KREHER ENGINEERING, INC.

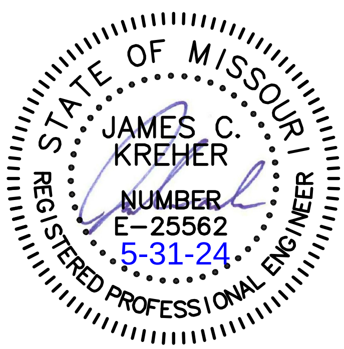
STRUCTURAL CALCULATIONS

PROPOSED NEW CARWASH
LOS 11 & 13 OF WEST PRYOR

LEE'S SUMMIT, MO

ARCHITECT:
ARCHITEXTURES SP
05/31/24

The Professional Engineers seal affixed to this sheet indicates that the named engineer has prepared or directed the preparation of the material shown only on the attached sheets pgs.1 thru 73 Other drawings and documents not exhibiting this seal shall not be considered prepared by or the responsibility of the undersigned



Exp 12-31-25

208 N. Main St., Suite H Columbia, IL 62236 ph (618)281-8505 fax (618)281-8515

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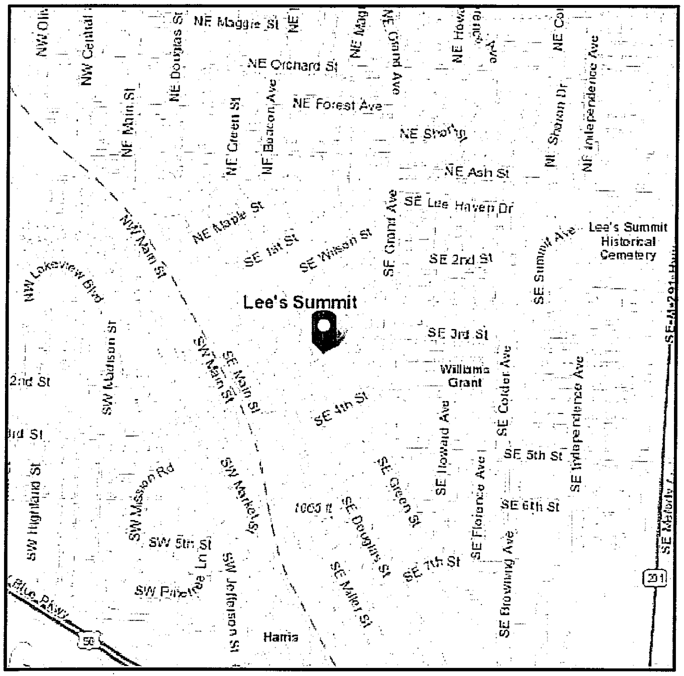
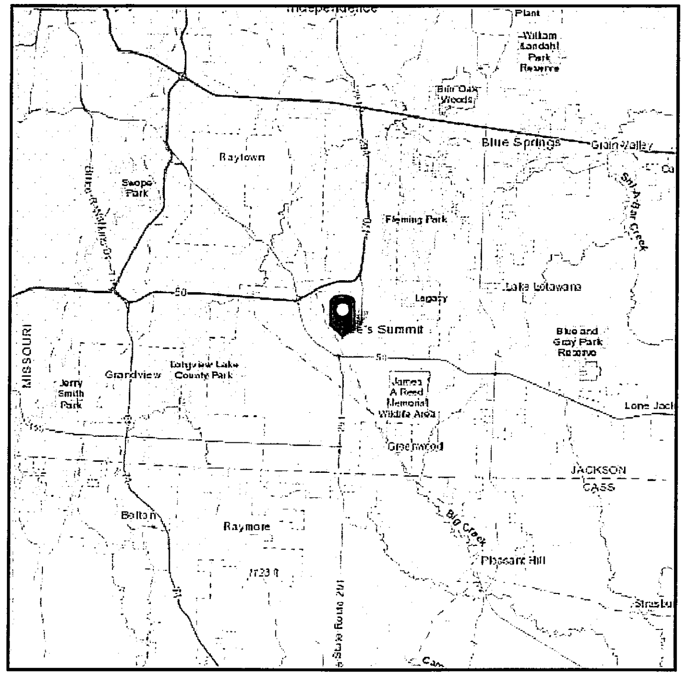
ASCE Hazards Report

Address:
Lee's Summit
Missouri,

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Latitude: 38.913214
Longitude: -94.374672
Elevation: 1019.1279459585498 ft (NAVD 88)





Wind

Results:

Wind Speed	~~109 Vmph~~ 115 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	88 Vmph
100-year MRI	94 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Mon May 27 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

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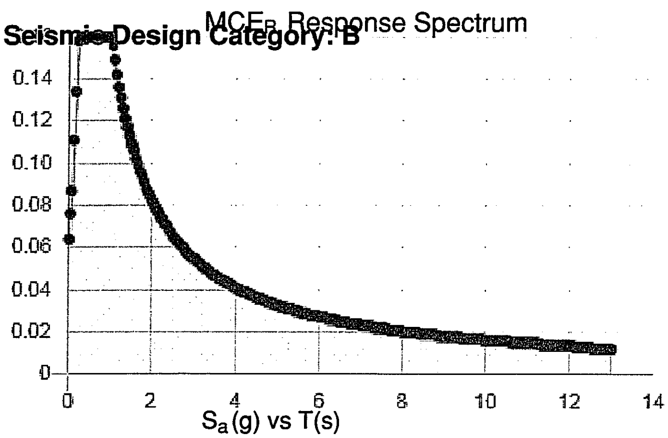
Seismic

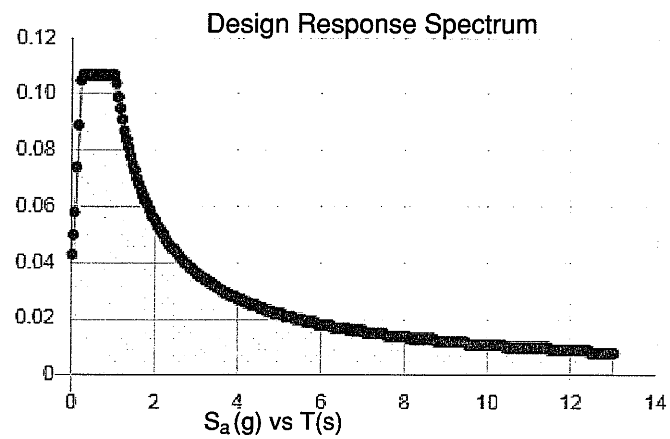
Site Soil Class: D - Default (see Section 11.4.3)

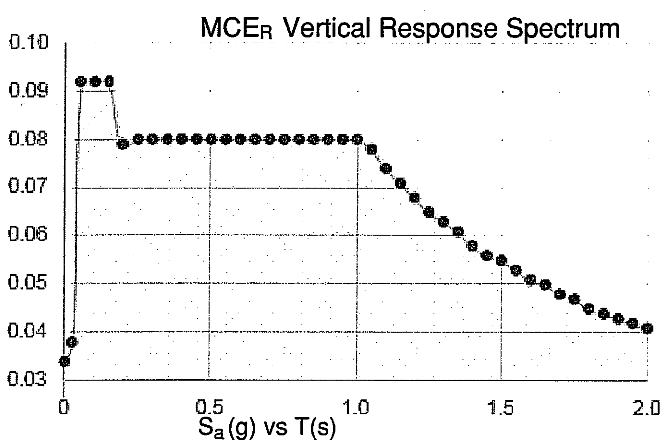
Results:

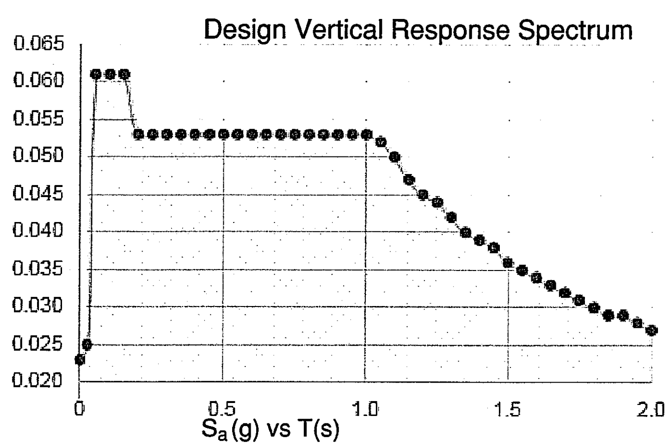
S_S :	0.1	S_{D1} :	0.109
S_1 :	0.068	T_L :	12
F_a :	1.6	PGA :	0.047
F_v :	2.4	PGA_M :	0.076
S_{MS} :	0.16	F_{PGA} :	1.6
S_{M1} :	0.164	I_e :	1
S_{DS} :	0.107	C_v :	0.7

Seismic Design Category: B









Data Accessed: **Mon May 27 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

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Ice

Results:

Ice Thickness:	1.50 in.
Concurrent Temperature:	5 F
Gust Speed	40 mph
Data Source:	Standard ASCE/SEI 7-16, Figs. 10-2 through 10-8
Date Accessed:	Mon May 27 2024

Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Values provided are equivalent radial ice thicknesses due to freezing rain with concurrent 3-second gust speeds, for a 500-year mean recurrence interval, and temperatures concurrent with ice thicknesses due to freezing rain. Thicknesses for ice accretions caused by other sources shall be obtained from local meteorological studies. Ice thicknesses in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Snow

Results:

Ground Snow Load, p_g :	20 lb/ft^2
Mapped Elevation:	1019.1 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Mon May 27 2024

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped elevation’ differ significantly from each other.

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Rain

Results:

15-minute Precipitation Intensity: 7.49 in./h

60-minute Precipitation Intensity: 3.52 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

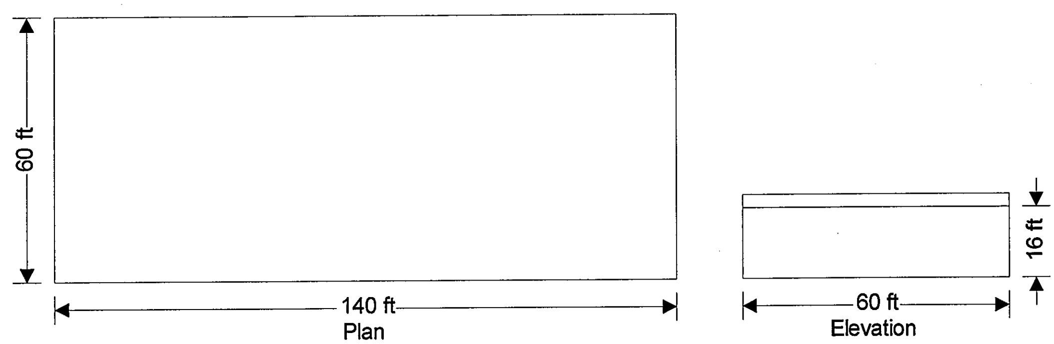
Date Accessed: Mon May 27 2024

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.09



Building data

Type of roof	Flat
Length of building	b = **140.00** ft
Width of building	d = **60.00** ft
Height to eaves	H = **16.00** ft
Height of parapet	h_p = **3.00** ft
Mean height	h = **16.00** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.4.7)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
15.00	0.85	24.46
16.00	0.86	24.75
19.00	0.89	25.61

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **24.75** psf

Parapet pressures and forces

Velocity pressure at top of parapet q_p = **25.61** psf

Combined net pressure coefficient, leeward GC_{pnl} = **-1.0**

Combined net parapet pressure, leeward $p_{pl} = q_p \times GC_{pnl}$ = **-25.61** psf

Combined net pressure coefficient, windward GC_{pnw} = **1.5**

Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw}$ = **38.42** psf

Wind direction 0 deg:

Leeward parapet force $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-10.8** kips

Windward parapet force $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **16.1** kips

Wind direction 90 deg:

Leeward parapet force $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-4.6** kips

Windward parapet force $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **6.9** kips

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A (-ve)	16.00	-0.90	24.75	-23.39	1120.00	-26.19
B (-ve)	16.00	-0.90	24.75	-23.39	1120.00	-26.19
C (-ve)	16.00	-0.50	24.75	-14.97	2240.00	-33.54
D (-ve)	16.00	-0.30	24.75	-10.77	3920.00	-42.20

Total vertical net force $F_{w,v}$ = **-128.13** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	24.46	12.18	2100.00	25.58
A_2	16.00	0.80	24.75	12.37	140.00	1.73
B	16.00	-0.50	24.75	-14.97	2240.00	-33.54
C	16.00	-0.70	24.75	-19.18	960.00	-18.41
D	16.00	-0.70	24.75	-19.18	960.00	-18.41

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times (H + h_p)$ = **2660.00** ft²

Projected vertical area of roof $A_{vert\_r\_0}$ = **0.00** ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **42.56** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_0}$ = **-44.3** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_0}$ = **43.4** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **87.7** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	16.00	-0.18	24.75	0.67	1120.00	0.75
B (+ve)	16.00	-0.18	24.75	0.67	1120.00	0.75
C (+ve)	16.00	-0.18	24.75	0.67	2240.00	1.50
D (+ve)	16.00	-0.18	24.75	0.67	3920.00	2.62

Total vertical net force $F_{w,v}$ = **5.61** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.46	21.09	2100.00	44.29
A_2	16.00	0.80	24.75	21.28	140.00	2.98
B	16.00	-0.50	24.75	-6.06	2240.00	-13.58
C	16.00	-0.70	24.75	-10.27	960.00	-9.86
D	16.00	-0.70	24.75	-10.27	960.00	-9.86

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times (H + h_p)$ = **2660.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **42.56** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_0}$ = **-24.3** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_0}$ = **63.4** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **87.7** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	16.00	-0.90	24.75	-23.39	480.00	-11.23
B (-ve)	16.00	-0.90	24.75	-23.39	480.00	-11.23
C (-ve)	16.00	-0.50	24.75	-14.97	960.00	-14.37
D (-ve)	16.00	-0.30	24.75	-10.77	6480.00	-69.76

Total vertical net force $F_{w,v}$ = **-106.59** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.46	12.18	900.00	10.96

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A_2	16.00	0.80	24.75	12.37	60.00	0.74
B	16.00	-0.28	24.75	-10.42	960.00	-10.00
C	16.00	-0.70	24.75	-19.18	2240.00	-42.96
D	16.00	-0.70	24.75	-19.18	2240.00	-42.96

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times (H + h_p) = \mathbf{1140.00}$ ft²

Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}$ ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{18.24}$ kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_90} = \mathbf{-14.6}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_90} = \mathbf{18.6}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{33.2}$ kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A (+ve)	16.00	-0.18	24.75	0.67	480.00	0.32
B (+ve)	16.00	-0.18	24.75	0.67	480.00	0.32
C (+ve)	16.00	-0.18	24.75	0.67	960.00	0.64
D (+ve)	16.00	-0.18	24.75	0.67	6480.00	4.33

Total vertical net force $F_{w,v} = \mathbf{5.61}$ kips

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	24.46	21.09	900.00	18.98
A_2	16.00	0.80	24.75	21.28	60.00	1.28
B	16.00	-0.28	24.75	-1.51	960.00	-1.45
C	16.00	-0.70	24.75	-10.27	2240.00	-23.01
D	16.00	-0.70	24.75	-10.27	2240.00	-23.01

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times (H + h_p) = \mathbf{1140.00}$ ft²

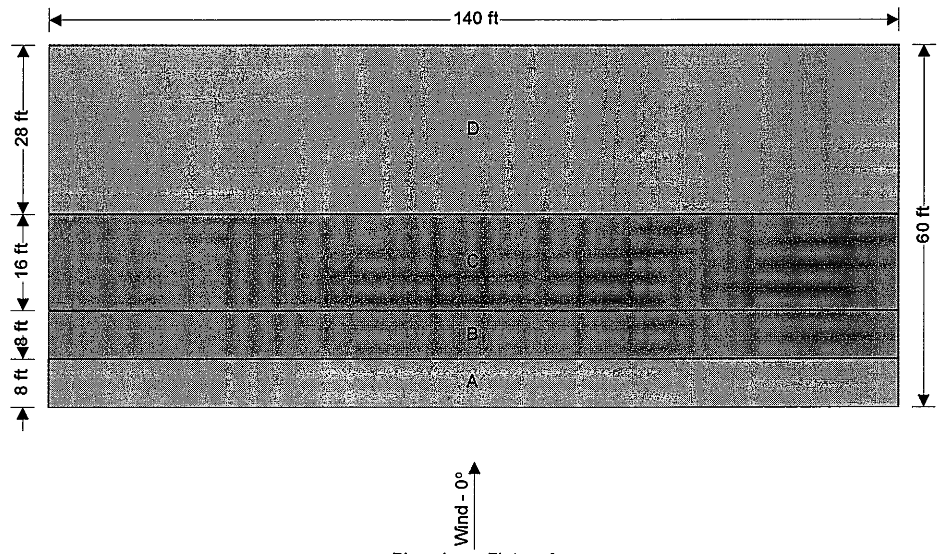
Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}$ ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{18.24}$ kips

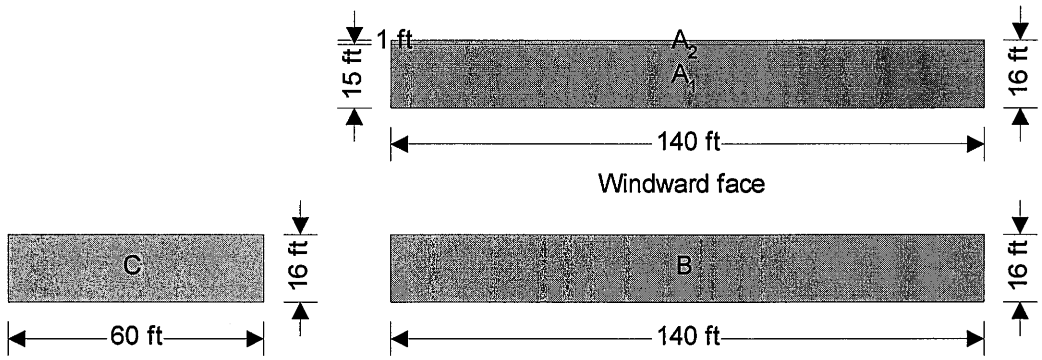
Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_90} = \mathbf{-6.1}$ kips

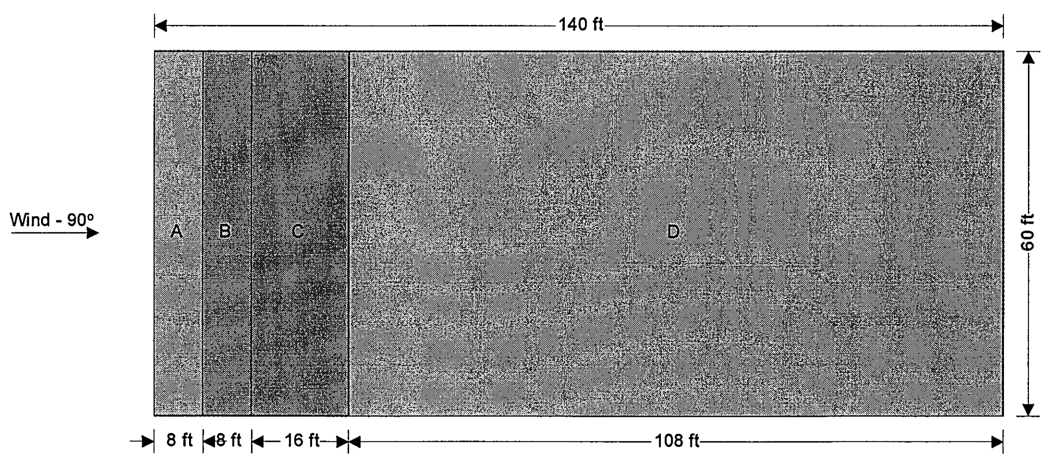
Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_90} = \mathbf{27.2}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{33.2}$ kips



Plan view - Flat roof





Plan view - Flat roof

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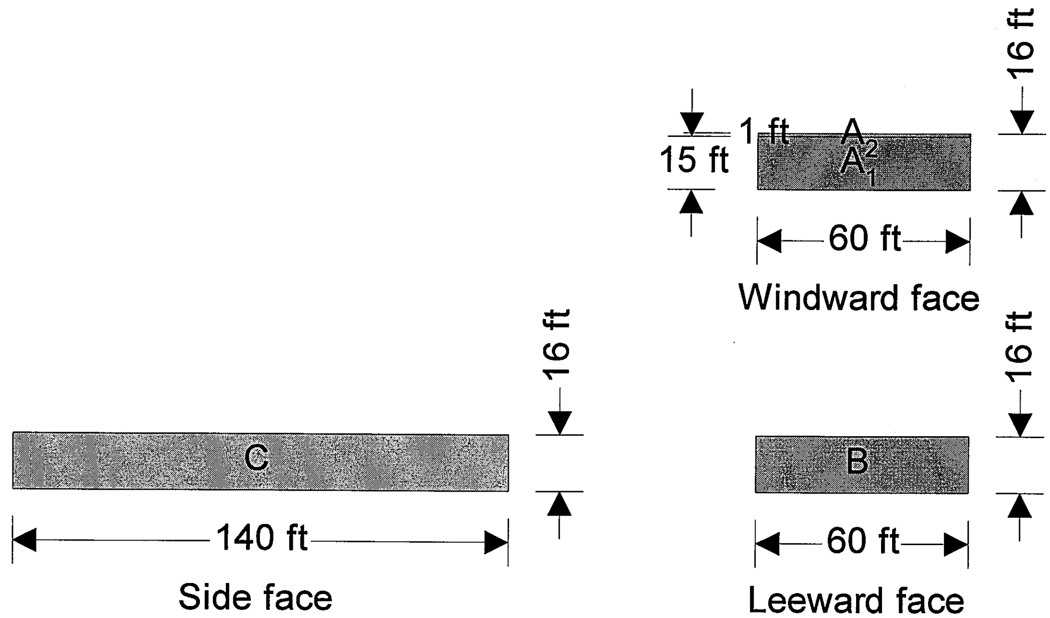

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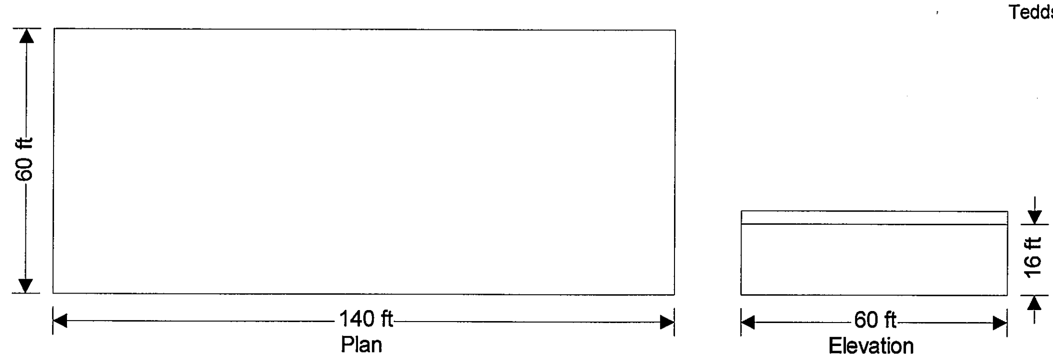
	Section				Sheet no./rev. 1	
	Calc. by O	Date 5/27/2024	Chk'd by	Date	App'd by	Date

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.09



Building data

Type of roof	Flat
Length of building	b = **140.00** ft
Width of building	d = **60.00** ft
Height to eaves	H = **16.00** ft
Height of parapet	h_p = **3.00** ft
Mean height	h = **16.00** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Parapet internal pressure coef +ve (Table 26.11-1)	GC_{pi_pp} = 0.0
Parapet internal pressure coef –ve (Table 26.11-1)	GC_{pi_np} = 0.0
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.86**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **24.7** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.89**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **25.6** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **24.75** psf

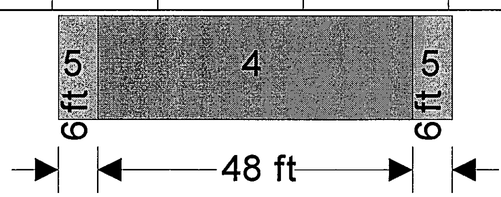
Equations used in tables

Net pressure $p = q_h \times [GC_p - GC_{pi}]$

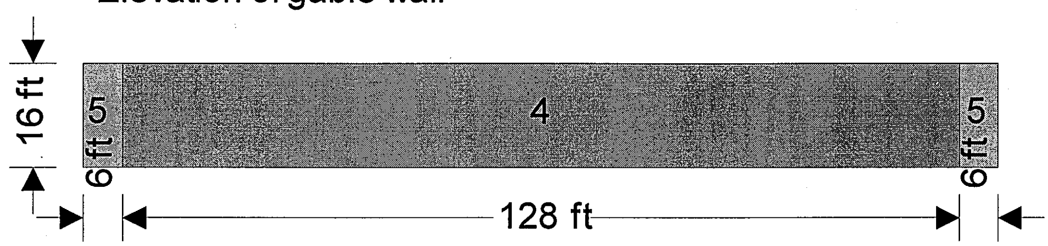
Parapet net pressure $p = q_p \times [GC_p - GC_{pi\_p}]$ 25.6 x (0.88) + 25.6(0.79)
22.52 20.22 = 42.75

Components and cladding pressures - Wall (Table 30.3-1 and (Figure 30.3-2A))

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GC$_p$	-GC$_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	0.90	-0.99	26.7	-29.0
50 sf	4	-	-	50.0	0.79	$\boxed{-0.88}$	24.0	-26.2
200 sf	4	-	-	200.0	0.69	-0.78	21.6	-23.8
>500 sf	4	-	-	500.1	0.63	-0.72	20.0	-22.3
<=10 sf	5	-	-	10.0	0.90	-1.26	26.7	-35.6
50 sf	5	-	-	50.0	$\boxed{0.79}$	-1.04	24.0	-30.1
200 sf	5	-	-	200.0	0.69	-0.85	21.6	-25.4
>500 sf	5	-	-	500.1	0.63	-0.72	20.0	-22.3



Elevation of gable wall



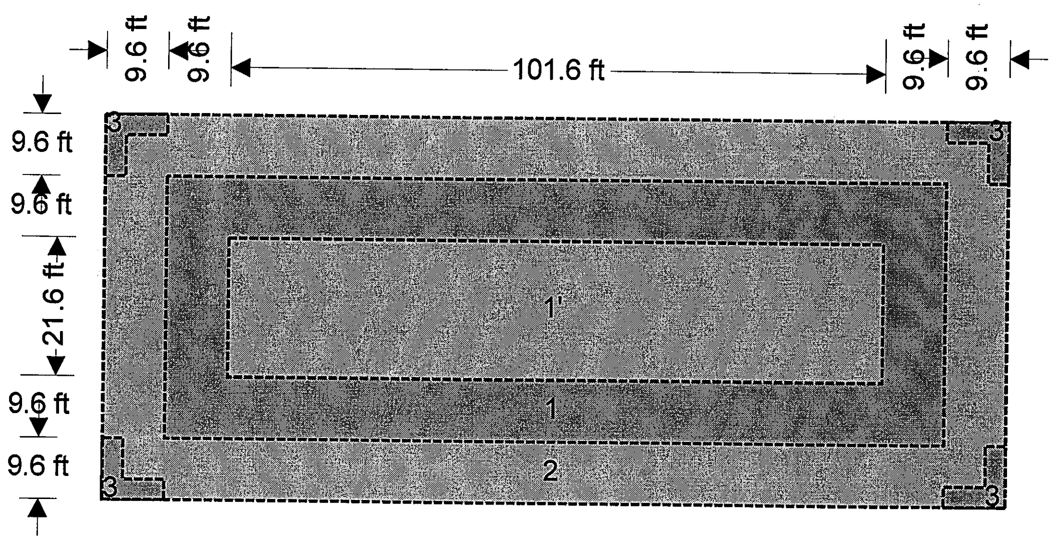
Elevation of side wall

Components and cladding pressures - Roof (Figure 30.3-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GC$_p$	-GC$_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.70	11.9 #	-46.5
100 sf	1	-	-	100.0	0.20	-1.29	9.4 #	-36.3
200 sf	1	-	-	200.0	0.20	-1.16	9.4 #	-33.3
>500 sf	1	-	-	500.1	0.20	-1.00	9.4 #	-29.2
<=10 sf	1'	-	-	10.0	0.30	-0.90	11.9 #	-26.7
100 sf	1'	-	-	100.0	0.20	-0.90	9.4 #	-26.7
500 sf	1'	-	-	500.0	0.20	-0.55	9.4 #	-18.1

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
>1000 sf	1'	-	-	1000.1	0.20	-0.40	9.4 #	-14.4 #
<=10 sf	2	-	-	10.0	0.90	-2.30	26.7	-61.4
100 sf	2	-	-	100.0	0.74	-1.77	22.8	-48.3
200 sf	2	-	-	200.0	0.69	-1.61	21.6	-44.3
>500 sf	2	-	-	500.1	0.63	-1.40	20.0	-39.1
<=10 sf	3	-	-	10.0	0.90	-2.30	26.7	-61.4
100 sf	3	-	-	100.0	0.74	-1.77	22.8	-48.3
200 sf	3	-	-	200.0	0.69	-1.61	21.6	-44.3
>500 sf	3	-	-	500.1	0.63	-1.40	20.0	-39.1

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



Plan on roof

Tekla Tedds	Project				Job Ref.	
Kreher Engineering, Inc	Section				Sheet no./rev. 1	
	Calc. by O	Date 5/27/2024	Chk'd by	Date	App'd by	Date

SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.03

Site parameters

Site class D, Soil properties not known

Mapped acceleration parameters (Section 11.4.2)

at short period S_S = **0.1**

at 1 sec period S_1 = **0.068**

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) $S_{MS} = F_a \times S_S = \mathbf{0.160}$

at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1 = \mathbf{0.163}$

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS} = \mathbf{0.107}$

at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1} = \mathbf{0.109}$

Seismic design category

Occupancy category (Table 1-1) II

Seismic design category based on short period response acceleration (Table 11.6-1)

A

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

B

Seismic design category B

Approximate fundamental period

Height above base to highest level of building h_n = **10** ft

From Table 12.8-2:

Structure type All other systems

Building period parameter C_t C_t = **0.02**

Building period parameter x x = **0.75**

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x = \mathbf{0.112}$ sec

Building fundamental period (Sect 12.8.2) $T = T_a = \mathbf{0.112}$ sec

Long-period transition period T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1) A. Bearing_Wall_Systems

9. Ordinary reinforced masonry shear walls

Response modification factor (Table 12.2-1) R = **2**

Seismic importance factor (Table 1.5-2) I_e = **1.000**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) $C_{s\_calc} = S_{DS} / (R / I_e) = \mathbf{0.0533}$

Maximum (Eq 12.8-3) $C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e)) = \mathbf{0.4837}$

Minimum (Eq.12.8-5)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) = \mathbf{0.0100}$
Seismic response coefficient	$C_s = \mathbf{0.0533}$

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure	$W = \mathbf{1000.0}$ kips
Seismic response coefficient	$C_s = \mathbf{0.0533}$
Seismic base shear (Eq 12.8-1)	$V = C_s \times W = \mathbf{53.3}$ kips

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.10

Building details

Roof type	Flat
Width of roof	$b = \mathbf{138.00}$ ft

Ground snow load

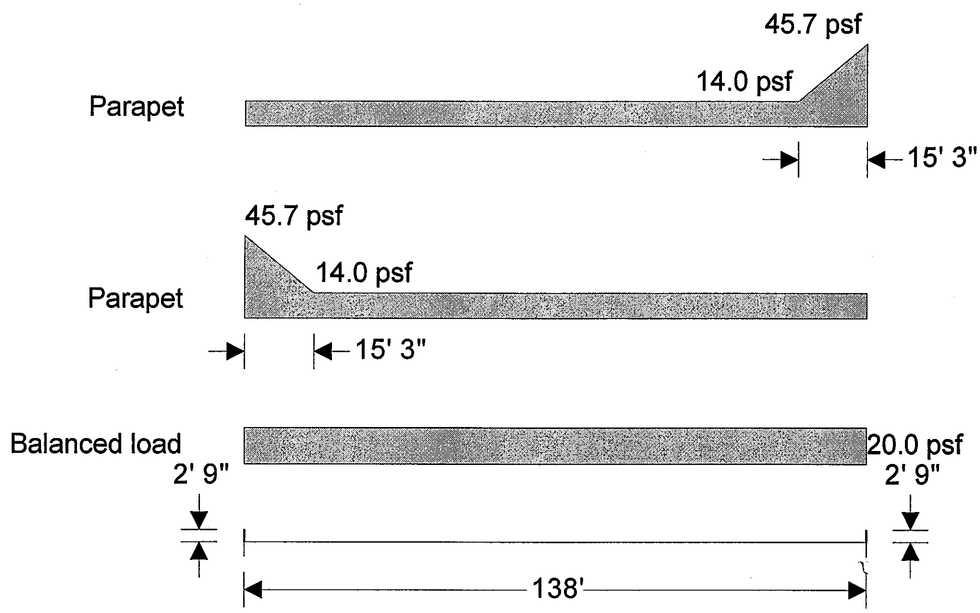
Ground snow load (Figure 7.2-1)	$p_g = \mathbf{20.00}$ lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) = \mathbf{16.60}$ lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	$C_e = \mathbf{1.00}$
Thermal condition (Table 7.3-2)	All
Thermal factor (Table 7.3-2)	$C_t = \mathbf{1.00}$
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	$I_s = \mathbf{1.00}$
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g = \mathbf{20.00}$ lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = \mathbf{14.00}$ lb/ft^2

Left parapet

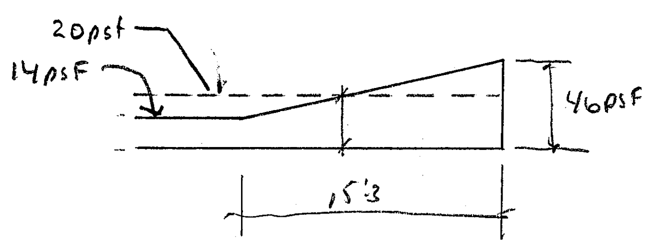
Balanced snow load height	$h_b = p_f / \gamma = \mathbf{0.84}$ ft
Height of left parapet	$h_{pptL} = \mathbf{2.75}$ ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b = \mathbf{1.91}$ ft
Length of roof - left parapet	$l_{u\_pptL} = b = \mathbf{138.00}$ ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) = \mathbf{2.78}$ ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b) = \mathbf{1.91}$ ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}^2 / h_{c\_pptL}, 8 \times (h_{pptL} - h_b), b) = \mathbf{15.25}$ ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma = \mathbf{31.65}$ lb/ft^2

Right parapet

Height of right parapet	$h_{pptR} = \mathbf{2.75}$ ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b = \mathbf{1.91}$ ft
Length of roof - right parapet	$l_{u\_pptR} = b = \mathbf{138.00}$ ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) = \mathbf{2.78}$ ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b) = \mathbf{1.91}$ ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}^2 / h_{c\_pptR}, 8 \times (h_{pptR} - h_b), b) = \mathbf{15.25}$ ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma = \mathbf{31.65}$ lb/ft^2



Roof elevation



Slope $\frac{32}{15.25} = \frac{26}{x}$ ∴ $x = 12.39$ ft

Use 12'6"

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.10

Building details

Roof type	Flat
Width of roof	b = **60.00** ft

Ground snow load

Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	All
Thermal factor (Table 7.3-2)	C_t = **1.00**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** 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** lb/ft^2

Left parapet

Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height of left parapet	h_{pptL} = **2.75** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **1.91** ft
Length of roof - left parapet	$l_{u\_pptL} = b$ = **60.00** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.83** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.83** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **7.32** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **30.37** lb/ft^2

Right parapet

Height of right parapet	h_{pptR} = **2.75** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **1.91** ft
Length of roof - right parapet	$l_{u\_pptR} = b$ = **60.00** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.83** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.83** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **7.32** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **30.37** lb/ft^2

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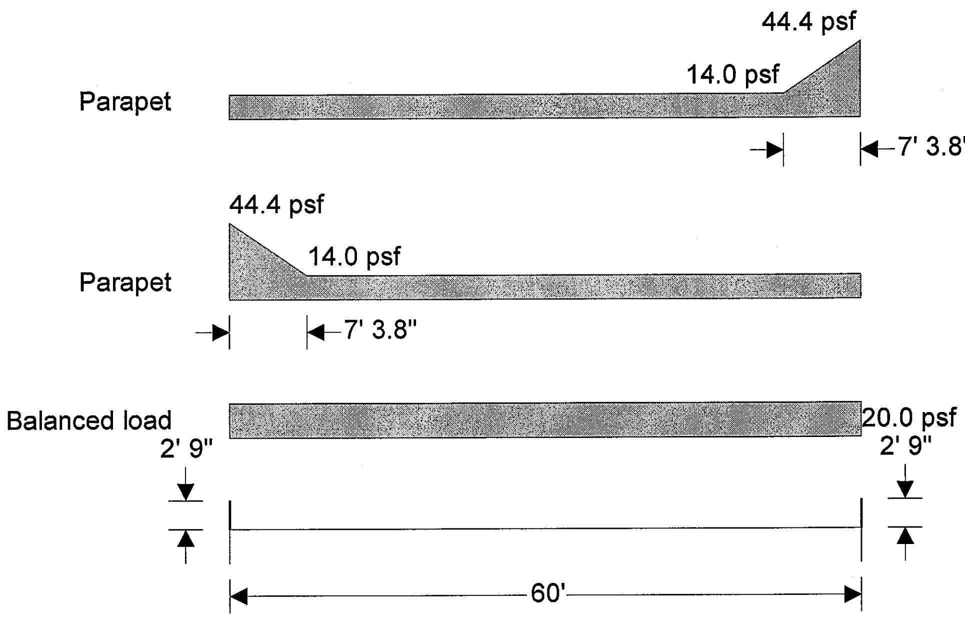
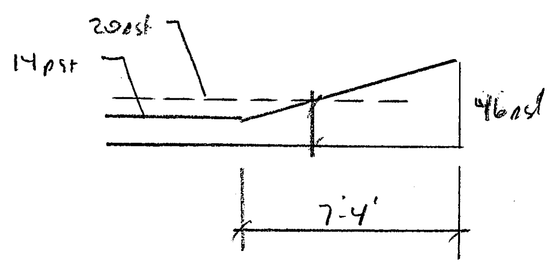


Roof elevation



$\frac{32}{7.4}$ $\frac{26}{x}$ → x = 5.96 ft

Us 6'0"

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JOB
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CALCULATED BY DATE
CHECKED BY DATE
SCALE

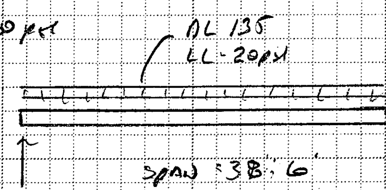
Roof Loading:

DL - Rfg/Insul + 6psf
Topping 3" 38psf
Ceil 3p 3psf
Mech - 5psf
Misc 3psf
55psf

Plank wt: 80psf

→ Plank Design:

WLL : 20psf



DL 2590 #/l
LL = 385 #/l
DL 1400 #/l
LL 210 #/l

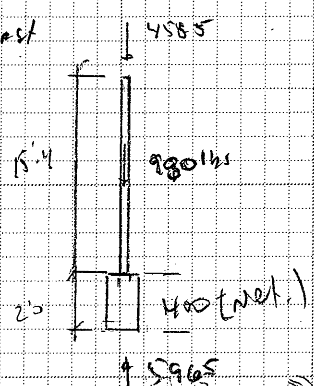
Span = 38'-6"
Span 20'-9"

Interior Wall:
10" CMU

Wall Load: DL = 3990 #/ft
LL = 595 #/ft

Wind Load - 10psf (ASD)
Seismic 0.4(SDS)(W)(E) SDS 0.107

Wall wt - 64psf



Rein. #5 @ 32"

⇒ Width $\frac{5965}{1500}$ 3.97 ft - use 4'-0

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Exterior Wall   Wall wt

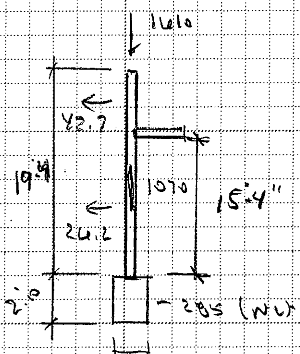
Short Span NORTH

Wall DL - 1460 #/ft
Wall LL 210 #/ft

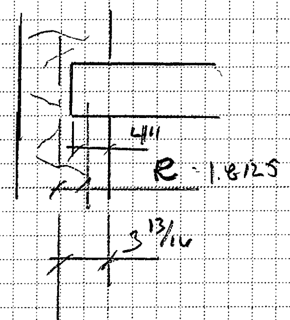
$A_i - (15.33)^2/3 = 78\ ft^2$
Use 50 ft^2

Wind - 26.2 psf
Seismic (0.4 SDS)(W)
Wall per 2 psf

UL 45° e32



$w = \frac{2965}{1500} = 1.98\ ft < 4'2"$

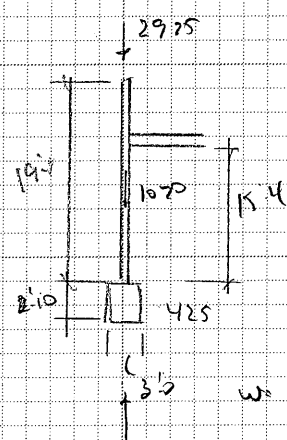


Parapet Wind: P_3 = 20.22
P_4 22.52
P_6 26.2
42.7 psf Parapet

Exterior Wall SOUTH

Long Span

Wall DL 2540
Wall LL - 385



Wind - 26.2 psf
Seis 0.4(SDS)(W)(I)
Wall per 2 psf

$w = \frac{4470}{1500} = 2.98\ ft$ → Use 3'0"

MASONRY WALL PANEL DESIGN (TMS 402/602-16)

INTERIOR BRG WALL

In accordance with strength design method

Tedds calculation version 2.2.08

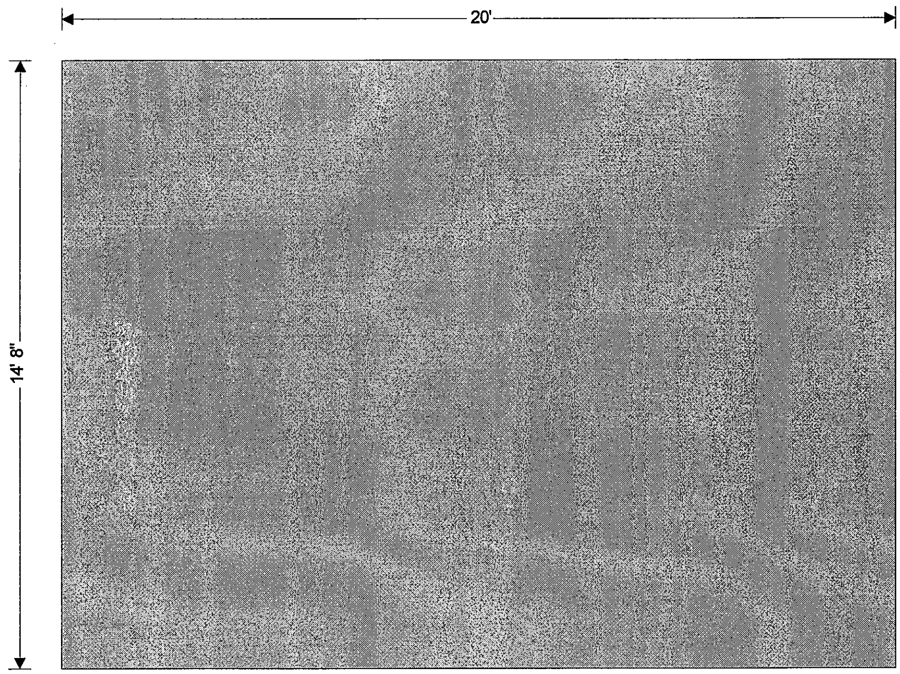
Masonry wall panel details

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 = **20** ft

Panel height — h = **14.667** ft



Seismic properties

Seismic design category — B

Seismic importance factor (ASCE7 Table 1.5-2) — I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **0.107**

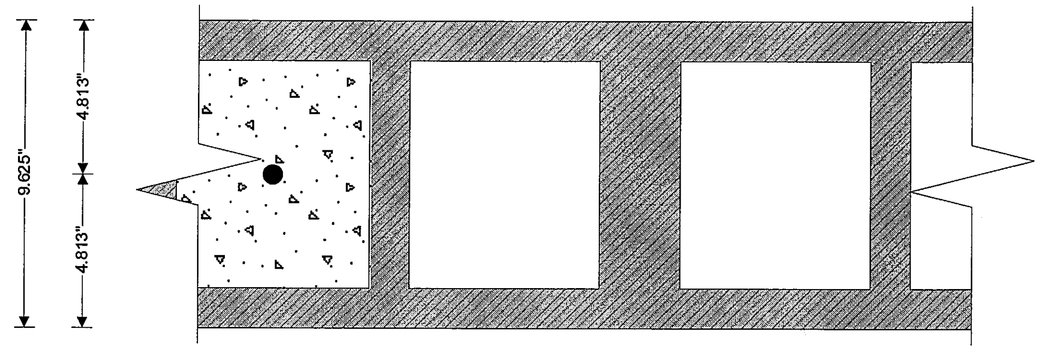
Seismic wall classification — Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load — ρ_E = **1.0**

Construction details

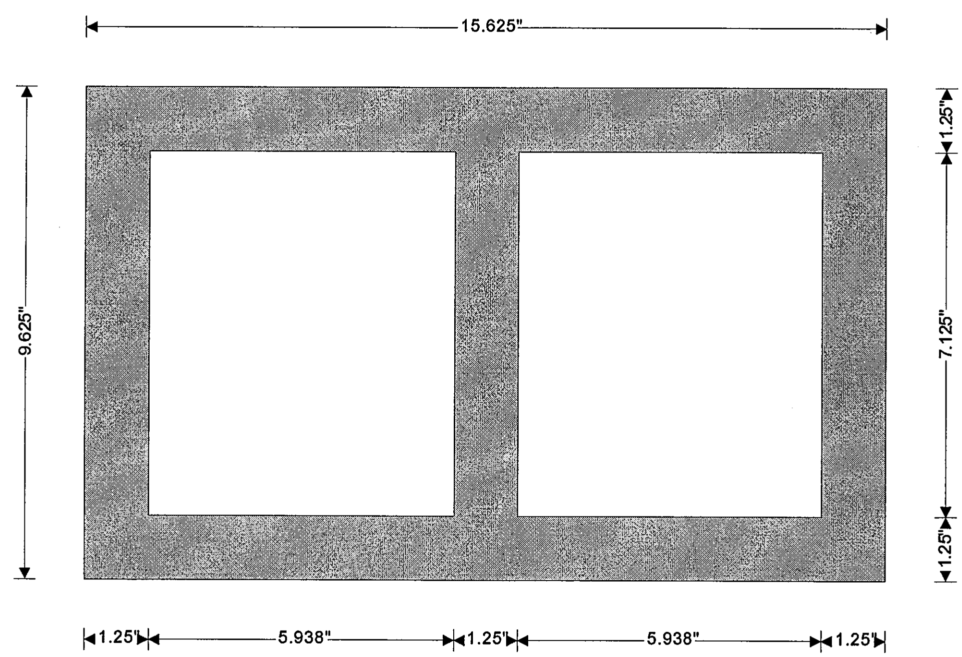
Wall thickness — t = **9.625** in



Masonry details

Hollow concrete units grouted at 32 in on center in running bond fully bedded with PCL class S mortar

Compressive strength of unit	f'_{cu} = **3250** psi
Density of masonry units	γ_{block} = **135** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **50.52** in^2/ft
Area of grout	$A_{grout} = [0.25 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **16.25** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **63.16** psf



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

Net compressive strength of masonry f'_m = **2500** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **2250000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **900000** psi

From TMS 402 -16 Table 9.1.9.2 - Modulus of rupture

Modulus of rupture normal to bed f_{r_norm} = **104** psi

Modulus of rupture parallel to bed f_{r_para} = **167** psi

Reinforcement details

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

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 32 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.12** in²/ft

Yield strength of horizontal reinforcement f_{yv} = **70000** psi

Allowable tensile stress in horizontal reinforcement F_{sv} = **30000** psi

Horizontal reinforcement provided (2) W1.7 wires at 16 in centers

Area of horizontal reinforcement $A_v = 2 \times \pi \times HDia^2 / (4 \times s_v)$ = **0.03** in²/ft

Minimum area of vertical reinf. (cl. 9.3.6.2) $A_{s\_min} = A_v / 3$ = **0.01** in²/ft

PASS - Area of vertical reinforcement provided exceeds the minimum

Lateral out-of-plane loads

Wind load on panel W = **10** psf

Wind load on parapet W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.043**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **6.3** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **6.3** psf

Lateral in-plane loads

Vertical loading details

Dead load at supported level DL = **3990** lb/ft

Live roof load at supported level LL_r = **595** lb/ft at an eccentricity of 2.667 in

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.021**

From ASCE 7-16 cl.2.3 - Combining factored loads using strength design (Utilization)

Load combination no.1 $1.4 \times DL$ (0.085)

Load combination no.2 $1.2 \times DL + 1.6 \times LL + 0.5 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.076)

Load combination no.3 $1.2 \times DL + LL + 1.6 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.084)

Load combination no.4 $1.2 \times DL + 1.6 \times (LL_r \text{ or } SL \text{ or } RL) + 0.5 \times W$ (0.084)

Load combination no.5 $1.2 \times DL + W + LL + 0.5 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.076)

Load combination no.6 $1.2 \times DL + LL + 0.2 \times SL + E_h + E_v$ (0.074)

Load combination no.7 $0.9 \times DL + W$ (0.070)

Load combination no.8 $0.9 \times DL + E_h - E_v$ (0.053)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.75 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **66.8** in²/ft

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Properties for walls loaded out-of-plane:

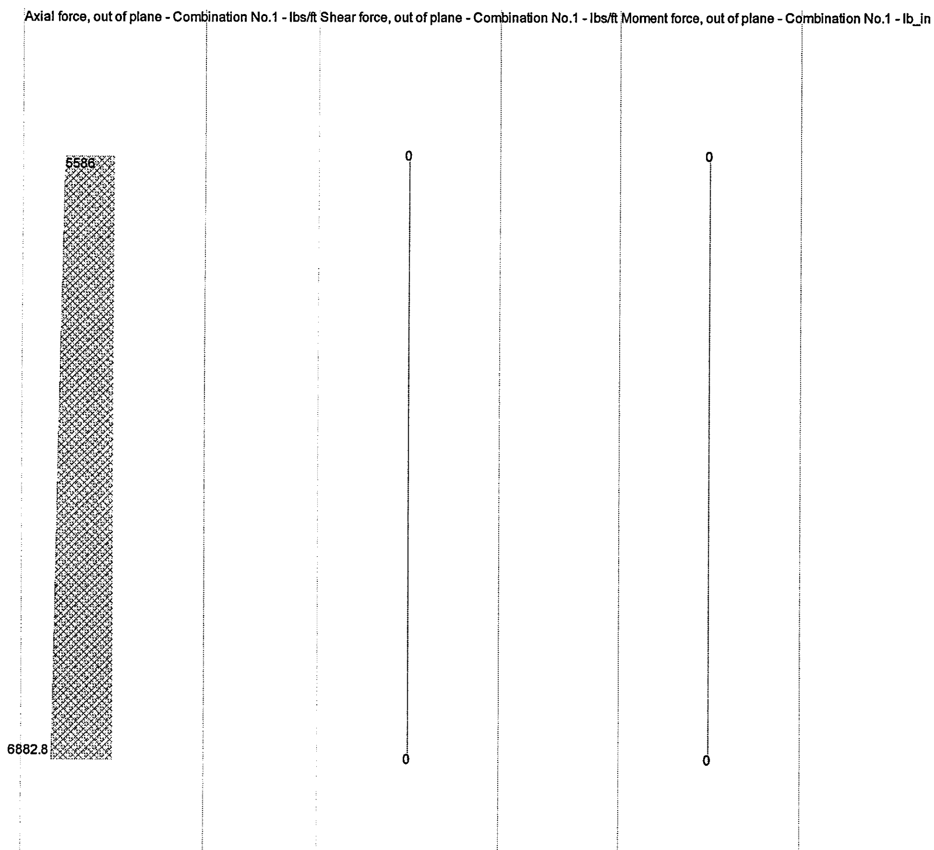
Moment of inertia $I = t^3 / 12 - 0.75 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **685.5** in^4/ft

Section modulus $S = I / c$ = **142.4** in^3/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **3.204** in

Effective height factor K = **1**

Consider wall at bottom under load combination no.1



Axial load at bottom of panel P = **6883** lb/ft

Slenderness ratio $(K \times h) / r$ = **54.927** < 99

Nominal axial strength $P_n = 0.8 \times (0.8 \times f'_m \times (A - A_s) + A_s \times 0\ ksi) \times [1 - ((K \times h) / (140 \times r))^2]$ = **90225** lb/ft

Strength reduction factor ϕ = **0.9**

Design axial strength $\phi \times P_n$ = **81203** lb/ft

$P / (\phi \times P_n)$ = **0.085**

PASS - Nominal axial strength exceeds axial load

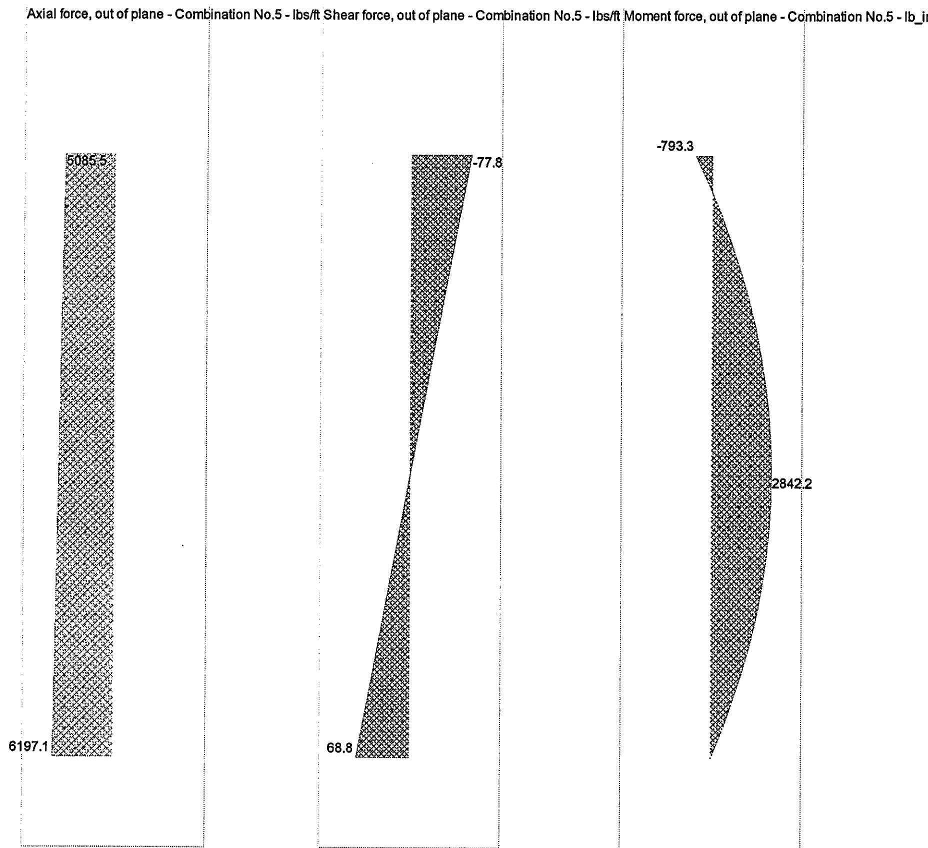
Nominal cracking moment strength $M_{cr} = S \times f_{r\_norm}$ = **14778** lb_in/ft

Modular ratio $n = E_s / E_m$ = **12.889**

Distance to neutral axis $c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m)$ = **0.718** in

Moment of inertia of cracked section $I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3$ = **51.1** in^4/ft

Consider wall at bottom under load combination no.5



Shear force	V = **69** lb/ft
Compressive force	N_u = **5900** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **14.099** in^2/ft
Nominal shear strength	$V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_u, 6 \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **4230** lb/ft
Strength reduction factor	ϕ_v = **0.8**
Design shear strength	$\phi_v \times V_n$ = **3384** lb/ft
	$V / (\phi_v \times V_n)$ = **0.020**

PASS - Design shear strength exceeds applied shear strength

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MASONRY WALL PANEL DESIGN (TMS 402/602-16)

NORTH BRG WALL

In accordance with strength design method

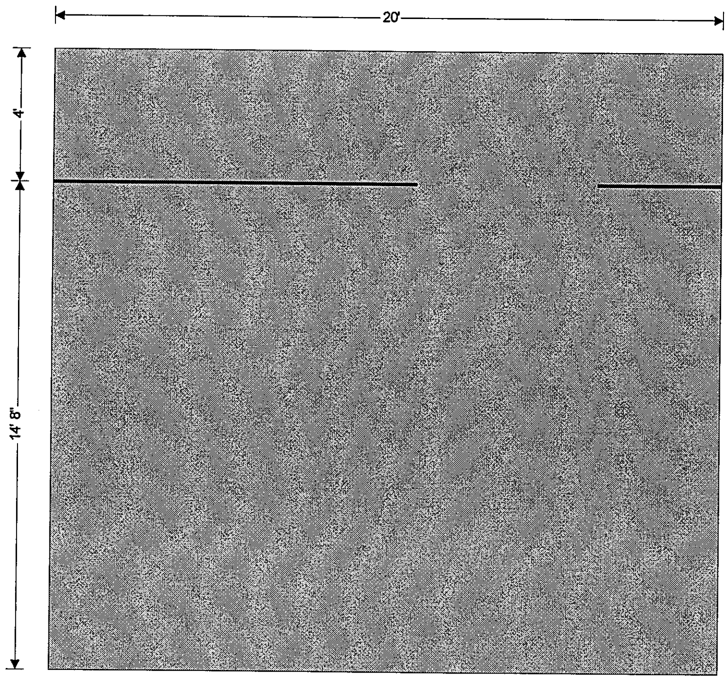
Tedds calculation version 2.2.08

Masonry wall panel details

Reinforced single-wythe wall with a parapet, 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 = **20** ft
Panel height	h = **14.667** ft
Parapet height	h_p = **4** ft



Seismic properties

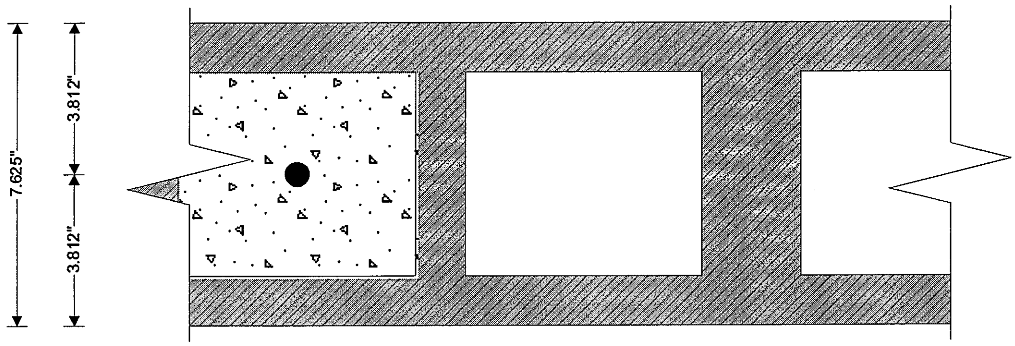
Seismic design category	B
Seismic importance factor (ASCE7 Table 1.5-2)	I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

	S_{DS} = **0.107**
Seismic wall classification	Nonparticipating
	No prescriptive minimum seismic reinforcement
Redundancy factor, on out-of-plane load	ρ_E = **1.0**

Construction details

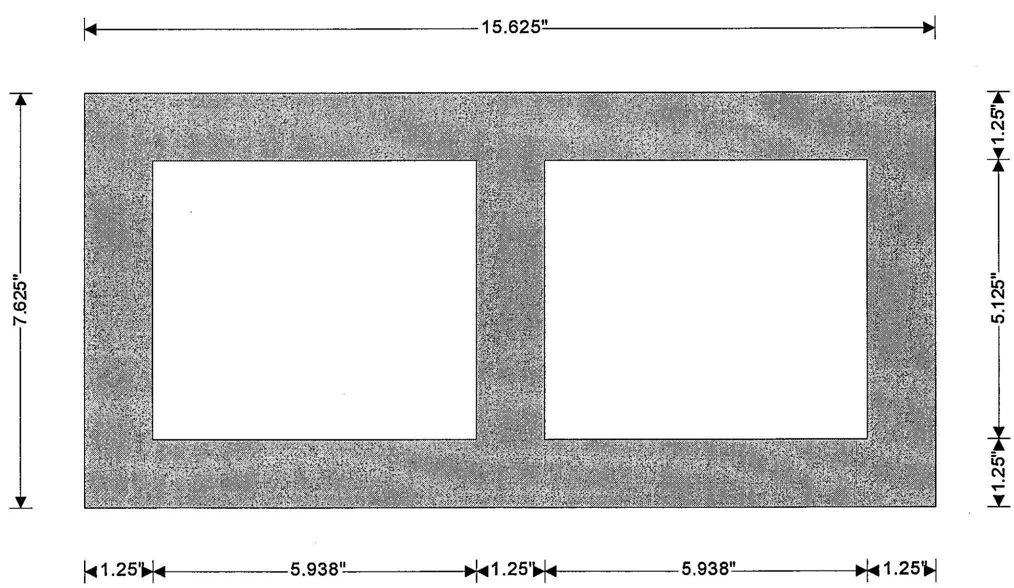
Wall thickness	t = **7.625** in



Masonry details

Hollow concrete units grouted at 32 in on center in running bond fully bedded with PCL class S mortar

Compressive strength of unit	f'_{cu} = **3250** psi
Density of masonry units	γ_{block} = **135** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** 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** in^2/ft
Area of grout	$A_{grout} = [0.25 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **11.69** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **53.32** psf



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

Net compressive strength of masonry	f'_m = **2500** psi
Modulus of elasticity for masonry	$E_m = 900 \times f'_m$ = **2250000** psi
Shear modulus of masonry	$G_v = 0.4 \times E_m$ = **900000** psi

From TMS 402 -16 Table 9.1.9.2 - Modulus of rupture

Modulus of rupture normal to bed	f_{r_norm} = **104** psi
Modulus of rupture parallel to bed	f_{r_para} = **167** psi

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Allowable tensile stress in reinforcement	F_s = **32000** psi
Modulus of elasticity for reinforcement	E_s = **29000000** psi
Vertical reinforcement provided	No.5 bars at 32 in centers
Area of vertical reinforcement	$A_s = \pi \times Dia^2 / (4 \times s)$ = **0.12** in^2/ft
Yield strength of horizontal reinforcement	f_{yv} = **70000** psi
Allowable tensile stress in horizontal reinforcement	F_{sv} = **30000** psi
Horizontal reinforcement provided	(2) W1.7 wires at 16 in centers
Area of horizontal reinforcement	$A_v = 2 \times \pi \times HDia^2 / (4 \times s_v)$ = **0.03** in^2/ft
Minimum area of vertical reinf. (cl. 9.3.6.2)	$A_{s\_min} = A_v / 3$ = **0.01** in^2/ft

PASS - Area of vertical reinforcement provided exceeds the minimum

Lateral out-of-plane loads

Wind load on panel	W = **26.2** psf
Wind load on parapet	W_p = **42.7** psf
Seismic load factor (ASCE7 12.11.1)	$F_p = 0.4 \times S_{DS} \times I_e$ = **0.043**
Seismic load from wall	$E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **5.3** psf
Additional seimic load	E_{add} = **0.1** psf
Seismic lateral load on panel	$E = E_{wall} + E_{add}$ = **5.4** psf

Lateral in-plane loads

Vertical loading details

Dead load at supported level	DL = **1400** lb/ft
Live roof load at supported level	LL_r = **210** lb/ft at an eccentricity of 1.813 in
Vertical seismic load factor applied to dead load	$F_{Ev} = 0.2 \times S_{DS}$ = **0.021**

From ASCE 7-16 cl.2.3 - Combining factored loads using strength design (Utilization)

Load combination no.1	$1.4 \times DL$ (0.055)
Load combination no.2	$1.2 \times DL + 1.6 \times LL + 0.5 \times (LL_r$ or SL or RL) (0.049)
Load combination no.3	$1.2 \times DL + LL + 1.6 \times (LL_r$ or SL or RL) (0.052)
Load combination no.4	$1.2 \times DL + 1.6 \times (LL_r$ or SL or RL) $+ 0.5 \times W$ (0.093)
Load combination no.5	$1.2 \times DL + W + LL + 0.5 \times (LL_r$ or SL or RL) (0.206)
Load combination no.6	$1.2 \times DL + LL + 0.2 \times SL + E_h + E_v$ (0.048)
Load combination no.7	$0.9 \times DL + W$ (0.226)
Load combination no.8	$0.9 \times DL + E_h - E_v$ (0.052)

Properties of masonry section

Cross-sectional area	$A = [t \times l_b - 0.75 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **56.4** in^2/ft

Properties for walls loaded out-of-plane:

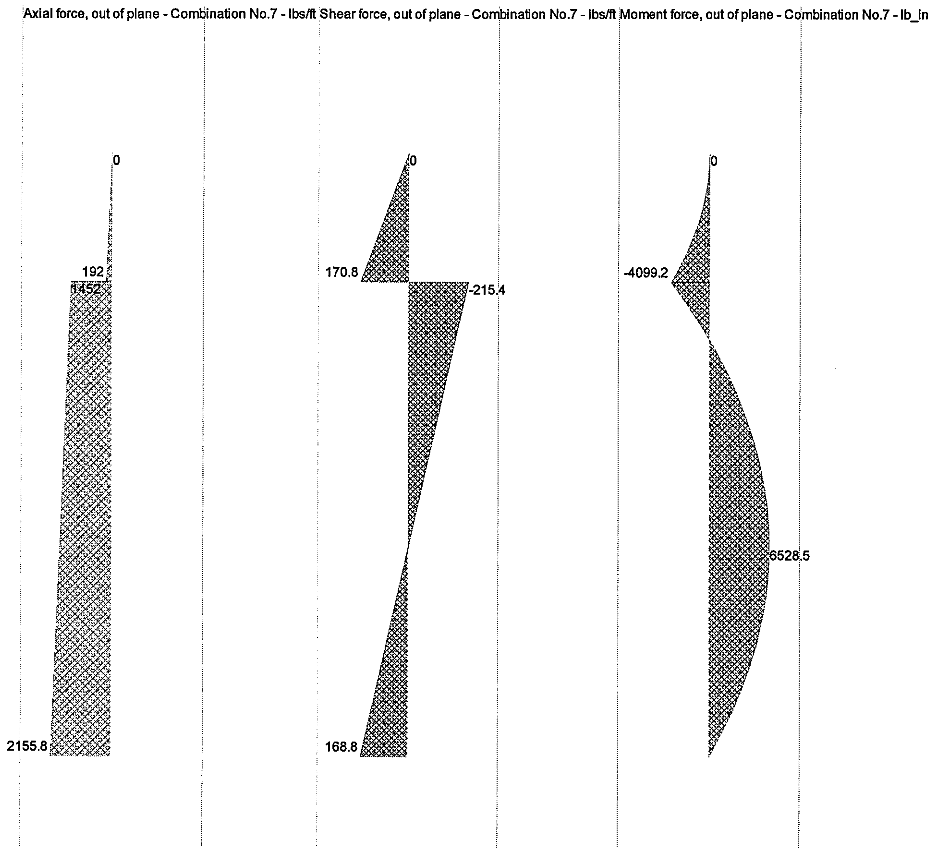
Moment of inertia: $I = t^3 / 12 - 0.75 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) = \mathbf{366.6}$ in^4/ft

Section modulus: $S = I / c = \mathbf{96.2}$ in^3/ft

Radius of gyration: $r = \sqrt{[I / A]} = \mathbf{2.548}$ in

Effective height factor: $K = \mathbf{1}$

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



Maximum moment location: 6.44 ft

Axial load at mid-height of panel: $P = \mathbf{1847}$ lb/ft

Slenderness ratio: $(K \times h) / r = \mathbf{69.061}$ < 99

Nominal axial strength: $P_n = 0.8 \times (0.8 \times f'_m \times (A - A_s) + A_s \times 0 \text{ ksi}) \times [1 - ((K \times h) / (140 \times r))^2] = \mathbf{68196}$ lb/ft

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

Design axial strength: $\phi \times P_n = \mathbf{61377}$ lb/ft

$P / (\phi \times P_n) = \mathbf{0.030}$

PASS - Nominal axial strength exceeds axial load

Factored axial stress: $P / t = \mathbf{20}$ psi

Factored axial stress limit: $0.2 \times f'_m = \mathbf{500}$ psi

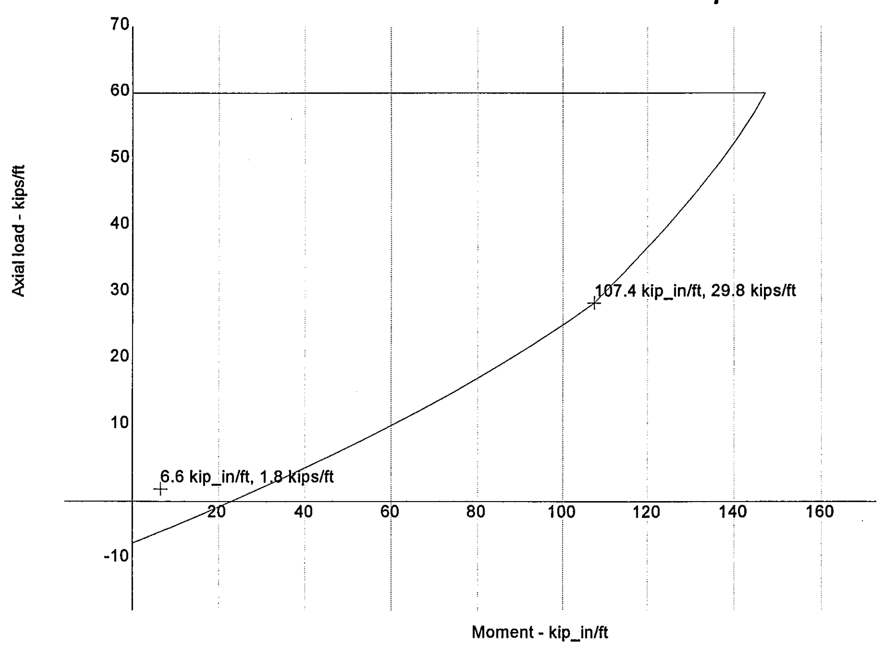
PASS - Allowable stress under out of plane loads exceeds factored axial stress

Nominal cracking moment strength	$M_{cr} = S \times f_{r\_norm} = \mathbf{9976}$ lb_in/ft
Modular ratio	$n = E_s / E_m = \mathbf{12.889}$
Distance to neutral axis	$c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m) = \mathbf{0.456}$ in
Moment of inertia of cracked section	$I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3 = \mathbf{21.6}$ in^4/ft
By iteration	$M_{u0} = M = \mathbf{6528}$ lb_in/ft
	$\delta_{u0} = 5 \times M_{u0} \times h^2 / (48 \times E_m \times I) = \mathbf{0.026}$ in
	$M_{u1} = M_{u0} + P \times \delta_{u0} = \mathbf{6576}$ lb_in/ft
	$\delta_{u1} = 5 \times M_{u1} \times h^2 / (48 \times E_m \times I) = \mathbf{0.026}$ in
Bending moment at mid-height of panel	$M = M_{u0} + P \times \delta_{u1} = \mathbf{6576}$ lb_in/ft
Depth of reinforcement	$d = \mathbf{3.813}$ in
Effective width per bar	$b_{eff} = \min(s, 6 \times t_{nom}, 72\text{ in}) = \mathbf{32}$ in
Strength reduction factor	$\phi = \mathbf{0.9}$
Tensile strain in reinforcement	$\varepsilon_s = f_y / E_s = \mathbf{0.0021}$
Maximum usable compressive strain of masonry	$\varepsilon_{mu} = \mathbf{0.0025}$
Fiber of max.compressive strain to neutral axis	$c_{bal} = \varepsilon_{mu} \times d / (\varepsilon_{mu} + \varepsilon_s) = \mathbf{2.086}$ in
Tensile force at balance point	$T_{bal} = A_s \times f_y = \mathbf{6903}$ lb/ft
	$\beta_1 = \mathbf{0.8}$
Compressive force at balance point	$C_{bal} = 0.8 \times f'_m \times \beta_1 \times c_{bal} = \mathbf{40053}$ lb/ft
Design axial force at balance point	$P_{bal} = \phi \times (C_{bal} - T_{bal}) = \mathbf{29835}$ lb/ft
Design moment at balance point	$M_{bal} = \phi \times [T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - \beta_1 \times c_{bal} / 2)] = \mathbf{107352}$ lb_in/ft
Maximum design moment from integration diagram	$M_c = \mathbf{29160}$ lb_in/ft
	$M / M_c = \mathbf{0.226}$

PASS - Combination of applied axial load and flexure is acceptable

Maximum area of tensile reinforcement (9.3.3.2)	$A_{s\_max} = 0.64 \times f'_m \times [\varepsilon_{mu} / (\varepsilon_{mu} + 1.5 \times \varepsilon_s)] \times d / f_y = \mathbf{0.544}$ in^2/ft

PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram

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Consider wall at bottom under load combination no.7

Shear force $V = \mathbf{169}$ lb/ft

Compressive force $N_u = \mathbf{2156}$ lb/ft

Net shear area $A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout}) = \mathbf{11.169}$ in^2/ft

Nominal shear strength $V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times N_u, 6 \times A_{nv} \times \sqrt{(f'_m \times 1\text{ psi})}) = \mathbf{2773}$ lb/ft

Strength reduction factor $\phi_v = \mathbf{0.8}$

Design shear strength $\phi_v \times V_n = \mathbf{2218}$ lb/ft

$V / (\phi_v \times V_n) = \mathbf{0.076}$

PASS - Design shear strength exceeds applied shear strength

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South Blk wall

MASONRY WALL PANEL DESIGN (TMS 402/602-16)

In accordance with strength design method

Tedds calculation version 2.2.08

Masonry wall panel details

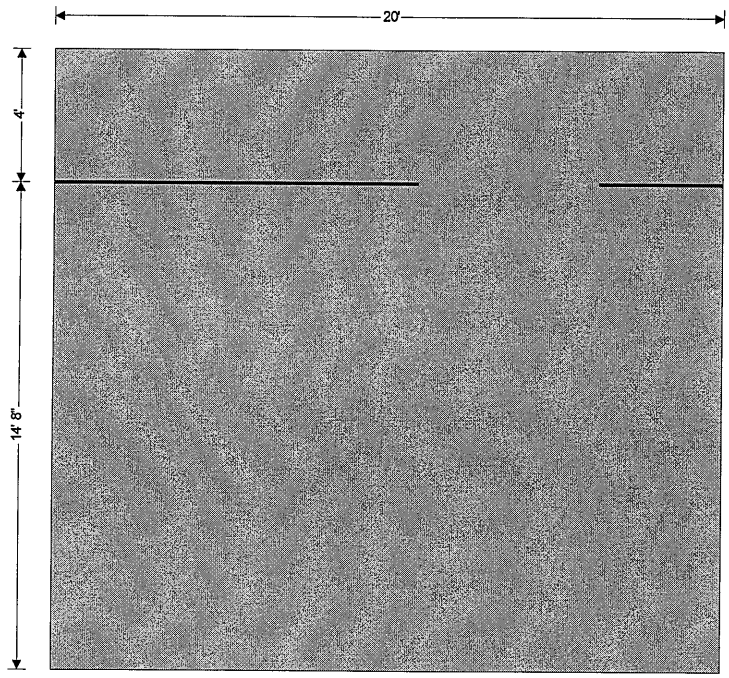
Reinforced single-wythe wall with a parapet, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **20** ft

Panel height h = **14.667** ft

Parapet height h_p = **4** ft



Seismic properties

Seismic design category B

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **0.107**

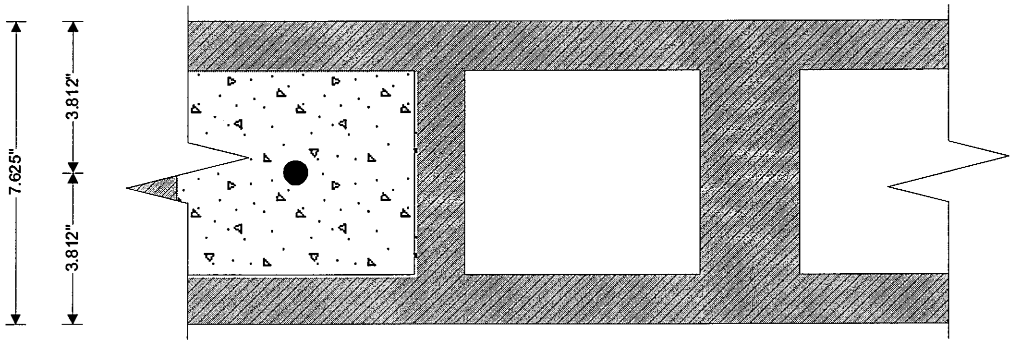
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

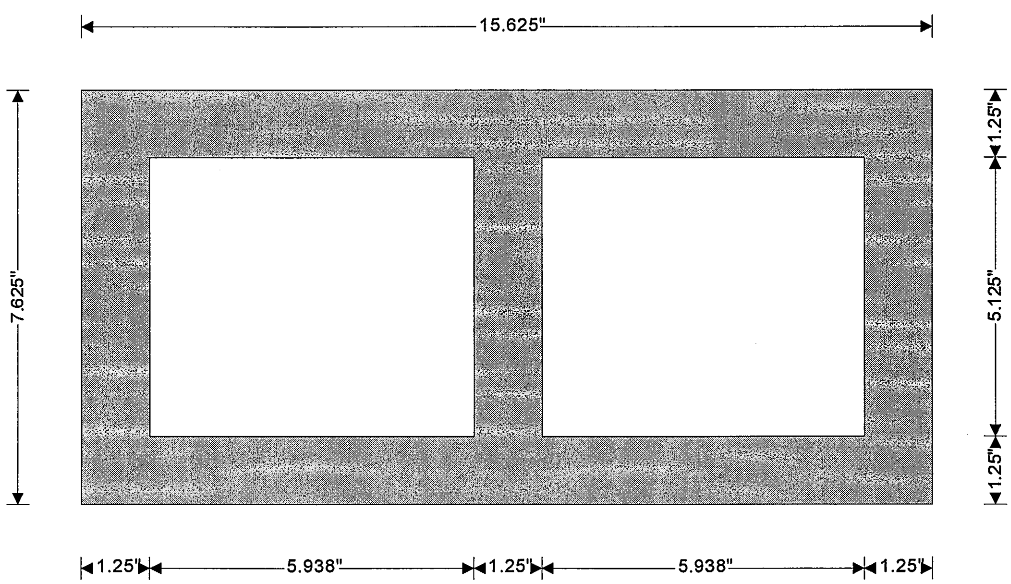
Wall thickness t = **7.625** in



Masonry details

Hollow concrete units grouted at 32 in on center in running bond fully bedded with PCL class S mortar

Compressive strength of unit	f'_{cu} = **3250** psi
Density of masonry units	γ_{block} = **135** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** 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** in^2/ft
Area of grout	$A_{grout} = [0.25 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **11.69** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **53.32** psf



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

Net compressive strength of masonry f'_m = **2500** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **2250000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **900000** psi

From TMS 402 -16 Table 9.1.9.2 - Modulus of rupture

Modulus of rupture normal to bed f_{r_norm} = **104** psi

Modulus of rupture parallel to bed f_{r_para} = **167** psi

Reinforcement details

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

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 32 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.12** in²/ft

Yield strength of horizontal reinforcement f_{yv} = **70000** psi

Allowable tensile stress in horizontal reinforcement F_{sv} = **30000** psi

Horizontal reinforcement provided (2) W1.7 wires at 16 in centers

Area of horizontal reinforcement $A_v = 2 \times \pi \times HDia^2 / (4 \times s_v)$ = **0.03** in²/ft

Minimum area of vertical reinf. (cl. 9.3.6.2) $A_{s\_min} = A_v / 3$ = **0.01** in²/ft

PASS - Area of vertical reinforcement provided exceeds the minimum

Lateral out-of-plane loads

Wind load on panel W = **26.2** psf

Wind load on parapet W_p = **42.7** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.043**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **5.3** psf

Additional seimic load E_{add} = **0.1** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **5.4** psf

Lateral in-plane loads

Vertical loading details

Dead load at supported level DL = **2590** lb/ft

Live roof load at supported level LL_r = **385** lb/ft at an eccentricity of 1.813 in

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.021**

From ASCE 7-16 cl.2.3 - Combining factored loads using strength design (Utilization)

Load combination no.1 $1.4 \times DL$ (0.082)

Load combination no.2 $1.2 \times DL + 1.6 \times LL + 0.5 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.073)

Load combination no.3 $1.2 \times DL + LL + 1.6 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.080)

Load combination no.4 $1.2 \times DL + 1.6 \times (LL_r \text{ or } SL \text{ or } RL) + 0.5 \times W$ (0.087)

Load combination no.5 $1.2 \times DL + W + LL + 0.5 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.176)

Load combination no.6 $1.2 \times DL + LL + 0.2 \times SL + E_h + E_v$ (0.071)

Load combination no.7 $0.9 \times DL + W$ (0.201)

Load combination no.8 $0.9 \times DL + E_h - E_v$ (0.051)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.75 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **56.4** in²/ft

Properties for walls loaded out-of-plane:

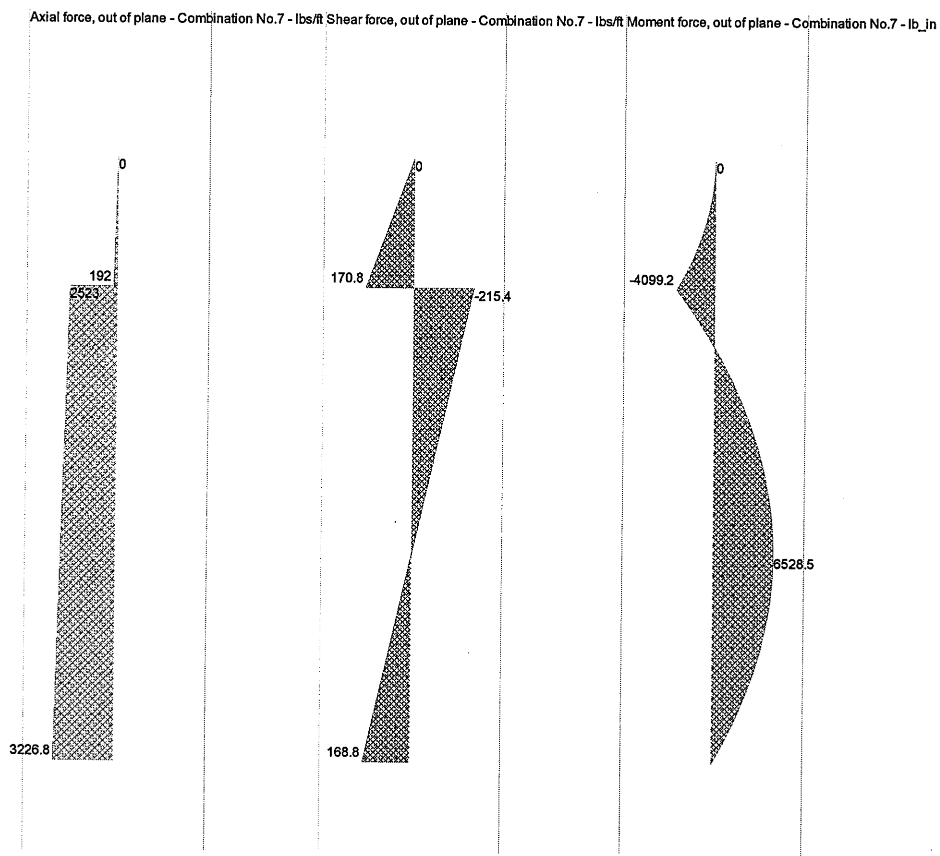
Moment of inertia $I = t^3 / 12 - 0.75 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) =$ **366.6** in^4/ft

Section modulus $S = I / c =$ **96.2** in^3/ft

Radius of gyration $r = \sqrt{[I / A]} =$ **2.548** in

Effective height factor $K =$ **1**

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



Maximum moment location 6.44 ft

Axial load at mid-height of panel $P =$ **2918** lb/ft

Slenderness ratio $(K \times h) / r =$ **69.061** < 99

Nominal axial strength $P_n = 0.8 \times (0.8 \times f'_m \times (A - A_s) + A_s \times 0 \text{ ksi}) \times [1 - ((K \times h) / (140 \times r))^2] =$ **68196** lb/ft

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

Design axial strength $\phi \times P_n =$ **61377** lb/ft

$P / (\phi \times P_n) =$ **0.048**

PASS - Nominal axial strength exceeds axial load

Factored axial stress $P / t =$ **32** psi

Factored axial stress limit $0.2 \times f'_m =$ **500** psi

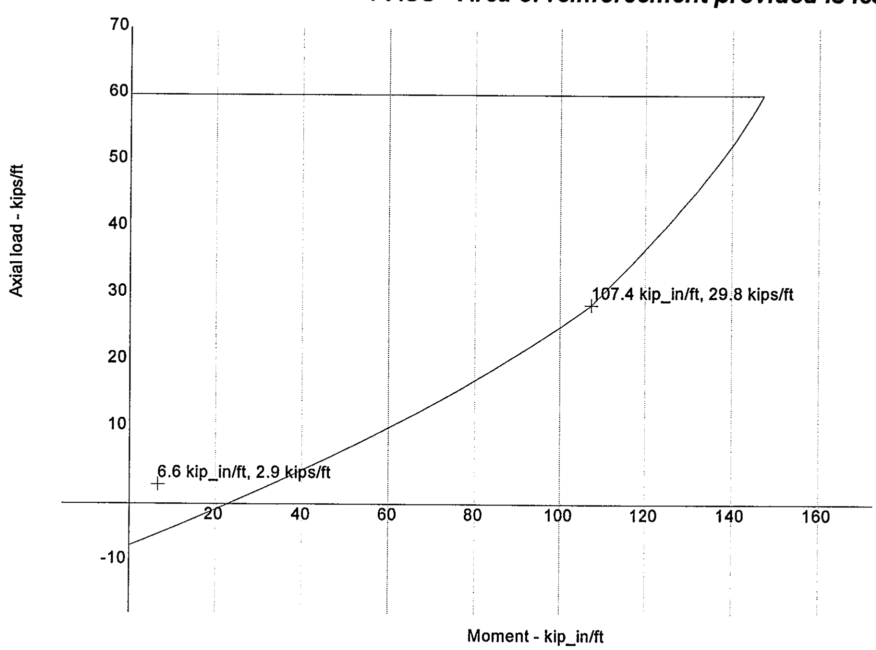
PASS - Allowable stress under out of plane loads exceeds factored axial stress

Nominal cracking moment strength	$M_{cr} = S \times f_{r\_norm}$ = **9976** lb_in/ft
Modular ratio	$n = E_s / E_m$ = **12.889**
Distance to neutral axis	$c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m)$ = **0.511** in
Moment of inertia of cracked section	$I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3$ = **23.5** in^4/ft
By iteration	$M_{u0} = M$ = **6528** lb_in/ft
	$\delta_{u0} = 5 \times M_{u0} \times h^2 / (48 \times E_m \times I)$ = **0.026** in
	$M_{u1} = M_{u0} + P \times \delta_{u0}$ = **6603** lb_in/ft
	$\delta_{u1} = 5 \times M_{u1} \times h^2 / (48 \times E_m \times I)$ = **0.026** in
Bending moment at mid-height of panel	$M = M_{u0} + P \times \delta_{u1}$ = **6604** lb_in/ft
Depth of reinforcement	d = **3.813** in
Effective width per bar	b_{eff} = min(s, 6 × t_{nom}, 72 in) = **32** in
Strength reduction factor	ϕ = **0.9**
Tensile strain in reinforcement	$\varepsilon_s = f_y / E_s$ = **0.0021**
Maximum usable compressive strain of masonry	ε_{mu} = **0.0025**
Fiber of max.compressive strain to neutral axis	$c_{bal} = \varepsilon_{mu} \times d / (\varepsilon_{mu} + \varepsilon_s)$ = **2.086** in
Tensile force at balance point	$T_{bal} = A_s \times f_y$ = **6903** lb/ft
	β_1 = **0.8**
Compressive force at balance point	$C_{bal} = 0.8 \times f'_m \times \beta_1 \times c_{bal}$ = **40053** lb/ft
Design axial force at balance point	$P_{bal} = \phi \times (C_{bal} - T_{bal})$ = **29835** lb/ft
Design moment at balance point	$M_{bal} = \phi \times [T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - \beta_1 \times c_{bal} / 2)]$ = **107352** lb_in/ft
Maximum design moment from integration diagram	M_c = **32835** lb_in/ft
	M / M_c = **0.201**

PASS - Combination of applied axial load and flexure is acceptable

Maximum area of tensile reinforcement (9.3.3.2)	$A_{s\_max} = 0.64 \times f'_m \times [\varepsilon_{mu} / (\varepsilon_{mu} + 1.5 \times \varepsilon_s)] \times d / f_y$ = **0.544** in^2/ft

PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram

Consider wall at bottom under load combination no.7

Shear force V = **169** lb/ft

Compressive force N_u = **3227** lb/ft

Net shear area $A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **11.169** in^2/ft

Nominal shear strength $V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_u, 6 \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **3041** lb/ft

Strength reduction factor ϕ_v = **0.8**

Design shear strength $\phi_v \times V_n$ = **2432** lb/ft

$V / (\phi_v \times V_n)$ = **0.069**

PASS - Design shear strength exceeds applied shear strength

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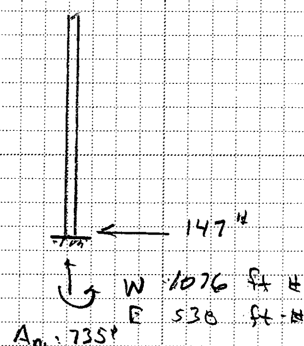
Interior Walls - Non Brg

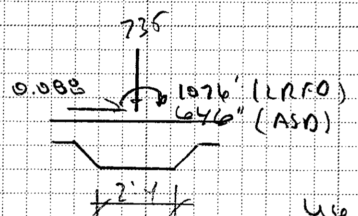
Wind Load 10psf (LRFD)

Seismic - SDS = 0.107 ∴ 5psf

Wall Height = 14'8"

Use 35' @ 48'oc





Use 2'-4' Wide Thickened Slab

MASONRY WALL PANEL DESIGN (TMS 402/602-16)

In accordance with strength design method

Tedds calculation version 2.2.08

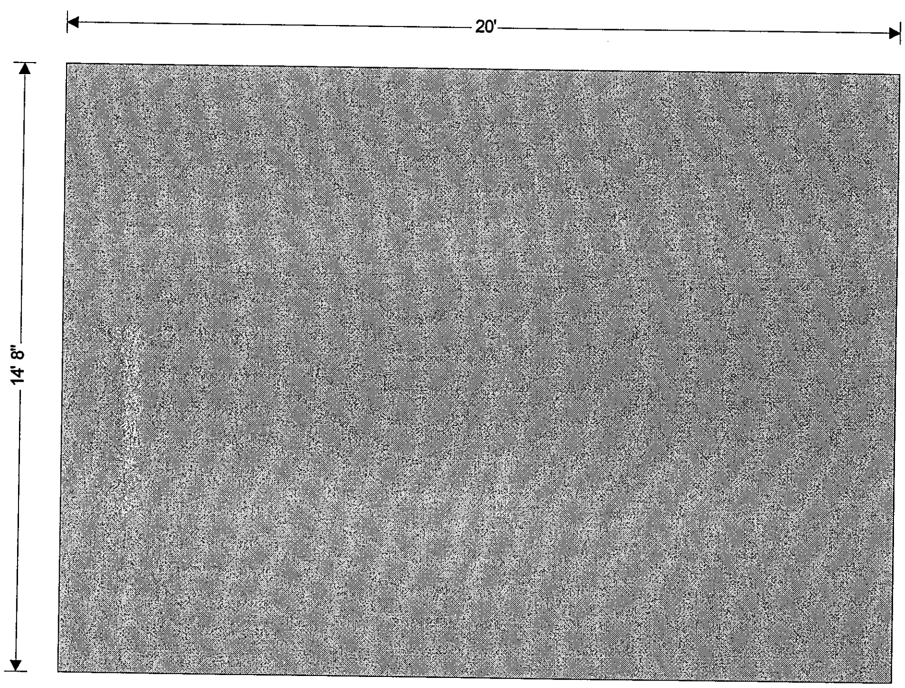
Masonry wall panel details

Reinforced single-wythe wall, the wall is free at the top and fixed 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 = **20** ft

Panel height h = **14.667** ft



Seismic properties

Seismic design category B

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **0.107**

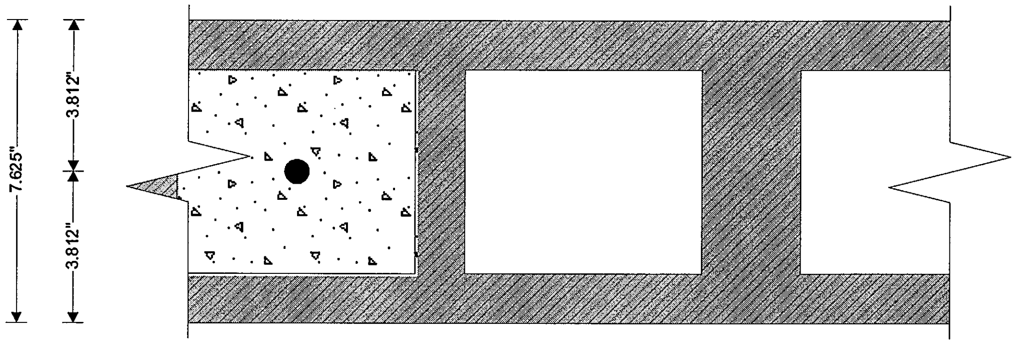
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

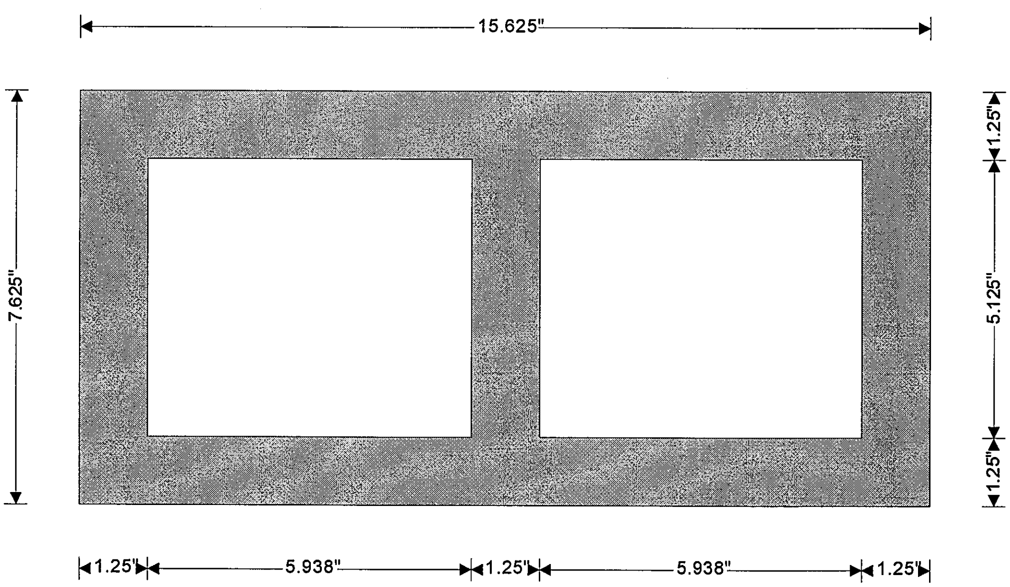
Wall thickness t = **7.625** in



Masonry details

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

Compressive strength of unit	f'_{cu} = **3250** psi
Density of masonry units	γ_{block} = **135** lb/ft^3
Height of masonry units	h_b = **7.625** in
Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** 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** in^2/ft
Area of grout	$A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **49.69** psf



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

Net compressive strength of masonry f'_m = **2500** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **2250000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **900000** psi

From TMS 402 -16 Table 9.1.9.2 - Modulus of rupture

Modulus of rupture normal to bed f_{r_norm} = **97** psi

Modulus of rupture parallel to bed f_{r_para} = **167** psi

Reinforcement details

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

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** 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** in²/ft

Yield strength of horizontal reinforcement f_{yv} = **70000** psi

Allowable tensile stress in horizontal reinforcement F_{sv} = **30000** psi

Horizontal reinforcement provided (2) W1.7 wires at 16 in centers

Area of horizontal reinforcement $A_v = 2 \times \pi \times HDia^2 / (4 \times s_v)$ = **0.03** in²/ft

Minimum area of vertical reinf. (cl. 9.3.6.2) $A_{s\_min} = A_v / 3$ = **0.01** in²/ft

PASS - Area of vertical reinforcement provided exceeds the minimum

Lateral out-of-plane loads

Wind load on panel W = **10** psf

Wind load on parapet W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.043**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **5** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **5** psf

Lateral in-plane loads

Vertical loading details

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.021**

From ASCE 7-16 cl.2.3 - Combining factored loads using strength design (Utilization)

Load combination no.1 $1.4 \times DL$ (0.018)

Load combination no.2 $1.2 \times DL + 1.6 \times LL + 0.5 \times (LL_r$ or SL or RL) (0.015)

Load combination no.3 $1.2 \times DL + LL + 1.6 \times (LL_r$ or SL or RL) (0.015)

Load combination no.4 $1.2 \times DL + 1.6 \times (LL_r$ or SL or RL) $+ 0.5 \times W$ (0.352)

Load combination no.5 $1.2 \times DL + W + LL + 0.5 \times (LL_r$ or SL or RL) (0.756)

Load combination no.6 $1.2 \times DL + LL + 0.2 \times SL + E_h + E_v$ (0.349)

Load combination no.7 $0.9 \times DL + W$ (0.773)

Load combination no.8 $0.9 \times DL + E_h - E_v$ (0.366)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **52.7** in²/ft

Properties for walls loaded out-of-plane:

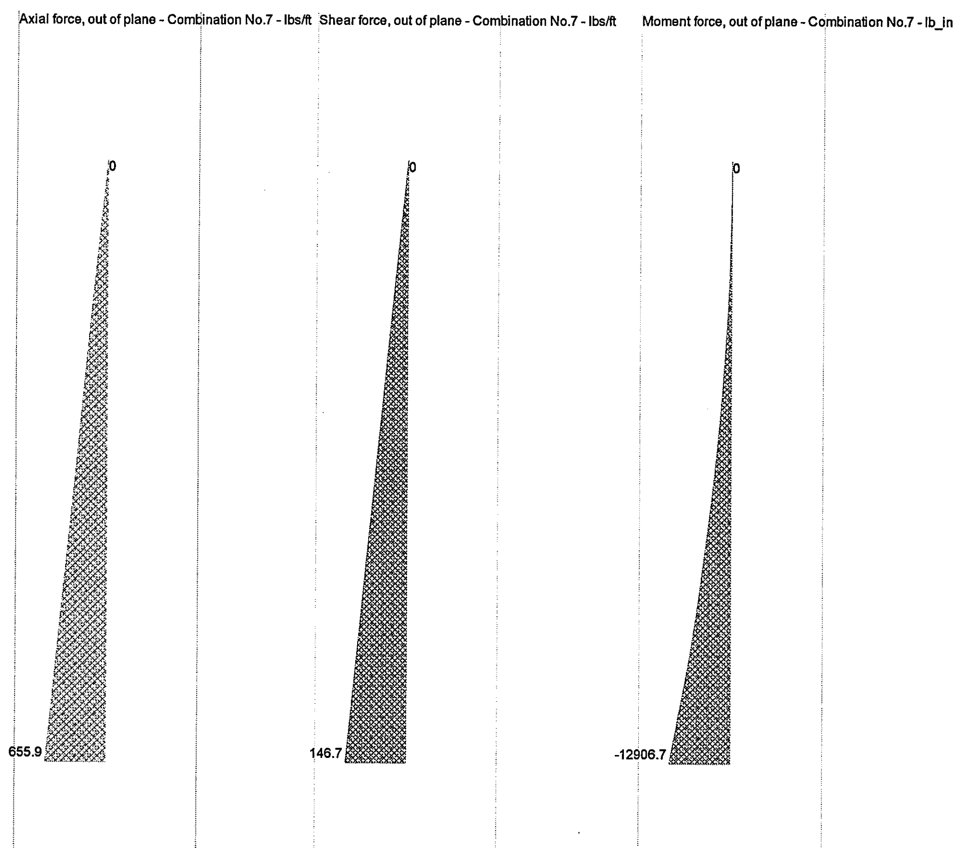
Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) =$ **358.4** in⁴/ft

Section modulus $S = I / c =$ **94** in^3/ft

Radius of gyration $r = \sqrt{[I / A]} =$ **2.608** in

Effective height factor $K =$ **1**

Consider wall at bottom under load combination no.7



Axial load at bottom of panel $P =$ **656** lb/ft

Slenderness ratio $(K \times h) / r =$ **67.492** < 99

Nominal axial strength $P_n = 0.8 \times (0.8 \times f'_m \times (A - A_s) + A_s \times 0 \text{ ksi}) \times [1 - ((K \times h) / (140 \times r))^2] =$ **64636** lb/ft

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

Design axial strength $\phi \times P_n =$ **58173** lb/ft

$P / (\phi \times P_n) =$ **0.011**

PASS - Nominal axial strength exceeds axial load

Factored axial stress $P / t =$ **7** psi

Factored axial stress limit $0.2 \times f'_m =$ **500** psi

PASS - Allowable stress under out of plane loads exceeds factored axial stress

Nominal cracking moment strength $M_{cr} = S \times f_{r\_norm} =$ **9135** lb_in/ft

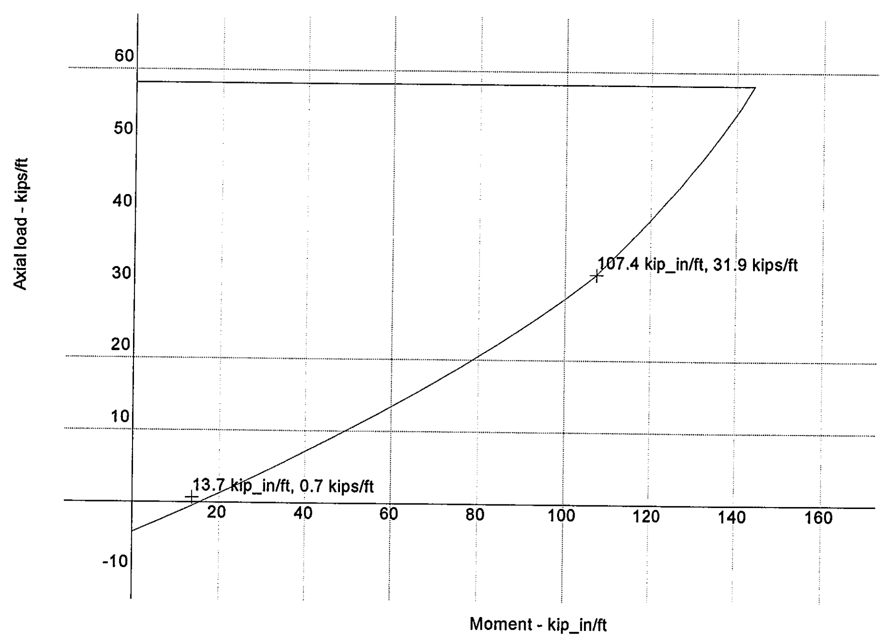
Modular ratio $n = E_s / E_m =$ **12.889**

Distance to neutral axis	$c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m) = \mathbf{0.274}$ in
Moment of inertia of cracked section	$I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3 = \mathbf{14.2}$ in^4/ft
By iteration	$M_{u0} = M = \mathbf{12907}$ lb_in/ft
	$\delta_{u0} = M_{cr} \times h^2 / (4 \times E_m \times I) + (M_{u0} - M_{cr}) \times h^2 / (4 \times E_m \times I_{cr}) = \mathbf{1.000}$ in
	$M_{u3} = M_{u0} + P \times \delta_{u2} = \mathbf{13683}$ lb_in/ft
	$\delta_{u3} = M_{cr} \times h^2 / (4 \times E_m \times I) + (M_{u3} - M_{cr}) \times h^2 / (4 \times E_m \times I_{cr}) = \mathbf{1.188}$ in
Bending moment at bottom of panel	$M = M_{u0} + P \times \delta_{u3} = \mathbf{13686}$ lb_in/ft
Depth of reinforcement	$d = \mathbf{3.813}$ in
Effective width per bar	$b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) = \mathbf{48}$ in
Strength reduction factor	$\phi = \mathbf{0.9}$
Tensile strain in reinforcement	$\varepsilon_s = f_y / E_s = \mathbf{0.0021}$
Maximum usable compressive strain of masonry	$\varepsilon_{mu} = \mathbf{0.0025}$
Fiber of max.compressive strain to neutral axis	$c_{bal} = \varepsilon_{mu} \times d / (\varepsilon_{mu} + \varepsilon_s) = \mathbf{2.086}$ in
Tensile force at balance point	$T_{bal} = A_s \times f_y = \mathbf{4602}$ lb/ft
	$\beta_1 = \mathbf{0.8}$
Compressive force at balance point	$C_{bal} = 0.8 \times f'_m \times \beta_1 \times c_{bal} = \mathbf{40053}$ lb/ft
Design axial force at balance point	$P_{bal} = \phi \times (C_{bal} - T_{bal}) = \mathbf{31906}$ lb/ft
Design moment at balance point	$M_{bal} = \phi \times [T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - \beta_1 \times c_{bal} / 2)] = \mathbf{107352}$ lb_in/ft
Maximum design moment from integration diagram	$M_c = \mathbf{17696}$ lb_in/ft
	$M / M_c = \mathbf{0.773}$

PASS - Combination of applied axial load and flexure is acceptable

Maximum area of tensile reinforcement (9.3.3.2) $A_{s\_max} = 0.64 \times f'_m \times [\varepsilon_{mu} / (\varepsilon_{mu} + 1.5 \times \varepsilon_s)] \times d / f_y = \mathbf{0.544}$ in^2/ft

PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram

Consider wall at bottom under load combination no.7

Shear force $V = \mathbf{147}$ lb/ft

Compressive force	N_u = **656** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **7.446** in^2/ft
Nominal shear strength	$V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_u, 4 \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **1002** lb/ft
Strength reduction factor	ϕ_v = **0.8**
Design shear strength	$\phi_v \times V_n$ = **801** lb/ft
	$V / (\phi_v \times V_n)$ = **0.183**

PASS - Design shear strength exceeds applied shear strength

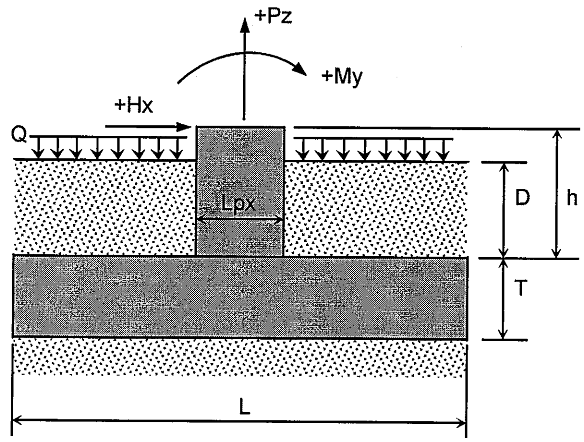
RECTANGULAR SPREAD FOOTING ANALYSIS
For Assumed Rigid Footing with from 1 To 8 Piers
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:		Subject:			
Job Number:		Originator:		Checker:	

Input Data:

Footing Data:

Footing Length, L =	2.330	ft.
Footing Width, B =	1.000	ft.
Footing Thickness, T =	12.000	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	0.000	ft.
Soil Unit Wt., γs =	0.120	kcf
Pass. Press. Coef., Kp =	2.040	
Coef. of Base Friction, μ =	0.300	
Uniform Surcharge, Q =	0.000	ksf

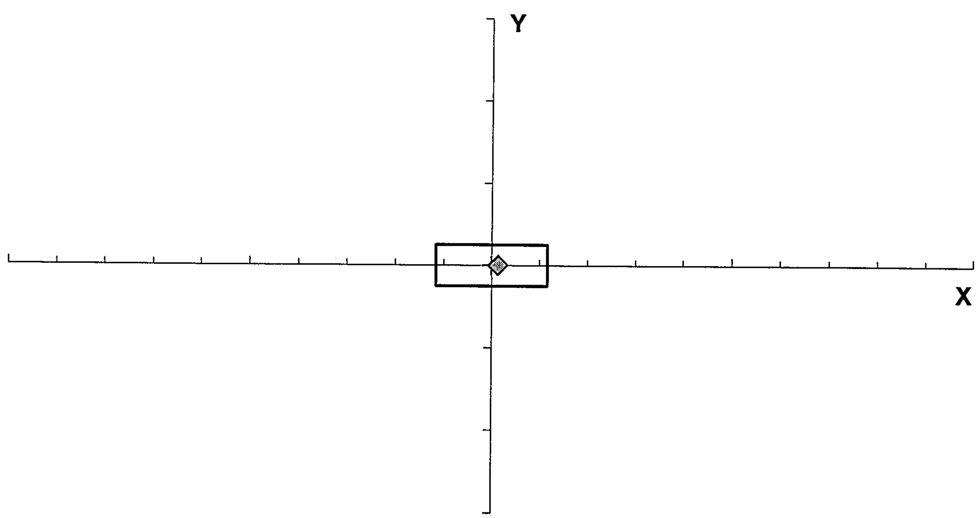


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	0.000							
Lpy (ft.) =	0.000							
h (ft.) =	0.000							
Pz (k) =	-0.74							
Hx (k) =	0.00							
Hy (k) =	0.00							
Mx (ft-k) =	0.00							
My (ft-k) =	0.65							



FOOTING PLAN

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-4.93	kips
ex =	0.13	ft. (<= L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	5.74	ft-kips
ΣMoy =	0.65	ft-kips
FS(ot)y =	8.889	(>= 1.5)

Sliding Check:

Pass(x) =	17.63	kips
Frict(x) =	1.48	kips
FS(slid)x =	N.A.	
Passive(y) =	41.07	kips
Frict(y) =	1.48	kips
FS(slid)y =	N.A.	

Uplift Check:

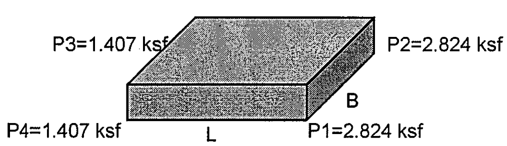
ΣPz(down) =	-4.93	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	2.330	ft.
Brg. Ly =	1.000	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P1 =	2.824	ksf
P2 =	2.824	ksf
P3 =	1.407	ksf
P4 =	1.407	ksf



CORNER PRESSURES

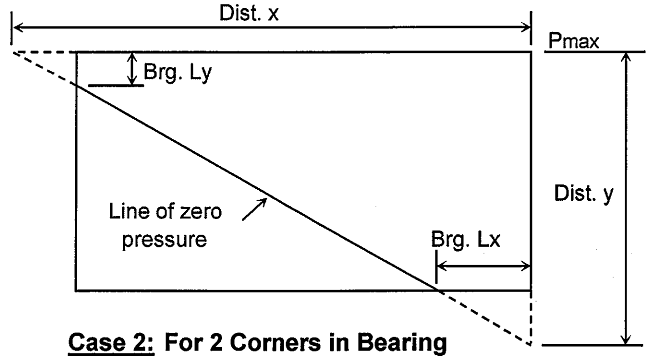
Maximum Net Soil Pressure:

$P_{max}(net) = P_{max}(gross)-(D+T)^*\gamma s$

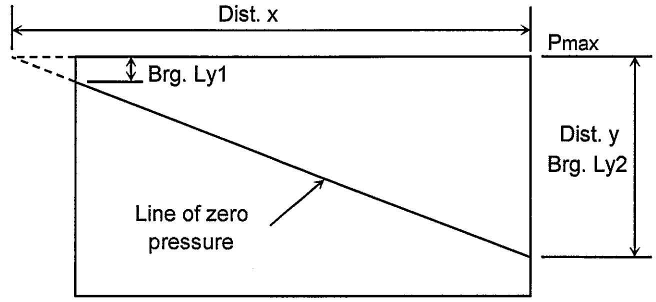
P_{max}(net) =	1.384	ksf

Nomenclature for Biaxial Eccentricity:

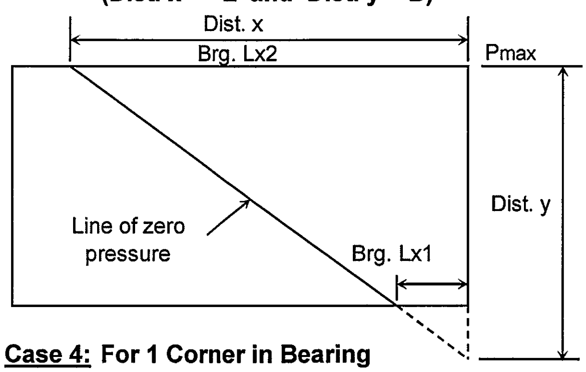
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



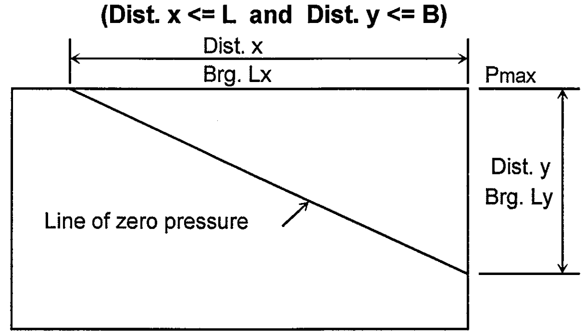
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)



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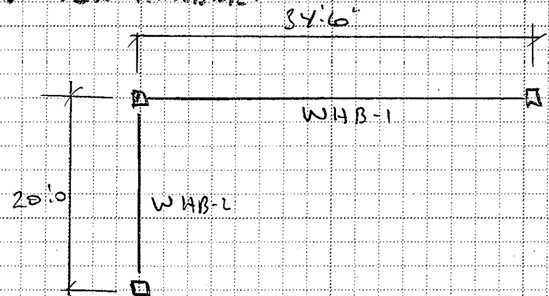
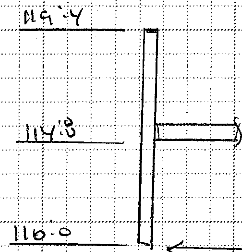
WINDOW HEADER:





WALL LOWATTR SOLID
W: 80 pcf (119.33 - 110.5) - 750
WRF 1400 #/ft DL
210 #/ft LL
WDSM: 2360 #/ft

WIND A: 150 #
W: 100 ft² ⇒ (25 psf)

W_y: 2.47 k/ft M_y: 367.5 ft·k V: 42.6 k

W_x: 0.19 k/ft M_x: 26.3 ft·k V: 3.3 k

@ SPAN: 20'-0

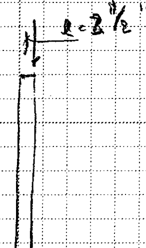
W: 750 #
Bm: 110 #/ft
SLAB: 80 (1)) 120 → WDL 0.98 k/ft = V_x = 10.1 k
36 (1) WLL 0.05 k/ft

HSS 20x6x5/8 - NO CAMBER

HSS 20x8x5/8 I_x: 1440

CAMBER 1/2" @ 34'-6"

COLUMN H: 10'-8 A_x: 10.1 + 42.6 = 52.7 k TL
3.6 k LL



HSS 6x6x3/8

FOOT: 6'-6"x6'-6"x 2'-10"

BASE PL - P: 52.7 k 12"x12"

l = 3.15

$$t = \sqrt{\frac{52.7(3.33)}{36(12)^2}} \times l = 0.57$$

USE 12x12x3/4 PL

Tekla Tedds Kreher Engineering, Inc	Project				Job Ref.	
	Section				Sheet no./rev. 1	
	Calc. by O	Date 5/29/2024	Chk'd by	Date	App'd by	Date

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 15th Edition published 2016 using the ASD method

Tedds calculation version 4.4.08

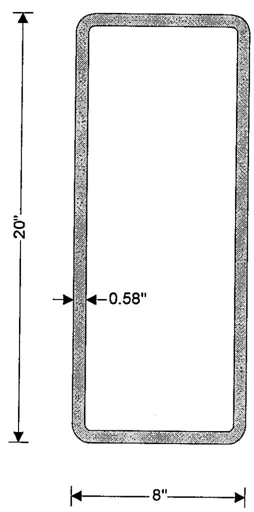
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Design section 1

Section details

Section type	HSS 20x8x5/8 (AISC 15th Edn (v15.0))
ASTM steel designation	User defined
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



HSS 20x8x5/8 (AISC 15th Edn (v15.0))
Section depth, d, 20 in
Section breadth, b_f, 8 in
Weight of section, Weight, 110.4 lbf/ft
Section thickness, t, 0.581 in
Area of section, A, 30.3 in^2
Radius of gyration about x-axis, r_x, 6.89 in
Radius of gyration about y-axis, r_y, 3.34 in
Elastic section modulus about x-axis, S_x, 144 in^3
Elastic section modulus about y-axis, S_y, 84.6 in^3
Plastic section modulus about x-axis, Z_x, 185 in^3
Plastic section modulus about y-axis, Z_y, 96.4 in^3
Second moment of area about x-axis, I_x, 1440 in^4
Second moment of area about y-axis, I_y, 338 in^4

Analysis results

Required flexural strength - Major axis	$M_{r,x}$ = **367.5** kips_ft
Required flexural strength - Minor axis	$M_{r,y}$ = **26.3** kips_ft
Required shear strength - Major axis	$V_{r,x}$ = **42.6** kips
Required shear strength - Minor axis	$V_{r,y}$ = **3.3** kips

Restraint spacing

Major axis lateral restraint	L_x = **34.5** ft
Minor axis lateral restraint	L_y = **34.5** ft
Torsional restraint	L_z = **34.5** ft

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 17)

Width to thickness ratio — $\max(d - 3 \times t, b_f - 3 \times t) / t =$ **31.42**

Limiting ratio for compact section — $\lambda_{pff} = 1.12 \times \sqrt{[E / F_y]} =$ **26.97**

Limiting ratio for non-compact section — $\lambda_{rff} = 1.40 \times \sqrt{[E / F_y]} =$ **33.72** Noncompact

Classification of web in flexure - Table B4.1b (case 19)

Width to thickness ratio — $\max(d - 3 \times t, b_f - 3 \times t) / t =$ **31.42**

Limiting ratio for compact section — $\lambda_{pwf} = 2.42 \times \sqrt{[E / F_y]} =$ **58.28**

Limiting ratio for non-compact section — $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]} =$ **137.27** Compact

Section is noncompact in flexure

Design of members for shear - Chapter G

Required shear strength — $V_{r,x} =$ **42.6** kips

Web area — $A_w = 2 \times (d - 3 \times t) \times t =$ **21.215** in^2

Web plate buckling coefficient — $k_v =$ **5**

$(d - 3 \times t) / t <= 1.10 \times \sqrt{(k_v \times E / F_y)}$

Web shear coefficient - eq G2-9 — $C_{v2} =$ **1.000**

Nominal shear strength - eq G4-1 — $V_{n,x} = 0.6 \times F_y \times A_w \times C_{v2} =$ **636.4** kips

Safety factor — $\Omega_v = 1.67$

Allowable shear strength — $V_{c,x} = V_{n,x} / \Omega_v =$ **381.1** kips

$V_{r,x} / V_{c,x} =$ **0.112**

PASS - Allowable shear strength exceeds required shear strength

Required shear strength — $V_{r,y} =$ **3.3** kips

Web area — $A_w = 2 \times (b_f - 3 \times t) \times t =$ **7.271** in^2

Web plate buckling coefficient — $k_v =$ **5**

$(b_f - 3 \times t) / t <= 1.10 \times \sqrt{(k_v \times E / F_y)}$

Web shear coefficient - eq G2-9 — $C_{v2} =$ **1.000**

Nominal shear strength - eq G4-1 — $V_{n,y} = 0.6 \times F_y \times A_w \times C_{v2} =$ **218.1** kips

Safety factor — $\Omega_v = 1.67$

Allowable shear strength — $V_{c,y} = V_{n,y} / \Omega_v =$ **130.6** kips

$V_{r,y} / V_{c,y} =$ **0.025**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength — $M_{r,x} =$ **367.5** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1 — $M_{n,yld,x} = M_{p,x} = F_y \times Z_x =$ **770.8** kips_ft

Lateral-torsional buckling - Section F7.4

Unbraced length — $L_b =$ **34.5** ft

Limiting unbraced length for yielding - eq F7-12 — $L_p = 0.13 \times E \times r_y \times \sqrt{(J \times A)} / (F_y \times Z_x) =$ **18.899** ft

Limiting unbraced length for inelastic LTB - eq F7-13 — $L_r = 2 \times E \times r_y \times \sqrt{(J \times A)} / (0.7 \times F_y \times S_x) =$ **533.62** ft

LTB modification factor — $C_b =$ **1.000**

Nominal flexural strength for lateral-torsional buckling - eq F7-10

$M_{n,ltb,x} = \min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x}) =$ **760.2** kips_ft

Compression flange local buckling - Section F7.2

Nominal flexural strength for compression flange local buckling - eq F7-2

$M_{n,flb,x} = \min(M_{p,x} - (M_{p,x} - F_y \times S_x) \times (3.57 \times (b_f - 3 \times t) / t \times \sqrt{(F_y / E)} - 4.0), M_{p,x})$ = **770.8** kips_ft

Web local buckling - Section F7.3

Nominal flexural strength for web local buckling - eq F7-5

$M_{n,wlb,x} = M_{p,x}$ = **770.8** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,flb,x})$ = **770.8** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **461.6** kips_ft

$M_{r,x} / M_{c,x}$ = **0.796**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,y}$ = **26.3** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1 $M_{n,yld,y} = M_{p,y} = F_y \times Z_y$ = **401.7** kips_ft

Compression flange local buckling - Section F7.2

Nominal flexural strength for compression flange local buckling - eq F7-2

$M_{n,flb,y} = \min(M_{p,y} - (M_{p,y} - F_y \times S_y) \times (3.57 \times (d - 3 \times t) / t \times \sqrt{(F_y / E)} - 4.0), M_{p,y})$ = **369.3** kips_ft

Web local buckling - Section F7.3

Nominal flexural strength for web local buckling - eq F7-5

$M_{n,wlb,y} = M_{p,y}$ = **401.7** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,y} = \min(M_{n,yld,y}, M_{n,flb,y})$ = **369.3** kips_ft

Allowable flexural strength $M_{c,y} = M_{n,y} / \Omega_b$ = **221.1** kips_ft

$M_{r,y} / M_{c,y}$ = **0.119**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1b $M_{r,x} / M_{c,x} + M_{r,y} / M_{c,y}$ = **0.915**

PASS - Combined flexure and axial force is within acceptable limits

FOOTING ANALYSIS

In accordance with ACI318-19

Tedds calculation version 3.3.03

Summary results

Overall design status PASS

Overall design utilisation 0.877

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	74.4			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.761	2.007	0.877	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	42.1	116.1	0.362	Pass
Moment, positive, y-direction	kip_ft	42.1	116.1	0.362	Pass
Shear, one-way, x-direction	kips	21.3	43.7	0.487	Pass
Shear, one-way, y-direction	kips	21.3	45.5	0.468	Pass
Shear, two-way, Col 1	psi	70.985	189.737	0.374	Pass
Min.area of reinf, bot., x-direction	in^2	1.966	2.480		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.1		Pass
Min.area of reinf, bot., y-direction	in^2	1.966	2.480		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.1		Pass

Pad footing details

Length of footing L_x = **6.5** ft

Width of footing L_y = **6.5** ft

Footing area $A = L_x \times L_y$ = **42.25** ft^2

Depth of footing h = **14** in

Depth of soil over footing h_{soil} = **28** in

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

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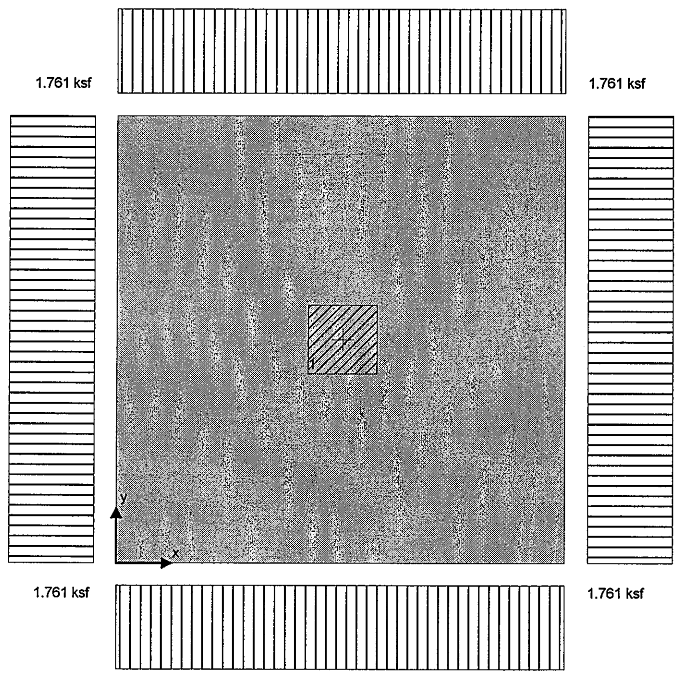


Column no.1 details

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

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

position in x-axis x_1 = **39.00** in

position in y-axis y_1 = **39.00** in

Soil properties

Net allowable bearing pressure q_{allow_Net} = **1.5** ksf using a soil factor of safety, FS_{soil}, of 1

Density of soil γ_{soil} = **145.0** lb/ft^3

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

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

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

Self weight $F_{swt} = h \times \gamma_{conc}$ = **175** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **338.3** psf

Column no.1 loads

Dead load in z F_{Dz1} = **49.1** kips

Live load in z F_{Lz1} = **3.6** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.835)

1.0D + 1.0L (0.877)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1}) + \gamma_L \times F_{Lz1}$ = **74.4 kips**

Moments on footing

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **241.8** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **241.8** kip_ft

Uplift verification

Vertical force F_{dz} = **74.388** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

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

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

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.761** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.761** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.761** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.761** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.761** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.761** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **2.007** ksf

q_{max} / q_{allow} = **0.877**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19

Tedds calculation version 3.3.03

Material details

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

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

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

Cover to top of footing c_{nom_t} = **3** in

Cover to side of footing c_{nom_s} = **3** in

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

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.444)

1.2D + 1.6L + 0.5Lr (0.435)
1.2D + 1.6L + 0.5S (0.435)
1.2D + 1.6L + 0.5R (0.435)
1.2D + 1.0L + 1.6Lr (0.421)
1.2D + 1.0L + 1.6S (0.421)
1.2D + 1.0L + 1.6R (0.421)
1.2D + 1.6Lr + 0.5W (0.396)
1.2D + 1.6S + 0.5W (0.396)
1.2D + 1.6R + 0.5W (0.396)
1.2D + 1.0L + 0.5Lr + 1.0W (0.421)
1.2D + 1.0L + 0.5S + 1.0W (0.421)
1.2D + 1.0L + 0.5R + 1.0W (0.421)
$(1.2 + 0.2 \times S_{DS})D + 1.0L + 0.2S + 1.0E$ (0.487)
0.9D + 1.0W (0.374)
$(0.9 - 0.2 \times S_{DS})D + 1.0E$ (0.374)

Combination 14 results: $(1.2 + 0.2 \times S_{DS})D + 1.0L + 0.2S + 1.0E$

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1}) + \gamma_L \times F_{Lz1}$ = **102.7** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **333.8** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **333.8** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.431** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.431** ksf

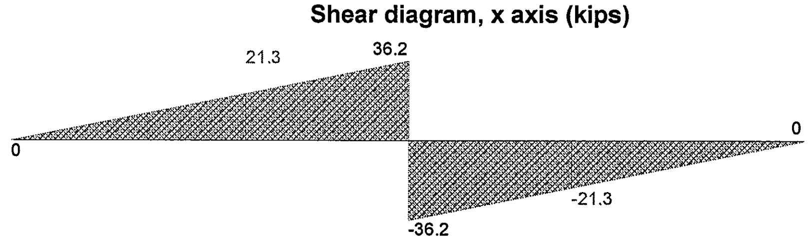
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.431** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.431** ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.431** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.431** ksf

Shear diagram, x axis (kips)



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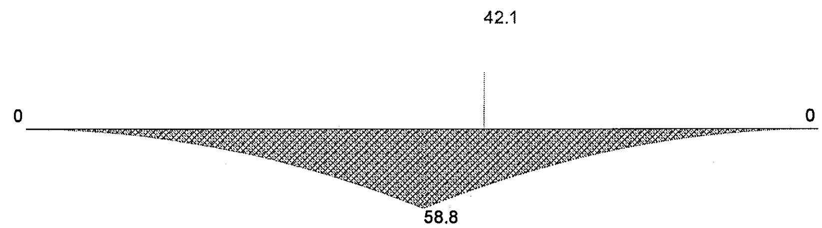
Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **42.082** kip_ft

Tension reinforcement provided 8 No.5 bottom bars (10.1 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **2.48** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h$ = **1.966** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **10.688** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.561** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.660** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04557**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **129.046** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **116.142** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.362**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **21.273** kips

Depth to reinforcement $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **10.688** in

Size effect factor (22.5.5.1.3) λ_s = **1**

Ratio of longitudinal reinforcement $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00297**

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v)$ = **60.662** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **45.497** kips

$V_{u.x} / \phi V_n$ = **0.468**

PASS - Design shear capacity exceeds ultimate shear load

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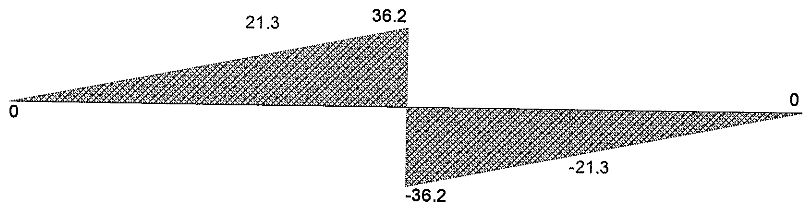
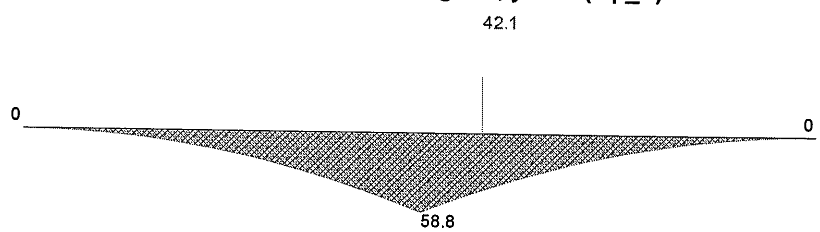
Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **42.082** kip_ft

Tension reinforcement provided 8 No.5 bottom bars (10.1 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **2.48** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **1.966** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **10.063** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.561** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.660** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04273**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **121.296** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **109.167** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.385**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **21.273** kips

Depth to reinforcement $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2)$ = **10.063** in

Size effect factor (22.5.5.1.3) λ_s = **1**

Ratio of longitudinal reinforcement $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v)$ = **0.00316**

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v) =$ **58.273** kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **43.705** kips

$V_{u.y} / \phi V_n =$ **0.487**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement $d_{v2} =$ **10.375** in

Shear perimeter length (22.6.4) $l_{xp} =$ **22.375** in

Shear perimeter width (22.6.4) $l_{yp} =$ **22.375** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **89.500** in

Shear area $A_p = l_{x,perim} \times l_{y,perim} =$ **500.641** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **356.641** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **2.431** ksf

Ultimate shear load $F_{up} = \gamma_D \times (F_{Dz1}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **65.914** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\text{ psi}) =$ **70.985** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1} =$ **1.00**

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

Size effect factor (22.5.5.1.3) $\lambda_s =$ **1**

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

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **419.753** psi

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

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

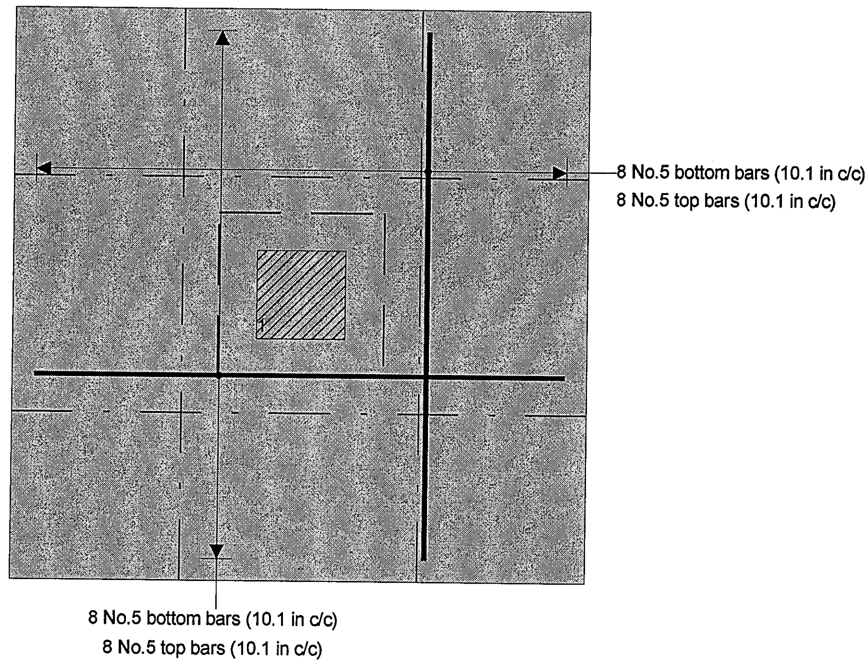
Shear strength reduction factor $\phi_v =$ **0.75**

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

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

$v_{ug} / \phi v_n =$ **0.374**

PASS - Design shear stress capacity exceeds ultimate shear stress load

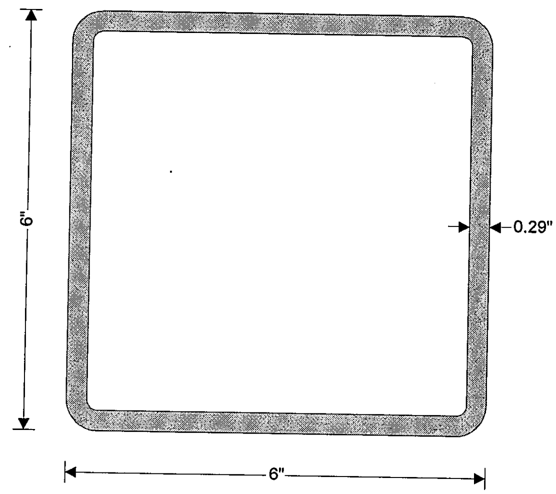


8 No.5 bottom bars (10.1 in c/c)
8 No.5 top bars (10.1 in c/c)
8 No.5 bottom bars (10.1 in c/c)
8 No.5 top bars (10.1 in c/c)

STEEL COLUMN DESIGN

In accordance with AISC360-10 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section — **HSS 6x6x5/16**

Design loading

Required axial strength — P_r = **53** kips (Compression)

Moment about x axis at end 1 — M_{x1} = **11.0** kips_ft

Moment about x axis at end 2 — M_{x2} = **11.0** kips_ft

Single curvature bending about x axis

Maximum moment about x axis — M_x = max(abs(M_{x1}), abs(M_{x2})) = **11.0** kips_ft

Moment about y axis at end 1 — M_{y1} = **11.0** kips_ft

Moment about y axis at end 2 — M_{y2} = **11.0** kips_ft

Single curvature bending about y axis

Maximum moment about y axis — M_y = max(abs(M_{y1}), abs(M_{y2})) = **11.0** kips_ft

Maximum shear force parallel to y axis — V_{ry} = **5.0** kips

Maximum shear force parallel to x axis — V_{rx} = **5.0** kips

Material details

Steel grade — **A500 Gr. C**

Yield strength — F_y = **50** ksi

Ultimate strength — F_u = **62** ksi

Modulus of elasticity — E = **29000** ksi

Shear modulus of elasticity — G = **11200** ksi

Unbraced lengths

For buckling about x axis — L_x = **120** in

For buckling about y axis — L_y = **120** in

For torsional buckling — L_z = **120** in

Effective length factors

For buckling about x axis K_x = **1.00**

For buckling about y axis K_y = **1.00**

For torsional buckling K_z = **1.00**

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f - 3 \times t$ = **5.127** in

Critical web width $h = d - 3 \times t$ = **5.127** in

Width to thickness ratio of flange (compression) $\lambda_{f\_c} = b / t$ = **17.619**

Width to thickness ratio of web (compression) $\lambda_{w\_c} = h / t$ = **17.619**

Width to thickness ratio of flange (major flexure) $\lambda_{f\_fx} = b / t$ = **17.619**

Width to thickness ratio of web (major flexure) $\lambda_{w\_fx} = h / t$ = **17.619**

Width to thickness ratio of flange (minor flexure) $\lambda_{f\_fy} = h / t$ = **17.619**

Width to thickness ratio of web (minor flexure) $\lambda_{w\_fy} = b / t$ = **17.619**

Compression

Limit for nonslender section $\lambda_{r\_c} = 1.40 \times \sqrt{(E / F_y)}$ = **33.716**

The section is nonslender in compression

Flexure

Limit for compact flange $\lambda_{pf\_f} = 1.12 \times \sqrt{(E / F_y)}$ = **26.973**

Limit for noncompact flange $\lambda_{rf\_f} = 1.40 \times \sqrt{(E / F_y)}$ = **33.716**

Limit for compact web $\lambda_{pw\_f} = 2.42 \times \sqrt{(E / F_y)}$ = **58.281**

Limit for noncompact web $\lambda_{rw\_f} = 5.70 \times \sqrt{(E / F_y)}$ = **137.274**

The section is compact in flexure about the major axis

The section is compact in flexure about the minor axis

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = K_x \times L_x / r_x$ = **51.9**

Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y$ = **51.9**

Second order effects

Second order effects for bending about x axis (cl. App 8.1)

Coefficient C_m $C_{mx} = 0.6 + 0.4 \times M_{x1} / M_{x2}$ = **1.000**

Coefficient α α = **1.6**

Elastic critical buckling stress $P_{e1x} = \pi^2 \times E \times I_x / (K_{1x} \times L_x)^2$ = **681.8** kips

P-δ amplifier $B_{1x} = \max(1.0, C_{mx} / (1 - \alpha \times P_r / P_{e1x}))$ = **1.142**

Required flexural strength $M_{rx} = B_{1x} \times M_x$ = **12.6** kips_ft

Second order effects for bending about y axis (cl. App 8.1)

Coefficient C_m $C_{my} = 0.6 + 0.4 \times M_{y1} / M_{y2}$ = **1.000**

Coefficient α α = **1.6**

Elastic critical buckling stress $P_{e1y} = \pi^2 \times E \times I_y / (K_{1y} \times L_y)^2$ = **681.8** kips

P-δ amplifier $B_{1y} = \max(1.0, C_{my} / (1 - \alpha \times P_r / P_{e1y})) = \mathbf{1.142}$

Required flexural strength $M_{ry} = B_{1y} \times M_y = \mathbf{12.6}$ kips_ft

Design of members for shear parallel to y axis - Chapter G

Required shear strength $V_{ry} = \mathbf{5.000}$ kips

Web area $A_w = 2 \times (d - 3 \times t) \times t = \mathbf{2.984}$ in^2

Web plate buckling coefficient $k_v = \mathbf{5}$

Web shear coefficient - eq G2-3 $C_v = \mathbf{1}$

Nominal shear strength – eq G2-1 $V_{ny} = 0.6 \times F_y \times A_w \times C_v = \mathbf{89.517}$ kips

Safety factor for shear $\Omega_v = \mathbf{1.67}$

Allowable shear strength $V_{cy} = V_{ny} / \Omega_v = \mathbf{53.603}$ kips

Design of members for shear parallel to x axis - Chapter G

Required shear strength $V_{rx} = \mathbf{5.000}$ kips

Web area $A_w = 2 \times (b_f - 3 \times t) \times t = \mathbf{2.984}$ in^2

Web plate buckling coefficient $k_v = \mathbf{5}$

Web shear coefficient - eq G2-3 $C_v = \mathbf{1}$

Nominal shear strength – eq G2-1 $V_{nx} = 0.6 \times F_y \times A_w \times C_v = \mathbf{89.517}$ kips

Safety factor for shear $\Omega_v = \mathbf{1.67}$

Allowable shear strength $V_{cx} = V_{nx} / \Omega_v = \mathbf{53.603}$ kips

Reduction factor for slender elements

Reduction factor for slender elements (E7)

The section does not contain any slender elements therefore:-

Slender element reduction factor $Q = \mathbf{1.0}$

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = \mathbf{106.1}$ ksi

Reduction factor $Q_x = Q = \mathbf{1.000}$

Flexural buckling stress about x axis $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y = \mathbf{41.0}$ ksi

Nominal flexural buckling strength $P_{nx} = F_{crx} \times A_g = \mathbf{263.9}$ kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (SR_y)^2 = \mathbf{106.1}$ ksi

Reduction factor $Q_y = Q = \mathbf{1.000}$

Flexural buckling stress about y axis $F_{cry} = Q_y \times (0.658^{Q_y \times F_y / F_{ey}}) \times F_y = \mathbf{41.0}$ ksi

Nominal flexural buckling strength $P_{ny} = F_{cry} \times A_g = \mathbf{263.9}$ kips

Allowable compressive strength (cl. E1)

Safety factor for compression $\Omega_c = \mathbf{1.67}$

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}) / \Omega_c = \mathbf{158.0}$ kips

PASS - The allowable compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F7.1)

Nominal flexural strength $M_{nx\_yld} = M_{px} = F_y \times Z_x = \mathbf{56.7}$ kips_ft

Allowable flexural strength about the major axis (cl. F1)

Safety factor for flexure Ω_b = **1.67**

Allowable flexural strength $M_{cx} = \min(M_{nx\_yld}) / \Omega_b$ = **33.932** kip_ft

PASS - The allowable flexural strength about the major axis exceeds the required flexural strength

Library item - Design flex str ASD x RHS**Flexural strength about the minor axis**

Yielding (cl. F7.1)

Nominal flexural strength $M_{ny\_yld} = M_{py} = F_y \times Z_y$ = **56.7** kips_ft

Allowable flexural strength about the minor axis (cl. F1)

Safety factor for flexure Ω_b = **1.67**

Allowable flexural strength $M_{cy} = M_{ny\_yld} / \Omega_b$ = **33.9** kips_ft

PASS - The allowable flexural strength about the minor axis exceeds the required flexural strength

Combined forces

Member utilization (cl. H1.1)

Equation H1-1a $UR = \mathrm{abs}(P_r) / P_c + 8 / 9 \times (M_{rx} / M_{cx} + M_{ry} / M_{cy})$ = **0.994**

PASS - The member is adequate for the combined forces

KREHER ENGINEERING, INC.
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FAX (618) 281-8515

JOB

SHEET NO. OF

CALCULATED BY DATE

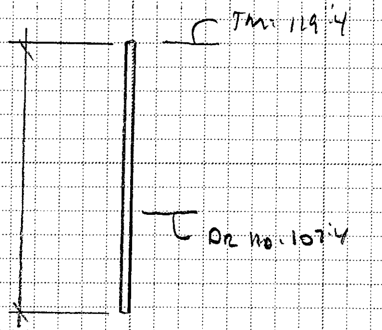
CHECKED BY DATE

SCALE

WALL (LINTEL)

8" CMU = 54 psf + Plaster = 2 psf = 56 psf

L1a



Wall wt. = 56 (119'4 - 107'4) 670 #/ft DL

Roof wt. 2950 #/ft DL

385 #/ft LL

3930 #/ft

L1a - Span: 3'4 - Design Sp. = 4'0

M_x = 7860 lb-ft

V_x 7860 lb

8" x 16 CMU w/ 2) #5 e BTM

W/ #3 STIRRUPS @ 4" o.c.

L2a

Wall wt. 65 psf

T.M. 114'8

Op. 107'4

7'4

w: 65 (114'8 - 107'4) = 480

L1b

Wall wt. 56 (114'8 - 107'4) = 415 #/ft

Span 3'4 - Design 4'0

M_x = 830 ft-lb

V_x 830 lb

8" x 16" CMU Bond Bm w/ 2) #5 e BL

Wr: DL 2950 + 1400

LL - 385 + 210

5425 lb/ft

Span 3'4 Design 4'0

M_x 10850

V_x 10850

L3a - Sp: 6'4 Design 7'0

w: 5426 #/ft M_x -

I_x -

W8x21 w/ Bm Pl

10" x 24" CMU w/ 2) #5

L3b: S. 14'6 D = 15'2"

w = 56 (119'4 - 110'0) = 525

w: 155 (2) + 26 (2) = 365

890 #/ft TL

90 #/ft LL

W8x28

L1a

MASONRY LINTEL DESIGN TO TMS/MSJC 2016

Using the allowable stress design method

Tedds calculation version 1.2.02

Masonry details

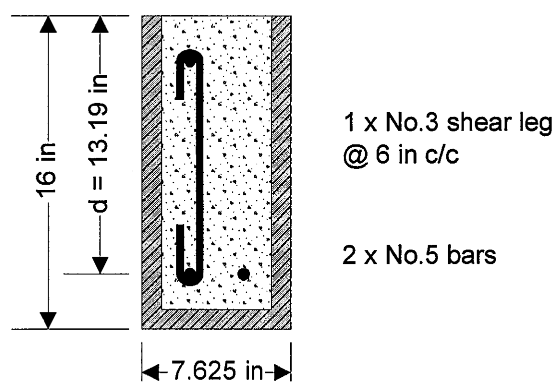
Masonry type	Concrete
Density of masonry unit	γ = 135 lb/ft^3
Pattern bond	Running
Mortar type	PCL Type S
Compressive strength of masonry unit	f'_{cu} = 3250 psi
Net compressive strength of masonry (Table 2)	f'_m = 2500 psi
Modulus of elasticity (4.2.2)	$E_m = 900 \times f'_m$ = **2250000** psi
Allowable flexural tensile stress (8.2.4.2)	F_t = 106 psi

Reinforcement details

Allowable tensile stress	F_s = 32000 psi
Modulus of elasticity of steel	E_s = 29000000 psi

Cover to reinforcement

Bottom cover to reinforcement	c_{nom_b} = 2 in
Side cover to reinforcement	c_{nom_s} = 1.5 in



Section properties

Modular ratio	$n = E_s / E_m$ = **12.89**
Section width	b = **7.625** in
Section depth	h = **16** in
Net shear area	$A_{nv} = b \times d$ = **100.57** in^2
Section modulus	$S = b \times h^2 / 6$ = **325.33** in^3
Depth to tension reinforcement	d = **13.19** in

Flexure design (Chapter 8)

Tension reinforcement	2 x No. 5 bars
Area of tension reinforcement	$A_s = N_{bot} \times BarArea_{bot}$ = **0.62** in^2
Reinforcement ratio	$\rho_{ratio} = A_s / (b \times d)$ = **0.00616**

Neutral axis factor	$k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n = \mathbf{0.327}$
Lever arm factor	$j = 1 - k / 3 = \mathbf{0.891}$
Cracking moment	$M_{cr} = 2.5 \times F_t \times S =$ **7.2** kip_ft
Design bending moment	M = **7.90** kip_ft
Tensile stress in reinforcement	$f_s = M / (A_s \times j \times d) =$ **13011** psi
Allowable tensile stress in reinf. (8.3.3.1)	$F_s =$ **32000** psi
Reinforcement stress ratio	$f_s / F_s = \mathbf{0.407}$

PASS - Allowable tensile stress exceeds tensile stress due to flexure

Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2) =$ **490.5** psi
Allowable stress in masonry (8.3.4.2.2)	$F_b = 0.45 \times f'_m =$ **1125.0** psi
Masonry stress ratio	$f_b / F_b = \mathbf{0.436}$

PASS - Allowable compressive stress exceeds compressive stress due to flexure

Shear design (Chapter 8)

Design shear force	V = **7.90** kips
Shear stress (8-21)	$f_v = V / A_{nv} =$ **78.5** psi
Maximum allowable shear stress (8-24)	$F_v = 2 \times \sqrt{(f'_m \times 1psi)} =$ **100.0** psi
Shear stress ratio	$f_v / F_v = \mathbf{0.785}$

PASS - Maximum allowable shear stress exceeds shear stress

Moment shear relationship, M/Vd	Assume $M\_Vd_{ratio} = \mathbf{1.0}$
Allowable shear stress resisted by masonry	$F_{vm} = 1/2 \times (4.0 - 1.75 \times M\_Vd_{ratio}) \times \sqrt{(f'_m \times 1psi)} =$ **56.3** psi
Allowable shear stress in steel	$F_{vs} = f_v - F_{vm} =$ **22.3** psi
Shear reinforcement	1 x No. 3 leg at 6 in c/c
Maximum shear reinf. spacing (8.3.5.2.1)	$s_{max} = \min(d / 2, 48 \text{ in}) =$ **6.6** in

PASS - Shear reinforcement spacing is less than maximum allowable

Area of shear reinforcement provided	$A_v = N_v \times BarArea_v =$ **0.11** in^2
Area of shear reinforcement required (8-27)	$A_{v\_req} = 2 \times F_{vs} \times A_n \times s / (F_s \times h) =$ **0.06** in^2

PASS - Area of shear reinforcement provided exceeds required shear reinforcement

Minimum longitudinal reinf. req'd (8.3.5.2.2)	$A_{long\_min} =$ **0.04** in^2

PASS - Flexural reinforcement satisfies minimum longitudinal reinforcement requirement

MASONRY LINTEL DESIGN TO TMS/MSJC 2016

L1b

Using the allowable stress design method

Tedds calculation version 1.2.02

Masonry details

Masonry type — Concrete

Density of masonry unit — γ = 135 lb/ft^3

Pattern bond — Running

Mortar type — PCL Type S

Compressive strength of masonry unit — f'_{cu} = 3250 psi

Net compressive strength of masonry (Table 2) — f'_m = 2500 psi

Modulus of elasticity (4.2.2) — $E_m = 900 \times f'_m$ = **2250000** psi

Allowable flexural tensile stress (8.2.4.2) — F_t = 106 psi

Reinforcement details

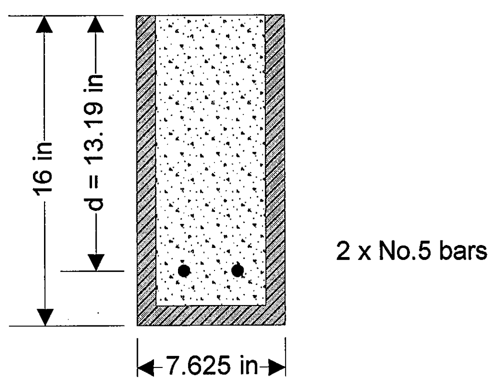
Allowable tensile stress — F_s = 32000 psi

Modulus of elasticity of steel — E_s = 29000000 psi

Cover to reinforcement

Bottom cover to reinforcement — c_{nom_b} = 2 in

Side cover to reinforcement — c_{nom_s} = 1.5 in



Section properties

Modular ratio — $n = E_s / E_m$ = **12.89**

Section width — b = **7.625** in

Section depth — h = **16** in

Net shear area — $A_{nv} = b \times d$ = **100.57** in^2

Section modulus — $S = b \times h^2 / 6$ = **325.33** in^3

Depth to tension reinforcement — d = **13.19** in

Flexure design (Chapter 8)

Tension reinforcement — 2 x No. 5 bars

Area of tension reinforcement — $A_s = N_{bot} \times BarArea_{bot}$ = **0.62** in^2

Reinforcement ratio — $\rho_{ratio} = A_s / (b \times d)$ = **0.00616**

Neutral axis factor $k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n = \mathbf{0.327}$

Lever arm factor $j = 1 - k / 3 = \mathbf{0.891}$

Cracking moment $M_{cr} = 2.5 \times F_t \times S = \mathbf{7.2}$ kip_ft

Design bending moment $M = \mathbf{0.83}$ kip_ft

Tensile stress in reinforcement $f_s = M / (A_s \times j \times d) = \mathbf{1367}$ psi

Allowable tensile stress in reinf. (8.3.3.1) $F_s = \mathbf{32000}$ psi

Reinforcement stress ratio $f_s / F_s = \mathbf{0.043}$

PASS - Allowable tensile stress exceeds tensile stress due to flexure

Compressive stress in masonry $f_b = 2 \times M / (j \times k \times b \times d^2) = \mathbf{51.5}$ psi

Allowable stress in masonry (8.3.4.2.2) $F_b = 0.45 \times f'_m = \mathbf{1125.0}$ psi

Masonry stress ratio $f_b / F_b = \mathbf{0.046}$

PASS - Allowable compressive stress exceeds compressive stress due to flexure

Shear design (Chapter 8)

Design shear force $V = \mathbf{0.83}$ kips

Depth of shear area $d_v = \mathbf{16.00}$ in

Moment shear relationship, M/Vd Assume $M\_Vd_{ratio} = 1$

Shear stress (8-21) $f_v = V / A_{nv} = \mathbf{8.3}$ psi

Allowable masonry shear stress (8-26) $F_v = 1/2 \times (4.0 - 1.75 \times M\_Vd_{ratio}) \times \sqrt{(f'_m \times 1psi)}$

$F_v = \mathbf{56.3}$ psi

Masonry shear stress ratio $f_v / F_v = \mathbf{0.147}$

PASS - Allowable shear stress exceeds shear stress in masonry

L2a

MASONRY LINTEL DESIGN TO TMS/MSJC 2016

Using the allowable stress design method

Tedds calculation version 1.2.02

Masonry details

Masonry type — Concrete

Density of masonry unit — γ = 135 lb/ft^3

Pattern bond — Running

Mortar type — PCL Type S

Compressive strength of masonry unit — f'_{cu} = 3250 psi

Net compressive strength of masonry (Table 2) — f'_m = 2500 psi

Modulus of elasticity (4.2.2) — $E_m = 900 \times f'_m =$ **2250000** psi

Allowable flexural tensile stress (8.2.4.2) — F_t = 106 psi

Reinforcement details

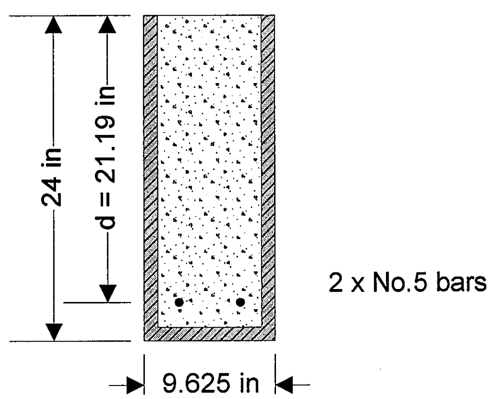
Allowable tensile stress — F_s = 32000 psi

Modulus of elasticity of steel — E_s = 29000000 psi

Cover to reinforcement

Bottom cover to reinforcement — c_{nom_b} = 2 in

Side cover to reinforcement — c_{nom_s} = 1.5 in



Section properties

Modular ratio — $n = E_s / E_m =$ **12.89**

Section width — b = **9.625** in

Section depth — h = **24** in

Net shear area — $A_{nv} = b \times d =$ **203.95** in^2

Section modulus — $S = b \times h^2 / 6 =$ **924** in^3

Depth to tension reinforcement — d = **21.19** in

Flexure design (Chapter 8)

Tension reinforcement — 2 x No. 5 bars

Area of tension reinforcement — $A_s = N_{bot} \times BarArea_{bot} =$ **0.62** in^2

Reinforcement ratio — $\rho_{ratio} = A_s / (b \times d) =$ **0.00304**

Neutral axis factor	$k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n$ = **0.243**
Lever arm factor	$j = 1 - k / 3$ = **0.919**
Cracking moment	$M_{cr} = 2.5 \times F_t \times S$ = **20.4** kip_ft
Design bending moment	M = **10.85** kip_ft
Tensile stress in reinforcement	$f_s = M / (A_s \times j \times d)$ = **10786** psi
Allowable tensile stress in reinf. (8.3.3.1)	F_s = **32000** psi
Reinforcement stress ratio	f_s / F_s = **0.337**

PASS - Allowable tensile stress exceeds tensile stress due to flexure

Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2)$ = **269.3** psi
Allowable stress in masonry (8.3.4.2.2)	$F_b = 0.45 \times f'_m$ = **1125.0** psi
Masonry stress ratio	f_b / F_b = **0.239**

PASS - Allowable compressive stress exceeds compressive stress due to flexure

Shear design (Chapter 8)

Design shear force	V = **10.83** kips
Depth of shear area	d_v = **24.00** in
Moment shear relationship, M/Vd	Assume $M\_Vd_{ratio} = 1$
Shear stress (8-21)	$f_v = V / A_{nv}$ = **53.1** psi
Allowable masonry shear stress (8-26)	$F_v = 1/2 \times (4.0 - 1.75 \times M\_Vd_{ratio}) \times \sqrt{(f'_m \times 1psi)}$
	F_v = **56.3** psi
Masonry shear stress ratio	f_v / F_v = **0.944**

PASS - Allowable shear stress exceeds shear stress in masonry

Gravity Beam Design

L3a

RAM SBeam v3.0
Licensed to: Kreher Engineering, Inc.

05/29/24 15:27:44

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (7.00,0.00)

Beam Size (User Selected) = W8X21 Fy = 50.0 ksi
Total Beam Length (ft) = 7.00

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL
1	0.000	0.021	0.000
	7.000	0.021	0.000
2	0.000	4.830	0.595
	7.000	4.830	0.595

SHEAR: Max V (DL+LL) = 19.06 kips fv = 9.21 ksi Fv = 20.00 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	Max +	33.4	3.5	7.0	1.00	21.99	30.00	21.99	30.00
Controlling		33.4	3.5	7.0	1.00	21.99	30.00	---	---

REACTIONS (kips):

	Left	**Right**
DL reaction	16.98	16.98
Max +LL reaction	2.08	2.08
Max +total reaction	19.06	19.06

DEFLECTIONS:

Dead load (in)	at	3.50 ft	=	-0.120	L/D	=	700
Live load (in)	at	3.50 ft	=	-0.015	L/D	=	5707
Net Total load (in)	at	3.50 ft	=	-0.135	L/D	=	623

Gravity Beam Design

RAM SBeam v3.0
Licensed to: Kreher Engineering, Inc.

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STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (15.17,0.00)

Beam Size (User Selected) = W8X28 Fy = 50.0 ksi
Total Beam Length (ft) = 15.17

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL
1	0.000	0.028	0.000
	15.170	0.028	0.000
2	0.000	0.800	0.090
	15.170	0.800	0.090

SHEAR: Max V (DL+LL) = 6.96 kips fv = 3.03 ksi Fv = 20.00 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	Max +	26.4	7.6	15.2	1.00	13.04	30.00	13.04	24.87
Controlling		26.4	7.6	15.2	1.00	---	---	13.04	24.87

REACTIONS (kips):

	Left	**Right**
DL reaction	6.28	6.28
Max +LL reaction	0.68	0.68
Max +total reaction	6.96	6.96

DEFLECTIONS:

Dead load (in)	at	7.59 ft	=	-0.347	L/D =	524
Live load (in)	at	7.59 ft	=	-0.038	L/D =	4824
Net Total load (in)	at	7.59 ft	=	-0.385	L/D =	473