1.67</b>
Flexure	Ω <sub>b</sub> = <b>1.67</b>
Tensile yielding	Ω <sub>t,y</sub> = <b>1.67</b>
Tensile rupture	Ω <sub>t,r</sub> = <b>2.00</b>
Compression	Ωc = <b>1.67</b>
Beam design	
Section details	
Section type	W 24x62 (AISC 15th Edn (v15.0))
ASTM steel designation	A992
Steel yield stress	F <sub>y</sub> = <b>50</b> ksi
Steel tensile stress	F <sub>u</sub> = <b>65</b> ksi
Modulus of elasticity	E = <b>29000</b> ksi



#### Lateral restraint

Top flange has full lateral restraint Bottom flange has lateral restraint at supports only

#### Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Limiting ratio for non-compact section

Classification of sections for local buckling - Section B4

23.7"

Classification of flanges in flexure - Table B4.1b (case 10)				
Width to thickness ratio	bf / (2 × tf) = <b>5.97</b>			
Limiting ratio for compact section	$\lambda_{\text{pff}}$ = 0.38 $\times$ $\sqrt{[\text{E}$ / $\text{F}_{\text{y}}]}$ = <b>9.15</b>			
Limiting ratio for non-compact section	$\lambda_{\text{rff}}$ = 1.0 × $\sqrt{[E / F_y]}$ = 24.08	Compact		
Classification of web in flexure - Table B4.1b (ca	ase 15)			
Width to thickness ratio	(d - 2 × k) / t <sub>w</sub> = <b>50.05</b>			
Limiting ratio for compact section	$\lambda_{\text{pwf}} = 3.76 \times \sqrt{[\text{E / F}_y]} = \textbf{90.55}$			

 $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]} = 137.27$ 

Compact
---------

Section is compact in flexure



Calc. By RJS

Checked By JH Date 3/1/2021

#### Check design at start of span

Design of members for shear - Chapter G	
Required shear strength	V <sub>r,x</sub> = <b>52</b> kips
Web area	$A_w = d \times t_w = 10.191 in^2$
Web plate buckling coefficient	k <sub>v</sub> = <b>5</b>
	(d - 2 $\times$ k) / t <sub>w</sub> <= 2.24 $\times$ $\sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C <sub>v</sub> = <b>1.000</b>
Nominal shear strength - eq G2-1	$V_{n,x}$ = 0.6 × F <sub>y</sub> × A <sub>w</sub> × C <sub>v</sub> = <b>305.7</b> kips
Safety factor	$\Omega_{\rm v}$ = 1.50
Allowable shear strength	$V_{c,x} = V_{n,x} / \Omega_v = 203.8$ kips
	$V_{r,x} / V_{c,x} = 0.255$
	PASS - Allowable shear strength exceeds required shear strength

#### Check design 11ft 10.2in along span

Design of members for flexure - Chapter F Required flexural strength

Yielding - Section F2.1 Nominal flexural strength for yielding - eq F2-1

Allowable flexural strength - F1 Nominal flexural strength

Allowable flexural strength

M<sub>r,x</sub> = 308.4 kips\_ft  $M_{n,yld,x} = M_{p,x} = F_y \times Z_x = 637.5 \text{ kips_ft}$ 

 $M_{n,x} = M_{n,yld,x} = 637.5$  kips ft  $M_{c,x} = M_{n,x} / \Omega_b = 381.7 \text{ kips_ft}$ Mr,x / Mc,x = 0.808

PASS - Allowable flexural strength exceeds required flexural strength

#### Check design 11ft 10.2in along span

Design of members for x-x axis deflection Maximum deflection Allowable deflection

 $\delta_x = 0.726$  in  $\delta_{x,Allowable} = L_{m1_{s1}} / 360 = 0.79$  in  $\delta_x / \delta_{x.Allowable} = 0.919$ 

PASS - Allowable deflection exceeds design deflection



Project Balderston Auto		Project No. 20-467	
Calc. By_RS	Checked By_JB	Date 1/1/21	

### **Summary**

The gravity structure system of the project referenced above consists primarily of steel joists and girders, load-bearing CMU walls and steel columns. CMU walls support all gravity and lateral loads along the perimeter of the building. CMU wall designs are performed in RAM Elements with Tedds calculations, as required.

The following section of calculations covers the complete design of the tilt CMU system for project referenced above. Refer to the "Loads" section of these calculations for the determination of all dead, live, roof live, and snow loads. Refer to the "Lateral Stability" section of these calculations for the determination of all wind and seismic loads.



Calc. By RJS

\_\_\_\_\_ Project No.-<del>20-467</del>\_\_\_\_

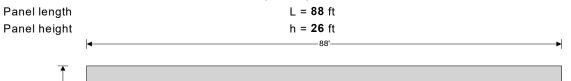
Checked By JH Date 3/1/2021

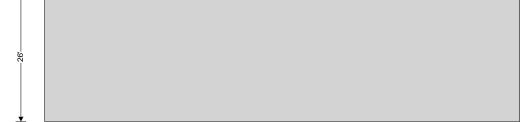
#### MASONRY WALL PANEL DESIGN TO MSJC-11

#### Using the strength design method

#### Masonry wall panel details

Typical Exterior Wall - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads The wall is fixed at the bottom and free at the top for in plane loads





#### **Seismic properties**

Seismic design category	В
Seismic importance factor (ASCE7 Table 1.5-2)	le = 1
Design spectral response acceleration parameter,	short periods (ASCE7 11.4.4)

Seismic wall classification

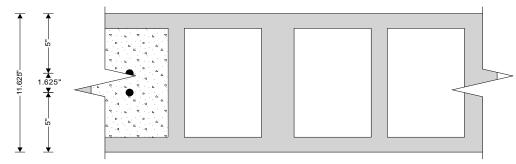
S<sub>DS</sub> = **0.345** Nonparticipating No prescriptive minimum seismic reinforcement p<sub>E</sub> = **1.0** 

Redundancy factor, on out-of-plane load

#### **Construction details**

Wall thickness

t = **11.625** in



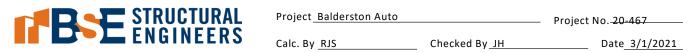
#### **Masonry details**

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

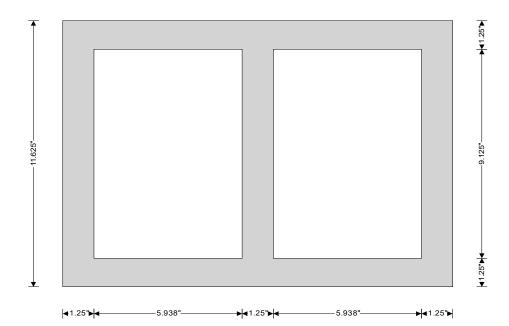
 $\begin{array}{ll} \mbox{Compressive strength of unit} & f'_{cu} = 1900 \mbox{ psi} \\ \mbox{Density of masonry units} & & & \\ \mbox{Height of masonry units} & & & \\ \mbox{h}_b = 7.625 \mbox{ in} \\ \mbox{Length of masonry units} & & & \\ \mbox{Number of internal webs} & & & \\ \mbox{Number of end webs} & & & \\ \mbox{Nend} = 2 \end{array}$ 

Sht. No. 1 C2 of 18 6

Tedds calculation version 2.2.04

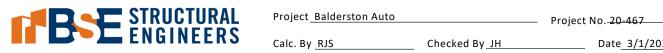


Internal web thickness	t <sub>bw</sub> = <b>1.25</b> in
Face shell thickness	t <sub>bf</sub> = <b>1.25</b> in
End web thickness	t <sub>be</sub> = <b>1.25</b> in
Area of block	$A_{block} = [t \times I_{b} - (I_{b} - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / I_{b} = 56.28 \text{ in}^{2}/\text{ft}$
Area of grout	$A_{grout} = [0.17 \times (I_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / I_b = 14.15 \text{ in}^2/\text{ft}$
Density of grout	$\gamma_{grout}$ = <b>140</b> lb/ft <sup>3</sup>
Self weight of wall	wsw = Ablock $\times \gamma$ block + Agrout $\times \gamma$ grout = 58.7 psf
	15.625"►



#### From TMS 602-11 Table 2 - Compressive strength of masonry

	an or masonry
Net compressive strength of masonry	f' <sub>m</sub> = <b>1500</b> psi
Modulus of elasticity for masonry	E <sub>m</sub> = 900 × f'm = <b>1350000</b> psi
Shear modulus of masonry	$G_v$ = 0.4 $\times$ Em = <b>540000</b> psi
From TMS 402 -11 Table 3.1.8.2 - Modulus of rug	oture
Modulus of rupture normal to bed	fr_norm = <b>80</b> psi
Modulus of rupture parallel to bed	f <sub>r_para</sub> = <b>125</b> psi
Reinforcement details	
Yield strength of reinforcement	f <sub>y</sub> = <b>60000</b> psi
Allowable tensile stress in reinforcement	Fs <b>= 32000</b> psi
Modulus of elasticity for reinforcement	Es <b>= 29000000</b> psi
Vertical reinforcement provided	No.5 bars at 48 in centers
Area of vertical reinforcement, per face	$A_s = \pi \times Dia^2 / (4 \times s) = 0.08 in^2/ft$
Lateral out-of-plane loads	
Wind load on panel	W = <b>28</b> psf
Wind load on parapet	W <sub>p</sub> = <b>70</b> psf
Seismic load factor (ASCE7 12.11.1)	$F_{p} = 0.4 \times S_{DS} \times I_{e} = \textbf{0.138}$
Seismic load from wall	$E_{wall} = max(F_{p}, 0.1) \times w_{SW} = 8.1 \text{ psf}$
Additional seimic load	E <sub>add</sub> = <b>0</b> psf



Seismic lateral load on panel		E = E <sub>wall</sub> + E <sub>add</sub> = <b>8.1</b> psf
Lateral in-plane loads		
Wind shear load on wall		Vw = 21800 lbs
Vertical loading details		
Dead load at supported level		DL = <b>276</b> lb/ft
Live load from above		LLabove = 290 lb/ft
Vertical seismic load factor ap	oplied to dead load	$F_{Ev} = 0.2 \times S_{DS} = 0.069$
From ASCE 7-10 cl.2.3.2 - C	-	oads using strength design (Utilization)
Load combination no.1	$1.4 \times DL$ (0.061)	
Load combination no.5	1.2 × DL + W + LL	+ 0.5 × (LL <sub>r</sub> or SL or RL) (0.881)
Load combination no.7	$0.9 \times DL + W$ (0.9	42)
Properties of masonry secti	on	
Cross-sectional area		$\label{eq:alpha} \begin{split} A &= [t \times I_b \text{ - } 0.83 \times (I_b \text{ - } N_{web} \times t_{bw} \text{ - } N_{end} \times t_{be}) \times (t \text{ - } 2 \times t_{bf})] \text{ / } I_b = \textbf{70.4} \\ in^2 / ft \end{split}$
Properties for walls loaded ou	t-of-plane:	
Moment of inertia		$I = t^3 / 12 - 0.83 \times (I_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times I_b) = 1091.7 \text{ in}^4/\text{ft}$
Section modulus		$S = I / c = 187.8 \text{ in}^3/\text{ft}$
Radius of gyration		r = √[l / A] = <b>3.937</b> in
Effective height factor		K = 1
Properties for walls loaded in-	plane:	
Net moment of inertia		$I_{x\_net} = t \times L^3 \ / \ 12 \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} \times x_{cell2^2} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} \times x_{cell2^2} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} \times x_{cell2^2} \times x_{cell2^2} \times x_{cell2^2} \times x_{cell2^2} \times x_{cell2^2}) \ - \ 2 \times (I_{x\_cell} \times x_{cell2^2} \times $
		$ imes$ (I <sub>x_cell</sub> + A <sub>cell</sub> $ imes$ X <sub>cell</sub> <sup>2</sup> ) - 2 $ imes$ (I <sub>x_cell</sub> + A <sub>cell</sub> $ imes$ X <sub>cell</sub> <sup>2</sup> ) - 2 $ imes$ (I <sub>x_cell</sub> + A <sub>cell</sub> $ imes$
		$x_{\text{cell5}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell7}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{\text{cell8}^2}) - 2 \times (I_{x\_\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{\text{cel8}^2} \times x_{\text{cell8}^2} \times x_{\text{cell8}^2} \times x_{cel8$
		$A_{\text{cell}} \times x_{\text{cell9}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell10}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell11}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell11}^2}) - 2 \times (I_{x\_\text{cell}} + A_{x\_\text{cell}} \times x_{x\_\text{cell11}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{x\_\text{cell111}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{x\_\text{cell1111}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2} \times x_{x\_\text{cell}^2$
		$(I_{x_{cell}} + A_{cell} \times x_{cell13}^2)$ - 2 × $(I_{x_{cell}} + A_{cell} \times x_{cell14}^2)$ - 2 × $(I_{x_{cell}} + A_{cell} \times x_{cell14}^2)$
		$x_{\text{cell15}^2}) \textbf{-} 2 \times (I_{x\_\text{cell}} \textbf{+} A_{\text{cell}} \times x_{\text{cell16}^2}) \textbf{-} 2 \times (I_{x\_\text{cell}} \textbf{+} A_{\text{cell}} \times x_{\text{cell17}^2}) \textbf{-} 2 \times (I_{x\_\text{cell}} \times x_{\text{cell}} \times x_{\text{cell17}^2}) \textbf{-} 2 \times (I_{x\_\text{cell}} \times x_{\text{cell}} $
		+ $A_{cell} \times x_{cell19^2}$ ) - 2 × ( $I_{x_cell}$ + $A_{cell} \times x_{cell20^2}$ ) - 2 × ( $I_{x_cell}$ + $A_{cell} \times x_{cell21^2}$ ) - 2
		$ imes$ (Ix_cell + Acell $ imes$ Xcell22 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$ Xcell23 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$
		$x_{\texttt{cell25}^2}) \textbf{-} 2 \times (I_{x\_\texttt{cell}} \textbf{+} A_{\texttt{cell}} \times x_{\texttt{cell26}^2}) \textbf{-} 2 \times (I_{x\_\texttt{cell}} \textbf{+} A_{\texttt{cell}} \times x_{\texttt{cell27}^2}) \textbf{-} 2 \times (I_{x\_\texttt{cell}} \times x_{\texttt{cell27}^2} \times x_{\texttt{cell27}^2}) \textbf{-} 2 \times (I_{x\_\texttt{cell}} \times x_{\texttt{cell27}^2} \times x_{ce$
		+ $A_{cell} \times x_{cell28}^2$ ) - 2 × ( $I_{x_cell}$ + $A_{cell} \times x_{cell29}^2$ ) - 2 × ( $I_{x_cell}$ + $A_{cell} \times x_{cell31}^2$ ) - 2
		$ imes$ (Ix_cell + Acell $ imes$ Xcell32 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$ Xcell33 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$
		$x_{\text{cell34}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell35}^2}) - 2 \times (I_{x\_\text{cell}} + A_{\text{cell}} \times x_{\text{cell37}^2}) - 2 \times (I_{x\_\text{cell}} \times x_{\text{cell37}^2})$
		+ $A_{cell} \times x_{cell38}^2$ ) - 2 × ( $I_{x_cell}$ + $A_{cell} \times x_{cell39}^2$ ) - 2 × ( $I_{x_cell}$ + $A_{cell} \times x_{cell40}^2$ ) - 2
		$ imes$ (Ix_cell + Acell $ imes$ Xcell41 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$ Xcell43 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$
		$x_{cell44^2}$ ) - 2 × (Ix_cell + Acell × $x_{cell45^2}$ ) - 2 × (Ix_cell + Acell × $x_{cell46^2}$ ) - 2 × (Ix_cell
		+ Acell $\times$ Xcell47 <sup>2</sup> ) - 2 $\times$ (Ix_cell + Acell $\times$ Xcell49 <sup>2</sup> ) - 2 $\times$ (Ix_cell + Acell $\times$ Xcell50 <sup>2</sup> ) - 2
		$ imes$ (Ix_cell + Acell $ imes$ Xcell51 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$ Xcell52 <sup>2</sup> ) - 2 $ imes$ (Ix_cell + Acell $ imes$
		$x_{cell53^2}$ ) - 2 × (Ix_cell + Acell × $x_{cell55^2}$ ) - 2 × (Ix_cell + Acell × $x_{cell56^2}$ ) - 2 × (Ix_cell
		+ Acell $\times$ Xcell57 <sup>2</sup> ) - 2 $\times$ (I <sub>x_cell</sub> + Acell $\times$ Xcell58 <sup>2</sup> ) - 2 $\times$ (I <sub>x_cell</sub> + Acell $\times$ Xcell59 <sup>2</sup> ) - 2
		$\times$ (I <sub>x_cell</sub> + A <sub>cell</sub> $\times$ X <sub>cell61</sub> <sup>2</sup> ) - 2 $\times$ (I <sub>x_cell</sub> + A <sub>cell</sub> $\times$ X <sub>cell62</sub> <sup>2</sup> ) - 2 $\times$ (I <sub>x_cell</sub> + A <sub>cell</sub> $\times$
		$x_{cell63^2}$ ) - 2 × ( $I_{x_cell}$ + $A_{cell}$ × $x_{cell64^2}$ ) - 2 × ( $I_{x_cell}$ + $A_{cell}$ × $x_{cell65^2}$ ) =
		587318944 in <sup>4</sup>
Net section modulus		S <sub>x_net</sub> = I <sub>x_net</sub> / (L / 2) = <b>1112346</b> in <sup>3</sup>



#### Consider wall at maximum moment location under load combination no.7

Axial force, out of plane - Combination No.7 - Ibs/ItShear force, out of plane - Combination No.7 - Ibs/ItMoment force, out of plane - Combination No.7 - Ib\_in 364 28392 1622 13 ft Maximum moment location P = 935 lb/ft Axial load at mid-height of panel Slenderness ratio  $(K \times h) / r = 79.244 < 99$ Nominal axial strength  $P_n = 0.8 \times (0.8 \times f'_m \times (A - 2 \times A_s) + 2 \times A_s \times 0 \text{ ksi}) \times [1 - ((K \times h) / (140 \times A_s) \times (K \times h) + (140 \times A_s) \times (K \times h) \times (K \times$ × r))<sup>2</sup>] = 45849 lb/ft φ = **0.9** Strength reduction factor Design axial strength φ × Pn = 41264 lb/ft  $P / (\phi \times P_n) = 0.023$ PASS - Nominal axial strength exceeds axial load Factored axial stress P / t = 7 psi Factored axial stress limit  $0.2 \times f'_m$  = 300 psi PASS - Allowable stress under out of plane loads exceeds factored axial stress Nominal cracking moment strength  $M_{cr} = S \times f_{r_norm} = 14963$  lb in/ft n = Es / Em = 21.481 Modular ratio Distance to neutral axis  $c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m) = 0.481$  in Moment of inertia of cracked section  $I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3 = 73.7 \text{ in}^4/\text{ft}$ Mu0 = M = 28392 lb in/ft By iteration  $\delta_{\text{u0}} = 5 \times M_{\text{cr}} \times h^2 / (48 \times E_m \times I) + 5 \times (M_{\text{u0}} - M_{\text{cr}}) \times h^2 / (48 \times E_m \times I_{\text{cr}}) =$ 1.471 in  $M_{u3} = M_{u0} + P \times \delta_{u2} = 29911 \text{ lb_in/ft}$ 

\_\_\_\_\_ Project No. <u>20-467</u>\_\_\_\_

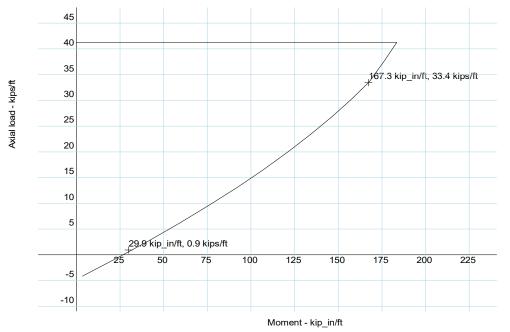


Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>3/1/2021</u>

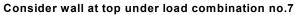
	$\delta_{\tt u3}$ = 5 × M <sub>cr</sub> × h <sup>2</sup> / (48 × E <sub>m</sub> × I) + 5 × (M <sub>u3</sub> - M <sub>cr</sub> ) × h <sup>2</sup> / (48 × E <sub>m</sub> × I <sub>cr</sub> ) =
	<b>1.626</b> in
Bending moment at mid-height of panel	M= M <sub>u0</sub> + P × δ <sub>u3</sub> = <b>29912</b> lb_in/ft
Depth of reinforcement	d = <b>6.625</b> in
Strength reduction factor	$\phi = 0.9$
Tensile strain in reinforcement	$\epsilon_{s} = f_{y} / E_{s} = 0.0021$
Maximum usable compressive strain of masonry	εmu = <b>0.0025</b>
Fiber of max.compressive strain to neutral axis	$c_{bal} = \epsilon_{mu} \times d / (\epsilon_{mu} + \epsilon_s) = 3.625$ in
Tensile force at balance point	$T_{bal} = A_s \times f_y = 4602 \text{ lb/ft}$
	$\beta_1 = 0.8$
Compressive force at balance point	$C_{\text{bal}} = 0.8 \times f'_{\text{m}} \times \beta_1 \times c_{\text{bal}} = 41760 \text{ lb/ft}$
Design axial force at balance point	$P_{bal} = \phi \times (C_{bal} - T_{bal}) = 33442 \text{ lb/ft}$
Design moment at balance point	$M_{\text{bal}} = \phi \times [T_{\text{bal}} \times (d - t / 2) + C_{\text{bal}} \times (t / 2 - \beta_1 \times c_{\text{bal}} / 2)] = 167325 \text{ lb_in/ft}$
Maximum design moment from integration diagram	Mc = <b>31767</b> lb_in/ft
	M / Mc = 0.942
	PASS - Combination of applied axial load and flexure is acceptable

Maximum area of tensile reinforcement (3.3.3.5)

5)  $A_{s_max} = 0.64 \times f'_m \times [\epsilon_{mu} / (\epsilon_{mu} + 1.5 \times \epsilon_s)] \times d / f_y = 0.568 in^2/ft$ PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram



Shear force	V = <b>364</b> lb/ft
Compressive force	N <sub>u</sub> = <b>248</b> lb/ft
Net shear area	$A_{nv}$ = d $\times$ I <sub>b</sub> / ((N <sub>web</sub> + 1) $\times$ s <sub>grout</sub> ) = <b>12.939</b> in <sup>2</sup> /ft
Nominal shear strength	$V_{n} = min([4 - 1.75 \times min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_{m} \times 1 \text{ psi})} + 0.25 \times 10^{-1} \text{ m}$
	$N_u$ , 6 × $A_{nv}$ × $\sqrt{(f'_m × 1 psi)}$ = 2067 lb/ft
Strength reduction factor	$\phi_{V} = 0.8$
Design shear strength	$\phi_V \times V_n$ = 1653 lb/ft
Nominal shear strength Strength reduction factor	$\begin{split} &V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times \\ &N_u, \ 6 \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} = \textbf{2067} \text{ lb/ft} \\ &\varphi_v = \textbf{0.8} \end{split}$



Project <u>Balderston Auto</u> Project No. <del>20-467</del> Calc. By <u>RJS</u> Checked By <u>JH</u> Date <u>3/1/2021</u>

 $V / (\phi_v \times V_n) = 0.220$ 

PASS - Design shear strength exceeds applied shear strength



Calc. By RJS

\_\_\_\_\_ Project No. <u>20-467</u>\_\_\_\_

\_\_\_\_\_ Checked By\_JH\_\_\_\_ Date\_3/1/2021\_

Tedds calculation version 2.2.04

#### MASONRY WALL PANEL DESIGN TO MSJC-11

Using the strength design method

#### Masonry wall panel details

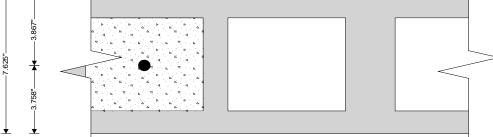
GL4 Wall - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads The wall is fixed at the bottom and free at the top for in plane loads

Panel length	L = <b>62</b> ft
Panel height	h = <b>26</b> ft
	l◀62'▶



#### **Seismic properties**

Seismic design category	В
Seismic importance factor (ASCE7 Table 1.5-2)	l <sub>e</sub> = 1
Design spectral response acceleration parameter,	short periods (ASCE7 11.4.4)
	S <sub>DS</sub> = 0.345
Seismic wall classification	Nonparticipating
	No prescriptive minimum seismic reinforcement
Redundancy factor, on out-of-plane load	ρε <b>= 1.0</b>
Construction details	
Wall thickness	t = <b>7.625</b> in



#### **Masonry details**

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit

Density of masonry units

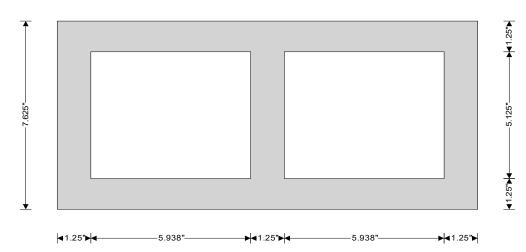
Height of masonry units

f'<sub>cu</sub> = **1900** psi γ<sub>block</sub> = **115** lb/ft<sup>3</sup> h<sub>b</sub> = **7.625** in

Sht. No. 1 C8<sup>of</sup> 18 5



Length of masonry units	l₀ = <b>15.625</b> in
Number of internal webs	N <sub>web</sub> = 1
Number of end webs	$N_{end} = 2$
Internal web thickness	t <sub>bw</sub> = <b>1.25</b> in
Face shell thickness	t <sub>bf</sub> = <b>1.25</b> in
End web thickness	t <sub>be</sub> = <b>1.25</b> in
Area of block	$A_{block} = [t \times l_{b} - (l_{b} - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_{b} = 44.76 \text{ in}^2/ft$
Area of grout	$A_{\text{grout}} = [0.17 \times (I_{\text{b}} - N_{\text{web}} \times t_{\text{bw}} - N_{\text{end}} \times t_{\text{be}}) \times (t - 2 \times t_{\text{bf}})] / I_{\text{b}} = 7.95 \text{ in}^2/\text{ft}$
Density of grout	$\gamma_{\text{grout}} = 140 \text{ lb/ft}^3$
Self weight of wall	wsw = $A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout} = 43.47 \text{ psf}$
◀	—15.625"►



#### From TMS 602-11 Table 2 - Compressive strength of masonry

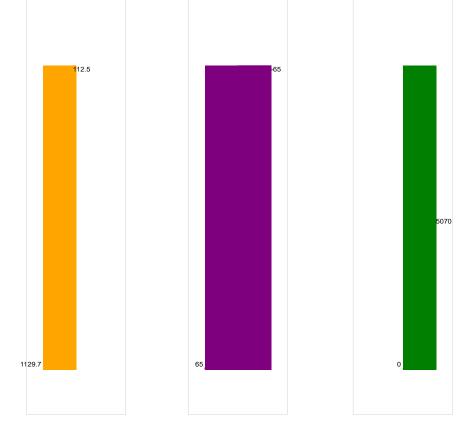
	gin or macomy
Net compressive strength of masonry	f' <sub>m</sub> = <b>1500</b> psi
Modulus of elasticity for masonry	E <sub>m</sub> = 900 × f'm = <b>1350000</b> psi
Shear modulus of masonry	$G_v = 0.4 \times E_m = $ <b>540000</b> psi
From TMS 402 -11 Table 3.1.8.2 - Modulus of ru	upture
Modulus of rupture normal to bed	fr_norm <b>= 80</b> psi
Modulus of rupture parallel to bed	f <sub>r_para</sub> = <b>125</b> psi
Reinforcement details	
Yield strength of reinforcement	fy = <b>60000</b> psi
Allowable tensile stress in reinforcement	F <sub>s</sub> = <b>32000</b> psi
Modulus of elasticity for reinforcement	Es = <b>29000000</b> psi
Vertical reinforcement provided	No.5 bars at 48 in centers
Area of vertical reinforcement	$A_s = \pi \times Dia^2 / (4 \times s) = 0.08 in^2/ft$
Lateral out-of-plane loads	
Wind load on panel	W = <b>5</b> psf
Wind load on parapet	W <sub>p</sub> = <b>70</b> psf
Seismic load factor (ASCE7 12.11.1)	$F_{p} = 0.4 \times S_{\text{DS}} \times I_{e} = \textbf{0.138}$
Seismic load from wall	$E_{wall} = max(F_{p}, 0.1) \times w_{SW} = 6 psf$
Additional seimic load	E <sub>add</sub> = <b>0</b> psf
Seismic lateral load on panel	E = E <sub>wall</sub> + E <sub>add</sub> = <b>6</b> psf



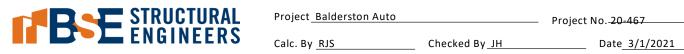


Lateral in-plane loads		
Vertical loading details		
Dead load at supported level		DL = <b>125</b> lb/ft
Live load from above		LL <sub>above</sub> = <b>250</b> lb/ft
Vertical seismic load factor a	oplied to dead load	$F_{Ev} = 0.2 \times S_{DS} = 0.069$
From ASCE 7-10 cl.2.3.2 - C	ombining factored I	oads using strength design (Utilization)
Load combination no.1	$1.4 \times DL$ (0.113)	
Load combination no.5	1.2 × DL + W + LL	+ $0.5 \times (LL_r \text{ or } SL \text{ or } RL) (0.272)$
Load combination no.7	$0.9 \times DL + W$ (0.29)	94)
Properties of masonry sect	ion	
Cross-sectional area		$\label{eq:A} A = [t \times I_b - 0.83 \times (I_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] \ / \ I_b = \textbf{52.7}$ in²/ft
Properties for walls loaded ou	ıt-of-plane:	
Moment of inertia		I = t <sup>3</sup> / 12 - 0.83 × (I <sub>b</sub> - N <sub>web</sub> × t <sub>bw</sub> - N <sub>end</sub> × t <sub>be</sub> ) × (t - 2 × t <sub>bf</sub> ) <sup>3</sup> / (12 × I <sub>b</sub> ) =
		<b>358.4</b> in <sup>4</sup> /ft
Section modulus		S = I / c = <b>94</b> in <sup>3</sup> /ft
Radius of gyration		r = √[I / A] = <b>2.608</b> in
Effective height factor		K = 1
Consider wall at maximum	moment location un	der load combination no.7

Axial force, out of plane - Combination No.7 - Ibs/ftShear force, out of plane - Combination No.7 - Ibs/ftMoment force, out of plane - Combination No.7 - Ib\_in

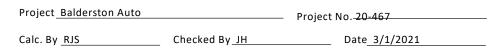


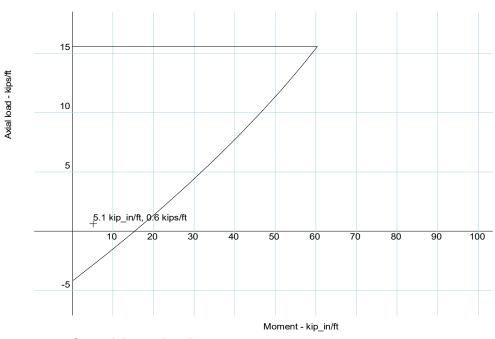
Maximum moment location



Axial load at mid-height of panel	P = <b>621</b> lb/ft
Slenderness ratio	(K × h) / r = <b>119.645</b> > 99
Nominal axial strength	$P_{n} = 0.8 \times (0.8 \times f'_m \times (A - A_s) + A_s \times 0 \text{ ksi}) \times (70 \times r \ / \ (K \times h))^2 = \textbf{17294}$
	lb/ft
Strength reduction factor	$\phi = 0.9$
Design axial strength	$\phi \times P_n$ = <b>15565</b> lb/ft
	P / (φ × P <sub>n</sub> ) = <b>0.040</b>
	PASS - Nominal axial strength exceeds axial load
Factored axial stress	P / t = <b>7</b> psi
Factored axial stress limit	0.05 × f'm = <b>75</b> psi
PASS - Allo	owable stress under out of plane loads exceeds factored axial stress
Nominal cracking moment strength	$M_{cr} = S \times f_{r_norm} = 7489 \text{ Ib_in/ft}$
Modular ratio	$n = E_s / E_m = 21.481$
Distance to neutral axis	$c_{cr} = (A_s \times f_y + P) / (0.64 \times f_m) = 0.453$ in
Moment of inertia of cracked section	$I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3 = 22.1 \text{ in}^4/\text{ft}$
By iteration	$M_{u0} = M = 5070 \text{ lb_in/ft}$
	$\delta_{u0}$ = 5 × M <sub>u0</sub> × h <sup>2</sup> / (48 × E <sub>m</sub> × I) = <b>0.106</b> in
	$M_{u1} = M_{u0} + P \times \delta_{u0} = 5136 \text{ lb_in/ft}$
	$\delta_{u1}$ = 5 $\times$ Mu1 $\times$ h² / (48 $\times$ Em $\times$ I) = 0.108 in
Bending moment at mid-height of panel	$M=M_{u0} + P \times \delta_{u1} = 5137 \text{ Ib_in/ft}$
Depth of reinforcement	d = <b>3.867</b> in
Strength reduction factor	$\boldsymbol{\varphi} = 0.9$
Tensile strain in reinforcement	$\epsilon_{s} = f_{y} / E_{s} = 0.0021$
Maximum usable compressive strain of masonry	εmu = <b>0.0025</b>
Fiber of max.compressive strain to neutral axis	$c_{bal} = \epsilon_{mu} \times d / (\epsilon_{mu} + \epsilon_s) = 2.116$ in
Tensile force at balance point	$T_{bal} = A_s \times f_y = 4602 \text{ lb/ft}$
	$\beta_1 = 0.8$
Compressive force at balance point	$C_{\text{bal}}$ = 0.8 × f'm × $\beta_1$ × $c_{\text{bal}}$ = 24375 lb/ft
Design axial force at balance point	$P_{bal} = \phi \times (C_{bal} - T_{bal}) = 17796 \text{ lb/ft}$
Design moment at balance point	$M_{\text{bal}} = \phi \times [T_{\text{bal}} \times (d - t / 2) + C_{\text{bal}} \times (t / 2 - \beta_1 \times c_{\text{bal}} / 2)] = \textbf{65296} \text{ Ib_in/ft}$
Maximum design moment from integration diagram	n Mc = <b>17473</b> Ib_in/ft
	$M / M_c = 0.294$
	PASS - Combination of applied axial load and flexure is acceptable
Maximum area of tensile reinforcement (3.3.3.5)	$A_{s\_max} = 0.64 \times f'_m \times [\epsilon_{mu} / (\epsilon_{mu} + 1.5 \times \epsilon_s)] \times d / f_y = 0.331 \text{ in}^2/\text{ft}$
PA	SS - Area of reinforcement provided is less than maximum allowable







Strength interaction diagram

#### Consider wall at top under load combination no.7

Shear force	V = <b>65</b> lb/ft
Compressive force	Nu = <b>112</b> lb/ft
Net shear area	$A_{nv} = d \times I_b / ((N_{web} + 1) \times s_{grout}) = 7.553 \text{ in}^2/\text{ft}$
Nominal shear strength	$V_n = min([4 - 1.75 \times min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times 10^{-1} \text{ m}$
	Nu, $6 \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} = 1198 \text{ lb/ft}$
Strength reduction factor	$\phi_v = 0.8$
Design shear strength	$\phi_v \times V_n = 959 \text{ lb/ft}$
	$V / (\phi_{V} \times V_{n}) = 0.068$
	PASS - Design shear strength exceeds applied shear strength



Calc. by <u>RS</u>

\_\_\_\_\_ Job Ref. 20-467\_\_\_\_

\_\_\_\_\_ Chk'd by \_\_\_\_\_ Date<u>3/1/2021</u>

### MASONRY LINTEL DESIGN TO TMS/MSJC 2013

#### Using the allowable stress design method

#### Masonry details

Masonry type	Concrete
Density of masonry unit	$\gamma = 135 \text{ lb/ft}^3$
Pattern bond	Running
Mortar type	PCL Type M
Compressive strength of masonry unit	f'cu = 1900.0 psi
Net compressive strength of masonry (Table 2)	f'm = 1900.0 psi
Modulus of elasticity (4.2.2)	$E_m$ = 900 × f' <sub>m</sub> = <b>1710000</b> psi
Allowable flexural tensile stress (8.2.4.2)	Ft = 106.0 psi
Deinfersement deteile	

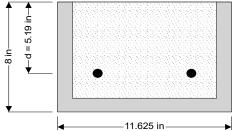
#### Reinforcement details

Allowable tensile stress Modulus of elasticity of steel

#### Cover to reinforcement

Bottom cover to reinforcement Side cover to reinforcement c<sub>nom\_b</sub> = 1.5 in c<sub>nom\_s</sub> = 1.5 in

F<sub>s</sub> = 32000 psi E<sub>s</sub> = 29000000 psi



2 x No.5 bars

Section properties	
Modular ratio	n = E <sub>s</sub> / E <sub>m</sub> = <b>16.96</b>
Section width	b = <b>11.625</b> in
Section depth	h = 8 in
Net shear area	$A_{nv} = b \times h = 93 \text{ in}^2$
Section modulus	$S = b \times h^2 / 6 = 124 in^3$
Depth to tension reinforcement	d = <b>5.19</b> in
Flexure design (Chapter 8)	
Tension reinforcement	2 x No. 5 bars
Area of tension reinforcement	As = Nbot × BarAreabot = 0.62 in <sup>2</sup>
Reinforcement ratio	$\rho_{ratio}$ = As / (b × d) = <b>0.01028</b>
Neutral axis factor	k = $\sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n = 0.441$
Lever arm factor	j = 1 - k / 3 = <b>0.853</b>
Cracking moment	$M_{cr} = 2.5 \times F_t \times S = 2.7 \text{ kip}_ft$
Design bending moment	M = 1.00 kip_ft
Tensile stress in reinforcement	f <sub>s</sub> = M / (A <sub>s</sub> × j × d) = <b>4372</b> psi
Allowable tensile stress in reinf. (8.3.3.1)	Fs = <b>32000</b> psi
Reinforcement stress ratio	fs / Fs = 0.137

Tedds calculation version 1.2.01



Calc. by <u>RS</u> Chk'd by \_\_\_\_\_ Date<u>3/1/2021</u>

	PASS - Allowable tensile stress exceeds tensile stress due to flexure
Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2) = 203.6 \text{ psi}$
Allowable stress in masonry (8.3.4.2.2)	F <sub>b</sub> = 0.45 × f' <sub>m</sub> = <b>855.0</b> psi
Masonry stress ratio	f <sub>b</sub> / F <sub>b</sub> = <b>0.238</b>
PASS -	Allowable compressive stress exceeds compressive stress due to flexure
Shear design (Chapter 8)	
Design shear force	V = <b>0.55</b> kips
Depth of shear area	d <sub>v</sub> = <b>8.00</b> in
Moment shear relationship, M/Vd	Assume M_Vd <sub>ratio</sub> = 1
Shear stress (8-24)	f <sub>v</sub> = V / A <sub>nv</sub> = <b>5.9</b> psi
Allowable masonry shear stress (8-29)	$F_v = 1/2 \times (4.0 - 1.75 \times M_V d_{ratio}) \times \sqrt{(f'_m \times 1psi)}$
	F <sub>v</sub> = <b>49.0</b> psi
Masonry shear stress ratio	f <sub>v</sub> / F <sub>v</sub> = <b>0.121</b>

PASS - Allowable shear stress exceeds shear stress in masonry



Project Balderston

Calc. By RJS

\_\_\_\_\_ Project No.<u>-20-467</u>\_\_\_\_

Checked By\_\_\_\_\_ Date 2/10/2021

Tedds calculation version 1.2.01

#### MASONRY LINTEL DESIGN TO TMS/MSJC 2013

#### Using the allowable stress design method

#### Masonry details

Masonry type	Concrete
Density of masonry unit	$\gamma$ = 135 lb/ft <sup>3</sup>
Pattern bond	Running
Mortar type	PCL Type M
Compressive strength of masonry unit	f'cu = 1900.0 psi
Net compressive strength of masonry (Table 2)	f'm = 1900.0 psi
Modulus of elasticity (4.2.2)	E <sub>m</sub> = 900 × f' <sub>m</sub> = <b>1710000</b> psi
Allowable flexural tensile stress (8.2.4.2)	Ft = 106.0 psi
Reinforcement details	
Allowable tensile stress	F <sub>s</sub> = 32000 psi

# Allowable tensile stress

Modulus of elasticity of steel

#### **Cover to reinforcement**

Bottom cover to reinforcement Side cover to reinforcement

 $c_{nom b} = 1.5 in$  $c_{nom_s} = 1.5$  in

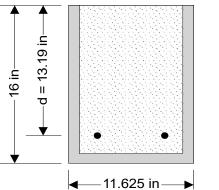
Es = 29000000 psi

n = Es / Em = 16.96

 $A_{nv} = b \times h = 186 \text{ in}^2$ 

b = **11.625** in

h = **16** in



2 x No.5 bars

#### Section properties

Modular ratio Section width Section depth Net shear area Section modulus Depth to tension reinforcement

### Flexure design (Chapter 8)

Tension reinforcement Area of tension reinforcement Reinforcement ratio Neutral axis factor Lever arm factor Cracking moment

 $S = b \times h^2 / 6 = 496 in^3$ d = **13.19** in 2 x No. 5 bars As = Nbot × BarAreabot = 0.62 in<sup>2</sup>  $\rho_{ratio} = A_s / (b \times d) = 0.00404$  $k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n = 0.308$ j = 1 - k / 3 = **0.897**  $M_{cr}$  = 2.5 × F<sub>t</sub> × S = **11.0** kip\_ft

Sht. No. <u>1</u> C15 of 18 2

 Project Balderston
 Project No. -20-467

 Calc. By RJS
 Checked By
 Date 2/10/2021

Design bending moment	M = <b>15.80</b> kip_ft
Tensile stress in reinforcement	fs = M / (As × j × d) = <b>25838</b> psi
Allowable tensile stress in reinf. (8.3.3.1)	Fs <b>= 32000</b> psi
Reinforcement stress ratio	fs / Fs = <b>0.807</b>
	PASS - Allowable tensile stress exceeds tensile stress due to flexure
Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2) = 678.3 \text{ psi}$
Allowable stress in masonry (8.3.4.2.2)	Fb = 0.45 × f'm = <b>855.0</b> psi
Masonry stress ratio	f <sub>b</sub> / F <sub>b</sub> = <b>0.793</b>
PASS - AI	lowable compressive stress exceeds compressive stress due to flexure
Shear design (Chapter 8)	
Design shear force	V = <b>4.80</b> kips
Depth of shear area	d <sub>v</sub> = <b>16.00</b> in
Moment shear relationship, M/Vd	Assume M_Vd <sub>ratio</sub> = 1
Shear stress (8-24)	f <sub>v</sub> = V / A <sub>nv</sub> = <b>25.8</b> psi

F<sub>v</sub> = **49.0** psi

 $f_v / F_v = 0.526$ 

Masonry shear stress ratio

Allowable masonry shear stress (8-29)

PASS - Allowable shear stress exceeds shear stress in masonry

 $F_v = 1/2 \times (4.0 - 1.75 \times M_V d_{ratio}) \times \sqrt{(f'_m \times 1psi)}$ 



\_\_\_\_\_ Project No. <u>20-467</u>

Calc. By RJS

\_\_\_\_\_ Checked By\_\_\_\_\_ Date\_2/10/2021\_

#### MASONRY LINTEL DESIGN TO TMS/MSJC 2013

#### Using the allowable stress design method

### Masonry details

Masonry type	Concrete
Density of masonry unit	$\gamma$ = 135 lb/ft <sup>3</sup>
Pattern bond	Running
Mortar type	РСL Туре М
Compressive strength of masonry unit	f'cu = 1900.0 psi
Net compressive strength of masonry (Table 2)	f'm = 1900.0 psi
Modulus of elasticity (4.2.2)	$E_m = 900 \times f'_m = 1710000 \text{ psi}$
Allowable flexural tensile stress (8.2.4.2)	Ft = 106.0 psi
Reinforcement details	
Allowable tensile stress	F <sub>s</sub> = 32000 psi

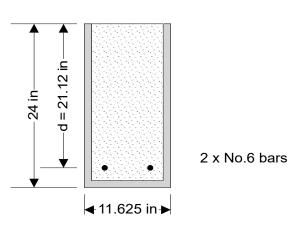
Allowable tensile stress Modulus of elasticity of steel

#### Cover to reinforcement

Bottom cover to reinforcement Side cover to reinforcement

 $c_{nom b} = 1.5 in$  $c_{nom_s} = 1.5$  in

Es = 29000000 psi



#### Section properties

Modular ratio Section width Section depth Net shear area Section modulus Depth to tension reinforcement

#### Flexure design (Chapter 8)

Tension reinforcement Area of tension reinforcement Reinforcement ratio Neutral axis factor Lever arm factor Cracking moment

n = Es / Em = 16.96 b = **11.625** in h = **24** in  $A_{nv} = b \times h = 279 \text{ in}^2$  $S = b \times h^2 / 6 = 1116 in^3$ d = **21.12** in 2 x No. 6 bars As = Nbot × BarAreabot = 0.88 in<sup>2</sup>  $\rho_{ratio}$ = As / (b × d) = 0.00358 k =  $\sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n = 0.293$ j = 1 - k / 3 = **0.902**  $M_{cr}$  = 2.5 × F<sub>t</sub> × S = **24.6** kip\_ft

Tedds calculation version 1.2.01

Project Balderston Auto Project No.-20-467

PASS - Allowable shear stress exceeds shear stress in masonry

Calc. By <u>RJS</u> Checked By\_\_\_\_\_ Date <u>2/10/2021</u>

Design bending moment	M = <b>28.00</b> kip_ft
Tensile stress in reinforcement	fs = M / (As × j × d) = <b>20036</b> psi
Allowable tensile stress in reinf. (8.3.3.1)	Fs <b>= 32000</b> psi
Reinforcement stress ratio	fs / Fs = <b>0.626</b>
	PASS - Allowable tensile stress exceeds tensile stress due to flexure
Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2) = 490.0 \text{ psi}$
Allowable stress in masonry (8.3.4.2.2)	F <sub>b</sub> = 0.45 × f' <sub>m</sub> = <b>855.0</b> psi
Masonry stress ratio	f <sub>b</sub> / F <sub>b</sub> = <b>0.573</b>
PASS - A	llowable compressive stress exceeds compressive stress due to flexure
Shear design (Chapter 8)	
Design shear force	V = <b>5.00</b> kips
Depth of shear area	d <sub>v</sub> = <b>24.00</b> in
Moment shear relationship, M/Vd	
Moment shear relationship, M/Vu	Assume M_Vd <sub>ratio</sub> = 1
Shear stress (8-24)	Assume $M_V d_{ratio} = 1$ $f_v = V / A_{nv} = 17.9$ psi
	—
Shear stress (8-24)	$f_v = V / A_{nv} = 17.9$ psi

 $f_v / F_v = 0.365$ 

Masonry shear stress ratio

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Project Balderston	Auto	Project No. 20-467
Calc. By RS	Checked By_JB	Date_02/17/2021

### **Summary**

The lateral stability system of the project referenced above consists of metal deck diaphragms and CMU shear walls. All roof diaphragms are designed to transfer lateral loads from the exterior walls, and those induced into the deck by seismic loads, into the shear wall system. Lateral resisting members directly transfer all lateral forces into the foundation system.

The following section of calculations covers the complete design of the lateral stability system for the project referenced above including the distribution of lateral forces into the individual elements. Refer to the "Loads" section of these calculations for the determination of all wind and seismic loads.



Calc. By RJS

### Wind Load Distribution - Single Diaphragm Design, Envelope Method

#### General Building Info :

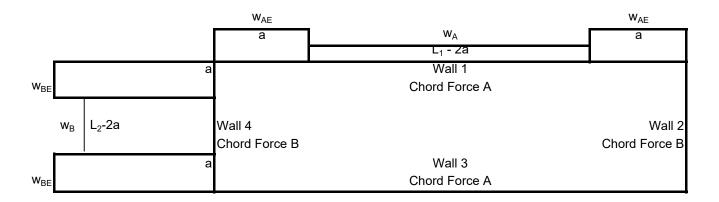
		Wall Height to		Total Wall
	Length	Roof	Parapet Height	Height
Wall 1 =	220.00 ft	25.00 ft	3.00 ft	28.00 ft
Wall 2 =	58.00 ft	25.00 ft	3.00 ft	28.00 ft
Wall 3 =	220.00 ft	25.00 ft	3.00 ft	28.00 ft
Wall 4 =	58.00 ft	25.00 ft	3.00 ft	28.00 ft

#### **General Wind Loading Info :**

ASD Factor = a = Zone 1 Pressure = Zone 1E Pressure = Zone 4 Pressure =	0.6 7.25 ft 5.60 psf 10.90 psf 11.90 psf 15.40 psf	(1 for ACSE 7-05, 0.6 for ASCE 7-10) (Length of End Zone per ACSE) (Refer to Tedd's Calculations for Pressures)
Zone 4E Pressure = WW Parapet Pressure = LW Parapet Pressure =	15.40 psf 39.89 psf 26.59 psf	

#### Wind Loading @ Roof:

L <sub>1</sub> - 2a = Distributed Force, w <sub>A</sub> = Distributed Force, w <sub>AE</sub> =	205.50 ft 418.19 plf 528.19 plf	(Pressures are Based on Entered Wind Pressures)
L <sub>2</sub> - 2a = Distributed Force, w <sub>B</sub> = Distributed Force, w <sub>BE</sub> =	43.50 ft 418.19 plf 528.19 plf	(Pressures are Based on Entered Wind Pressures)



#### Shear Wall Loads:

Wall 1 =	7.8 k ASD	x Factor =	12.9 k LRFD	(Distributed Load x Effective Trib)
Wall 2 =	28.1 k ASD	x Factor =	46.8 k LRFD	(Distributed Load x Effective Trib)
Wall 3 =	7.8 k ASD	x Factor =	12.9 k LRFD	(Distributed Load x Effective Trib)
Wall 4 =	28.1 k ASD	x Factor =	46.8 k LRFD	(Distributed Load x Effective Trib)



Project.	Balderston Auto		Project No. 20-4	67
Calc. By	RJS	Checked By	Date	2/20/2021

56.15808 0.255264 0.246461793

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ENGINEERS	Calc.

Project Balderston Auto		Project No. 20-4	467	
Calc. By RJS	Checked By	Date	2/20/2021	

### Diaphragm Loads:

Wall 1 =	35.3 plf ASD	x Factor =	58.8 plf LRFD	(Wall 1 Load*1000/Wall 1 Length)
Wall 2 =	484.1 plf ASD	x Factor =	806.9 plf LRFD	(Wall 2 Load*1000/Wall 2 Length)
Wall 3 =	35.3 plf ASD	x Factor =	58.8 plf LRFD	(Wall 3 Load*1000/Wall 3 Length)
Wall 4 =	484.1 plf ASD	x Factor =	806.9 plf LRFD	(Wall 4 Load*1000/Wall 4 Length)

### Deck Design:

Deck =	22 GA Type B Metal Roof deck	(Per Vulcraft Catalog)
Weld Pattern =	36/7	(Per Vulcraft Catalog)
Sidelap Fasteners =	(2) #10 TEK Screws	(Per Vulcraft Catalog)
Allowable Load =	328.0 plf ASD	(Per Vulcraft Catalog)
Max Diaphragm Load =	484.1 plf ASD	
	N.G.	

### Chord Loads:

Chord Force A =	26.2 k ASD	x Factor =	43.7 k LRFD	( M <sub>wind</sub> against A / Wall B Length )
Chord Force B =	0.5 k ASD	x Factor =	0.8 k LRFD	( M <sub>wind</sub> against B / Wall A Length )

### Chord Design:

Chord A Design:		
Angle =	L3x3x1/2	
Angle Area =	2.8 in^2	(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	89.4 k LRFD	(AISC 360 Eq. D2-1Φ⊺ = 0.9*Area *36 ksi )
Comp. Capacity, ФР =	39.7 k LRFD	(AISC Manual Table 4-11 )
	N.G.	
Chord B Design:		
Angle =	L3x3x1/4	
Angle Area =	2.8 in^2	(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	89.4 k LRFD	(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi )
Comp. Capacity, ΦP =	32.6 k LRFD	(AISC Manual Table 4-11)
	ОК	



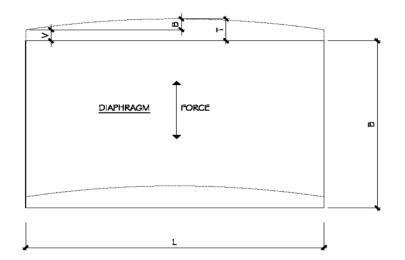
Project Balderston Auto Project No. 20-467
Calc. By RJS Checked By Date 2/20/2021

### **Diaphragm Deflection Calculations : Simply Supported**

<u>Span</u>	<u>Max Joist</u> Spacing (ft)	<u>L (ft)</u>	<u>B (ft)</u>	<u>E (ksi)</u>	<u>Chord Area A</u> (in <sup>2</sup> )
Wall 2 to 4	6	220.00	58.00	29000	2.76
Wall 1 to 3	6	58.00	220.00	29000	2.76

Values From Vulcraft Deck Catalog		Loading	Deflections			
<u>K<sub>2</sub></u>	<u>DB</u>	<u>K₁</u>	<u>q(lb/ft)</u>	<u>∆B (in)</u>	<u>∆V (in)</u>	<u>∆T (in)</u>
870	129	0.204	250.914	0.68	0.418225305	1.10
870	129	0.204	250.914	0.00	0.007663484	0.01

- $\Delta B$ = Deflection due to moment in inches
- $\Delta V$ = Deflection due to shear in inches
- ΔT= Total deflection in inches
- q(lb/ft)= distributed load on diaphragm
  - $K_1$ = Factor from steel deck catalog
  - DB= Factor from steel deck catalog
  - $K_2$  = Factor from steel deck catalog
- A(in<sup>2</sup>)= Area of chord member
- E (ksi)= Modulus of elasticity of steel
- B (ft)= Diaphragm Depth
- L (ft)= Diaphragm Span
- I (in<sup>4</sup>)= Moment of Inertia of diaphragm
- G' (k/in) = Effective shear modulus of decking
  - Span = Max Joist Spacing



5qL<sup>4</sup>/384El qL<sup>2</sup>/8BG' ΔB+ΔV

2A(B/2)<sup>2</sup> K<sub>2</sub>/(3.78+(0.38\*DB/Span)+3\*K<sub>1</sub>\*Span)

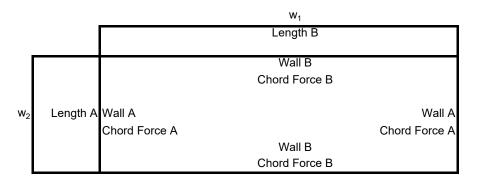


Project Balderston Auto		Project No. 20-	467
Calc. By_RJS	Checked By	Date_	2/20/2021

## Seismic Load Distribution - Single Diaphragm Design

### General Info :

Seismic Response Coeff. C <sub>s</sub> = Roof Dead Load =	0.0475 15.00 psf	(from Tedds Seismic Load Calculations)
Wall Length A = Wall Weight A = Wall Height A = Parapet Height A = Distributed Force, w <sub>1</sub> =	183.00 ft 12.00 psf 16.67 ft 1.00 ft 148.63 plf	(Roof to FFE) (Top of Parapet to Roof) ((Length A*Dead+(0.5*Height B+Parapet)*Weight B) * C <sub>S</sub> )
Wall Length B = Wall Weight B = Wall Height B = Parapet Height B = Distributed Force, w <sub>2</sub> =	225.00 ft 12.00 psf 16.00 ft 8.00 ft 170.95 plf	(Roof to FFE) (Top of Parapet to Roof) ((Length B*Dead+(0.5*Height A+Parapet)*Weight A) * C <sub>S</sub> )



### Shear Wall Loads from Diaphragm:

	/all A = /all B =		/0.7 = /0.7 =	16.7 k LRFD 15.6 k LRFD	( (0.7*w <sub>1</sub> *Length B*0.5)/1000 ) ( (0.7*w <sub>2</sub> *Length A*0.5)/1000 )
Total Shear	Wall Loads (	Includes Wall W	leight):		
	/all A = /all B =		/0.7 = /0.7 =	18.6 k LRFD 18.7 k LRFD	( Dia. A+Length A*Weight A*Height A*C <sub>S</sub> ) ( Dia. B+Length B*Weight B*Height B*C <sub>S</sub> )
<u>Diaphragm L</u>	<u>_oads:</u>				
				91.4 plf LRFD 69.5 plf LRFD	(Wall A Load*1000/Wall A Length) (Wall B Load*1000/Wall B Length)

### Deck Design:

Project Balderston Auto		Project No. 20-	467
Calc. By_ <b>RJS</b>	Checked By	Date	2/20/2021

	Deck = Weld Pattern = Sidelap Fasteners = Allowable Load = Max Diaphragm Load =	22 GA Type B Meta 36/7 (2) #10 TEK Screw 328.0 plf ASD 64.0 plf ASD OK		k (Per Vulcraft Catalog) (Per Vulcraft Catalog) (Per Vulcraft Catalog) (Per Vulcraft Catalog)
Chord Lo	bads:			
<u>Chord De</u>	Chord Force A = Chord Force B = esign:	2.2 k ASD 3.6 k ASD	/0.7 = /0.7 =	3.2 k LRFD ( M <sub>wind</sub> against A / Wall B Length ) 5.1 k LRFD ( M <sub>wind</sub> against B / Wall A Length )
	Chord A Design: Angle = Angle Area = Tensile Capacity, $\Phi T$ = Comp. Capacity, $\Phi P$ = Chord B Design: Angle = Angle Area = Tensile Capacity, $\Phi T$ = Comp. Capacity, $\Phi P$ =	L3x3x1/4 1.4 in^2 46.7 k LRFD 20.9 k LRFD k OK L3x3x1/4 1.4 in^2 46.7 k LRFD 32.6 k LRFD k OK		(AISC Manual Table 1-7) (AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi ) (AISC Manual Table 4-11) (AISC Manual Table 1-7) (AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi ) (AISC Manual Table 4-11)



 Project\_\_\_\_\_\_
 Job Ref.
 \_\_\_\_\_\_

 Calc. by <u>RS</u>
 Chk'd by \_\_\_\_\_\_
 Date\_3/1/2021

#### MASONRY WALL PANEL DESIGN TO TMS 402/602-16

#### Using the allowable stress design method

#### Masonry wall panel details

Solid masonry wall without parapet - Reinforced single-wythe wall, the wall is pinned at the top and fixed at the bottom for out of plane loads

The wall is	fixed at	t the bottom	and free	at the to	o for in	plane loads
ino wan io	inkou ui		i una noo			plune loude

Panel length	L = <b>58</b> ft	
Panel height	h = <b>21</b> ft	
◀		



#### **Seismic properties**

Seismic design category	A		
Seismic importance factor (ASCE7 Table 1.5-2)	le = 1		
Design spectral response acceleration parameter,	short periods (ASCE7 11.4.4)		
	S <sub>DS</sub> = 1		
Seismic wall classification	Nonparticipating		
	No prescriptive minimum seismic reinforcement		
Redundancy factor, on out-of-plane load	ρε = <b>1.0</b>		
Construction details			
Wall thickness	t = <b>7.625</b> in		

#### Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit

Density of masonry units

Height of masonry units

f'<sub>cu</sub> = **2800** psi γ<sub>block</sub> = **115** lb/ft<sup>3</sup> h<sub>b</sub> = **7.625** in

Sht. No. <u>1</u> <u>D8</u> of <u>13</u> 4

Tedds calculation version 2.2.04

	Project		Job Ref	
ENGINEERS	Calc. by <u>RS</u>	Chk'd by	Date <u>3/1/2021</u>	
Length of masonry units	Ib =	<b>15.625</b> in		
Number of internal webs	Nwet	= 1		
Number of end webs	Nend	= 2		
Internal web thickness	t <sub>bw</sub> = <b>1.25</b> in			
Face shell thickness	t <sub>bf</sub> =	<b>1.25</b> in		
End web thickness	t <sub>be</sub> =	<b>1.25</b> in		
Area of block	Abloo	$_{\rm k}$ = [t $\times$ lb - (lb - Nweb $\times$ tbw - Ne	$t_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / I_b = 44.76 \text{ in}^2/\text{ft}$	
Area of grout	Agro	$t = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{web})$	$\times$ t <sub>be</sub> ) $\times$ (t - 2 $\times$ t <sub>bf</sub> )] / I <sub>b</sub> = <b>7.95</b> in <sup>2</sup> /ft	
Density of grout	γgrou			
Self weight of wall	Wsw	= Ablock $\times \gamma$ block + Agrout $\times \gamma$ grou	ut = <b>43.47</b> psf	
, 1, 625"			▲1.25 <b>)</b> ▲1.25 <b>)</b> ▲1.25 <b>)</b>	



# From TMS 602-16 Table 2 - Compressive strength of masonry

Net compressive strength of masonry	f'm = <b>2250</b> psi
Modulus of elasticity for masonry	$E_m = 900 \times f'_m =$

Shear modulus of masonry

= 2024584 psi G<sub>v</sub> = 0.4 × E<sub>m</sub> = **809834** psi

### From TMS 402 -16 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed	Ft_norm = <b>38</b> psi
Allowable flexural tensile stress parallel to bed	F <sub>t_para</sub> <b>= 66</b> psi
Reinforcement details	
Yield strength of reinforcement	fy = <b>60000</b> psi
Allowable tensile stress in reinforcement	Fs <b>= 32000</b> psi
Modulus of elasticity for reinforcement	Es = <b>29000000</b> psi
Vertical reinforcement provided	No.5 bars at 48 in centers
Area of vertical reinforcement	As = $\pi \times \text{Dia}^2 / (4 \times \text{s}) = 0.08 \text{ in}^2/\text{ft}$
Lateral out-of-plane loads	
Wind load on panel	W = <b>5</b> psf
Wind load on parapet	W <sub>p</sub> = <b>18</b> psf
Seismic load factor (ASCE7 12.11.1)	$F_p = 0.4 \times S_{DS} \times I_e = 0.4$
Seismic load from wall	$E_{wall} = max(F_{p}, 0.1) \times w_{SW} = 17.4 \text{ psf}$
Additional seimic load	E <sub>add</sub> = <b>0</b> psf
Seismic lateral load on panel	E = E <sub>wall</sub> + E <sub>add</sub> = <b>17.4</b> psf

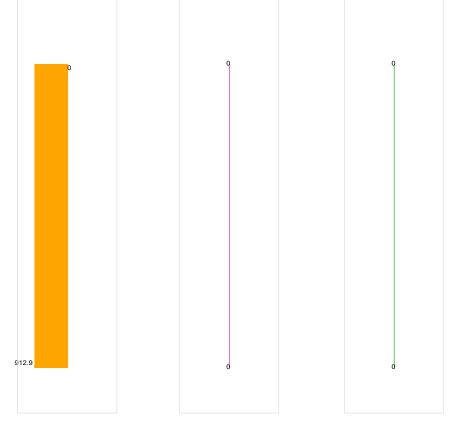


Calc. by <u>RS</u> Chk'd by \_\_\_\_\_ Date\_<u>3/1/2021</u>

\_\_\_\_\_ Job Ref. \_\_\_\_

Lateral in-plane loads							
Vertical loading detailsVertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS} = 0.2$							
From ASCE 7-16 cl.2.4 - Combining nominal loads using allowable stress design (Utilization) Load combination no.1 DL (0.059)							
Properties of masonry section							
Cross-sectional area	A = [t × l <sub>b</sub> - 0.83 × (l <sub>b</sub> - N <sub>web</sub> × t <sub>bw</sub> - N <sub>end</sub> × t <sub>be</sub> ) × (t - 2 × t <sub>bf</sub> )] / l <sub>b</sub> = <b>52.7</b> in <sup>2</sup> /ft						
Properties for walls loaded out-of-plane:							
Moment of inertia	$I = t^{3} / 12 - 0.83 \times (I_{b} - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^{3} / (12 \times I_{b}) = 358.4 \text{ in}^{4}/\text{ft}$						
Section modulus	S = I / c = <b>94</b> in <sup>3</sup> /ft						
Radius of gyration	r = √[I / A] = <b>2.608</b> in						
Effective height factor	K = 1						
Consider wall at bottom under load combination	Consider wall at bottom under load combination no.1						

Axial force, out of plane - Combination No.1 - Ibs/ftShear force, out of plane - Combination No.1 - Ibs/ftMoment force, out of plane - Combination No.1 - Ib\_in



P = 913 lb/ft Axial load at bottom of panel Compressive stress due to axial load Slenderness ratio Allowable compressive stress due to axial load

fa = P / A = **17.3** psi (K × h) / r = **96.636** < 99  $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] = 294.4 \text{ psi}$ fa / Fa = **0.059** 



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### PASS - Allowable compressive stress exceeds compressive stress due to axial loads

Allowable compressive force

 $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] = 15496 \text{ lb/ft}$ 

P / Pa = 0.059

PASS - Allowable compressive force exceeds axial load



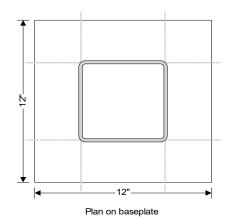
 Project
 Project No.

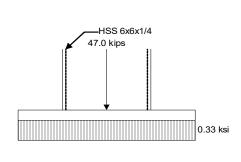
 Calc. By <u>RS</u>
 Checked By

Date <u>1/9/2019</u>

### COLUMN BASE PLATE DESIGN







Elevation on baseplate

Design forces and moments	
Axial force	Pu = <b>47.0</b> kips (Compression)
Bending moment	M <sub>u</sub> = <b>0.0</b> kip_in
Shear force	F <sub>v</sub> = <b>0.0</b> kips
Column details	
Column section	HSS 6x6x1/4
Depth	d = <b>6.000</b> in
Breadth	bf = <b>6.000</b> in
Thickness	t = <b>0.233</b> in
Baseplate details	
Depth	N = <b>12.000</b> in
Breadth	B = <b>12.000</b> in
Thickness	t <sub>p</sub> = <b>0.750</b> in
Design strength	F <sub>y</sub> = <b>36.0</b> ksi
Foundation geometry	
Member thickness	h <sub>a</sub> = <b>34.000</b> in
Dist center of baseplate to left edge foundation	x <sub>ce1</sub> = <b>48.000</b> in
Dist center of baseplate to right edge foundation	x <sub>ce2</sub> = <b>48.000</b> in
Dist center of baseplate to bot edge foundation	y <sub>ce1</sub> = <b>48.000</b> in
Dist center of baseplate to top edge foundation	y <sub>ce2</sub> = <b>48.000</b> in
Minimum tensile strength, base plate	F <sub>y</sub> = <b>36</b> ksi
Minimum tensile strength, column	F <sub>yCol</sub> = <b>50</b> ksi
Compressive strength of concrete	f'c <b>= 4</b> ksi
Strength reduction factors	
Compression	$\phi_{\rm c} = 0.60$
Flexure	$\phi_{\rm b} = 0.90$
Weld shear	$\phi_{\rm V}=0.75$
Plate cantilever dimensions	
Area of base plate	A <sub>1</sub> = B × N = <b>144.000</b> in <sup>2</sup>
Maximum area of supporting surface	$A_2 = (N + 2 \times I_{min}) \times (B + 2 \times I_{min}) = 9216.000 \text{ in}^2$
Nominal strength of concrete under base plate	$P_{p}$ = 0.85 $\times$ f'c $\times$ A1 $\times$ min( $\sqrt{(A_{2}$ / A1), 2)} = 979.2 kips

Tedds calculation version 2.1.01 Flange/base weld - 0.3" Web/base weld - 0.3"



Project\_\_\_\_\_

Project No.\_\_\_\_

Calc. By <u>RS</u> Checked By Date <u>1/9/2019</u>

Bending line cantilever distance m Bending line cantilever distance n Maximum bending line cantilever

#### Plate thickness

Required plate thickness Specified plate thickness

### Design bearing strength (AISC 360-05-J8)

Design bearing strength Factored bearing strength

#### Flange weld

Flange weld leg length Tension capacity of flange Force in tension flange Critical force in flange Flange weld force per in Electrode classification number Design weld stress Design strength of weld per in  $m = (N - 0.95 \times d) / 2 = 3.150 \text{ in}$ n = (B - 0.95 × bf) / 2 = 3.150 in I = max(m, n) = 3.150 in

$$\label{eq:tp,req} \begin{split} t_{p,req} = I \times \sqrt{((2 \times P_u) \: / \: (\varphi_b \times F_y \times B \times N))} = \textbf{0.447} \text{ in} \\ t_p = \textbf{0.750} \text{ in} \end{split}$$

PASS - Thickness of plate exceeds required thickness

 $P_p$  = 979.20 kips  $\phi_c P_p$  = 587.52 kips PASS - Allowable bearing stress exceeds applied bearing stress

$$\begin{split} t_{wf} &= \textbf{0.3126 in} \\ P_{tf} &= b_f \times t \times F_{yCol} = \textbf{69.9 kips} \\ F_{tf} &= M_u / (d - t) - P_u \times (b_f \times t) / A_{col} = \textbf{-12.5 kips} \\ F_f &= \min(P_{tf}, \max(F_{tf}, 0kips)) = \textbf{0.0 kips} \\ R_{wf} &= F_f / b_f = \textbf{0.0 kips/in} \\ F_{EXX} &= \textbf{70.0 ksi} \\ \phi F_w &= \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5}) = \textbf{47.250ksi} \\ \phi R_{nf} &= \phi F_w \times t_{wf} / \sqrt{2} = \textbf{10.4 kips/in} \end{split}$$

PASS - Available strength of flange weld exceeds force in flange weld



Project Balderston A	uto P	Project No. 20-467
Calc. By_RS	Checked By_JB	Date 2/18/2021

### <u>Summary</u>

The gravity structure system of the project referenced above consists primarily of steel joists and girders, load-bearing CMU walls and steel columns. The load bearing walls are supported at grade by continuous shallow foundations. Interior steel framing is supported by steel columns. All columns are supported by shallow spread foundations. The locations of all footings are indicated on the structural foundation plans.

The following section of calculations covers the complete design of the foundation system for project referenced above, including the design of retaining walls located on the site. Refer to the "Loads" section of these calculations for the determination of all dead, live, roof live, and snow loads. Refer to the "Roof Framing" and "Tilt Panel" section of these calculations for the design of the sector being supported by the continuous and spread footings.



20-467 **Balderston Auto** Project\_ \_\_ Project No.\_\_

Calc. By RJS Checked By JH Date 03/01/21

### Footing Designation: F1

Footing Designation:	F1						P/Pu
Footing Location:	Interior w/ sola	ar					B is into the page M/Mu
General Information:							V
Footing Length, L = Footing Width, B = Footing Depth, H = Location = Steel Depth, d = Typical Slab Depth = Slab Depth Above Footing = Area of Footing = Soil Bearing Pressure = Allowable or Effective SBC? Concrete Strength = Column Size = Base Plate Size = Critical Section =	5 5 12 Interior 8.0625 5 8 25 2.5 Allowable 3 B Direction 8.00 in 14.00 in 11.00 in		x x x	L Direction 8.00 in 14.00 in 11.00 in	(B*L)	· 1.5*Bar Dia	Qmin L Length of Soil Pressure
Loading:							
Vertical Loads: Applied Dead Load = Slab + Wall +Footing Weight = Applied Live Load = ASD Total Load, P = LRFD Total Load, Pu = ASD Uplift Load = LRFD Uplift Load =	9.7 5.3125 10.3 25.3125 34.495 6.7 10.72	k k k k k k k		LRFD Factors: Dead = Live = Uplift=		1.2 1.6 1.6	( ASCE 7 Combo )
Moments:				LRFD Factors:			
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu =	0 0 0 0	k-ft k-ft k-ft k-ft		Dead = Wind =		1.2 1.6	(ASCE 7 Combo )
ASD Soil Pressures:							
e = Kern = e > = < Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmax = Is Qmax <sbc?< th=""><th>0.000 0.833 Less Than 5.000 1.013 1.013 <b>YES</b></th><th>ft ft ksf ksf</th><th></th><th></th><th>("Less Th ("Less Th "Equal To</th><th>r Than", Len ian",Qmin = ian",Qmax = o", Qmax = (2</th><th>gth = 3*(L/2 -e) ; Otherwise = L ) (P/L*B) - (6*M / B*L^2), Otherwise = 0 ) (P/L*B) + (6*M / B*L^2), 2*P) / (L*B) x = (4*P) / (3*B*(L - 2*e) )</th></sbc?<>	0.000 0.833 Less Than 5.000 1.013 1.013 <b>YES</b>	ft ft ksf ksf			("Less Th ("Less Th "Equal To	r Than", Len ian",Qmin = ian",Qmax = o", Qmax = (2	gth = 3*(L/2 -e) ; Otherwise = L ) (P/L*B) - (6*M / B*L^2), Otherwise = 0 ) (P/L*B) + (6*M / B*L^2), 2*P) / (L*B) x = (4*P) / (3*B*(L - 2*e) )
LRFD Soil Pressures: e =	0.000	ft			( ASD M	/ Pu )	
e – Kern = e > = < Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmax = Qcritical = Critical Length =	0.833 Less Than	ft ft ksf ksf ksf ft			(L/6) ("Greate ("Less Th ("Less Th "Equal To "Greater (Qcritical	r Than", Len han",Qmin = han",Qmax = b", Qmax = (2 Than", Qmax = pressure (	gth = 3*(L/2 -e) ; Otherwise = L ) (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0 ) (Pu/L*B) + (6*Mu / B*L^2), 2*Pu) / (L*B) x = (4*Pu) / (3*B*(L - 2*e) ) @ critical section of footing ) - Critical Section/2-d/2)



Project_	Balderston Auto		Pro	20-467 Dject No
Calc. By_	RJS	Checked By	JH	Date 03/01/21

One-Way Shear Check: Vu1 =	11.77	k		( Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5 )
ΦVn =	39.74	k		(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	YES	ĸ		$(A01310-00 Equation 11-3, \Psi M = 0.73 \times 301(10) \times 1000)$
	•			
Two-Way Shear Check:				
b1 =	19.06	in		( Critical Section B + d )
b2 =	19.06	in		(Column Height L + d)
b0 =	76.25	in		(2*b1 + 2*b2 )
Vu2 =	31.01	k		(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2) )
α =	40			(ACI 318-08 Section 11.11.2.1)
β =	1			(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim )
ΦVn =	151.52	k		(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(fc)*bo*d*(2+4/Beta))
ΦVn =	157.32	k		(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(fc)*bo*d*(2+Alpha*d/bo))
ΦVn =	101.02	k		(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	YES			
Column Bearing Check:				
ΦPn =	649.74	k		( ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*fc*Plate Area*2) )
Adequate in Bearing?	YES	ĸ		(A01310-000000000000000000000000000000000
Adequate in Dealing	120			
Uplift Check:				
ASD Combo for Uplift = (	0.6D + Uplift			(ASCE 7)
Uplift Force =	6.7	k		( From Above )
Required Dead Load =	11.17	k		( Uplift / 0.6 )
Applied Dead Load + Slab + Ftg =	15.0125	k		
Additional Slab Used =	0	ft		( Length of Additional Slab in Each Direction )
Wall Weight Over Footing =	0	klf		
Length Parallel to Slab Edge =	0	ft		( B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft		( B or L Depending on the Case)
Area of Cont. Footing =	0	ft <sup>2</sup>		
Length of Cont. Footing Used =	0	ft		
Total Dead Load =	15.0125	k		(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg )
Adequate for Uplift?	Footing is A	dequate to Resist I	Jplift	(Calculation assumes wall is above the cont. ftg.)
Tan Otaala				
Top Steel: Mu =	0.62	k-ft / ft		( Mu = (Pu/A)*0.5*Crit. L^2 )
	23.529	K-II / II		(m = fy/(0.85  fr))
m = Ru =	0.010	ksi		$(Ru = Mu/(0.9*12 inches*d^2))$
		N51		
ρ Req'd =	0.0002			$(\rho = (1/m)^{*}(1-\operatorname{sqrt}(1-2^{*}\operatorname{Ru}^{*}m/fy)))$
ρ Min. =	0.0027			(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy )
4/3*Mu ρ Req'd =	0.0002			$(\rho = (1/m)^{*}(1-sqrt(1-2^{*}1.33^{*}Ru^{*}m/fy)))$
Governing ρ =	0.0002	. 2.0		( If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.022	in²/ft		( As = Governing ρ*12 inches*d )
Bar # =	4			
Bar Spacing =	12	in		
As Provided =	0.20	in²/ft	=	6 Bars in B Direction
Bottom Steel:			=	6 Bars in L Direction
Bottom Steel: Mu =	2.01	k-ft / ft		(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	23.529	K-II / II		(m = fy/(0.85 fc))
	0.034	ksi		$(Ru = Mu/(0.9*12 inches*d^2))$
ρ Req'd =		1/21		
	0.0006			$(\rho = (1/m)^{*}(1-\operatorname{sqrt}(1-2^{*}\operatorname{Ru}^{*}m/fy)))$
ρ Min. =	0.0027			(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy )
4/3*Mu ρ Req'd =	0.0008			$(\rho = (1/m)^{*}(1-sqrt(1-2^{*}1.33^{*}Ru^{*}m/fy)))$
Governing ρ =	0.0008	200		( If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.074	in²/ft		( As = Governing ρ*12 inches*d )
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in²/ft	=	6 Bars in B Direction
			=	6 Bars in L Direction



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teel:		
0.2592	in²/ft	( T&S Steel = 0.0018*
0.20	in²/ft	
0.31	in²/ft	
0.51	in²/ft	
YES		
5	ft	
5	ft	
12	in	
	0.20 0.31 0.51 YES 5 5	0.2592 in <sup>2</sup> /ft 0.20 in <sup>2</sup> /ft 0.31 in <sup>2</sup> /ft 0.51 in <sup>2</sup> /ft <b>YES</b> 5 ft 5 ft

Top Steel =	#4 bars	@12 inches O.C.
Bottom Steel =	#5 bars	@12 inches O.C.

3\*12 inches\*H)



\_\_\_\_\_ Project No.\_ 20-467 Balderston Auto Project\_

Calc. By\_\_\_\_RJS\_\_\_\_\_Checked By\_\_\_\_JH\_\_\_\_Date\_\_03/01/21

# Footing Designation F2 Footing Location: Interior at Mezzanin

Footing Designation	F2						P/Pu
Footing Location:	Interior at Me	zzanine					B is into M/Mu
General Information:							¥
Footing Length, L = Footing Width, B = Footing Depth, H = Location = Steel Depth, d = Typical Slab Depth = Slab Depth Above Footing = Area of Footing =	8 8 12 Interior 8.0625 4 8 64	ft ft in in in ft^2			(H - 3 in ( B*L )	- 1.5*Bar Dia.	Qmin Qmax L Length of Soil Pressure
Soil Bearing Pressure = Allowable or Effective SBC? Concrete Strength =	2.5 Allowable 3	ksf ksi			(02)		
	B Direction			L Directior	ו		
Column Size =	6.00 in		Х	6.00 in			
Base Plate Size = Critical Section =	12.00 in 9.00 in		X X	12.00 in 9.00 in			
	0100						
Loading:							
Vertical Loads: Applied Dead Load =	45	k		LRFD Factors: Dead =		1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	12.8	k		Live =		1.6	
Applied Live Load =	51	k		Uplift=		1.6	
ASD Total Load, P =	108.8	k		opint			
LRFD Total Load, Pu =	150.96	k					
ASD Uplift Load =	0	k					
LRFD Uplift Load =	0	k					
Moments:				LRFD Factors:			
Dead Load Moment =	0	k-ft		Dead =	-	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft		Wind =		1.6	
ASD Total Moment, M =	0	k-ft		Villa		1.0	
LRFD Total Moment, Mu =	0	k-ft					
ASD Soil Pressures:							
e =	0.000	ft			( ASD M	(P)	
Kern =	1.333	ft			(L/6)	,.,	
e > = < Kern ?	Less Than				( )		
Length of Pressure =	8.000	ft			( "Greate	er Than", Leng	gth = 3*(L/2 -e) ; Otherwise = L )
Minimum Pressure, Qmin =	1.700	ksf			•	,	P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.700	ksf			•		(P/L*B) + (6*M / B*L^2),
Is Qmax <sbc?< td=""><td>YES</td><td></td><td></td><td></td><td></td><td>o", Qmax = (2 Than", Omax</td><td>/^P) / (L^B) : = (4*P) / (3*B*(L - 2*e) )</td></sbc?<>	YES					o", Qmax = (2 Than", Omax	/^P) / (L^B) : = (4*P) / (3*B*(L - 2*e) )
					Greater		(- (+ 1)) (3 D (L - 2 e))
LRFD Soil Pressures:							
e =	0.000	ft			( ASD M	/ Pu )	
Kern =	1.333	ft			(L/6)		
e > = < Kern ?	Less Than	<i>с</i> і			(		
Length of Pressure =	8.000	ft kof					$gth = 3^{*}(L/2 - e)$ ; Otherwise = L)
Minimum Pressure, Qmin = Maximum Pressure, Qmax =	2.359 2.359	ksf ksf					Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0) (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	2.359	ksi			•	o", Qmax = (2	
Critical Length =	3.289	ft			•		x = (4*Pu) / (3*B*(L - 2*e) )
ontour Longth	0.200						(critical section of footing)
					•		Critical Section/2-d/2)



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Calc. By_	RJS	Checked By	JH	Date 03/01/21

One-Way Shear Check:				
Vu1 =	62.06	k		(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	63.59	k		( ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	YES			
Two-Way Shear Check:				
b1 =	17.06	in		(Critical Section B + d )
b2 =	17.06	in		(Column Height L + d )
b0 =	68.25	in		(2*b1 + 2*b2)
Vu2 =	87.71	k		(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2) )
α =	40	N.		(ACI 318-08 Section 11.11.2.1)
β =	1			(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim )
φ- ΦVn =	135.63	k		(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(fc)*bo*d*(2+4/Beta))
ΦVn =	152.02	k		$(ACI 318-08 \text{ Eq } 11-32, \Phi Vn = 0.75 \text{ sqrt}(10) \text{ bo } d(2^+4/\text{Deta}))$ (ACI 318-08 Eq 11-32, $\Phi Vn = 0.75^{\circ} \text{sqrt}(fc)^{\circ} \text{bo}^{\circ} d^{\circ}(2^+\text{Alpha*d/bo}))$
ΦVn =	90.42	k k		
Φνη – Adequate in Two-Way Shear?	90.42 YES	К		(ACI 318-08 Eq 11-33,
	120			
Column Bearing Check:				
ΦPn =	477.36	k		( ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2) )
Adequate in Bearing?	YES			
Uplift Check:				
ASD Combo for Uplift = (	0.6D + I Inlif	t		(ASCE 7)
Uplift Force =	0.00 · Opin 0	k		(From Above )
Required Dead Load =	0.00	k		( Uplift / 0.6 )
Applied Dead Load + Slab + Ftg =	57.8			( <b>Opin</b> ( 7 0.0 )
	4	k ft		(Longth of Additional Slab in Each Direction )
Additional Slab Used =				( Length of Additional Slab in Each Direction )
Wall Weight Over Footing =	0	klf		( Dan L Dan and in n and the Orace)
Length Parallel to Slab Edge =	0	ft		(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft		( B or L Depending on the Case)
Area of Cont. Footing =	0	ft <sup>2</sup>		
Length of Cont. Footing Used =	0	ft		
Total Dead Load =	67.4	k		(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg )
Adequate for Uplift?	Footing is <i>I</i>	Adequate to F	Resist Uplift	(Calculation assumes wall is above the cont. ftg.)
Top Steel:				
Mu =	0.00	k-ft / ft		( Mu = (Pu/A)*0.5*Crit. L^2 )
m =	23.529			(m = fy/(0.85*f'c))
Ru =	0.000	ksi		$(Ru = Mu/(0.9*12 inches*d^2))$
ρ Req'd =	0.0000			$(\rho = (1/m)^{*}(1-sqrt(1-2^{*}Ru^{*}m/fy)))$
	0.0000			(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy )
ρ Min. = 4/3*Mu ρ Req'd =	0.0027			
				$(\rho = (1/m)^{*}(1-\operatorname{sqrt}(1-2^{*}1.33^{*}\operatorname{Ru}^{*}m/fy)))$
Governing ρ =	0.0000	. 2		( lf ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in²/ft		( As = Governing ρ*12 inches*d )
Bar # =	4			
Bar Spacing =	12	in		
As Provided =	0.20	in²/ft	=	9 Bars in B Direction
			=	9 Bars in L Direction
Bottom Steel:	10 70			
Mu =	12.76	k-ft / ft		(Mu= Qcrit*0.5*Lcrit*2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit*2)
m =	23.529			(m = fy/(0.85*f'c))
Ru =	0.218	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0038			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027			(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy )
4/3*Mu ρ Req'd =	0.0051			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0038			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.368	in²/ft		(As = Governing $\rho^{*12}$ inches*d)
Bar # =	5			
Bar Spacing =	12	in		
		in <sup>2</sup> /ft	_	0 Pore in P Direction
As Provided =	0.31	111 / IL	=	•
			=	9 Bars in L Direction



Project	Balderston Auto		Project No	20-467
Calc. Bv	RJS	Checked By J	H Date	03/01/21

Temperature & Shrinkage	Steel:
Minimum Steel -	0 2502

Temperature & Shrinkage Si	eer:		
Minimum Steel =	0.2592	in²/ft	( T&S Steel = 0.0018*12 inches*H )
As Provided Top =	0.20	in²/ft	
As Provided Bott =	0.31	in²/ft	
As Provided Total =	0.51	in²/ft	
T&S Steel Provided?	YES		
Final Footing Design:			
Footing Width, B =	8	ft	
Footing Length L =	8	ft	
Footing Depth, H =	12	in	
Top Steel =	#4 bar	rs @12 inches O.C.	

Bottom Steel = #5 bars @12 inches O.C.



Project\_ Balderston Auto

\_\_\_\_\_ Project No.\_\_ 20-467

P/Pu

Calc. By RJS Checked By JH Date 03/01/21

## Footing Designation F3

Footing Location:	Interior at Me	zzanine					B is into M/Mu the page	
General Information:							¥	
Footing Length, L =	4	ft						
Footing Width, B =	4	ft					Qmin	
Footing Depth, H =	34	in					Qm	av
Location =	Interior	in					L	
Steel Depth, d =	30.0625	in			(H - 3 in -	1.5*Bar Dia. )	Length of Soil Pressure	
Typical Slab Depth =	5	in				,	Lengtror Son Pleasure	
Slab Depth Above Footing =	8	in						
Area of Footing =	16	ft^2			(B*L)			
Soil Bearing Pressure =	2.5	ksf						
Allowable or Effective SBC?	Allowable							
Concrete Strength =	3	ksi						
	B Direction			L Direction				
Column Size =	6.00 in		Х	6.00 in				
Base Plate Size =	12.00 in		Х	12.00 in				
Critical Section =	9.00 in		Х	9.00 in				
Loading:								
Vertical Loads:				LRFD Factors:				
Applied Dead Load =	9.7	k		Dead =		1.2	(ASCE 7 Combo)	
Slab + Wall +Footing Weight =	7.8	k		Live =		1.6		
Applied Live Load =	10.3	k		Uplift=		1.6		
ASD Total Load, P =	27.8	k		opint				
LRFD Total Load, Pu =	37.48	k						
ASD Uplift Load =	0	k						
LRFD Uplift Load =	0	k						
Moments:				LRFD Factors:				
Dead Load Moment =	0	k-ft		Dead =		1.2	(ASCE 7 Combo)	
Dead Load Moment = Live Load Moment =	0	k-ft				1.2 1.6	(ASCE 7 Combo)	
Dead Load Moment = Live Load Moment = ASD Total Moment, M =	<mark>0</mark> 0	k-ft k-ft		Dead =			(ASCE 7 Combo)	
Dead Load Moment = Live Load Moment =	0	k-ft		Dead =			(ASCE 7 Combo)	
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu =	<mark>0</mark> 0	k-ft k-ft		Dead =			(ASCE 7 Combo )	
Dead Load Moment = Live Load Moment = ASD Total Moment, M =	<mark>0</mark> 0	k-ft k-ft		Dead =		1.6	(ASCE 7 Combo )	
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu =	0 0 0	k-ft k-ft k-ft		Dead =		1.6	(ASCE 7 Combo )	
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu = ASD Soil Pressures: e =	0 0 0	k-ft k-ft k-ft ft		Dead =	(ASD M	1.6	(ASCE 7 Combo )	
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu = ASD Soil Pressures: e = Kern =	0 0 0.000 0.667	k-ft k-ft k-ft ft		Dead =	(ASD M) (L/6)	1.6 /P)	( ASCE 7 Combo ) th = 3*(L/2 -e) ; Otherwise = L )	
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Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu = <b>ASD Soil Pressures:</b> e = Kern = e > = < Kern ? Length of Pressure =	0 0 0 0.000 0.667 Less Than 4.000	k-ft k-ft k-ft ft ft		Dead =	( ASD M / ( L / 6 ) ( "Greater ("Less Th	1.6 / P ) r Than", Lengt pan",Qmin = (F	h = 3*(L/2 -e) ; Otherwise = L )	
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu = <b>ASD Soil Pressures:</b> e = Kern = e > = < Kern ? Length of Pressure = Minimum Pressure, Qmin =	0 0 0 0.000 0.667 Less Than 4.000 1.738	k-ft k-ft ft ft ft ksf		Dead =	( ASD M / ( L / 6 ) ( "Greatel ("Less Th ("Less Th "Equal Tc	1.6 / P ) r Than", Lengt aan",Qmin = (F aan",Qmax = (I v", Qmax = (2*	h = 3*(L/2 -e) ; Otherwise = L ) ?/L*B) - (6*M / B*L^2), Otherwise = 0 ) P/L*B) + (6*M / B*L^2), P) / (L*B)	
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Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu = <b>ASD Soil Pressures:</b> e = Kern = e > = < Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmax = Is Qmax <sbc? <b>LRFD Soil Pressures:</b> e = Kern = e &gt; = &lt; Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmin = Maximum Pressure, Qmax = Qcritical =</sbc? 	0 0 0 0.000 0.667 Less Than 4.000 1.738 1.738 <b>YES</b> 0.000 0.667 Less Than 4.000 2.343 2.343 2.343	k-ft k-ft ft ft ksf ksf ft ft ksf ksf ksf ksf		Dead =	( ASD M / ( L / 6 ) ( "Greate ("Less Th ("Less Th "Equal Tc "Greater" ( ASD M / ( L / 6 ) ( "Greate ("Less Th ("Less Th "Equal Tc	1.6 / P ) r Than", Lengt nan",Qmin = (F nan",Qmax = (A ', Qmax = (2* Than", Qmax = (2* / Pu ) r Than", Lengt nan",Qmin = (F nan",Qmax = (4*)	ch = 3*(L/2 -e) ; Otherwise = L ) P/L*B) - (6*M / B*L^2), Otherwise = 0 ) P/L*B) + (6*M / B*L^2), P) / (L*B) = (4*P) / (3*B*(L - 2*e) ) ch = 3*(L/2 -e) ; Otherwise = L ) Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0 ) Pu/L*B) + (6*Mu / B*L^2), Pu) / (L*B)	
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu = <b>ASD Soil Pressures:</b> e = Kern = e > = < Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmax = Is Qmax <sbc? <b>LRFD Soil Pressures:</b> e = Kern = e &gt; = &lt; Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmax =</sbc? 	0 0 0 0.000 0.667 Less Than 4.000 1.738 1.738 YES 0.000 0.667 Less Than 4.000 2.343 2.343	k-ft k-ft ft ft ft ksf ksf ft ft ksf ksf ksf		Dead =	( ASD M / ( L / 6 ) ( "Greate ("Less Th ("Less Th "Equal Tc "Greater" ( ASD M / ( L / 6 ) ( "Greater ("Less Th ("Less Th "Equal Tc "Greater"	1.6 / P ) r Than", Lengt ian",Qmin = (F ian",Qmax = (I o", Qmax = (2* Than", Qmax = (2* / Pu ) r Than", Qmax = (F ian",Qmin = (F ian",Qmax = (2* Than", Qmax = (2*	$\begin{aligned} h &= 3^*(L/2 - e) ; \ Otherwise = L \ ) \\ P/L^*B) &- (6^*M / B^*L^2), \ Otherwise = 0 \ ) \\ P/L^*B) &+ (6^*M / B^*L^2), \\ P) / (L^*B) \\ &= (4^*P) / (3^*B^*(L - 2^*e) \ ) \end{aligned}$ $\begin{aligned} h &= 3^*(L/2 - e) ; \ Otherwise = L \ ) \\ Pu/L^*B) &- (6^*Mu / B^*L^2), \ Otherwise = 0 \ ) \\ Pu/L^*B) &+ (6^*Mu / B^*L^2), \\ Pu/L^*B) &= (4^*Pu) / (L^*B) \\ &= (4^*Pu) / (3^*B^*(L - 2^*e) \ ) \end{aligned}$	
Dead Load Moment = Live Load Moment = ASD Total Moment, M = LRFD Total Moment, Mu = <b>ASD Soil Pressures:</b> e = Kern = e > = < Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmax = Is Qmax <sbc? <b>LRFD Soil Pressures:</b> e = Kern = e &gt; = &lt; Kern ? Length of Pressure = Minimum Pressure, Qmin = Maximum Pressure, Qmin = Maximum Pressure, Qmax = Qcritical =</sbc? 	0 0 0 0.000 0.667 Less Than 4.000 1.738 1.738 <b>YES</b> 0.000 0.667 Less Than 4.000 2.343 2.343 2.343	k-ft k-ft ft ft ksf ksf ft ft ksf ksf ksf ksf		Dead =	( ASD M / ( L / 6 ) ( "Greate ("Less Th ("Less Th "Equal To "Greater" ( ASD M / ( L / 6 ) ( "Greate ("Less Th ("Less Th "Equal To "Greater" (Qcritical	1.6 / P ) r Than", Lengt ian",Qmin = (F ian",Qmax = (1 )", Qmax = (2* Than", Qmax = (2* / Pu ) r Than", Qmax = (7 ian",Qmin = (F ian",Qmax = (1 )", Qmax = (2* Than", Qmax = (2*	ch = 3*(L/2 -e) ; Otherwise = L ) P/L*B) - (6*M / B*L^2), Otherwise = 0 ) P/L*B) + (6*M / B*L^2), P) / (L*B) = (4*P) / (3*B*(L - 2*e) ) ch = 3*(L/2 -e) ; Otherwise = L ) Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0 ) Pu/L*B) + (6*Mu / B*L^2), Pu) / (L*B)	



Project	Balderston Auto			Project No	20-467	
Calc. By	RJS	Checked By	JH	Date	03/01/21	

One-Way Shear Check:			
Vu1 =	3.49	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	118.55	k	( ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	YES		
Two-Way Shear Check:			
b1 =	39.06	in	(Critical Section B + d)
b2 =	39.06	in	(Column Height L + d)
b0 =	156.25	in	(2*b1 + 2*b2 )
Vu2 =	12.66	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim )
ΦVn =	1157.76	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1870.94	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	771.84	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d )
Adequate in Two-Way Shear?	YES		
Column Bearing Check:			
ΦPn =	477.36	k	( ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2) )
Adequate in Bearing?	YES		
Uplift Check:			
ASD Combo for Uplift =	0.6D + Uplif	t	(ASCE 7)
Uplift Force =	0	k	( From Above )
Required Dead Load =	0.00	k	( Uplift / 0.6 )
Applied Dead Load + Slab + Ftg =	17.5	k	
Additional Slab Used =	4	ft	( Length of Additional Slab in Each Direction )
Wall Weight Over Footing =	0	klf	
Length Parallel to Slab Edge =	0	ft	( B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft <sup>2</sup>	
	0	ft	
Length of Cont Footing Used =			
Length of Cont. Footing Used = Total Dead Load =			(Applied Dead + Slab + Wall Weight + Add_Slab + Cont_Etg.)
Total Dead Load =	25.5	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg ) (Calculation assumes wall is above the cont. ftg.)
Total Dead Load =	25.5		(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg ) (Calculation assumes wall is above the cont. ftg.)
Total Dead Load = Adequate for Uplift?	25.5	k	
Total Dead Load =	25.5	k	(Calculation assumes wall is above the cont. ftg.)
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b>	25.5 Footing is <i>I</i>	k Adequate to Resist Uplift	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 )
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b> Mu =	25.5 Footing is <i>J</i> 0.00	k Adequate to Resist Uplift	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 ) ( m = fy/(0.85*fc) )
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru =	25.5 Footing is A 0.00 23.529 0.000	k Adequate to Resist Uplift k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 ) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2))
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b> Mu = m = Ru = ρ Req'd =	25.5 Footing is A 0.00 23.529 0.000 0.0000	k Adequate to Resist Uplift k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 ) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) (ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b> Mu = m = Ru = ρ Req'd = ρ Min. =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0027	k Adequate to Resist Uplift k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 ) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) (ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy))) (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy )
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b> Mu = m = Ru = ρ Req'd = ρ Min. = 4/3*Mu ρ Req'd =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0027 0.0000	k Adequate to Resist Uplift k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc) )$ ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*Ru^*m/fy))$ ) (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*1.33^*Ru^*m/fy))$ )
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b> Mu = m = Ru = ρ Req'd = ρ Min. = 4/3*Mu ρ Req'd = Governing ρ =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0027 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc) )$ ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*Ru^*m/fy))$ ) (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru = $\rho$ Req'd = $\rho$ Min. = 4/3*Mu $\rho$ Req'd = Governing $\rho$ = A's Required =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc) )$ ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*Ru^*m/fy))$ ) (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*1.33^*Ru^*m/fy))$ )
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b> Mu = m = Ru = ρ Req'd = ρ Min. = 4/3*Mu ρ Req'd = Governing ρ = A's Required = Bar # =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5	k Adequate to Resist Uplift k-ft / ft ksi in²/ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc) )$ ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*Ru^*m/fy))$ ) (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru = $\rho$ Req'd = $\rho$ Min. = 4/3*Mu $\rho$ Req'd = Governing $\rho$ = A's Required = Bar # = Bar Spacing =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10	k Adequate to Resist Uplift k-ft / ft ksi in <sup>2</sup> /ft in	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*Ru^*m/fy))$ ) (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd ( As = Governing $\rho$ *12 inches*d )
Total Dead Load = Adequate for Uplift? <b>Top Steel:</b> Mu = m = Ru = ρ Req'd = ρ Min. = 4/3*Mu ρ Req'd = Governing ρ = A's Required = Bar # =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08$ Equation 10-3, Smaller of: $3^*\text{sqrt}(fc)/fy \& 200/fy$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( $If \rho \text{ Req'd} < 4/3^*Mu \rho \text{ Req'd} < \rho \text{ Min, Use } 4/3^*Mu \rho \text{ Req'd}$ ( $As = \text{Governing } \rho^*12 \text{ inches}^*d$ ) = 6 Bars in B Direction
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru = $\rho \text{ Req'd} =$ $\rho \text{ Min.} =$ $4/3*Mu \rho \text{ Req'd} =$ $\text{Governing } \rho =$ A's  Required = Bar # = Bar  Spacing = As  Provided =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*Ru^*m/fy))$ ) (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy) ( $\rho = (1/m)^*(1\text{-sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd ( As = Governing $\rho$ *12 inches*d )
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru = $\rho \text{ Req'd} =$ $\rho \text{ Min.} =$ $4/3*Mu \rho \text{ Req'd} =$ $Governing \rho =$ A's  Required = Bar # = Bar  Spacing = As  Provided =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-sqrt(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08$ Equation 10-3, Smaller of: $3^*sqrt(fc)/fy \& 200/fy$ ) ( $\rho = (1/m)^*(1-sqrt(1-2^*1.33^*Ru^*m/fy))$ ) ( $If \rho \text{ Req'd} < 4/3^*Mu \rho \text{ Req'd} < \rho \text{ Min, Use } 4/3^*Mu \rho \text{ Req'd}$ ( $As = \text{Governing } \rho^*12 \text{ inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru = $\rho$ Req'd = $\rho$ Min. = 4/3*Mu $\rho$ Req'd = Governing $\rho$ = A's Required = Bar # = Bar Spacing = As Provided = Mu =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37 0.16	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*f^c) )$ ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\text{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08$ Equation 10-3, Smaller of: $3^*\text{sqrt}(f^c)/fy \& 200/fy )$ ( $\rho = (1/m)^*(1-\text{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho$ Req'd < $4/3^*Mu \rho$ Req'd < $\rho$ Min, Use $4/3^*Mu \rho$ Req'd ( $As = \text{Governing } \rho^*12 \text{ inches}^*d )$ = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu= \text{Qcrit}^*0.5^*\text{Lcrit}^2 + (\text{Qmax-Qcrit})^*0.5^*(2/3)^*\text{Lcrit}^2)$
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru = $\rho \operatorname{Req'd} =$ $\rho \operatorname{Min} =$ $4/3*Mu \rho \operatorname{Req'd} =$ $\operatorname{Governing} \rho =$ $A's \operatorname{Required} =$ $\operatorname{Bar} \# =$ $\operatorname{Bar} \operatorname{Spacing} =$ $\operatorname{As} \operatorname{Provided} =$ $\operatorname{Bottom Steel:}$ Mu = m =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*f^c)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08$ Equation 10-3, Smaller of: $3^*\operatorname{sqrt}(f^*c)/fy \& 200/fy$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho$ Req'd < $4/3^*Mu \rho$ Req'd < $\rho$ Min, Use $4/3^*Mu \rho$ Req'd ( $As = \text{Governing } \rho^*12 \text{ inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu= \operatorname{Qcrit}^*0.5^*L\operatorname{Crit}^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*L\operatorname{Crit}^2)$ ( $m = fy/(0.85^*f^c)$ )
Total Dead Load = Adequate for Uplift? Top Steel: Mu = m = Ru = $\rho \operatorname{Req'd} =$ $\rho \operatorname{Min} =$ $4/3*Mu \rho \operatorname{Req'd} =$ $\operatorname{Governing} \rho =$ $A's \operatorname{Required} =$ $\operatorname{Bar} \# =$ $\operatorname{Bar} \operatorname{Spacing} =$ $\operatorname{As} \operatorname{Provided} =$ Bottom Steel: Mu = m = $\operatorname{Ru} =$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*f^c)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^2)$ )) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ )) ( $ACI 318-08 \text{ Equation } 10-3$ , Smaller of: $3^*\operatorname{sqrt}(f^c)/fy \& 200/fy$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ )) ( If $\rho \operatorname{Req'd} < 4/3^*Mu \ \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \ \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing} \rho^*12 \text{ inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu= \operatorname{Qcrit}^*0.5^*Lcrit^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*Lcrit^2)$ ( $m = fy/(0.85^*f^c)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^2)$ ))
$\begin{tabular}{lllllllllllllllllllllllllllllllllll$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \text{ Equation } 10-3, \text{ Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy $ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho \operatorname{Req'd} < 4/3^*Mu \ \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \ \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing } \rho^*12 \text{ inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu= \operatorname{Qcrit}^*0.5^*Lcrit^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*Lcrit^2)$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ )
$\begin{tabular}{lllllllllllllllllllllllllllllllllll$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \text{ Equation } 10-3, \text{ Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy )$ ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho \operatorname{Req'd} < 4/3^*Mu \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing } \rho^*12 \operatorname{inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu=\operatorname{Qcrit}^*0.5^*Lcrit^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*Lcrit^2)$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \operatorname{inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \operatorname{Equation } 10-3, \operatorname{Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy$ )
Total Dead Load = Adequate for Uplift? Top Steel: $Mu = m =$ $Ru =$ $\rho \operatorname{Req'd} =$ $\rho \operatorname{Min} =$ $4/3^*\operatorname{Mu} \rho \operatorname{Req'd} =$ $\operatorname{Governing} \rho =$ $A's \operatorname{Required} =$ $\operatorname{Bar} \# =$ $\operatorname{Bar} \operatorname{Spacing} =$ $\operatorname{As} \operatorname{Provided} =$ $Bottom \operatorname{Steel:}$ $Mu =$ $m =$ $Ru =$ $\rho \operatorname{Req'd} =$ $\rho \operatorname{Min} =$ $4/3^*\operatorname{Mu} \rho \operatorname{Req'd} =$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \text{ Equation } 10-3, \text{ Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy $ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( $If \rho \operatorname{Req'd} < 4/3^*Mu \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing} \rho^*12 \operatorname{inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu= \operatorname{Qcrit}^*0.5^*Lcrit^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*Lcrit^2)$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \operatorname{inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \operatorname{Equation } 10-3, \operatorname{Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ )
$\begin{tabular}{lllllllllllllllllllllllllllllllllll$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi in <sup>2</sup> /ft in in <sup>2</sup> /ft k-ft / ft ksi	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \text{ Equation } 10-3, \text{ Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy )$ ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( If $\rho \operatorname{Req'd} < 4/3^*Mu \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing } \rho^*12 \operatorname{inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu=\operatorname{Qcrit}^*0.5^*Lcrit^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*Lcrit^2)$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \operatorname{inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \operatorname{Equation } 10-3, \operatorname{Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy$ )
Total Dead Load = Adequate for Uplift? Top Steel: $Mu = m =$ $Ru =$ $\rho \operatorname{Req'd} =$ $\rho \operatorname{Min} =$ $4/3^*\operatorname{Mu} \rho \operatorname{Req'd} =$ $\operatorname{Governing} \rho =$ $A's \operatorname{Required} =$ $\operatorname{Bar} \# =$ $\operatorname{Bar} \operatorname{Spacing} =$ $\operatorname{As} \operatorname{Provided} =$ $Bottom \operatorname{Steel:}$ $Mu =$ $m =$ $Ru =$ $\rho \operatorname{Req'd} =$ $\rho \operatorname{Min} =$ $4/3^*\operatorname{Mu} \rho \operatorname{Req'd} =$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft k-ft / ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \text{ Equation } 10-3, \text{ Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy $ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( $If \rho \operatorname{Req'd} < 4/3^*Mu \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing} \rho^*12 \operatorname{inches}^*d$ ) = 6 Bars in B Direction = 6 Bars in L Direction ( $Mu= \operatorname{Qcrit}^*0.5^*Lcrit^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*Lcrit^2)$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \operatorname{inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \operatorname{Equation } 10-3, \operatorname{Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ )
Total Dead Load = Adequate for Uplift? Top Steel: $Mu = m = m = Ru = p Req'd = p Min. = 4/3*Mu \rho Req'd = Governing \rho = A's Required = Bar # = Bar Spacing = As Provided = Bar # = Bar Spacing = As Provided = m = Ru = p Req'd = m = Ru = \rho Req'd = p Min. = 4/3*Mu \rho Req'd = Governing \rho = dot red to the second $	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi in <sup>2</sup> /ft in in <sup>2</sup> /ft k-ft / ft ksi	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 ) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*\text{Ru*m/fy})))$ (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*1.33*\text{Ru*m/fy})))$ ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd ( As = Governing $\rho$ *12 inches*d ) = 6 Bars in B Direction = 6 Bars in L Direction (Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*\text{Ru*m/fy})))$ (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*1.33*\text{Ru*m/fy})))$ ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd
Total Dead Load = Adequate for Uplift? Top Steel: $Mu = m = m = Ru = p Req'd = p Min. = 4/3*Mu \rho Req'd = Governing \rho = A's Required = Bar # = Bar Spacing = As Provided = Bar # = Bar Spacing = As Provided = m = Ru = p Req'd = m = Ru = \rho Req'd = p Min. = 4/3*Mu \rho Req'd = Governing \rho = A's Required = 0 Kar Required = K$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0002 0.0000 0.0000 0.0000 0.0000	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft k-ft / ft ksi in²/ft in²/ft	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 ) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*\text{Ru*m/fy})))$ (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*1.33*\text{Ru*m/fy})))$ ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd ( As = Governing $\rho$ *12 inches*d ) = 6 Bars in B Direction = 6 Bars in L Direction (Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*\text{Ru*m/fy})))$ (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*1.33*\text{Ru*m/fy})))$ ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd
Total Dead Load = Adequate for Uplift? Top Steel: $Mu = m =$ $Ru =$ $\rho Req'd =$ $\rho Min. =$ $4/3*Mu \rho Req'd =$ Governing $\rho =$ A's Required = Bar # = Bar Spacing = As Provided = Bottom Steel: $Mu =$ $m =$ $Ru =$ $\rho Req'd =$ $\rho Min. =$ $4/3*Mu \rho Req'd =$ Governing $\rho =$ A's Required = Bar # =	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0002 0.0000 0.0002 5	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in in²/ft k-ft / ft ksi in²/ft in	(Calculation assumes wall is above the cont. ftg.) ( Mu = (Pu/A)*0.5*Crit. L^2 ) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*\text{Ru*m/fy})))$ (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*1.33*\text{Ru*m/fy})))$ ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd ( As = Governing $\rho$ *12 inches*d ) = 6 Bars in B Direction = 6 Bars in L Direction (Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2) ( m = fy/(0.85*fc) ) (Ru = Mu/(0.9*12 inches*d^2)) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*\text{Ru*m/fy})))$ (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(fc)/fy & 200/fy ) ( $\rho = (1/m)*(1-\text{sqrt}(1-2*1.33*\text{Ru*m/fy})))$ ( If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd
Total Dead Load = Adequate for Uplift? Top Steel: $Mu = m = m = Ru = p Req'd = p Min. = 4/3*Mu \rho Req'd = Governing \rho = A's Required = Bar # = Bar Spacing = As Provided = Bottom Steel: Mu = m = m = Ru = p Req'd = p Min. = 4/3*Mu \rho Req'd = p Min. = 4/3*Mu \rho Req'd = Governing \rho = A's Required = Bar # = Bar Spacing = Marcelet = Bar # = Bar Spacing = Marcelet = Bar # = Bar Spacing = Marcelet = Marcelet$	25.5 Footing is A 0.00 23.529 0.000 0.0000 0.0000 0.0000 0.0000 5 10 0.37 0.16 23.529 0.000 0.0000 0.0000 0.0000 0.0000 0.0002 5 10	k Adequate to Resist Uplift k-ft / ft ksi in²/ft in²/ft k-ft / ft ksi in²/ft in²/ft	(Calculation assumes wall is above the cont. ftg.) ( $Mu = (Pu/A)^*0.5^*Crit. L^2 )$ ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \text{ inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \text{ Equation } 10-3, \text{ Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy $ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( $If \rho \operatorname{Req'd} < 4/3^*Mu \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing} \rho^*12 \operatorname{inches}^*d$ ) = 6 Bars in B Direction ( $Mu= \operatorname{Qcrit}^*0.5^*Lcrit^2 + (\operatorname{Qmax-Qcrit})^*0.5^*(2/3)^*Lcrit^2$ ) ( $m = fy/(0.85^*fc)$ ) ( $Ru = Mu/(0.9^*12 \operatorname{inches}^*d^22)$ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*Ru^*m/fy))$ ) ( $ACI 318-08 \operatorname{Equation } 10-3, \operatorname{Smaller of: } 3^*\operatorname{sqrt}(fc)/fy \& 200/fy $ ) ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*1.33^*Ru^*m/fy))$ ) ( $If \rho \operatorname{Req'd} < 4/3^*Mu \rho \operatorname{Req'd} < \rho \operatorname{Min}$ , Use $4/3^*Mu \rho \operatorname{Req'd}$ ( $As = \operatorname{Governing} \rho^*12 \operatorname{inches}^*d$ )



Project_	Balderston Auto			20-467 Project No
Calc. By_	RJS	Checked By	JH	Date 03/01/21

Temperature & Shrinkage S	teel:		
Minimum Steel =	0.7344	in²/ft	( T&S Steel = 0.0018*12 inches*H )
As Provided Top =	0.37	in²/ft	
As Provided Bott =	0.37	in²/ft	
As Provided Total =	0.74	in²/ft	
T&S Steel Provided?	YES		
Final Footing Design:			
Footing Width, B =	4	ft	
Footing Length L =	4	ft	
Footing Depth, H =	34	in	

Top Steel =	#5 bars	@10 inches O.C.
Bottom Steel =	#5 bars	@10 inches O.C.



	Project	Balderston Auto		F	20-467 Project No
<b>BE</b> STRUCTURAL ENGINEERS	Calc. By_	RJS	Checked By	JH	Date 03/01/21

### Footing Designation F4

Footing Location:         B is into the page         M/Mu         General Information:         Footing Length, L =       4       ft         Footing Width, B =       4       ft         Footing Depth, H =       12       in         Location =       Interior       in         Location =       Interior       in         Steel Depth, d =       8.0625       in         Typical Slab Depth =       5       in         Slab Depth Above Footing =       8       in         Area of Footing Pressure =       2.5       ksf         Allowable or Effective SBC?       Allowable       Concrete Strength =       3         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       9.00 in         Critical Section =       9.00 in       X       9.00 in         Critical Section =       9.01 in       X       12.00 in         Vertical Loads:       LRFD Factors:       Applied Dead Load =       9.7         Applied Dead Load =       9.7       K       Dead =       1.2
Footing Length, L = 4 ft Footing Width, B = 4 ft Footing Depth, H = 12 in Location = Interior in Steel Depth, d = 8.0625 in (H - 3 in - 1.5*Bar Dia.) Typical Slab Depth = 5 in Slab Depth Above Footing = 8 in Area of Footing = 16 ft^2 (B*L) Soil Bearing Pressure = 2.5 ksf Allowable or Effective SBC? Allowable Concrete Strength = 3 ksi B Direction L Direction Column Size = 6.00 in X 6.00 in Base Plate Size = 12.00 in X 12.00 in Critical Section = 9.00 in X 9.00 in LEADING: Vertical Loads: LRFD Factors:
Footing Width, B = 4 ft Footing Depth, H = 12 in Location = Interior in Steel Depth, d = 8.0625 in (H - 3 in - 1.5*Bar Dia.) Typical Slab Depth = 5 in Slab Depth Above Footing = 8 in Area of Footing = 16 ft*2 (B*L) Soil Bearing Pressure = 2.5 ksf Allowable or Effective SBC? Allowable Concrete Strength = 3 ksi B Direction L Direction Column Size = 6.00 in X 6.00 in Base Plate Size = 12.00 in X 12.00 in Critical Section = 9.00 in X 9.00 in Loading: Vertical Loads: LRFD Factors:
Footing Width, B = 4 ft Footing Depth, H = 12 in Location = Interior in Steel Depth, d = 8.0625 in (H - 3 in - 1.5*Bar Dia.) Typical Slab Depth = 5 in Slab Depth Above Footing = 8 in Area of Footing = 16 ft*2 (B*L) Soil Bearing Pressure = 2.5 ksf Allowable or Effective SBC? Allowable Concrete Strength = 3 ksi B Direction L Direction Column Size = 6.00 in X 6.00 in Base Plate Size = 12.00 in X 12.00 in Critical Section = 9.00 in X 9.00 in Loading: Vertical Loads: LRFD Factors:
Footing Depth, H =       12       in         Location =       Interior       in         Steel Depth, d =       8.0625       in       (H - 3 in - 1.5*Bar Dia.)         Typical Slab Depth =       5       in         Slab Depth Above Footing =       8       in         Area of Footing =       16       ft^2       (B*L)         Soil Bearing Pressure =       2.5       ksf         Allowable or Effective SBC?       Allowable         Concrete Strength =       3       ksi         B Direction       L Direction         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       9.00 in         Critical Section =       9.00 in       X       9.00 in
Location = Interior in Steel Depth, d = 8.0625 in (H - 3 in - 1.5*Bar Dia.) Typical Slab Depth = 5 in Slab Depth Above Footing = 8 in Area of Footing = 16 ft^2 (B*L) Soil Bearing Pressure = 2.5 ksf Allowable or Effective SBC? Allowable Concrete Strength = 3 ksi B Direction L Direction Column Size = 6.00 in X 6.00 in Base Plate Size = 12.00 in X 12.00 in Critical Section = 9.00 in X 9.00 in LRFD Factors:
Steel Depth, d = 8.0625 in (H - 3 in - 1.5*Bar Dia.)   Typical Slab Depth = 5 in   Slab Depth Above Footing = 8 in   Area of Footing = 16 ft^2   Soil Bearing Pressure = 2.5 ksf   Allowable or Effective SBC? Allowable   Concrete Strength = 3 ksi   B Direction L Direction   Column Size = 6.00 in   X 6.00 in   Base Plate Size = 12.00 in   Yertical Loads: LRFD Factors:
Typical Slab Depth =       5       in         Slab Depth Above Footing =       8       in         Area of Footing =       16       ft*2       (B*L)         Soil Bearing Pressure =       2.5       ksf         Allowable or Effective SBC?       Allowable         Concrete Strength =       3       ksi         B Direction       L Direction         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       12.00 in         Critical Section =       9.00 in       X       9.00 in
Slab Depth Above Footing =       8       in         Area of Footing =       16       ft*2       (B*L)         Soil Bearing Pressure =       2.5       ksf         Allowable or Effective SBC?       Allowable         Concrete Strength =       3       ksi         B Direction       L Direction         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       12.00 in         Critical Section =       9.00 in       X       9.00 in
Area of Footing =       16       ft <sup>A</sup> 2       (B*L)         Soil Bearing Pressure =       2.5       ksf         Allowable or Effective SBC?       Allowable         Concrete Strength =       3       ksi         B Direction       L Direction         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       12.00 in         Critical Section =       9.00 in       X       9.00 in
Soil Bearing Pressure =       2.5       ksf         Allowable or Effective SBC?       Allowable         Concrete Strength =       3       ksi         B Direction       L Direction         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       12.00 in         Critical Section =       9.00 in       X       9.00 in
Allowable or Effective SBC?       Allowable         Concrete Strength =       3       ksi         B Direction       L Direction         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       12.00 in         Critical Section =       9.00 in       X       9.00 in
Concrete Strength =       3       ksi         B Direction       L Direction         Column Size =       6.00 in       X       6.00 in         Base Plate Size =       12.00 in       X       12.00 in         Critical Section =       9.00 in       X       9.00 in
B Direction         L Direction           Column Size =         6.00 in         X         6.00 in           Base Plate Size =         12.00 in         X         12.00 in           Critical Section =         9.00 in         X         9.00 in
Column Size =         6.00 in         X         6.00 in           Base Plate Size =         12.00 in         X         12.00 in           Critical Section =         9.00 in         X         9.00 in
Base Plate Size =       12.00 in       X       12.00 in         Critical Section =       9.00 in       X       9.00 in         Loading:       Vertical Loads:       LRFD Factors:
Critical Section = 9.00 in X 9.00 in Loading: Vertical Loads: LRFD Factors:
Loading: Vertical Loads: LRFD Factors:
Vertical Loads: LRFD Factors:
Applied Dead Load = $9.7$ k Dead = $1.2$ (ASCE 7 Combo)
Slab + Wall +Footing Weight = 3.4 k Live = 1.6
Applied Live Load = 10.3 k Uplift= 1.6
ASD Total Load, P = 23.4 k
LRFD Total Load, Pu = 32.2 k
ASD Uplift Load = 0 k
LRFD Uplift Load = 0 k
Moments: LRFD Factors:
Dead Load Moment = 0 k-ft Dead = 1.2 (ASCE 7 Combo)
Live Load Moment = 0 k-ft Wind = 1.6
ASU LOTAL MOMENT M = 10 K-TT
ASD Total Moment, M = 0 k-ft
LRFD Total Moment, $Mu = 0$ k-ft
LRFD Total Moment, Mu =         0         k-ft           ASD Soil Pressures:         e =         0.000         ft         (ASD M / P )
LRFD Total Moment, Mu = 0 k-ft ASD Soil Pressures:
LRFD Total Moment, Mu =         0         k-ft           ASD Soil Pressures:         e =         0.000         ft         (ASD M / P )
LRFD Total Moment, Mu = 0 k-ft ASD Soil Pressures:
LRFD Total Moment, Mu = 0 k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern ? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 -e); Otherwise = L)
LRFD Total Moment, Mu = 0 k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern ? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 -e); Otherwise = L)
LRFD Total Moment, Mu =0k-ftASD Soil Pressures:(ASD M / P) $e =$ 0.000ftKern =0.667ft $e > = < Kern$ ?Less ThanLength of Pressure =4.000ftMinimum Pressure, Qmin =1.463ksfMaximum Pressure, Qmax =1.463ksf("Less Than", Qmax = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
LRFD Total Moment, Mu =0k-ftASD Soil Pressures:(ASD M / P) $e =$ 0.000ftKern =0.667ft $e > = < Kern$ ?Less ThanLength of Pressure =4.000ftMinimum Pressure, Qmin =1.463ksfMaximum Pressure, Qmax =1.463ksf("Less Than", Qmax = (P/L*B) - (6*M / B*L^2),
LRFD Total Moment, Mu =0k-ftASD Soil Pressures: $e = 0.000$ ft $(ASD M / P)$ Kern =0.667ft $(L/6)$ $e > = < Kern ?Less Than(U/6)Length of Pressure =4.000ft("Greater Than", Length = 3*(L/2 - e); Otherwise = L)Minimum Pressure, Qmin =1.463ksf("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)Maximum Pressure, Qmax =1.463ksf("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Otherwise = 0)Is QmaxYES"Equal To", Qmax = (2*P) / (L*B)"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))$
LRFD Total Moment, Mu =       0       k-ft         ASD Soil Pressures: <ul> <li>e =</li> <li>0.000</li> <li>ft</li> <li>(L76)</li> <li>e &gt; = &lt; Kern ?</li> <li>Less Than</li> <li>Length of Pressure =</li> <li>4.000</li> <li>ft</li> <li>("Greater Than", Length = 3*(L/2 -e); Otherwise = L)</li> <li>("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)</li> <li>Maximum Pressure, Qmax =</li> <li>1.463</li> <li>ksf</li> <li>("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Otherwise = 0)</li> <li>("Less Than", Qmax = (2*P) / (L*B)</li> <li>"Greater Than", Qmax = (2*P) / (3*B*(L - 2*e))</li> </ul> LRFD Soil Pressures:
LRFD Total Moment, $Mu = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (2*P) / (L*B) Is Qmax < SBC? YES "Equal To", Qmax = (2*P) / (3*B*(L - 2*e)) LRFD Soil Pressures: e = 0.000 ft (ASD M / Pu)
LRFD Total Moment, $Mu = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Is Qmax <sbc? "equal="" (l*b)<br="" qmax="(2*P)" to",="" yes="">"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e)) LRFD Soil Pressures: e = 0.000 ft (ASD M / Pu) Kern = 0.667 ft (L / 6)</sbc?>
LRFD Total Moment, $Mu = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Is Qmax <sbc? "equal="" (l*b)<br="" qmax="(2*P)" to",="" yes="">"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e)) LRFD Soil Pressures: e = 0.000 ft (ASD M / Pu) Kern = 0.667 ft (L / 6)</sbc?>
LRFD Total Moment, $Mu = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Is Qmax < SBC? YES "Equal To", Qmax = (2*P) / (L*B) Is Qmax < SBC? YES "Equal To", Qmax = (2*P) / (L*B) "Greater Than", Qmax = (4*P) / (3*B*(L - 2*e)) LRFD Soil Pressures: e = 0.000 ft (ASD M / Pu) Kern = 0.667 ft (L / 6) e > = < Kern ? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L)
LRFD Total Moment, $Mu = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Is Qmax < SBC? YES "Equal To", Qmax = (2*P) / (L*B) "Greater Than", Qmax = (4*P) / (3*B*(L - 2*e)) LRFD Soil Pressures: e = 0.000 ft (ASD M / Pu) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) ("Less Than", Qmin = (Pu/L*B) - (6*M / B*L^2), Otherwise = 0)
LRFD Total Moment, Mu =0k-ftASD Soil Pressures: $e = 0.000$ ft (L/6)(ASD M / P) (L/6) $e = -6.67$ ft $e > = < Kern ?$
LRFD Total Moment, $Mu = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmax = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Is Qmax <sbc? "equal="" (l*b)<br="" qmax="(2*P)" to",="" yes="">"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e)) LRFD Soil Pressures: e = 0.000 ft (ASD M / Pu) Kern = 0.667 ft (L / 6) e &gt; = &lt; Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf "Equal To", Qmax = (2*Pu) / (L*B)</sbc?>
LRFD Total Moment, $\dot{M}u = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmin = (P/L*B) - (6*M / B*L^2), Utherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmin = (P/L*B) + (6*M / B*L^2), Is Qmax <sbc? <b="">YES "Equal To", Qmax = (2*P) / (L*B) "Greater Than", Qmax = (4*P) / (3*B*(L - 2*e)) <b>LRFD Soil Pressures:</b> e = 0.000 ft (ASD M / Pu) Kern = 0.667 ft (L / 6) e &gt; = &lt; Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 2.013 ksf ("Less Than", Qmin = (PuL*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmin = (PuL*B) - (6*M / B*L^2), Otherwise = 0) Qeritical = 2.013 ksf ("Less Than", Qmin = (PuL*B) - (6*M / B*L^2), Otherwise = 0) Critical Length = 1.289 ft "Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))</sbc?>
LRFD Total Moment, $Mu = 0$ k-ft ASD Soil Pressures: e = 0.000 ft (ASD M / P) Kern = 0.667 ft (L / 6) e > = < Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 1.463 ksf ("Less Than", Qmax = (P/L*B) - (6*M / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 1.463 ksf ("Less Than", Qmax = (P/L*B) + (6*M / B*L^2), Is Qmax <sbc? "equal="" (l*b)<br="" qmax="(2*P)" to",="" yes="">"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e)) LRFD Soil Pressures: e = 0.000 ft (ASD M / Pu) Kern = 0.667 ft (L / 6) e &gt; = &lt; Kern? Less Than Length of Pressure = 4.000 ft ("Greater Than", Length = 3*(L/2 - e); Otherwise = L) Minimum Pressure, Qmin = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf ("Less Than", Qmax = (Pu/L*B) + (6*Mu / B*L^2), Otherwise = 0) Maximum Pressure, Qmax = 2.013 ksf "Equal To", Qmax = (2*Pu) / (L*B)</sbc?>

P/Pu



Project	Balderston Auto		Pro	20-467 Dject No	
Calc. By_	RJS	Checked By	JH	Date 03/01/21	

One-Way Shear Check: $Vu1 = 10.38 k \qquad (Qcrit*CritL + (Qmax-Qcrit*CritL*0.5)  dvn = 31.80 k \qquad (Qcrit*CritL + (Qmax-Qcrit*CritL*0.5)  Adequate in One-Way Shear? YES  Two-Way Shear Check: b1 = 17.06 in \qquad (Critical Section B + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 17.06 in \qquad (Column Height L + d)  b2 = 1 \qquad (ACI 318-08 Section 11.11.2.1)  a = 40 \qquad (ACI 318-08 Section 11.11.2.1, Larger Fig Dim / Smaller Fig  dvn = 135.63 k \qquad (ACI 318-08 Section 11.11.2.1, Larger Fig Dim / Smaller Fig  dvn = 152.02 k \qquad (ACI 318-08 Section 11.11.2.1, Larger Fig Dim / Smaller Fig  dvn = 152.02 k \qquad (ACI 318-08 Section 11.11.2.1, Larger Fig Dim / Smaller Fig  dvn = 152.02 k \qquad (ACI 318-08 Section 11.11.2.1, Larger Fig Dim / Smaller Fig  dvn = 152.02 k \qquad (ACI 318-08 Section 10.14.1 dvn = 0.65^{\circ}0.85^{\circ}tc^{\circ}Plate Arc  Adequate in Two-Way Shear? YES  Uplift Check: ASD Combo for Uplift = 0.6D + Uplift (ASCE 7)  dvitt = 0.6D k \qquad (Uplift - 0.6)  Additional Slab Used = 4 ft (Length of Additional Slab in Each Direction )  Wall Weight Over Footing = 0 ktif (B or L Depending on the Case)  Length Perpendicular to Slab Edge = 0 ft (B or L Depending on the Case)  Area of Cont. Footing = 0 ktif (B or L Depending on the Case)  Area of Cont. Footing = 0 ft2  Length Perpendicular to Slab Edge = 0 ft (B or L Depending on the Case)  Area of Cont. Footing = 0 ft2  Adequate for Uplift? Footing is Adequate to Resist Uplift (Calculation assumes wall is above the cont. ftg.)  Top Steel: \frac{Mu = 0.000 ktif / ft}{Mu = (Pu/A)^{\circ}0.5^{\circ}Crit.L^{\circ}2) (m = 4y(0.85^{\circ}Cr))$	g Dim ) )) id/bo))
$ \begin{array}{c} \operatorname{dv} n = 31.80  k & (ACI 318-08 Equation 11-5, \ensuremath{\Phi Vn} = 0.75^{+2} \operatorname{sqrt}(fc)^{+}B^{+}d / 10^{+} (ACI 318-08 Equation 11-5, \ensuremath{\Phi Vn} = 0.75^{+2} \operatorname{sqrt}(fc)^{+}B^{+}d / 10^{+} (ACI 318-08 Equation 11-5, \ensuremath{\Phi Vn} = 0.75^{+2} \operatorname{sqrt}(fc)^{+}B^{+}d / 10^{+} (ACI 318-08 Equation 11-12, \ensuremath{\Phi Vn} = 0.75^{+2} \operatorname{sqrt}(fc)^{+}B^{+}d / 10^{+} (ACI 318-08 Equation 11-12, \ensuremath{\Phi Vn} = 0.75^{+2} \operatorname{sqrt}(fc)^{+}B^{+}d / 10^{+} (ACI 318-08 Equation 11-12, \ensuremath{\Phi Vn} = 0.75^{+2} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Equation 11-12, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Equation 11-11, 2, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Equation 11-11, 2, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.75^{+} \operatorname{sqrt}(fc)^{+}D^{+}d / 10^{+} (ACI 318-08 Eq 11-32, \ensuremath{\Phi Vn} = 0.65^{+} \operatorname{sqrt}(fc)^{+}D$	g Dim ) )) id/bo))
Adequate in One-Way Shear? YES Two-Way Shear Check: b1 = 17.06 in (Critical Section B + d) b2 = 17.06 in (Column Height L + d) (Column Height Check: (ACI 318-08 Eq 11-32, $\Phi$ Vn = 0.75*sqtt(fc)*bo*d) Adequate in Height G + Uplift (ACI 318-08 Section 10.14.1 $\Phi$ Pn = 0.65*0.85*fc*Plate Are Adequate in Bearing? YES Uplift Check: ASD Combo for Uplift = 0.6D + Uplift (ASCE 7) (Uplift / 0.6) Applied Dead Load = 0.00 k (Uplift / 0.6) Applied Dead Load = 0.00 k ((Uplift / 0.6) Applied Dead Load + Slab + Ftg = 13.1 k (Length of Additional Slab Used = 4 ft (Length of Additional Slab in Each Direction ) Wall Weight Over Footing = 0 ktif Length Parallel to Slab Edge = 0 ft (H = (Depending on the Case) Area of Cont. Footing = 0 ft <sup>4</sup> Length of Cont. Footing Used = 0 ft (Length of Cont. Footing Used = 0 ft (Length of Cont. Footing Used = 0 ft (Length of Cont. Footing Used = 0 ft (Calculation assumes wall is above the cont. ftg.) Top Steel: Mu = 0.00 k-ft / ft (Mu = (PulA)^0.5*Crit. L^2) (m = fy(0.85*fc))	g Dim ) )) id/bo))
b) = 17.06 in (Critical Section B + d) b) = 17.06 in (Column Height L + d) b) = 68.25 in (2'b'1 + 2'b2) Vu2 = 28.13 k (Vu2 = (Qmax+Qmin)/2 * (Fig Area - b'tb2)) a = 40 (ACI 318-08 Section 11.11.2.1) b) = 1 (ACI 318-08 Section 11.11.2.1) c) a = 40 (ACI 318-08 Section 11.11.2.1) c) a = 1 (ACI 318-08 Section 11.11.2.1) c) a = 0.02 k (ACI 318-08 Section 10.11.11.2) c) a = 0.05°0.85°fc*Plate Area a dequate in Bearing Pteck: a Adequate in Bearing ? YES Column Bearing Check: a Adequate in Bearing ? YES Column Height A = 0.00 k (Uplit / 0.6) A pieled Dead Load = 0.00 k (Uplit / 0.6) A pieled Dead Load = 0.00 k (If (Length of Additional Slab in Each Direction ) Wall Weight Over Footing = 0 kIf Length Parallel to Slab Edge = 0 ft (B or L Depending on the Case) Length Parallel to Slab Edge = 0 ft Adequate for Uplit? Footing is Adequate to Resist Uplit! Total Dead Load = 21.1 k (Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft (Caculation assumes wall is above the cont. ftg.) m = 23.529 (m = fy(0.85°c)) (m = fy(0.85°c)) (m = fy(0.85°c))	)) d/bo))
b) = 17.06 in (Critical Section B + d) b) = 17.06 in (Column Height L + d) b) = 68.25 in (2'b'1 + 2'b2) Vu2 = 28.13 k (Vu2 = (Qmax+Qmin)/2 * (Fig Area - b'tb2)) a = 40 (ACI 318-08 Section 11.11.2.1) b) = 1 (ACI 318-08 Section 11.11.2.1) c) a = 40 (ACI 318-08 Section 11.11.2.1) c) b) = 1 (ACI 318-08 Section 10.11.11.2) c) b) = 1 (ACI 318-08 Section 10.11.1.1 (D'Pn = 0.65*0.85*fc*Plate Area c) Adequate in Bearing (P ES) Column Bearing Check: c) b) = 477.36 k (ACI 318-08 Section 10.11.1 (D'Pn = 0.65*0.85*fc*Plate Area c) Adequate in Bearing (P ES) Column	)) d/bo))
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	)) d/bo))
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	)) d/bo))
$\begin{array}{rrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrr$	)) d/bo))
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	)) d/bo))
$ \begin{array}{c} \beta = 1 \\ (ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg \\ \Phi Vn = 135.63 k \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75*sqrt(fc)*bo*d*(2+A)Beta \\ \Phi Vn = 90.42 k \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75*sqrt(fc)*bo*d*(2+A)Beta \\ \Phi Vn = 90.42 k \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75*sqrt(fc)*bo*d*(2+A)Beta \\ \Phi Vn = 90.42 k \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75*sqrt(fc)*bo*d*(2+A)Beta \\ \Phi Vn = 90.42 k \\ (ACI 318-08 Section 10.14.1 \Phi Pn = 0.65*0.85*fc*Plate Are Adequate in Bearing? YES \\ \hline Uplift Check: \\ Abequate in Bearing? YES \\ \hline Uplift Check: \\ AsD Combo for Uplift = 0.6D + Uplift \\ Additional Slab Used = 0 k \\ Additional Slab Used = 4 ft \\ Additional Slab Used = 4 ft \\ Additional Slab Used = 4 ft \\ Length Parallel to Slab Edge = 0 ft \\ Length Parallel to Slab Edge = 0 ft \\ B or L Depending on the Case) \\ Length of Cont. Footing = 0 ft^2 \\ Length of Cont. Footing is Adequate to Resist Uplift \\ Total Dead Load = 21.1 k \\ Adequate for Uplift? Footing is Adequate to Resist Uplift \\ Total Dead Load = 21.1 k \\ Mu = 0.00 k \cdot ft / ft \\ Mu = 0.00 k \cdot ft / ft \\ Mu = 0.00 k \cdot ft / ft \\ Mu = (Pu/A)*0.5*Crit. L^2) \\ m = 23.529 \\ (m = fy/(0.85*fc)) \end{array}$	)) d/bo))
$ \begin{array}{c} \Phi Vn = 135.63  k & (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ \Phi Vn = 152.02  k & (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^* bo^* d^*(2+4)Beta \\ (ASCE 7) & (Upiff 10-6) \\ (ASC$	)) d/bo))
$\begin{array}{c} \Phi Vn = & 152.02 & k \\ \Phi Vn = & 90.42 & k \end{array} \qquad (ACI 318-08 Eq 11-32, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ACI 318-08 Eq 11-33, \Phi Vn = 0.75^* sqrt(fc)^*bo^*d^*(2^+Alpha^* \\ (ASCE 7) \\ (Vn = 0.00 & k (f + f f f f f f f f f f f f f f f f f $	d/bo))
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	
Adequate in Two-Way Shear? YES Column Bearing Check: $\Phi Pn = 477.36$ k (ACI 318-08 Section 10.14.1 $\Phi Pn = 0.65^{\circ}0.85^{\circ}fc^{\circ}Plate Are Adequate in Bearing? YES Uplift Check: Adequate in Bearing? YES Uplift Check: ASD Combo for Uplift = 0.6D + Uplift (ASCE 7) Uplift Force = 0 k (From Above) Required Dead Load = 0.00 k (Uplift / 0.6) Applied Dead Load + Slab + Ftg = 13.1 k Additional Slab used = 4 ft (Length of Additional Slab in Each Direction ) Wall Weight Over Footing = 0 klf Length Parallel to Slab Edge = 0 ft (B or L Depending on the Case) Length Perpendicular to Slab Edge = 0 ft (B or L Depending on the Case) Area of Cont. Footing Used = 0 ft (Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft Adequate for Uplift? Footing is Adequate to Resist Uplift Total Dead Load = 21.1 k (Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft (Calculation assumes wall is above the cont. ftg.) Top Steel: Mu = 0.00 \ k-ft / ft (Mu = (Pu/A)^{\circ}0.5^{\circ}Crit. L^{2}) (m = fy/(0.85^{\circ}fc))$	ea*2) )
$\begin{array}{c} \Phi Pn = & 477.36  k \\ Adequate in Bearing?  \textbf{YES} \end{array} \qquad (ACI 318-08 \ Section 10.14.1 \ \Phi Pn = 0.65^{\circ}0.85^{\circ}fc^{\circ}Plate \ Area \ Adequate in Bearing?  \textbf{YES} \end{array}$	ea*2))
$\begin{array}{c} \Phi Pn = & 477.36  k \\ Adequate in Bearing?  \textbf{YES} \end{array} \qquad (ACI 318-08 \ Section 10.14.1 \ \Phi Pn = 0.65^{\circ}0.85^{\circ}fc^{\circ}Plate \ Area \ Adequate in Bearing?  \textbf{YES} \end{array}$	ea*2))
Adequate in Bearing?YESUplift Check: ASD Combo for Uplift = $0.6D + Uplift$ (ASCE 7) (Pplift Force = $0$ kRequired Dead Load = $0.00$ k(From Above) (Uplift / $0.6$ )Applied Dead Load + Slab + Ftg = 13.1 k(Length of Additional Slab in Each Direction )Additional Slab Used = $4$ ft(Length of Additional Slab in Each Direction )Wall Weight Over Footing = $0$ klf(B or L Depending on the Case)Length Parallel to Slab Edge = $0$ ft(B or L Depending on the Case)Area of Cont. Footing = $0$ ft²(B or L Depending on the Case)Area of Cont. Footing = $0$ ft²(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft (Calculation assumes wall is above the cont. ftg.)Top Steel:Mu = $0.00$ k-ft / ft(Mu = (Pu/A)*0.5*Crit. L^2 ) (m = fy/(0.85*fc))	a~2))
Uplift Check: ASD Combo for Uplift = 0.6D + Uplift(ASCE 7) Uplift Force = 0 kRequired Dead Load = 0.00 k(From Above) Required Dead Load = 0.00 k(Uplift / 0.6)Applied Dead Load + Slab + Ftg = 13.1 k(Length of Additional Slab in Each Direction )Mall Weight Over Footing = 0 klf Length Parallel to Slab Edge = 0 ft(B or L Depending on the Case)Length Perpendicular to Slab Edge = 0 ft(B or L Depending on the Case)Area of Cont. Footing = 0 ft <sup>2</sup> Length of Cont. Footing Used = 0 ft(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft (Calculation assumes wall is above the cont. ftg.)Top Steel:Mu = 0.00 k-ft / ft m = 23.529Mu = (Pu/A)*0.5*Crit. L^2) (m = fy/(0.85*fc))	
ASD Combo for Uplift = $0.6D + Uplift$ (ASCE 7) Uplift Force = 0 k (From Above) Required Dead Load = 0.00 k (Uplift / 0.6) Applied Dead Load + Slab + Ftg = 13.1 k Additional Slab Used = 4 ft (Length of Additional Slab in Each Direction) Wall Weight Over Footing = 0 klf Length Parallel to Slab Edge = 0 ft (B or L Depending on the Case) Length Perpendicular to Slab Edge = 0 ft (B or L Depending on the Case) Area of Cont. Footing = 0 ft <sup>2</sup> Length of Cont. Footing used = 0 ft Total Dead Load = 21.1 k (Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft Adequate for Uplift? Footing is Adequate to Resist Uplift Top Steel: $Mu = 0.00  k-ft / ft (Mu = (Pu/A)*0.5*Crit. L^2)  m = 23.529  (m = fy/(0.85*fc))$	
Uplift Force =0k(From Above)Required Dead Load =0.00k(Uplift / 0.6)Applied Dead Load + Slab + Ftg =13.1kAdditional Slab Used =4ft(Length of Additional Slab in Each Direction)Wall Weight Over Footing =0klfLength Parallel to Slab Edge =0ft(B or L Depending on the Case)Length Perpendicular to Slab Edge =0ft(B or L Depending on the Case)Area of Cont. Footing =0ft²(B or L Depending on the Case)Length of Cont. Footing Used =0ft²(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft (Calculation assumes wall is above the cont. ftg.)Top Steel:Mu =0.00k-ft / ft(Mu = (Pu/A)*0.5*Crit. L^2 ) (m = fy/(0.85*fc))	
Required Dead Load =0.00k(Uplift / 0.6)Applied Dead Load + Slab + Ftg =13.1kAdditional Slab Used =4ft(Length of Additional Slab in Each Direction )Wall Weight Over Footing =0klfLength Parallel to Slab Edge =0ft(B or L Depending on the Case)Length Perpendicular to Slab Edge =0ft(B or L Depending on the Case)Area of Cont. Footing =0ft²(B or L Depending on the Case)Length of Cont. Footing Used =0ft(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. FtAdequate for Uplift?Footing is Adequate to Resist Uplift(Calculation assumes wall is above the cont. ftg.)Top Steel:Mu =0.00k-ft / ft(Mu = (Pu/A)*0.5*Crit. L^2 )m =23.529(m = fy/(0.85*fc))	
Applied Dead Load + Slab + Ftg =13.1kAdditional Slab Used =4ft(Length of Additional Slab in Each Direction )Wall Weight Over Footing =0klf(B or L Depending on the Case)Length Parallel to Slab Edge =0ft(B or L Depending on the Case)Length Perpendicular to Slab Edge =0ft(B or L Depending on the Case)Area of Cont. Footing =0ft²(B or L Depending on the Case)Length of Cont. Footing Used =0ft(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. FtAdequate for Uplift?Footing is Adequate to Resist Uplift(Calculation assumes wall is above the cont. ftg.)Top Steel:Mu =0.00k-ft / ft(Mu = (Pu/A)*0.5*Crit. L^2 )m =23.529(m = fy/(0.85*fc))	
Additional Slab Used =4ft( Length of Additional Slab in Each Direction )Wall Weight Over Footing =0klf(B or L Depending on the Case)Length Parallel to Slab Edge =0ft(B or L Depending on the Case)Length Perpendicular to Slab Edge =0ft(B or L Depending on the Case)Area of Cont. Footing =0ft²Length of Cont. Footing Used =0ftTotal Dead Load =21.1kAdequate for Uplift?Footing is Adequate to Resist UpliftTop Steel:Mu =0.00k-ft / ftMu =0.00K-ft / ft(Mu = (Pu/A)*0.5*Crit. L^2 )Mu =0.00K-ft / ft(Mu = fy/(0.85*fc) )	
Wall Weight Over Footing =0klfLength Parallel to Slab Edge =0ft(B or L Depending on the Case)Length Perpendicular to Slab Edge =0ft(B or L Depending on the Case)Area of Cont. Footing =0ft²(B or L Depending on the Case)Length of Cont. Footing Used =0ft(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. FtAdequate for Uplift?Footing is Adequate to Resist Uplift(Calculation assumes wall is above the cont. ftg.)Top Steel:Mu =0.00k-ft / ft(Mu = (Pu/A)*0.5*Crit. L^2 )m =23.529(m = fy/(0.85*fc))	
Length Parallel to Slab Edge =0ft(B or L Depending on the Case)Length Perpendicular to Slab Edge =0ft(B or L Depending on the Case)Area of Cont. Footing =0ft²Length of Cont. Footing Used =0ftTotal Dead Load =21.1kAdequate for Uplift?Footing is Adequate to Resist Uplift(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. FtTop Steel:Mu =0.00k-ft / ft(Mu = (Pu/A)*0.5*Crit. L^2 )m =23.529(m = fy/(0.85*fc) )	
Length Perpendicular to Slab Edge = 0 ft (B or L Depending on the Case) Area of Cont. Footing = 0 ft <sup>2</sup> Length of Cont. Footing Used = 0 ft Total Dead Load = 21.1 k (Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft Adequate for Uplift? Footing is Adequate to Resist Uplift (Calculation assumes wall is above the cont. ftg.) Top Steel: $Mu = 0.00  k-ft / ft (Mu = (Pu/A)*0.5*Crit. L^2) (m = fy/(0.85*fc))$	
Length Perpendicular to Slab Edge = 0 ft (B or L Depending on the Case) Area of Cont. Footing = 0 ft <sup>2</sup> Length of Cont. Footing Used = 0 ft Total Dead Load = 21.1 k (Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft Adequate for Uplift? Footing is Adequate to Resist Uplift (Calculation assumes wall is above the cont. ftg.) Top Steel: $Mu = 0.00  k-ft / ft (Mu = (Pu/A)*0.5*Crit. L^2) (m = fy/(0.85*fc))$	
Area of Cont. Footing =0 $ft^2$ Length of Cont. Footing Used =0ftTotal Dead Load =21.1kAdequate for Uplift?Footing is Adequate to Resist Uplift(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. FtTop Steel:(Au =Mu =0.00k-ft / ftMu =0.00k-ft / ftm =23.529(Mu = (Pu/A)*0.5*Crit. L^2 )m =(m = fy/(0.85*fc) )	
Length of Cont. Footing Used = 0 ft Total Dead Load = 21.1 k (Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ft Adequate for Uplift? Footing is Adequate to Resist Uplift (Calculation assumes wall is above the cont. ftg.) Top Steel: Mu = 0.00 k-ft / ft (Mu = (Pu/A)*0.5*Crit. L^2) m = 23.529 (m = fy/(0.85*fc))	
Total Dead Load = $21.1$ k(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. FtAdequate for Uplift?Footing is Adequate to Resist Uplift(Calculation assumes wall is above the cont. ftg.)Top Steel: $Mu = 0.00$ k-ft / ft( $Mu = (Pu/A)^*0.5^*Crit. L^2$ )m = $23.529$ (m = fy/(0.85^*fc))	
Adequate for Uplift? Footing is Adequate to Resist Uplift(Calculation assumes wall is above the cont. ftg.)Top Steel: $Mu = 0.00  ext{ k-ft / ft}$ $(Mu = (Pu/A)^*0.5^*Crit. L^2)$ m = 23.529 $(m = fy/(0.85^*fc))$	a)
Top Steel: Mu = 0.00 k-ft / ft (Mu = (Pu/A)*0.5*Crit. L^2) m = 23.529 (m = fy/(0.85*fc))	9/
Mu =0.00k-ft / ft( Mu = (Pu/A)*0.5*Crit. L^2 )m =23.529( m = fy/(0.85*fc) )	
m = 23.529 ( $m = fy/(0.85*fc)$ )	
$Ru = 0.000$ ksi $(Ru = Mu/(0.9^{-1}2 \text{ inches}^{-1}2))$	
$\rho \operatorname{Req'd} = 0.0000$ ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*\operatorname{Ru}^*m/\operatorname{fy}))$ )	
ρ Min. = 0.0027 (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/f	у)
$4/3^{*}$ Mu $\rho$ Req'd = 0.0000 ( $\rho = (1/m)^{*}(1-sqrt(1-2^{*}1.33^{*}Ru^{*}m/fy)))$	
Governing $\rho$ = 0.0000 (If $\rho$ Req'd < 4/3*Mu $\rho$ Req'd < $\rho$ Min, Use 4/3*Mu $\rho$ Req'd	
A's Required = 0.000 $in^2/ft$ (As = Governing $\rho^*12$ inches*d)	
Bar # = 4	
Bar Spacing = <mark>12</mark> in	
As Provided = 0.20 in <sup>2</sup> /ft = 5 Bars in B Direction	
= 5 Bars in L Direction	
Bottom Steel:	
Mu = 1.67 k-ft / ft (Mu= Qcrit*0.5*Lcrit*2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit*2)	
m = 23.529 ( $m = fy/(0.85*fc)$ )	
$Ru = 0.029$ ksi $(Ru = Mu/(0.9*12 inches*d^2))$	
$\rho \text{ Reg'd} = 0.0005$ ( $\rho = (1/m)^*(1-\operatorname{sqrt}(1-2^*\operatorname{Ru}^*m/fy))$ )	
ρ Min. = 0.0027 (ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/f	y)
4/3*Mu $\rho$ Req'd = 0.0006 ( $\rho = (1/m)^*(1-\text{sqrt}(1-2^*1.33^*\text{Ru}^*m/\text{fy}))$ )	
$Governing \rho = 0.0006    (If \rho \text{ Reg'd} < 4/3*\text{Mu} \rho \text{ Reg'd} < 2/3*\text{Mu} \rho \text{ Reg'd} < 4/3*\text{Mu} $	
A's Required = $0.062 \text{ in}^2/\text{ft}$ (As = Governing p*12 inches*d)	
Bar # = 5	
Bar Spacing = 12 in $i^{2/4}$	
As Provided = $0.31$ in <sup>2</sup> /ft = 5 Bars in B Direction	
= 5 Bars in L Direction	



Project	Balderston Auto			Project No	20-467	
Calc. Bv	RJS	Checked By	JH	Date	03/01/21	

Temperature &	Shrinkage	Steel:

Minimum Steel =	0.2592	in²/ft	( T&S Steel = 0.0018*12 inches*H )
As Provided Top =	0.20	in²/ft	
As Provided Bott =	0.31	in²/ft	
As Provided Total =	0.51	in²/ft	
T&S Steel Provided?	YES		
Final Footing Design:			
Footing Width, B =	4	ft	
Footing Length L =	4	ft	
Footing Depth, H =	12	in	
Top Steel =	#4 bar	s @12 inches O.C.	

Bottom Steel = **#5 bars** @12 inches O.C.



Project	<b>Balderston Auto</b>		Pro	oject No	20-467	
Calc. By	RJS	Checked By	JH	Date_	03/01/21	

Grade Beam Location:	Thickened Sla	ab NLB 8	' CMU
General Information:			WIDTH
Footing Width, B =	18	in	TOP STEEL
Footing Depth, H =	12	in	4
Steel Depth, d =	8.0625	in	(H - 3 in - 1.5*Bar Dia. )
Wall Width =	8.625	in	<b>T</b>
Soil Bearing Pressure =	2.5	ksf	
Allowable or Effective SBC?	Allowable		le a al BOTTOW STEEL
Concrete Strength =	3	ksi	
			В
Loading:			
Vertical Loads:			LRFD Factors: (ASCE 7 Combo)
Applied Dead Load =	0.9	klf	
Wall Weight =	56	psf	Dead = 1.2
Wall Height =	22	ft	Live = 1.6
Total Wall Weight =	1.232	klf	
Footing Weight =	0.225	klf	
Applied Live Load =	0.10	klf	
ASD Total Load, W =	2.457	klf	
LRFD Total Load, Wu =	2.9884	klf	
ASD Soil Pressures:			
Required Footing Width =	0.9828	ft	(Footing Width = W / Soil Bearing Pressure)
Chosen Footing Width =	1.5	ft	
Assumed Footing Span =	6	ft	
Is Footing Width Adequate?	YES		
One-Way Shear Check:			
Vu1 =	8.97	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	11.92	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000 )
Adequate in One-Way Shear?	YES	i di	(710101000 Equation 110, \$111 0.70 E oqu(10) B u 71000 )
Are Stirrups Reg'd?	YES		( ACI 318-08 Section 11.4.6.1 If
	Calculate Sti	rrups	(Provide minimum stirrups to support steel)
Wall Bearing Check:			
ΦPn =	28.591875	klf	( ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*fc*Wall Width*1'*2)
Adequate in Bearing?	YES		
Bottom Steel:			
Mu =	13.45	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	23.529		(m = fy/(0.85 * fc))
Ru =	0.153	ksi	$(Ru = Mu/(0.9*B*d^2))$
ρ Req'd =	0.0026		$(\rho = (1/m)^{(1.5 \text{ J} + 1/2)})$
ρ Min. =	0.0020		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy )
4/3*MuρReg'd =	0.0027		$(\rho = (1/m)^{(1-sqrt(1-2^{*}1.33^{*}Ru^{*}m/fy)))$
			(μ – (1/11) (1-sqr((1-2 1.33 Ku 11/19))) ( If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
Governing p =	0.0027	in <sup>2</sup>	
A's Required =	0.397	in <sup>2</sup>	( As = Governing ρ*B*d )
Bar # =	5	her-	
Number of Bars =	3	bars	
As Provided =	0.93	in <sup>2</sup>	
Is Steel Adequate ?	YES		
Top Steel:			
Top Steel: Bar # =	5		
-	5 2	bars in²	



Project	Balderston Auto			Project No	20-467
Calc. By	RJS	Checked By	JH	Date_	03/01/21

### Temperature & Shrinkage Steel:

Minimum Steel = As Provided Top = As Provided Bott = As Provided Total = T&S Steel Provided?	0.3888 0.62 0.93 1.55 <b>YES</b>	in <sup>2</sup> /ft in <sup>2</sup> /ft in <sup>2</sup> /ft in <sup>2</sup> /ft	( T&S Steel = 0.0018*B*H )
Final Footing Design: Footing Width, B = Footing Depth, H =	18 12	in in 5) #5 bars	

Top Steel =	(5) #5 bars
Bottom Steel =	(3) #5 bars
Stirrups = Calculat	e Stirrups



Project	Balderston Auto			Project No	20-467
Calc. By	RJS	Checked By	JH	Date	03/01/21

Footing Design			
Grade Beam Design			w/wu I
Grade Beam Location:	NLB w/ 12" C	MU	
General Information:			WALL
Footing Width, B =	30	in	TOP STEEL
Footing Depth, H =	34	in	
Steel Depth, d =	30.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	11.625	in	
Soil Bearing Pressure =	2.5	ksf	
Allowable or Effective SBC?	Allowable	NOI	BOTTOM STEEL
Concrete Strength =	3	ksi	- 40 <sup>d</sup>
	5	151	В
Loading:			
Vertical Loads:			LRFD Factors: (ASCE 7 Combo)
Applied Dead Load =	0.05	klf	
Wall Weight =	83	psf	Dead = 1.2
Wall Height =	27	ft	Live = 1.6
Total Wall Weight =	2.241	klf	
Footing Weight =	1.0625	klf	
Applied Live Load =	0.13	klf	
ASD Total Load, W =	3.48575	klf	
LRFD Total Load, Wu =	4.2358	klf	
ASD Soil Pressures:			
Required Footing Width =	1.3943	ft	(Footing Width = W / Soil Bearing Pressure)
Chosen Footing Width =	2.5	ft	(·······
Assumed Footing Span =	6	ft	
Is Footing Width Adequate?	YES		
One-Way Shear Check:			
Vu1 =	12.71	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	74.10	klf	
	YES	KII	( ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(fc)*B*d / 1000 )
Adequate in One-Way Shear? Are Stirrups Req'd?	NO		( ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otherwise "Yes")
	Use #3 Stirru	ips at 1	
Wall Bearing Check:			
ΦPn =	38.536875	klf	( ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall Width*1'*2) )
Adequate in Bearing?	YES		
Bottom Steel:			
Mu =	19.06	k-ft	(Wu*Assumed Footing Span^2 / 8 )
m =			(m = fy/(0.85*fc))
Ru =	0.009	ksi	$(Ru = Mu/(0.9*B*d^2))$
ρ Req'd =	0.0002		$(\rho = (1/m)^{*}(1-sqrt(1-2^{*}Ru^{*}m/fy)))$
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy )
4/3*Mu ρ Req'd =	0.0002		$(\rho = (1/m)^{*}(1-sqrt(1-2^{*}1.33^{*}Ru^{*}m/fy)))$
Governing $\rho$ =	0.0002		( If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.188	in <sup>2</sup>	(As = Governing $\rho^*B^*d$ )
Bar # =	5		(··································
Number of Bars =	3	bars	
As Provided =	0.93	in <sup>2</sup>	
Is Steel Adequate ?	0.93 YES		
Top Steel:			
Bar # =	5		
Number of Bars =	3	bars	
As Provided =	0.93	in <sup>2</sup>	
As Hovided =	0.00		



Project	Balderston Auto			Project No	20-467
Calc. By	RJS	Checked By	JH	Date_	03/01/21

Footing Design			
Temperature & Shrinkage St	teel:		
Minimum Steel =	1.836	in²/ft	( T&S Steel = 0.0018*B*H )
As Provided Top =	0.93	in²/ft	
As Provided Bott =	0.93	in²/ft	
As Provided Total =	1.86	in²/ft	
T&S Steel Provided?	YES		
Final Footing Design:			
Footing Width, B =	30	in	
Footing Depth, H =	34	in	
Top Steel = Bottom Steel =	•	3) #5 bars 3) #5 bars	

Stirrups = #3 Stirrups at 18 in. O.C.



As Provided =

1.55

in<sup>2</sup>

Project	Balderston Auto			Project No	20-467
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#### Footing Design W/Wu Grade Beam Design Grade Beam Location: Mezzanine WALL WIDTH **General Information:** TOP STEEL Footing Width, B = 42 in Footing Depth, H = 34 in Steel Depth, d = 30.0625 in (H - 3 in - 1.5\*Bar Dia.) STIRRUPS Wall Width = 11.625 in Soil Bearing Pressure = 2.5 ksf BOTTOM STEEL Allowable or Effective SBC? Allowable Concrete Strength = 3 ksi в Loading: LRFD Factors: (ASCE 7 Combo) Vertical Loads: Applied Dead Load = 1.4 klf Wall Weight = Dead = 1.2 83 psf Wall Height = 24 ft Live = 1.6 Total Wall Weight = 1.992 klf Footing Weight = 1.4875 klf Applied Live Load = 1.50 klf ASD Total Load, W = 6.3795 klf LRFD Total Load, Wu = 8.2554 klf **ASD Soil Pressures:** Required Footing Width = 2.5518 ft (Footing Width = W / Soil Bearing Pressure) Chosen Footing Width = 3.5 ft Assumed Footing Span = 6 ft Is Footing Width Adequate? YES **One-Way Shear Check:** Vu1 = 24.77 klf (Wu\*Assumed Footing Span / 2) ΦVn = 103.74 (ACI 318-08 Equation 11-5, ΦVn = 0.75\*2\*sqrt(fc)\*B\*d / 1000) klf Adequate in One-Way Shear? YES (ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otherwise "Yes") Are Stirrups Req'd? NO Use #3 Stirrups at 18 in. O.C. (Provide minimum stirrups to support steel) Wall Bearing Check: ΦPn = 38.536875 klf (ACI 318-08 Section 10.14.1 ΦPn = 0.65\*0.85\*f'c\*Wall Width\*1'\*2)) Adequate in Bearing? YES **Bottom Steel:** Mu = 37.15 k-ft (Wu\*Assumed Footing Span^2 / 8) ( m = fy/(0.85\*f'c) ) m = 23.529 0.013 $(Ru = Mu/(0.9*B*d^2))$ Ru = ksi $\rho$ Req'd = 0.0002 $(\rho = (1/m)^{*}(1-sqrt(1-2^{*}Ru^{*}m/fy)))$ ρ Min. = 0.0027 (ACI 318-08 Equation 10-3, Smaller of: 3\*sqrt(f'c)/fy & 200/fy ) 4/3\*Mu ρ Req'd = 0.0003 $(\rho = (1/m)^*(1-sqrt(1-2^*1.33^*Ru^*m/fy)))$ Governing $\rho$ = 0.0003 ( If $\rho \text{Req'd} < 4/3^*\text{Mu} \rho \text{Req'd} < \rho \text{Min}$ , Use $4/3^*\text{Mu} \rho \text{Req'd}$ in<sup>2</sup> A's Required = 0.366 (As = Governing $\rho^*B^*d$ ) Bar # = 5 5 Number of Bars = bars in<sup>2</sup> As Provided = 1.55 Is Steel Adequate ? YES Top Steel: 5 Bar # = Number of Bars = 5 bars

SHT. NO.



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Footing Design			
Temperature & Shrinkage S	teel:		
Minimum Steel =	2.5704	in²/ft	( T&S Steel = 0.0018*B*H )
As Provided Top =	1.55	in²/ft	
As Provided Bott =	1.55	in²/ft	
As Provided Total =	3.10	in²/ft	
T&S Steel Provided?	YES		
Final Footing Design:			
Footing Width, B =	42	in	
Footing Depth, H =	34	in	
Top Steel =	(!	5) #5 bars	
Bottom Steel =	•	5) #5 bars	
Stirrups = #	3 Stirrups	at 18 in. O.C.	



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Footing Design			
Grade Beam Design			w/wu I
Grade Beam Location:	12" CMU		
General Information:			WIDTH
Footing Width, B =	24	in	TOP STEEL
Footing Depth, H =	34	in	
Steel Depth, d =	30.0625	in	(H - 3 in - 1.5*Bar Dia. )
Wall Width =	11.625	in	<b>T</b>
Soil Bearing Pressure =	2.5	ksf	
Allowable or Effective SBC?	Allowable		BOTTOM STEEL
Concrete Strength =	3	ksi	
Loading:			<b>B</b>
Vertical Loads:			LRFD Factors: (ASCE 7 Combo)
Applied Dead Load =	0.24	klf	
Wall Weight =	83	psf	$Dead = \frac{1.2}{2}$
Wall Height =	27	ft	Live = 1.6
Total Wall Weight =	2.241	klf	
Footing Weight =	0.85	klf	
Applied Live Load =	0.40	klf	
ASD Total Load, W =	3.731	klf	
LRFD Total Load, Wu =	4.6372	klf	
ASD Soil Pressures:			
Required Footing Width =	1.4924	ft	(Footing Width = W / Soil Bearing Pressure)
Chosen Footing Width =	2	ft	
Assumed Footing Span =	6	ft	
Is Footing Width Adequate?	YES		
One-Way Shear Check:			
Vu1 =	13.91	klf	( Wu*Assumed Footing Span / 2 )
ΦVn =	59.28	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(fc)*B*d / 1000 )
Adequate in One-Way Shear?	YES		
Are Stirrups Req'd?	NO		( ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otherwise "Yes")
	Use #3 Stirru	ups at 1	8 in. O.C. (Provide minimum stirrups to support steel)
Wall Bearing Check:			
ΦPn =	38.536875	klf	( ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*fc*Wall Width*1'*2) )
Adequate in Bearing?	YES		
Bottom Steel:			
Mu =	20.87	k-ft	(Wu*Assumed Footing Span^2 / 8 )
m =	23.529		( m = fy/(0.85*fc) )
Ru =	0.013	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0002		$(\rho = (1/m)^{*}(1-sqrt(1-2^{*}Ru^{*}m/fy)))$
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy )
4/3*Mu ρ Req'd =	0.0003		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0003		( If ρ Req'd < 4/3*Μu ρ Req'd < ρ Min, Use 4/3*Μu ρ Req'd
A's Required =	0.206	in <sup>2</sup>	(As = Governing $\rho^*B^*d$ )
Bar # =	5		(
Number of Bars =	3	bars	
As Provided =	0.93	in <sup>2</sup>	
Is Steel Adequate ?	YES		
Top Steel:			
• Bar # =	5		
Number of Bars =	3	bars	
As Provided =	0.93	in <sup>2</sup>	



Project	Balderston Auto			Project No	20-467
Calc. By	RJS	Checked By	JH	Date_	03/01/21

Lesting Design			
Footing Design			
Temperature & Shrinkage St	teel:		
Minimum Steel =	1.4688	in²/ft	( T&S Steel = 0.0018*B*H )
As Provided Top =	0.93	in²/ft	
As Provided Bott =	0.93	in²/ft	
As Provided Total =	1.86	in²/ft	
T&S Steel Provided?	YES		
Final Footing Design:			
Footing Width, B =	24	in	
Footing Depth, H =	34	in	
Top Steel = Bottom Steel =	•	3) #5 bars 3) #5 bars	

Stirrups = **#3 Stirrups at 18 in. O.C.**